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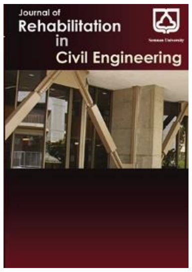

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Experimental Study for Strength Capacity of Cold-Formed Steel Joists Connections with Consideration of Various Bolts Arrangements

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ABSTRACT

With growth the construction technologies, Cold-Formed Steel, CFS, sections are widely used in ordinary steel buildings because of some advantages such as light weight, ease of installation, decrease in cost, and increase in speed of operation. Using the bolted connections for CFS joist is one of the best details for steel structures. The main objective of this study is to conduct an experimental research to evaluate the load carrying capacity of bolted connections based on various bolts arrangement. Ten full scale joist-beam connections are tested under the incremental gravity load. The variable parameters are the arrangement of bolts, and thickness of CFS sheets. The joist sections made of two C-shaped, which are back-to-back connected using self-drilling screw bolts in the web. The arrangement of bolts connection and steel sheet thickness are considered as two major factors to improve the load carrying capacity. Base on the obtained results, it was observed that increasing the number of the bolts and their spacing from the neutral axes led to the additional load carrying capacity. Furthermore, it can be concluded that the thickness of CFS sheets play an effective role for load carrying capacity of connections.

1. Introduction

The joists and girders in flooring systems carry the gravity loads and transfer them to columns. Since 1850s, the CFS members have been considering for steel structures, however, the major development of CFS was initiated by Winter [1]. The failure mechanism of bolted connections for CFS joists cannot be similar to the bolted connection for the hot-rolled steel joists [2-

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4]. Unlike hot-rolled steel constructions, utilizing the combination of both slip-critical and bearing type connection, by considering the bearing connection the CFS bolted connection capacity can be satisfied. However, extreme dishing of connected sheets and bolt rotation can be detected in CFS. According to the North American Specification for the Design of Cold-Formed Steel Structural Members, AISI S100 [5], four failure modes were considered for CFS bolted connections. These failure modes have been categorized as follows: 1-shear failure of sheet, 2-bearing failure of sheet, 3-tearing/rupture in net section, and 4-shear failure in bolt.

Today, the Cold-Formed Steel, CFS, sections are widely used in steel structures as a non-bearing structural elements, because of some inherent advantages of CFS, such as light weighting, ease of construction, high speed of construction, and reduction of the construction cost [6]. This paper mainly focuses on the bearing failure of sheet [7]. The assessment of bearing strength of bolted connections was developed by Yu [8], Zadanfarrokh and Bryan [9], LaBoube and Yu [10], and Wallace et al [11, 12], which were considered as a fundamental researches for AISI S100 [5]. Following Yu and Panyanouvong [13] conducted the experimental study to evaluate the bearing strength of CFS bolt connections with consideration of the edge distance from the bolted connections as a main variable. Recently, a new generation of the reliability index has been defined by Ghasemi and Nowak (2016-2017). Screws are widespread kinds of connections which are utilized for cold-formed steel. Because of the tiny thickness of the cold-formed steel, the screw connectors can provide a decent advantage in terms of the straightforward design protocols and quick putting in place.

Daudet and LaBoube firstly development and testing of self-drilling screw for CFS. They [14] conducted series of the experimental investigation of CFS connections using drilling screws with applying a particular shear. Several researches [15-17] scrutinized the moment carrying capacity and rigidity of the self-drilling screws to evaluate the resistance of self-drilling screws, in most of the cases they concluded that the moment carrying capacity of self-drilling screws is significantly higher than the ordinary used connectors.

Peköz [18] was the first researcher who derives a design formula for screwed connectors. Accordingly, Serrette and D. Peyton figured out the safety level of the CFS connectors. However, the safety level of this type of the connectors should be evaluated using reliability analysis. To do So, Ghasemi and Nowak developed a reliability approach to evaluate the safety level of the structural members. The advantage of Ghasemi-Nowak [20-21] formula is to measure the safety level of a non-normal limit state function using the simplified convolution concepts. For evaluation of the required reliability index of structures, there is a need for expression of the optimum safety level, which is known as target reliability. Ghasemi and Nowak [22] and Yanaka et al. [23] proposed several approaches to estimate the target reliability for bridges with respect to the minimization of the cost.

The bolted connections for joists should be designed based on transferred forces due to gravity load, which transfer the shear force or bending moment to the connections. In this research, load carrying capacity of the bolted connections in CFS joists were experimentally evaluated based on the variation of bolts arrangement and thickness of CFS sheets. 10 full-scaled bolted connections of the CFS joists were tested under incremental gravity load. Finally, load-carrying capacity of the connections was assessed.

2. Methodology

In order to determine the bearing capacity of the connection, regardless of the holes deformation, AISI S100 [5] proposes the following formula

$P_n = m_f C d t F_u$ (1)

where, Fu is the ultimate stress, d is the width of plate, t is the thickness of the plate. m_f indicates a modification factor of connection type. C is a bearing coefficient which depends on d/t. Both coefficients (m_f and C) can be determined based on the study by Wallace et al [11, 12]. Also, Wallace et al [11, 12] were conducted a several tensional experiments on bolted connections of CFS to determine the m_f and C. Schafer [16] provided a vast study to review the direct strength method of cold-formed steel member design.

3. Experimental Program

In this study, the joist sections made of two C-shaped, which their webs were back-to-back attached by self-drilling screw bolts, were examined [14] (see Fig.1).

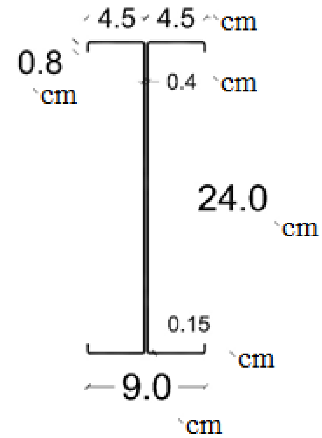

Fig. 1. C-shape cross section.

The joists were connected through the web/flange with steel bolts and nutson two sides using one/two hot-rolled steel plates. These hot-rolled plates made of one/two welded L-shaped. Finally, the L-shaped were attached to the girder with bolts.

3.1. Basic Design

In this study, a 6.0 m CFS joist, which made of double C-shaped sections, was investigates. The tributary width was 5.0 m. The design calculation was computed based on ASCE 7-16 [24]. The considered geometric properties of the joists were investigated as follows: the thickness of the sheets were considered about 1.0, 1.5, and 2.0 mm. The web is 240 mm in height, the flange is 45 mm in width, and the edge of flange is 8 mm (see Fig.2).

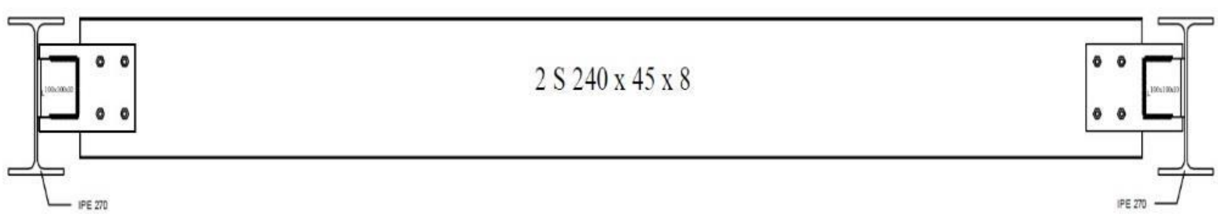

Fig. 2. Cold Formed Steel (CFS) joist.

In this experiment, the scale of cross-section was considered the same as the real one. However, based on the lab's limitations, the lengths of the joist were considered 600 mm, without any hole in the web. It is worth mentioning that the experiment was considered only one of the supports. The 600 mm length was divided into two parts: 400 mm and 200 mm. As a matter of fact, the 400 mm is included the range of 1.5 to 2 times the height of the joists, where the plastic hinge in the joists can be formed, where the related failure modes is occurred. The 200 mm is reserved for the steel sheet in order to transfer the distributed loads on the joists rather than the concentrated loads.

3.2. Material Properties

The thickness of the sheets is considered about 1.0, 1.5, and 2.0 mm. The properties of sheet material illustrates in Table1.The hammer drive bolts with 4/8 diameters and 19 mm length were used; in order to back-to-back attach the C-shaped. Also, the 8.8 grade bolts and nuts with 10 mm diameters and 40 mm length were utilized to attach the joist to the sheets. It is noteworthy that the yield stress and ultimate stress of 8.8 grade bolts and nuts was 640 MPa and 800 MPa, consequently. The material properties tabulates in Table 1.

Table 1. Material properties.

Properties	**Steel**
Density(kg/m$^{3)}$	7850
Elastic Modulus (MPa)	2.03×10^5
Poisson`s Ratio	0.3
Yield Strength (MPa)	240
Ultimate Strength (MPa)	370

3.3. Test Specimens

In this study, 10 types of the connections were recommended and tested. The number and arrangements of bolts and thickness of the sheets were the variable parameters. Table 2 and Fig. 3 illustrate the details of each specimen.

Table 2. The specimen specifications.

Specimen Number	Bolt Number	Specimen section	Bolted arrangement
1	1-2-3	Two C shaped with thickness of 1.5 mm	Three holes arranged in a perpendicular line (perpendicular to the longitudinal direction of the joists).
2	1-5-3	Two C shaped with thickness of 1.5 mm	Three holes arranged in a staggered pattern.
3	1-3-4-6	Two C shaped with thickness of 1 mm	Four holes arranged in two lines of a symmetric and parallel relation to each other and in relation to the horizontal direction.
4	1-3-4-6	Two C shaped with thickness of 1.5 mm	Four holes arranged in two lines of a symmetric and parallel relation to each other and in relation to the horizontal direction.
5	1-3-4-6	Two C shaped with thickness of 2 mm	Four holes arranged in two lines of a symmetric and parallel relation to each other and in relation to the horizontal direction.
6	1-3-4-6	Two C shaped with thickness of 1.5 mm	Four holes arranged in two lines of a symmetric and parallel relation to each other and in relation to the horizontal direction and the connection of this type of specimen was supplied just using one L-shaped.
7	8-10-11-13	Two C shaped with thickness of 1.5 mm	Four holes arranged in a rhomboidal pattern.
8	1-2-3-4-5-6	Two C shaped with thickness of 1.5 mm	Six holes arranged in two lines perpendicular to the longitudinal direction of the joists.
9	7-8-9-12-13-14	Two C shaped with thickness of 1.5 mm	Six holes arranged in two lines of a symmetric and parallel relation to each other and in relation to the horizontal direction.
10	15-16-17-18	Two C shaped with thickness of 1.5 mm	Two hot-rolled plates and 8 bolts and nuts, which they connect the top and bottom flanges of joist to the top and bottom flanges of girder.

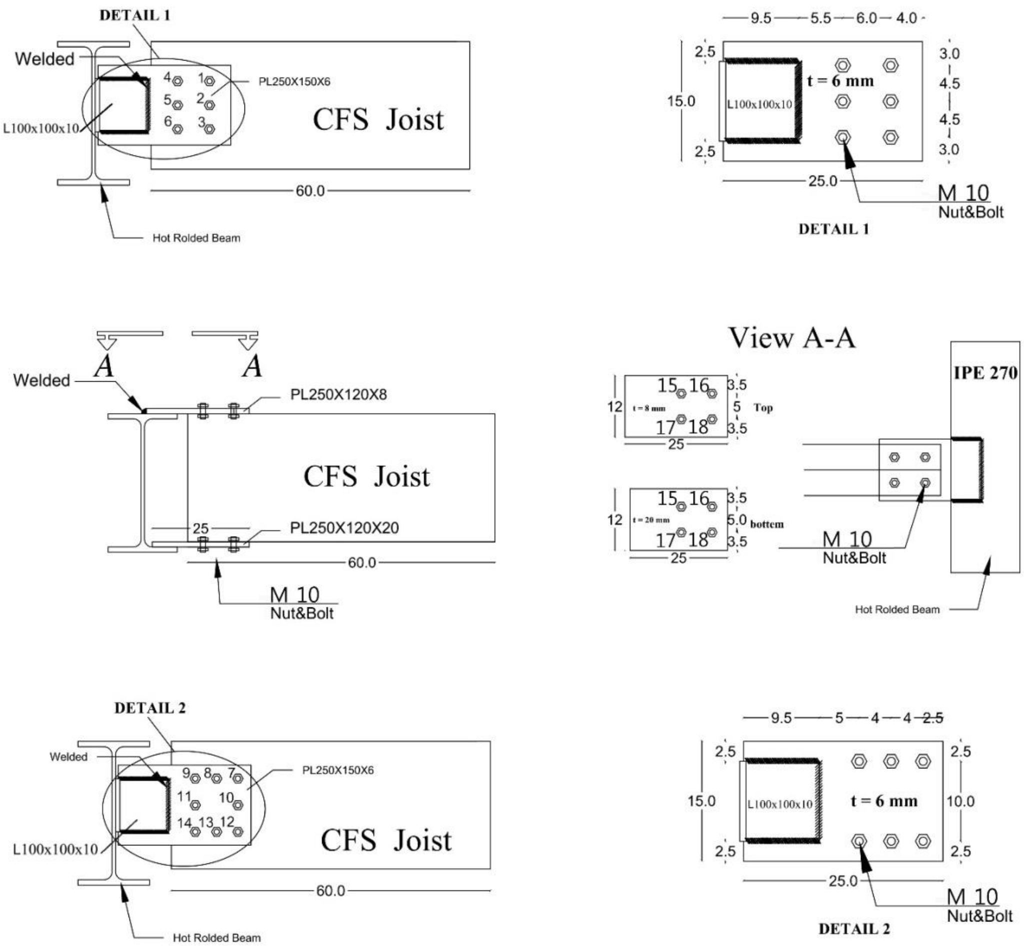

Fig. 3. Bolted arrangement of the test specimens.

3.4. Loading

The applied load was considered as a static load with deliberation of the incremental gravity loads behavior. In order to prevent the concentrated stress under the applied load, the load was applied on the steel plates with 200 mm length and 10 mm thickness. The distance from the center of applied load to the end of the joist was 500 mm and to the tip was 100 mm (see Fig. 4).

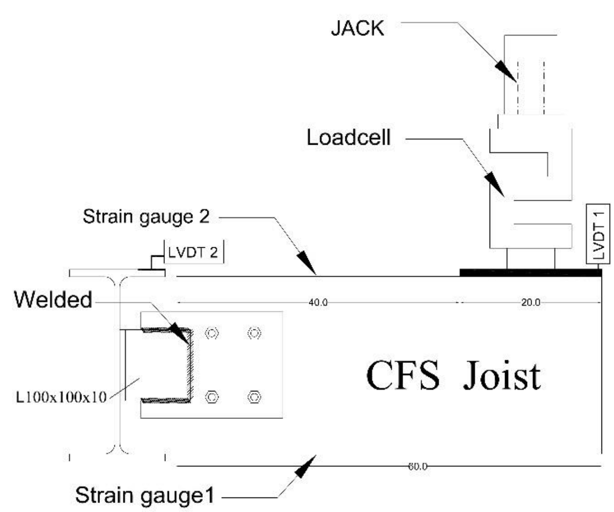

Fig. 4. Bolted arrangement of the test specimens.

3.5. Measuring Instruments

In this study, the loadcell was used to determine the magnitude of the applied load. The maximum loadcell capacity is about 100 KN. Two strain gauges were used, which one of them was placed on the top of the tip of the joist closed to the applied load, and the second one was attached to the flange of the joist, to measure the displacement and rotation. (See Fig. 5)

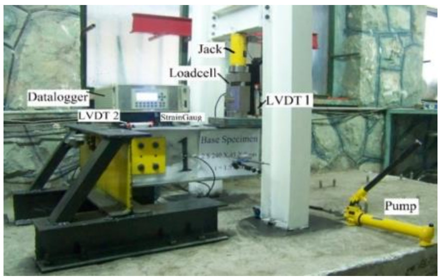

Fig. 5. Measurement instruments layout and test setup.

3.6. Determination of Rotation and Moment Carrying Capacity of Connections

The rotations of specimens can be appeared, because of the applied load at the tip of the joists. The relative displacement of joists flange were measured by using LVDT, where was placed on the top flange of each specimen. Since the joists rotation were small enough, the joists rotations (θ) can be considered as a ratio of the horizontal displacement (Δ) to the half of the height of the joist (H/2), which shows in Fig. 6 and demonstrates in Eq. 2. The test results tabulate in Table 3.

The computed rotations of the joists display in Table 3. Also, the moment carrying capacity for each specimen was computed based on the multiplication of the maximum applied load by the moment's arm (the distance from the center of the applied load to the geometric center of the bolts). The results of the moment carrying capacity exhibits in Table 3.

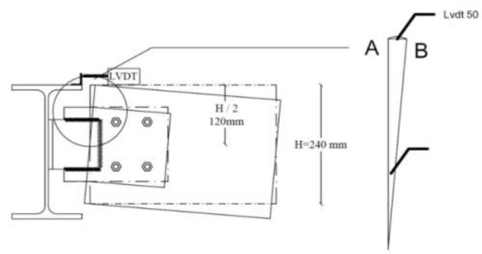

Fig. 6. Measuring instruments layout and test setup.

$$\boldsymbol{\theta} = \frac{\overline{AB}}{\frac{H}{2}} = \frac{\Delta}{120} \text{ (rad)} \quad (2)$$

4. Test Results

4.1. Bolts Arrangements

The load-carrying capacity, displacement and rotation at the tip of the joist tabulates in Table 3.

Table 3. Test Results including Flexural moment at the support, tip displacement, Relative displacement of joist flange, and rotation.

Specimen	F (KN) Applied load	M (KN.m) Flexural moment at the support	LV DT 1 (mm) Tip displacement	LVDT 2 (mm) Relative displacement of joist flange	θ (rotation)
No. 1	8.021	3.13	100	19.98	0.1665
No. 2	10.375	4.25	89.75	19.14	0.1595
No. 3	2.353	0.99	17.14	2.15	0.0179
No. 4	7.913	3.323	55.86	10.57	0.0881
No. 5	9.679	4.07	63.18	10.89	0.0908
No. 6	4.452	1.87	55.96	11.18	0.0932
No. 7	9.806	4.07	65.63	11.91	0.0993
No. 8	6.874	2.89	36.52	7.81	0.0651
No. 9	7.972	3.31	37.79	5.15	0.0429
No. 10	17.925	7.53	12.74	0.01	0.0001

4.1.1 Deformation Mechanism of Test Specimens

Specimens No. 1 and No.2: in these two tests, the joists tolerated the elastic rotation and displacement without considerable damage, but the upper and lower holes were suffered the bearing failure (Fig. 7. and Fig. 8.). Also, the force-displacement diagram of specimen No. 1 and No. 2 was displayed in Fig. 9. As can be seen in Fig. 9, the load carrying capacity of specimen No.2 was about 30 percent higher than specimen No. 1, which demonstrates the influence of the bolts arrangements on the load carrying capacity of connections.

Fig. 7. Deformation mechanism of testspecimen No. 1.

Fig. 8. Deformation mechanism of testspecimen No. 2.

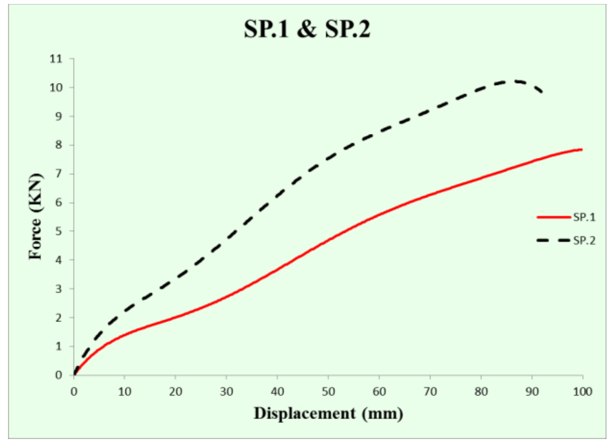

Fig. 9. Force vs. displacement diagram for specimen No. 1 and No. 2.

Specimens No.4 and No.7: in these two tests, the joists tolerated the less rotation and displacement compare to the Specimens No. 1 and No. 2. However, because of the lack of lateral bracing, they experienced web buckling and the plastic deformation, moreover, the bearing failure was detected (Fig. 10. and Fig. 11.). Fig. 12 shows the force-displacement diagram of specimen No. 4 and No. 7. As shown in Fig. 12, the load carrying capacity of specimen No.4 was about 25 percent higher than specimen No. 7, which represents the effect of the bolts arrangements on load carrying capacity of connections.

Specimens No. 8 and No. 9: in these two tests, the joists experienced the less rotation and displacement than those of the previous four specimens.

Fig. 10. Deformation mechanism of test specimen No. 7.

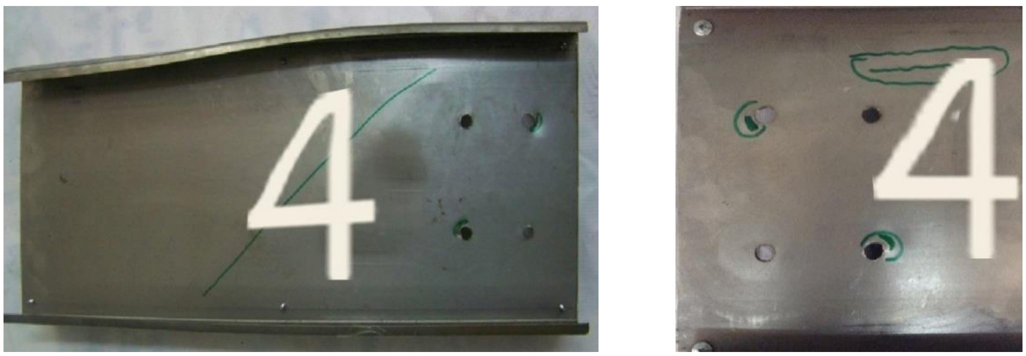

Fig. 11. Deformation mechanism of test specimen No. 4.

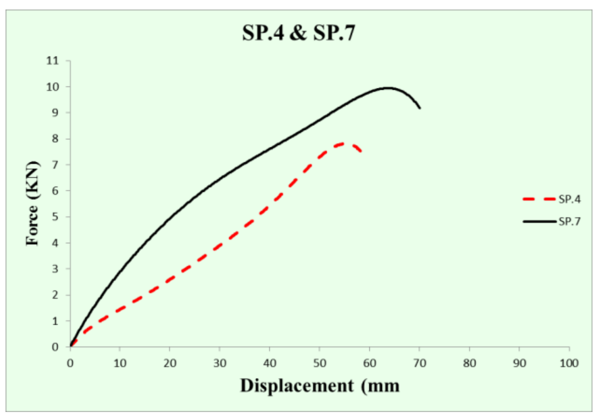

Fig. 12. Force vs. displacement diagram for specimen No. 4 and No.7.

However, specimen No. 5 suffered plastic deformation and specimen No. 6 underwent the elastic deformation, therefore, the holes remained completely undamaged (Fig. 13. and Fig. 14). As depicted in Fig. 15, the load carrying capacity of specimen No.8 was about the same as the specimen No. 9.

Fig. 13. Deformation mechanism of test specimen No. 8.

Fig. 14. Deformation mechanism of test specimen No. 9.

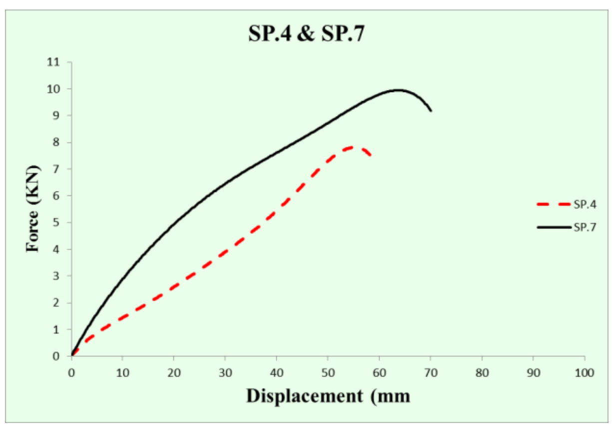

Fig. 15. Force vs. displacement diagram for specimen No. 8 and No. 9.

4.2. Thickness Variation

Although, as an arrangement point of the view, specimens No. 3 and No. 5 are similar to the specimens No. 4, the thickness of No.3 was 1.0 mm, and the thickness of specimen No. 5 was 2.0 mm.

Specimen No. 3: because of the thin thickness of the sheet of specimen No. 3 (Fig. 16), the insignificant load magnitude caused the buckling and torsion in web (Fig. 17). However, no bearing failure was detected.

Specimen No. 5: since the thickness of No. 5 was ticker than specimen No. 4, it gradually incurred the vertical displacement and rotation (Fig. 18), without any noticeable damage in joist and holes (Fig. 19). Accordingly, the load-displacement diagram was displayed for specimen No. 3, No. 4, and

No.5. As a result, it was observed that the load carrying capacity of specimen No. 3 is just about 30 percent of specimen No. 4, and load carrying capacity of specimen No. 4, is just about 80 percent of specimen No. 5. Fig. 20 shows the force-displacement diagram of specimen No. 3, No. 4, and No. 5.

Fig. 16(a). After loading (plastic buckling).

Fig. 16(b). Specimen No.3 subjected to the gravity load.

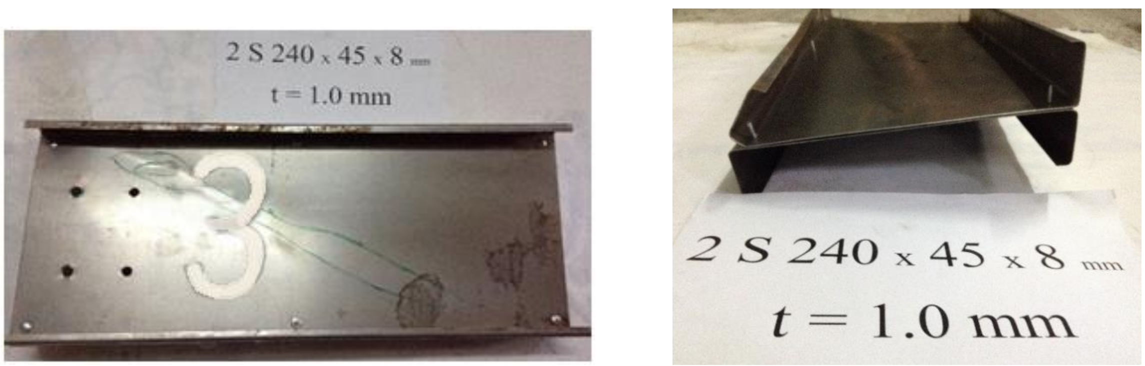

Fig. 17. Web buckling.

Fig. 18. Specimen rotation without considerable damage.

Fig. 19. Undamaged specimens after loading (elastic buckling), web elastic buckling without damage in holes.

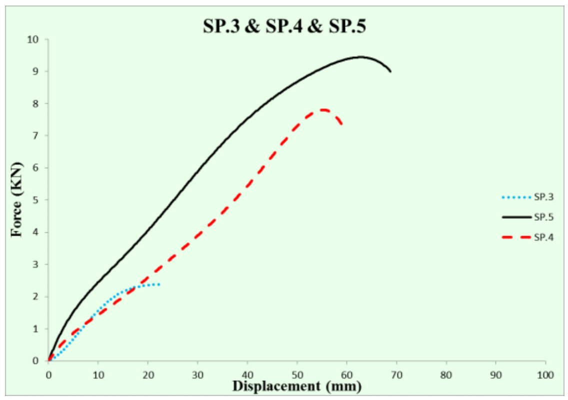

Fig. 20. Force vs. displacement diagram for specimen No. 3, No. 4, and No. 5.

Specimen No. 10: this connection consisted of two hot-rolled plates and 8 bolts and nuts, which they connected the top and bottom flanges of joist to the top and bottom flanges of the girder. This specimen presented a proper withstand-ability against the applied load. As experienced, for the examined specimen, up to 5000 N the vertical displacement at the tip was negligible. However, after that, the vertical displacement began to be gradually increased. Once the load reached to about 18000 N, the vertical displacement exceeded 12 mm. Accordingly, the *longitudinal* plastic buckling in web was detected. Also, the displacement of specimen No.10 was about 20-30 percent of specimen No. 4. Moreover, the load carrying capacity of this specimen was 2-3 times higher than specimen No. 4.

Fig. 21. Web plastic buckling without considerable damage in flanges and holes.

Specimen No.6: the connection of this type of specimen was supplied using one L-shaped. The considerable vertical displacement and rotation were experienced, immediately after applying load. Once the applied load reached to about 4500 N, the elastic buckling with torsion in the joist web has been occurred. The load carrying capacity of two L-shaped connector is about two times of one L-shaped connector. Fig. 22 displays the force-displacement diagram of specimen No. 4, No. 6, and No. 10.

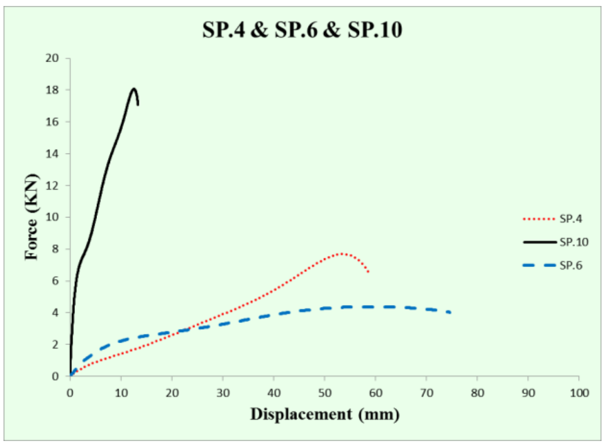

Fig. 22. Force vs. displacement diagram for specimen No. 4, No. 6, and No. 10.

5. Conclusions

This paper conducted an experimental study to evaluate the impact of bolt arrangement and thickness of CFS sheet on the load carrying capacity for the joist. To do so, ten types of joist-beam connections were presented and examined.

Eventually, as it was expected, it can be concluded that using the thicker sheet, both side of the joists are increased the load carrying capacity of the connections. In addition, the not-in-line arrangement of the screw can significantly increase the load carrying capacity and the rigidity of the connectors with the same amount of the screws. The detailed obtained tests results can be summarized as follows

1. If the bolts placed closed to the neutral axes, the flexural load carrying capacity decreases. However, once bolts were placed far from the neutral axes, the flexural load carrying capacity increases.

2. Approaching to the simply supported connection (specimen No. 1 and No. 2), the rotations of the joists and vertical displacements are increased. Therefore, the released moment is intensified, which can be concluded to the less damage on joist. However, the bearing stress in holes is increased, which is more considerable failure for those holes which far from the neural axes.

3. Using the proper bolts arrangement (specimen No. 8 and No. 9), the vertical displacements and the joists rotations are decreased. Therefore, the released moment is also decreased, which leads to the more damage in joist. This type of damage can be appeared in shape of the diagonal buckling in the web with a noticeable torsion. As a result of that, there is no specific bearing stress around the holes.

4. Regarding the sheets thickness influence on the load carrying capacity, it was observed that the load carrying capacity of the sheets with 1 mm thickness (specimen No. 3) is just about 30 percent of the same connection with the 1.5 mm thickness (specimen No. 4), and load carrying capacity of the specimen No. 4, is about 80 percent of the same connection with 2.0 mm thickness (specimen No. 5).

5. As a result of the specimen No. 6 (one L-shaped connector) and No. 4 (two L-shaped connector), the load carrying capacity of two L-shaped connector is about two times of one L-shaped connector.

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