Finite Element Analysis of the Screw Arrangement Effects on the Connection Behaviour for Cold Formed Steel Truss Systems

Fatimah De'nan, Kok Keong Choong, Jessica Si Jia Loo and Nor Salwani Hashim

Abstract Due to advantages of lightweight section with high strength to weight ratio, easy for transportation, can be manufactured in different configurations and shapes, the demand for cold formed steel (CFS) has rapidly increased. There were several types of connections such as bolt, welded, rivet and screw. Recently, limited researches were noticed especially when it was used on steel trusses. Rectangular hollow section (RHS) and lipped channel (LCS) were selected as material of cold formed steel (CFS) truss where consist of interconnected small elements such as web, top and bottom chord and other components. In this research, the screw connections of Fink and Howe truss at the side and peak location were being modelled using LUSAS. The behaviors in different cases through modification of screws arrangement were being compared. The identification of connection strength based on the results such as displacement, shear, stress and moment were discussed through finite element analysis. Thereafter, according to procedures prescribed in Eurocode 3 design checking on the shear and tension resistance of screw connection were carried out as a validation of design procedure for connection of cold formed steel (CFS). From the outcomes results, it was observed that both types of cross sections gave different performances. In general, when the arrangement of screws was emphasized the function of screw connection necessarily improved but was not crucially affected. However, such arrangement of screws was appropriate to use in truss system design.

Keywords Cold formed steel (CFS) · Connection · Finite element analysis (FEA) · Truss

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F. Mohamed Nazri (ed.), *Proceedings of AICCE'19*, Lecture Notes in Civil
Engineering 53, https://doi.org/10.1007/978-3-030-32816-0_14

1 Introduction

The connection problems in cold formed was an issue that caused buckling and instability due to thin-wall behaviour. However, CFS was extensively used in structural components such as roof trusses, frame, wall panels and etc. Due to the increment of the CFS application in construction, more researches were performed to ensure the stabilization of the structure design. This relates to the safety issues of structure, stiffness of members [1], the failure modes of steel members and the strength of CFS. Besides that, more research began to take place in the design of cold formed members, wall and systems and seismic design of CFS structure. CFS were then utilized in important structural members in low rise to midrise building construction [2].

Due to the increment of demand for CFS connection in roof truss systems, it was important to investigate the acceptable connection for trusses to confirm an effective connection between truss members. Besides that, the thickness of flange cleat towards the rotational initial stiffness were also observed [3]. Pitched roof such as and Howe truss and Fink truss were very commonly available in market due to the nature of section which transfer the applied load to the supports without the any presence of web members. The use of self-drilling screws was also economical as the fastening process does not require special tools. Self-drilling screws was very effective in CFS connection because it was easily operated and clamped process of two or more thin steel sheets without any aid of special tools. However, the head structure were required to be changes in order to protect truss chord due to compression from random torsion [4]. Compression member mainly fails due to slender truss section, but maybe overcome by replacement of short top chord section or curved section [5]. It was also previously observed that the distance between the lateral bracing was more than the buckling length of a compressed truss chord [6]. With the cut-curved strengthening method of it is shown that higher value of ultimate load was found with the member were fully weld along the cut section and adds with two self-drilling screws (F7A) [7].

In steel structure, connection was classified as an important component to be considered. Since 1946 and 1949 when the first specification and design were published, American Iron and Steel Institute (AISI) was taking the effort by continually providing research funding to widen the design coverage, improve the design technology, and educate on CFS research and design community to improve market share of CFS [8]. The failure in serviceability due to large deflections was occurred, if the connection was not properly designed [1]. This was clearly notified in the earthquake at Northridge, California (1994) and later the Kobe, Japan (1995) which caused many cases of structural failure due to failure of connection [9]. Due to these incidents, more attention was given to the design and detailing of all connections. Besides that, buckling failure modes failure especially in lipped channel in compression was normally occurred in CFS. Local buckling was particularly prevalent in CFS and characterized by the relatively short wavelength buckling of the individual plate element. Global buckling caused by the Euler (flexural) and flexural-torsional buckling of columns and lateral-torsional buckling of beams. Distortional buckling is buckling which takes place as a consequence of the distortion of the cross section [10]. This was normally due to initiation of plastic behaviour.

Moreover, crippling of web at concentrated load and supports position was a critical problem in CFS structure design. This was due to the rounded corners of the sections thus affected eccentric loads from the centreline. Often, this was the case for beams such as Z purlins and C purlins which may experience flexural-torsional buckling because of low torsional stiffness, if not properly braced. The webs were slender and unstiffened in CFS, unlike hot rolled where web stiffeners were used [11]. Moreover, the section was mainly undergo torques and distortional buckling because the sections were usually loaded eccentrically from shear center [12].

In another study, it was revealed that connection strength per screw was decrease as the number of screws in the connection increased. This is known through the investigation of the screw connection behavior from single screw to twelve screws in hot rolled steel by testing the shear and tension strength [13]. This was due to 'group effect' reduction, which was defined as the ratio of the connection strength per screw to the average strength for a single screw connection of the same thickness of steel sheet and same size of screw. Besides, elimination of tilt fastener was recommended during the test of screw shear strength for more accurate shear strength analysis [14]. Thus, load bearing requirements such as strength and stiffness, economic consideration, the durability and the appearance of connectors must be considered. This is why the thin wall component required different design procedures as compared to thicker element (t > 3 mm) [15]. Due to the CFS were categorized as thin-walled section, CFS exhibited large deformation and different mode of failure as the buckling was the major concern of the connection structural analysis [16].

When the number of screws increased the shear value was also increased [17]. Through the observation on graphs of load versus displacement, it was found that the connection became more elastic when more numbers of screws were used because no screws have been pulled out from the hole. Thus, the strength of screw connection increased when the number of screws was higher. In real situation, the screws were arranged in certain shapes such as box, diamond and diagonal shape. Although there was an increment in strength connection, the effect of different screw arrangement on the strength was less than 10% which was only minimal percentage of difference. According to Mujagic and Easterling [18], due to the increment of in situ machineries use while connecting members, the location of screw will be affected. Therefore, based on practical installation considerations and reliability of fastener, the thinnest connected parts of the structure should not exceed 3.2 mm.

Due to the group effect in screw connection as found in LaBoube and Sokol [13], a declining trend was observed that the connection strength per screw will decrease as the number of screws in a connection increase. The arrangement of screws did not significantly influence the strength of the screws as the group effect only varied less than 10% [19] as noticed through the test of shear and tensile strength. However, the screw connection strength for single screws was higher than the strength of single screw connection. Structural connection was improved by the welded connection because the behaviour of rigid joint [20].

In Malaysia, due to the raining season, the Howe and Fink truss were regularly used because the roofs were suitable designed for water evacuation during heavy rain. Therefore, the roof truss connection was a very important issue to ensure the structure were safe. In this research, the pitched roofs connection will be detailed analysed in this research. Screws were very effective to connect individual panels in order to fasten structural members. No precise tools were required therefore it was a suitable process to attached thin steel section [14]. Recently, through the fix design parameters Hamid and Harsad [19] carried a study on CFS single shear connection on screws number from two to six screws.

2 Finite Element Analysis

By assigning different arrangement of screws, the results of all these cases was compared. This study was done by using LUSAS to understand the behaviour of connection in truss system. The connection for Howe and Fink truss were investigated at side and peak location of the truss system for LCS and RHS. It is known as a pitched truss and very efficient to transfer the applied loads directly to the support through top chord members without the need for web members. Pitched roofs were also better when dealing with water evacuation due to the 'A' frame slopes. It was very durable if designed properly as compared to other types of truss system.

The angle for both types of truss was 25° with a span of 9 m. The overall height of the truss was 2.1 m. The details of truss dimension for Howe truss and Fink truss are shown in Figs. 1 and 2, respectively.

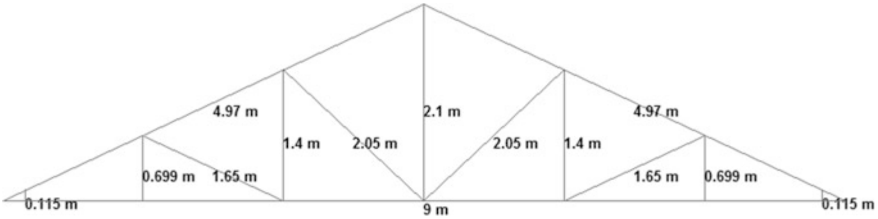


Fig. 1 Dimension of Howe truss

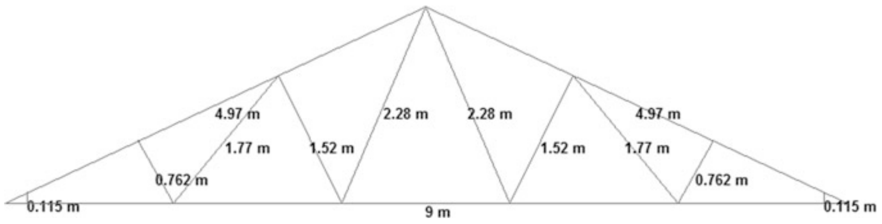


Fig. 2 Dimension of Fink truss

2.1 Section Properties

The section properties for the cold formed LCS and cold formed RHS are shown in Tables 1 and 2 and illustrated in Figs. 3 and 4 respectively. Both sections used the same thickness of 0.75 mm. The steel grade was hot dip galvanized steel. The yield stress was a minimum of 550 MPa for thickness less than 1.2 mm. The coating of steel was galvanized iron or real zinc or zincalume to avoid corrosion.

2.2 Screw Specification

The self-drilling screw used in the cold formed connection of truss was 4 mm hex washer head with drilling point, hardened, chromium VI Free Zinc Plated.

Table 1 Exact dimension of cold formed LCS and RHS

Exact dimension							
Cold formed LCS					Cold formed RHS		
Depth, *D* (mm)	Flange, *B*1 (mm)	Base, *B*2 (mm)	Lips, *S* (mm)	Thickness, *T* (mm)	Depth, *D* (mm)	Base, *B* (mm)	Thickness, *t* (mm)
75	35	37	7	0.75	75	40	0.75

Table 2 (**a**) LCS properties, (**b**) RHS properties

Weight, *w* (kg/m)	Area, *A* (mm^2)	Second moment of area		Radius of gyration		Elastic modulus	
		I_x (10^3 mm^4)	I_y (10^3 mm^4)	r_x (mm)	r_y (mm)	Z_x (mm^4)	Z_y (mm^4)
(**a**)							
0.954	117.06	108.5	19.5	30.4	12.9	2.844	0.763
(**b**)							
1.512	170.3	132.3	50.4	27.88	17.22	3.53	2.52

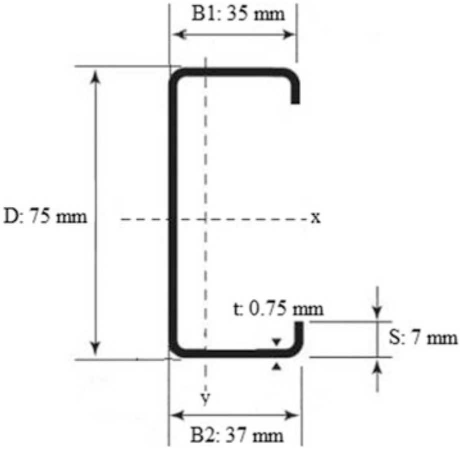


Fig. 3 Section properties of LCS

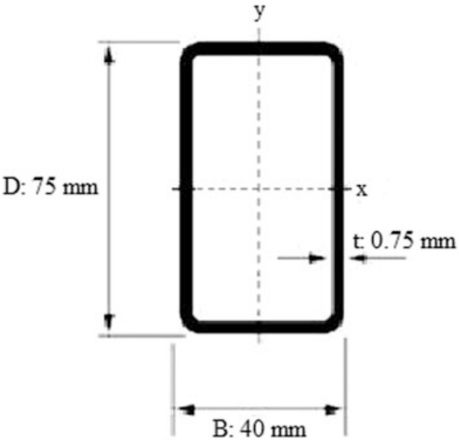


Fig. 4 Section properties of RHS

The screw specification is shown in Fig. 5. However, in modelling, screw connection was simplified to pinned support for ease of interpretation.

Table 3 show the different arrangement of screws that have been modelled using LUSAS. For the comparison of performance in screw connection, the 18 modelled based on arrangement of screw was modified from the typical arrangement similar to Case 1 for number of screw connection.

Two other different arrangements, namely Case 2 and Case 3 were modelled respectively, for the LCS and RHS. For modification of screw connection, by maintaining minimum spacing according to EC3, the changes of screw position were done in order to determine different arrangement and position of the screws effects to the behaviour of screw connection in the CFS truss system.

The self-drilling screw used in the cold formed connection of truss was 4 mm. The surface mesh with element size 10 mm was assigned to all the truss members.

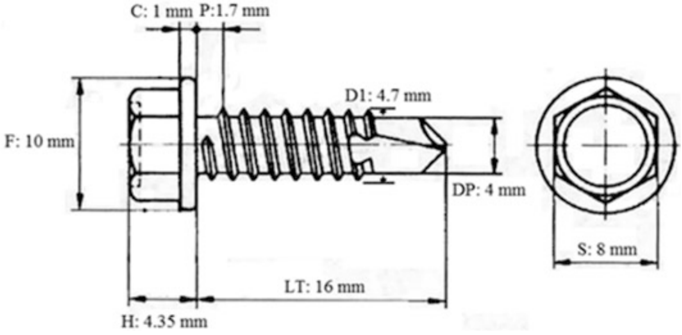


Fig. 5 Self-drilling screw dimension

All the supports were set as fixed support and pinned support. The loads assigned include live load of 0.25 kN and dead load of 0.598 kN. The material assigned to the model was similar to the actual truss member which was CFS with Young's Modulus of 200 GPa.

2.3 Procedures in LUSAS Modelling

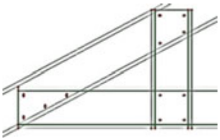
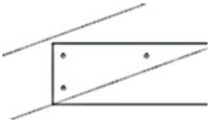
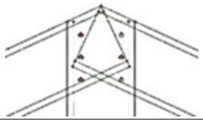
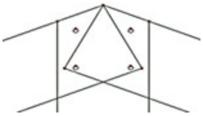
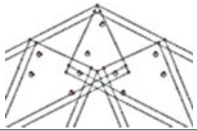
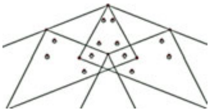
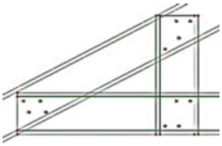
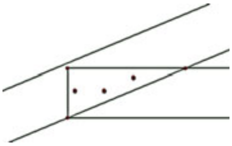
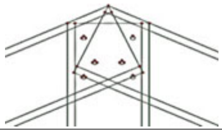
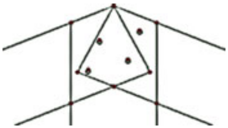
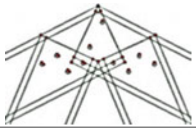
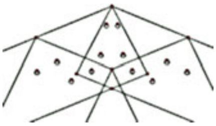
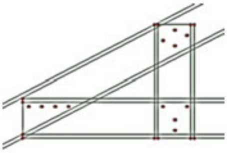
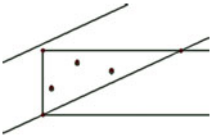
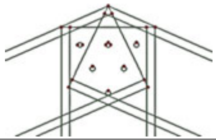
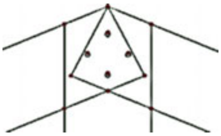
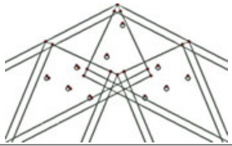
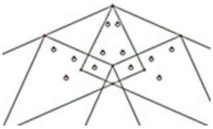
LUSAS 14 was used for finite element analysis in this research. All the dimension and other parameters of the models were mentioned in Chap. 3. Below were the procedures for constructing one of the models (side location of RHS with typical arrangement and number of screws) (Fig. 6).

Through convergence study of the meshing size, it was found that the size of 10 mm was suitable in this research because the analysis results for 10 mm was very accurate with the percentage difference of less than 5% as compared to 5 mm. The surface mesh with element size 10 mm was assigned to all the truss members. The quadrilateral thin shell element (QSI4) was set for the models.

2.4 Verification

To verify the results of FEA in LUSAS, theoretical calculation using the virtual work method was used to calculate the displacement of the truss. An example of a truss is shown in Fig. 7. The truss was simplified to A frame with 9 m long and 2.1 m height similar to the truss used in this research. The cross-sectional area of each member was 400 mm^2 and E = 200 GPa. It was required to get vertical displacement at point C when a load of 5 kN was applied to the truss by manual calculation and compared with the result obtained from LUSAS.

Table 3 Cases for arrangement of screw connection

	Lipped channel section (LCS)		Rectangular hollow section (RHS)	
Case 1: Typical arrangement of screw connection				
Side	a(i)		b(i)	
Peak (Howe truss)	a(ii)		b(ii)	
Peak (Fink truss)	a (iii)		b (iii)	
Case 2: Modified arrangement 1 of screw connection				
Side	a(i)		b(i)	
Peak (Howe truss)	a(ii)		b(ii)	
Peak (Fink truss)	a (iii)		b (iii)	
Case 3: Modified arrangement 2 of screw connection				
Side	a(i)		b(i)	
Peak (Howe truss)	a(ii)		b(ii)	
Peak (Fink truss)	a (iii)		b (iii)	

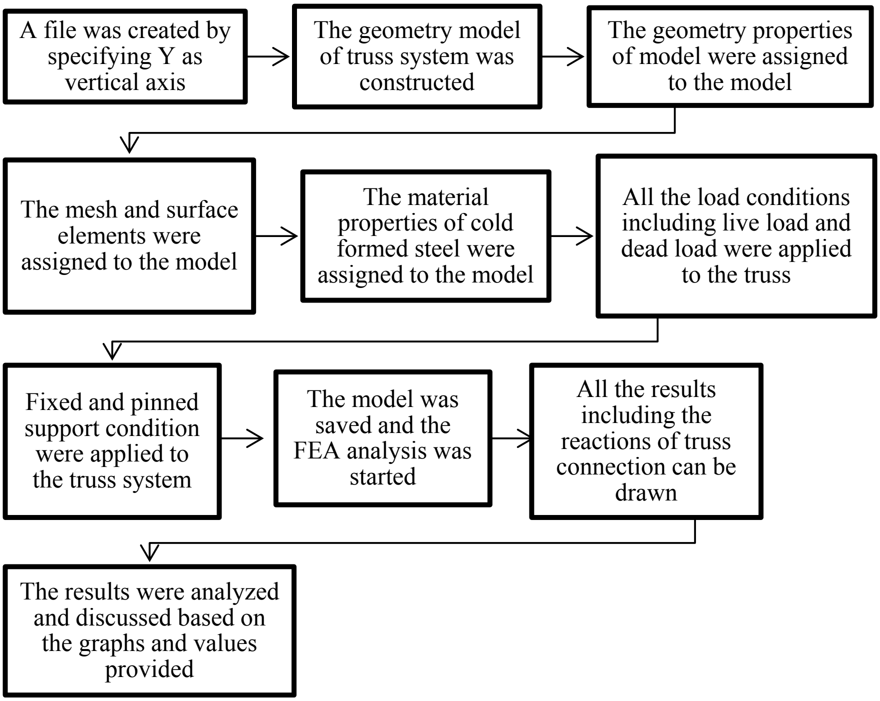
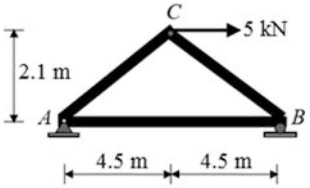


Fig. 6 Process of FEA

Fig. 7 Truss used for verification work



2.4.1 Virtual Work Method

By applying the formula of virtual work method, the solution for displacement is shown in Fig. 8a–d for better understanding.

$$1 \times \Delta = \sum_{i=1}^{n} \frac{nNL}{AE} \tag{1}$$

By applying nNL force from Fig. 8d to Eq. 1, the vertical displacement at point C was calculated. The vertical displacement at point C using virtual work method is 0.141 m (141 mm) downwards.

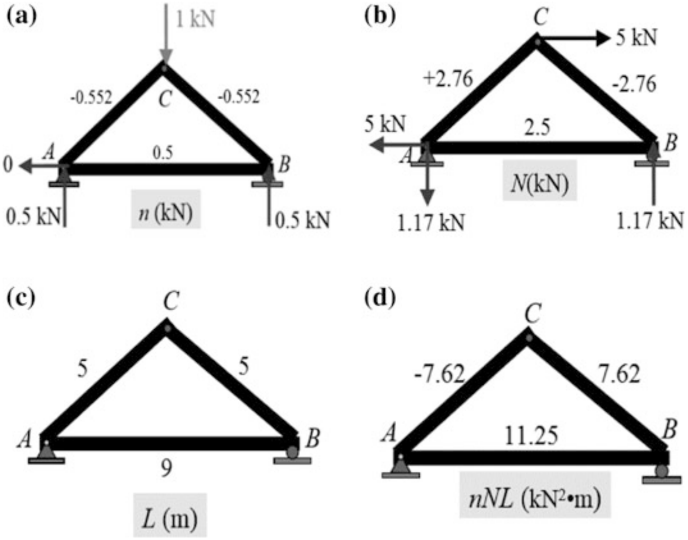


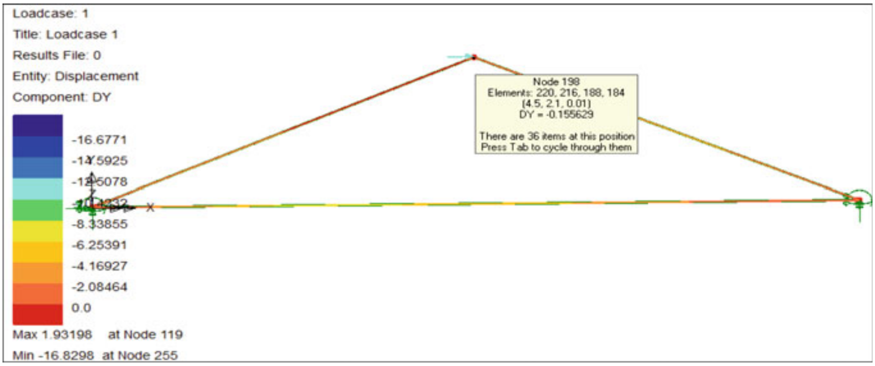
Fig. 8 **a** Virtual force of truss member, **b** real force of truss member, **c** length of truss member, **d** nNL force of truss member

2.4.2 LUSAS Software

Since the virtual work method only applicable for deflection checking, the displacement of the truss system was obtained from LUSAS for verification purpose. Similar truss as shown in Fig. 7 was modelled in LUSAS and the vertical displacement, *D*y at Point C obtained at node 198 was −0.155629 m (155.6 mm) downwards as displayed in Fig. 9. As compared to the answer from calculation which was 140.6 mm, it shows a difference of 10.6% from displacement in LUSAS. Therefore, from this verification, LUSAS was a trustable tool for analysis which can be used to represent the realistic structure.

3 Results and Discussion

The outcomes of the research were based on the parametric study of the arrangement and number of screw connection. Rectangular hollow section (RHS) and lipped channel section (LCS) were used in the cross section of the truss system. It was predicted that the screw arrangement will affect the performance and strength of screw connection in truss system. As such, discussions were made in accordance with the results obtained such as displacement, stress, shear and moment after all analysis have been done using LUSAS software as shown in Fig. 10.


Loadcase: 1
Title: Loadcase 1
Results File: 0
Entity: Displacement
Component: DY

Node 198
Elements: 220, 216, 188, 184
(4.5, 2.1, 0.01)
DY = -0.155629
There are 36 items at this position
Press Tab to cycle through them

Max 1.93198 at Node 119
Min -16.8298 at Node 255

Fig. 9 Vertical displacement at node 198

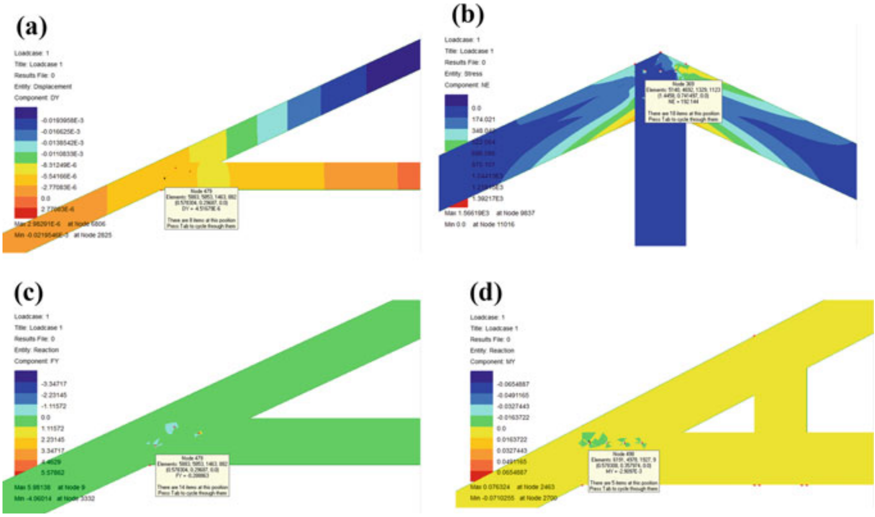


Fig. 10 **a** Displacement analysis, **b** stress analysis, **c** shear analysis, **d** moment analysis

3.1 Behavior of Connection Based on Different Screws Arrangement

Another parametric study is done by modifying the arrangement of screw connection for all the cases. Tables 4 and 5 show the results of RHS and LCS truss based on different screws arrangement.

The performance of screw connection in terms of displacement, stress, moment and shear were compared and discussed based on different arrangements of screws

Table 4 Result for different arrangement of screws in RHS

Location	Side			Peak (Howe)			Peak (Fink)		
Case	1	2	3	1	2	3	1	2	3
Displacement (m)	−4.36E −05	−5.15E −04	−5.43E −05	−1.89E −05	−1.86E −05	−1.83E −05	−1.72E −05	−1.66E −05	−1.13E −05
Stress (N/m^2)	1681.2	1345.1	1773.0	4433.4	3184.4	3910.6	2741.1	3682.1	829.7
Shear (N)	−4.7314	0	1.07826	10.3913	8.3446	−11.779	6.88215	21.7469	−7.6821
Moment (Nm)	−0.0012	0	−1.47E −03	7.75E−04	3.37E−05	4.77E−04	2.28E−04	1.20E−04	3.21E−06

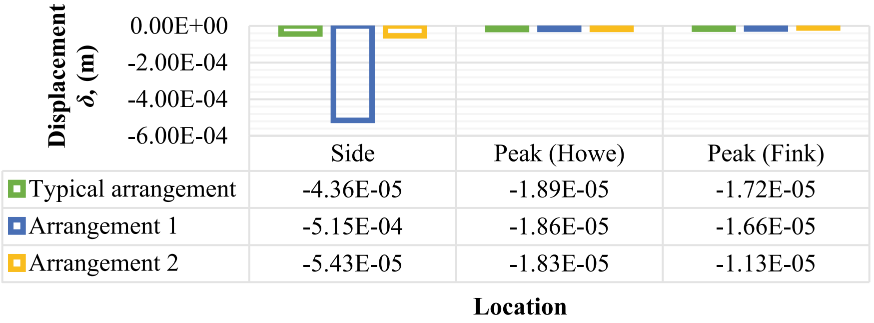
Table 5 Result for different arrangement of screws in LCS

Location	Side			Peak (Howe)			Peak (Fink)		
Case	1	2	3	1	2	3	1	2	3
Displacement (m)	6.11E−03	1.94E−04	1.75E−04	−9.87E−05	−9.91E−05	−6.80E−05	7.35E−05	4.28E−05	2.22E−06
Stress (N/m^2)	2879.3	1290.3	1798.6	19,641.6	16,156.9	16,300	97.3	712.9	212
Shear (N)	2.28865	2.91709	0	51.1432	73.0772	97.8395	−0.2450	−0.3592	−0.5766
Moment (Nm)	−2.77E−03	−2.21E−03	0	−5.33E−04	8.94E−03	1.22E−02	7.96E−05	1.48E−02	8.82E−05

in connection. As found in Figs. 11 and 12, Arrangement 2 was more suitable for lower displacement. Both Arrangement 1 and Arrangement 2 show lower stress and lower moment. However, the typical arrangement gives lower shear strength compared to other arrangements.

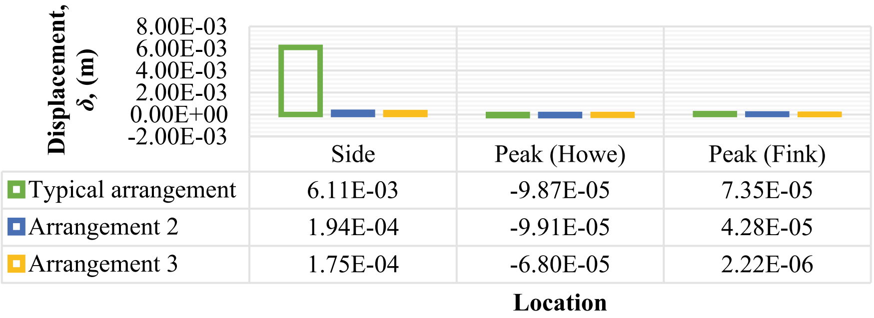
As shown in Figs. 13 and 14, different arrangement of the screws might affect its principal stress. Arrangement 1 of the RHS in has the lowest principal stress for the side and peak location of Fink truss but highest principal stress for Howe truss. Arrangement 2 was more suitable for Howe truss in RHS as the stress was 70% lower than typical arrangement. Arrangement 1 and 2 for LCS has a descending stress compared to the typical arrangement. Arrangement 1 has the lowest stress for the side and peak location of Howe truss. However, the stress climbed up from 97.4 N/m^2 with both modified screw arrangements for the peak location of Fink truss to 713 and 211.6 N/m^2 respectively.

The arrangement 1 at the peak location of Howe truss and typical arrangement of Fink truss has the least shear force. Instead, as found in Fig. 15 the side location of RHS shows zero shear force at Arrangement 1 which indicated that it has the



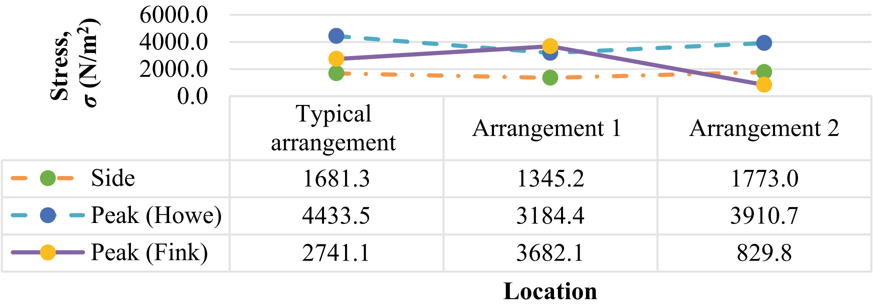
	Side	Peak (Howe)	Peak (Fink)
🟩 Typical arrangement	-4.36E-05	-1.89E-05	-1.72E-05
🟦 Arrangement 1	-5.15E-04	-1.86E-05	-1.66E-05
🟨 Arrangement 2	-5.43E-05	-1.83E-05	-1.13E-05

Fig. 11 Displacement for different arrangement of screws in RHS



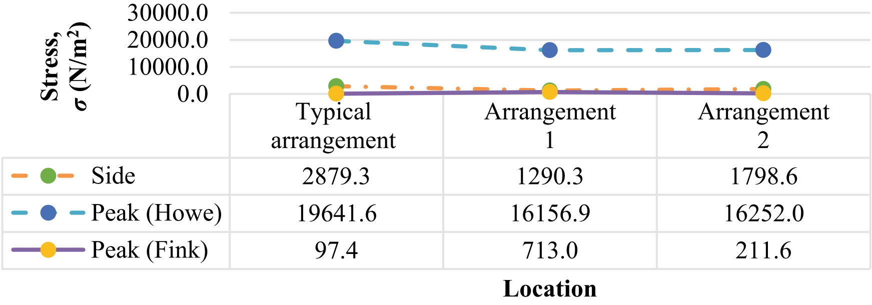
	Side	Peak (Howe)	Peak (Fink)
🟩 Typical arrangement	6.11E-03	-9.87E-05	7.35E-05
🟦 Arrangement 2	1.94E-04	-9.91E-05	4.28E-05
🟨 Arrangement 3	1.75E-04	-6.80E-05	2.22E-06

Fig. 12 Displacement for different arrangement of screws in LCS



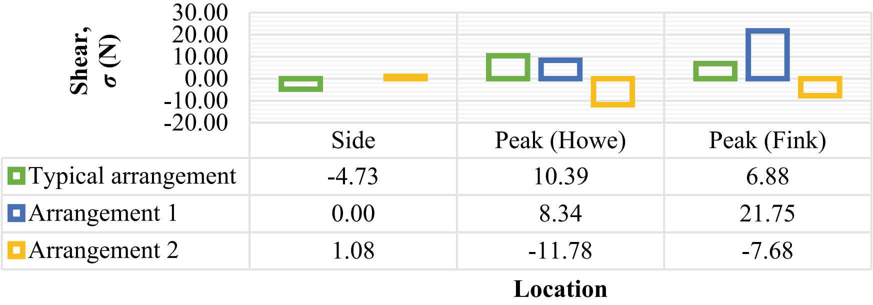
	Typical arrangement	Arrangement 1	Arrangement 2
– ● – Side	1681.3	1345.2	1773.0
– ● – Peak (Howe)	4433.5	3184.4	3910.7
—●— Peak (Fink)	2741.1	3682.1	829.8

Fig. 13 Stress for different arrangement of screws in RHS



	Typical arrangement	Arrangement 1	Arrangement 2
– ● – Side	2879.3	1290.3	1798.6
– ● – Peak (Howe)	19641.6	16156.9	16252.0
—●— Peak (Fink)	97.4	713.0	211.6

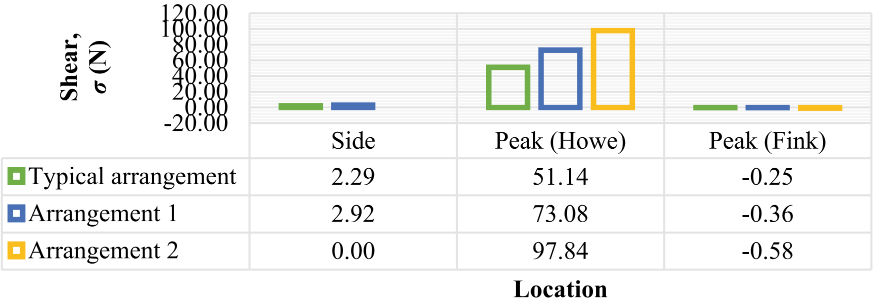
Fig. 14 Stress for different arrangement of screws in LCS



	Side	Peak (Howe)	Peak (Fink)
🟩 Typical arrangement	-4.73	10.39	6.88
🟦 Arrangement 1	0.00	8.34	21.75
🟨 Arrangement 2	1.08	-11.78	-7.68

Fig. 15 Shear for different arrangement of screws in RHS

maximum moment at this position. This situation also happened on Arrangement 2 at the side location of LCS as shown in Fig. 16. For the LCS, the shear force was relatively small for the side and peak location of Fink truss. Their difference in shear was not as readily apparent as the peak location of Howe truss which the



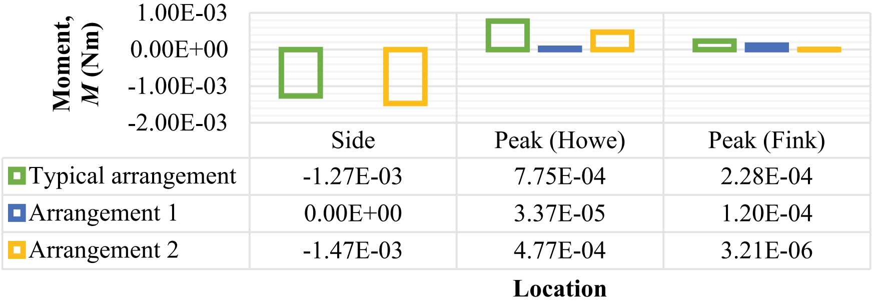
	Side	Peak (Howe)	Peak (Fink)
🟩 Typical arrangement	2.29	51.14	-0.25
🟦 Arrangement 1	2.92	73.08	-0.36
🟨 Arrangement 2	0.00	97.84	-0.58

Fig. 16 Shear for different arrangement of screws in LCS

shear force became larger with both modified arrangements; Thus, the typical arrangement was suitable to be implemented since the shear force was 91% smaller than the others.

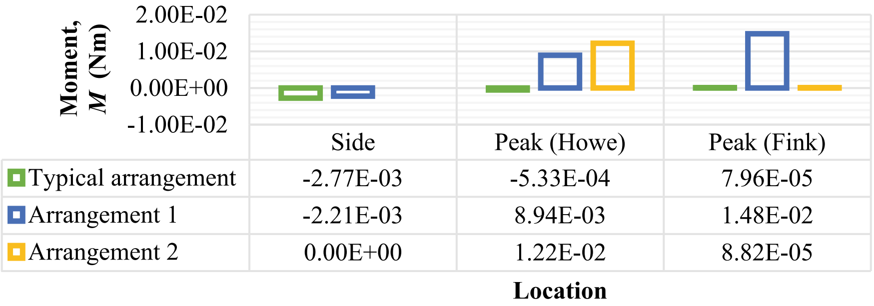
It was concluded that modified arrangements did improved the screw connection behavior but in a very small extend of values since all the screws arrangements have the similar number of rows. Usually, shear capacity of the screws was mainly focused when the connection strength was determined. Shear capacity in this research decreased with modified arrangements.

Generally, as demonstrated in Figs. 17 and 18 typical arrangement has the highest moment compared to Arrangement 1 and Arrangement for the connection at peak location of RHS. The percentage was more than 95% higher than the other cases. Meanwhile, at the side location, Arrangement 2 has the highest resisting moment while Arrangement 1 of side location displayed zero moment. For the LCS cases, Arrangement 2 at the peak location of Howe truss and Arrangement 1 of Fink truss improved the moment of connection dramatically around 180–2000%. In contrast, the moment declined with modified arrangement and became zero for Arrangement 2 at the side location of LCS, which indicated that typical arrangement is the best arrangement at this location.



	Side	Peak (Howe)	Peak (Fink)
🟩 Typical arrangement	-1.27E-03	7.75E-04	2.28E-04
🟦 Arrangement 1	0.00E+00	3.37E-05	1.20E-04
🟨 Arrangement 2	-1.47E-03	4.77E-04	3.21E-06

Fig. 17 Moment for different arrangement of screws in RHS



	Side	Peak (Howe)	Peak (Fink)
🟩 Typical arrangement	-2.77E-03	-5.33E-04	7.96E-05
🟦 Arrangement 1	-2.21E-03	8.94E-03	1.48E-02
🟨 Arrangement 2	0.00E+00	1.22E-02	8.82E-05

Fig. 18 Moment for different arrangement of screws in LCS

Ordinarily, three main region of moment capacity curves was normally focused based on the beam slenderness, which was elastic, plastic and inelastic buckling. For current codes of design, the range of beam slenderness of these three regions were not similar. beams with lateral restraints in short or long span act near to the plastic region and local buckling or yielding were mostly governed in this section. Yielding more were normally governs by short section which categorized under compact cross-sections. However, elastic and elastic local buckling were mostly controlled by slender or semi-compact sections, respectively. For moment capacity of slender cross-sections, elastic local buckling was truly governing and therefore decreased under the yield moment capacity.

This conclusion was compared with previous studies of Sapiee [17] and Hamid and Harsad [19]. These researchers found that the arrangement of screws did not impact much on the screw connection strength as the percentage difference was very small. However, they realized that the connection strength for double rows are better than the single row connection because the arrangement increased the rotational stability of the connection and thus offered more resistance to rotation [21]. While Sapiee [17] has investigated the behavior of screws by using different shapes of screw arrangements, she found that box shape screw arrangement did not demonstrate sudden dropping in load while other cases like diagonal and diamond screw arrangement experienced signs of shear off in screw connection. Her study is similar with the result of this research, which the shear strength for typical arrangement (box shape) was lower and did not experience sudden drop of load as compared to Arrangement 1 and Arrangement 2 (diagonal and diamond shape).

Thus, the box shape arrangement according to typical arrangement was proved to have better performance than other modified arrangements in the diagonal and diamond shape of the screws. It was recommended that the typical arrangement of screws was used for the connection in RHS and LCS truss system. In most situations, multiple rows of screws should be arranged for the connection instead of single row of screws as it can increase the strength of screw connection.

4 Conclusion

Through FEA, the screws behaviors at the side and peak location of RHS and LCS truss system were compared based on different screw arrangement. To conclude, different screws number arrangement did not necessarily enhance the connection strength of screws in the CFS truss system. Even though there were some cases that made the connection better, a reasonable consideration should be done to check the conservative of screws design. Since the connection did not show a large improvement in strength, it was advised to apply the arrangement of screws used on the site after cost and economic consideration since they are already appropriate. This achieved the objective by determining the screws behavior based on different screws arrangement using FEA.

It was recommended to carry out a nonlinear analysis by considering the material, boundary and geometric nonlinearity so that it can establish the factual results. The modelling process in LUSAS should be made easier so that the models can represent the real structure in the simplest form like line elements, in turn save the time of checking procedure in the design process.

In addition, with the permissions of time and financial considerations, it was strongly advised to get the proper model for material testing to compare the analysis in FEA software. Models with different arrangements of screws can be modelled for testing to check the behavior of connection.

Acknowledgements The authors gratefully recognize the financial support by the Bridging Research Grant 2017 (Account Number: 1001/PAWAM/6316234) provided by the Universiti Sains Malaysia (USM) during the course of this research.

References

1. Bucmys Z, Sauciuuvenas G (2013) The behaviour of cold formed steel structure connections. Eng Struct Technol 113–122
2. Schafer BW (2011) Cold-formed steel structures around the world—a review of recent advances in applications, analysis and design. Steel Constr 4(3):1–9
3. Lee YH, Tan CS, Mohammad S, Lee YL (2015) Mechanical properties prediction for cold-formed steel angle connection with various flange cleat thickness. Technol J 74(4):52–59
4. Gordziej-Zagorowska M, Urbanska-Galewska E, Deniziak P, Pyrzowski L (2017) Truss joint with positive eccentricity experimental research. Civ Environ Eng Rep 25(2):107–123
5. Mohd Sani MSH, Muftah F, Tan CS (2018) A state-of-the-art review on cold formed steel roof truss system. Int J Civ Eng Technol (IJCIET) 9(9):746–758
6. Konkong N, Aramraks T, Phuvoravan K (2017) Buckling length analysis for compression chord in cold-formed steel cantilever truss. Int J Steel Struct 17(2):775–787
7. Mohd Sani MSH, Muftah F, Muda MF, Tan CS (2017) Tension behaviour on the connection of the cold-formed cut-curved steel channel section. In: IOP conference series: materials science and engineering, vol 225, pp 1–9
8. Yu WW, Wolford DS, Johnson AL (1996) Golden anniversary of the AISI specification, recent research and developments in cold-formed steel design and construction. In: Proceedings of the 13th international specialty conference on cold-formed steel structures

9. Smolka ARE (1996) The earthquake of Northridge 1994 and Kobe 1995—Lessons for risk assessment and loss prevention with special reference to earthquake insurance. In: Eleventh world conference on earthquake engineering
10. Dubina D, Zaharia R (2014) Behaviour of cold formed steel truss bolted joints, pp 443–453
11. Young B, Hancock GJ (2001) Design of cold-formed channels subjected to web crippling. J Struct Eng 127(10):1137–1144
12. Yu WW (2000) Cold-formed steel design. Wiley, Canada
13. LaBoube RA, Sokol MA (2002) Behavior of screw connections in residential construction. J Struct Eng 128(1):115–118
14. Lee YH (2014) Review on cold formed steel connections. Sci World J
15. Davies JM (1991) Connections for cold-formed steelwork. In: Rhodes J (ed) Design of cold-formed steel members. London and New York, Elsevier Applied Science
16. Lee YH, Tan CS, Mahmood T, Shahrin M (2012) Numerical modelling and validation of light gauge steel top-seat flange-cleat connection. J Vibro Eng 14(3):1104–1112
17. Sapiee SF (2013) Behaviour and shear strength of screw connections in high strength cold-formed steel structures. MSc Thesis, Curtin University
18. Mujagic JRU, Easterling WS (2009) Connections in cold-formed steel framing—designing with AISI 2007: specification and application overview. Struct Mag 9–11
19. Hamid HA, Harsad MIS (2016) Behavior of self-drilling screw upon single shear loading on cold formed steel. Malays J Civ Eng 28(1):59–68
20. Komara I, Wahyuni E, Suprobo P (2017) A study on cold-formed steel frame connection: a review. J Technol Sci 28(3):83–89
21. Sokol MA, LaBoube RA, Yu WW (1998) Determination of the tensile and shear strengths of screws and the effect of screw patterns on cold-formed steel connections. In: Civil engineering study 98-3, Cold formed steel series, Department of Civil Engineering, University of Missouri-Rolla