Experimental study on the effect of heel plate thickness on the structural integrity of cold-formed steel roof trusses

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Abstract. Applications of Cold-Formed Steel (CFS) are widely used in buildings, bridge and etc. Many researchers initiate the study of CFS to be used as a roof truss system. There is a need to enhance the configurations of CFS roof trusses due to the uncertainty on structural stability and failures in terms of materials and rigidity of joints. The objectives of this research are to determine the effect of heel plate thickness related to the ultimate strength of the roof truss system. Three different thicknesses of heel plate specimens were fabricated and subjected to concentrated loads applied until the specimens failed. The highest ultimate capacity for the experiment was 27 kN. The results showed that the increment of the thickness of the heel plate had slightly increased the ultimate load and the maximum deflection at bottom chord. The top chords could not sustain the loading was the main factor contributed to the structural failure.

1. Introduction
Cold-formed steel (CFS) is steel products that are made by bending flat sheets of steel at room temperature. CFS is usually shaped from steel plate, sheet and strip material. It is widely used for construction, vehicles, machinery and much more. CFS is commonly used as a secondary framing material, such as roof purlin for wall cladding (Yu and LaBoube, 2010). However, with recent research and development of CFS, it can be used as primary steel members due to its stability and reliability of the constructed steel structures (Ye, Hajirasouliha, Becque and Pilakoutas, 2016). This is because of CFS inherits high strength to weight ratio and can be transported easily due to its lightweight. Other than that, it is excellent in its versatility and also non-combustibility characteristic compared to conventional timber material (Yu and LaBoube, 2010). CFS has been introduced in Malaysia in recent years. Based on its advantages, it has been an increased in the usage of CFS in new construction from housing areas to commercial buildings in Malaysia due to the advantages such as reduced cost, promote sustainable environment and prevail the disadvantage of other materials.

CFS compression member is crucial to the limit states, such as yielding, local buckling, overall column buckling and distortional buckling. All these limit states are depending on the cross-section geometry, thickness and length (Yu and LaBoube, 2010). Compression usually occurs at the chord members of trusses especially top chords and diagonal members in end panels of trusses. CFS compression members can indicate into three modes of instability: local buckling, distortional buckling and flexural or flexural-torsional buckling (Rondal, 2000). Craveiro, Rodrigues and Laim (2016) concluded that two lipped channels positioned back-to-back cover with two plain channels had the highest buckling load compare to other types of cross section. In addition, the authors stated that the failure modes consist of local buckling, flexural buckling, distortional buckling and flexural-torsional buckling.
buckling. Increase the number of channel profile will become risky due to insufficient of the Effective Width Method to deal with built-up profiles.

CFS tension member is based on yielding of the net cross section which exclude the region of member connection (Harper, LaBoube & Yu, 1995). Nominal tensile strength of axially loaded CFS member can controlled either by yielding of total sectional area or by fracture of the total area of cross section away from connections and effective total section at the connections (Yu and LaBoube, 2000). However, CFS tension members can easily cause a connection failure under tension. Due to the limiting strength consideration of tension member, there is a limit on deformation that a tension member can accomplish.

Mills and Laboube (2004) stated that self-drilling screws were easily installed and very rigid. These kind of screw connections gave a joint with non-slip, which is different from bolted joints where the holes for connections are bigger than the diameter of the bolt. Li, Ma and Yao (2010) stated the connections had more strength as screw spacing increases in a specific range and the connections had no changes on shear strength when go beyond the range. Dawe and Wood (2006) researched on the ultimate capacity of 23 small scale CFS roof trusses and subjected to a point load at the ridge. The main failure mode was local buckling of top chord, followed by distortion of the heel plate. The authors suggested that distortion of the heel plate could be reduced by adding edge stiffeners at the heel plate. Wood and Dawe (2006) studied the behaviour of the full-scale CFS roof trusses as the major failure mode was local buckling of the top chords, followed by crippling of the heel plate with the failure of the screws.

The purpose of this project is to eliminate the intermediate member within the truss and replacing with heel plate as a support near to the eaves. Moreover, horizontal members and ridge plates are installed at peak to joint both side of the top chords. With this replacement, it creates an additional living space within the truss system. Further study in detail of CFS roof trusses is needed. This is due to the details of CFS roof trusses that can affect its performance is yet to be determined, where different thicknesses can have different stresses and capacities. Researchers are expected to acquire such knowledge where they have to understand the effect of reinforcing methods on the roof truss system in order to increase the capacity of the system.

2. Material and Methods
In this study, three specimens were fabricated with different thicknesses of heel plates. Each specimen was fabricated into a 4:12 roof pitch by connecting CFS C-channel section such as top chord, bottom chord, horizontal member and purlin. Table 1 shows the dimensions for all the C-channel section.

| Web, d (mm) | Flange, b (mm) | Lip, c (mm) | Thickness, t (mm) |
|------------|---------------|-------------|------------------|
| 97         | 47            | 11          | 1                |

All the members held by using heel plate, ridge plate and additional plate are shown in Figure 1, Figure 2 and Figure 3 respectively. All the members were attached with Number 10 hexagonal head self-drilling screw. The thicknesses of the heel plates used were consist of 2 mm, 2.5 mm and 3 mm. Figure 4 shows the dimensions of heel plate truss specimen, Figure 5 shows the configurations of typical test setup for the heel plate truss specimen. In addition, one additional conventional roof truss with intermediate members was fabricated as shown in Figure 6. This truss was made to compare the ultimate load capacity between the conventional roof truss and the roof truss with heel plates.
Figure 1. Dimensions and configurations of heel plates.

Figure 2. Ridge plate.

Figure 3. Additional plate.

Figure 4. Dimensions for heel plate CFS truss specimen.

Figure 5. Typical test setup for the heel plate CFS truss specimen.
Figure 6. Conventional CFS roof truss specimen.

All the members were cut into tensile test specimens to test the material properties with a loading rate of 2mm/min by using Model 5582 Instron universal testing machine in accordance with ASTM A370 (ASTM, 2000). Table 2 shows the material properties for all the members. All the results were based on average of triplicate.

Table 2. Material Properties for all the members.

| Member            | Thickness (mm) | Young Modulus, E (GPa) | Yield Strength, $f_y$ (MPa) | Ultimate Strength, $f_u$ (MPa) |
|-------------------|----------------|------------------------|----------------------------|-------------------------------|
| CFS member        | 1.0            | 191                    | 623.65                     | 640.76                        |
| Additional plate  | 1.0            | 109                    | 200.81                     | 316.10                        |
| Ridge plate       | 1.5            | 176                    | 370.42                     | 525.40                        |
| Heel plate        | 2.0            | 170                    | 219.29                     | 304.46                        |
| Heel plate        | 2.5            | 143                    | 347.95                     | 460.21                        |
| Heel plate        | 3.0            | 163                    | 326.78                     | 474.68                        |

Load capacity, deflection and failure mode of the roof truss specimens were analysed and load was plotted against deflection. All the specimens were subjected to an area load at the ridge to determine the ultimate load capacity and the failure mode by using 300 kN structural reaction frame (Dawe and Wood, 2006).

The positions of LVDTs are shown in Figure 7 as referred to Mohammad, Tahir, Tan & Shek (2012). LVDT 3 was placed at below the middle of the bottom chord to determine the maximum vertical deflection. Besides, LVDT 2 and LVDT 4 were positioned with a distance of 722.9 mm away from LVDT 3 to check the load balancing. On the other hand, LVDT 1 and LVDT 5 were installed at the midspan of top chord to identify the vertical deflection of top chords.

Figure 7. Positions of LVDTs.
3. Experimental Results
Table 3 shows the ultimate load and deflection data for all the specimens.

Table 3. Ultimate load, deflection data and mode for all specimens

| Specimen | Ultimate Load (kN) | Ultimate Deflection (mm) | Deflection mode |
|----------|--------------------|---------------------------|-----------------|
|          |                    | LVDT  | Top Chord | Bottom Chord |
|          |                    | 1    | 2   | 3   | 4   | 5   | LVDT2 | LVDT3 | LVDT4 |
| Conventional | 21.0               | -7.472 | -1.316 | -11.155 | -2.008 | -7.710 | sagging | sagging | sagging |
| T2L150   | 26.0               | -3.480 | -0.588 | -0.486 | 0.080  | -1.042 | -3.688 | sagging | sagging | sagging |
| T2.5L150 | 25.0               | -2.925 | -0.246 | 0.130  | -0.626 | -3.288 | sagging | sagging | hogging | sagging |
| T3L150   | 27.0               | -4.260 | -0.938 | -0.910 | -1.612 | -4.818 | sagging | sagging | hogging | sagging |

* T2 indicates as 2 mm thickness of the heel plate, L150 indicates as 150 mm length of the heel plate.

Based on Table 3, the ultimate capacity for heel plate specimens was in the range of 25 to 27 kN. When the applied load reached around 15 kN, buckling waves formed at top chord webs adjacent to the heel plate. All the heel plate truss specimens failed with local buckling of top chords adjacent to the heel plates at all of the four top chords as shown in Figure 8. Harper, LaBoube and Yu (1995) stated that local buckling of top chords will occur when removing the intermediate member. As for conventional roof truss, the failure mode was distortional buckling of top chords at the ridge and local buckling of bottom chords at the support as shown in Figures 9(a) and 9(b).

Figure 8. Local buckling of top chord for heel plate truss specimen.

Figure 9(a). Distortional buckling of top chord for conventional truss specimen.   Figure 9(b). Local buckling of bottom chord for conventional truss specimen.

All the data were plotted with load against deflection as shown in Appendix A1 to A4. The results showed that both sides of top chords were in negative deflection, which was sagging deflection. Figure 10 shows the typical illustration of the heel plate truss specimens’ deflection for both sides of the top chords. Both LVDTs 2 and 4 were in sagging deflection. While, LVDT3 was in hogging deflection for heel plate truss specimens. On the other hand, the middle of bottom chords for conventional truss was in sagging deflection. Figure 11 illustrates the phenomena of the bottom chords deflections for heel plate truss specimen. Figures 12 and 13 show the illustration of the top chords and bottom chords deflections for conventional truss specimens respectively.
Generally, the ultimate load capacity and maximum deflection slightly increased as the thickness of the heel plate increased from 2 mm to 3 mm. These might be due to the ultimate strength of the heel plate increase whereas the thickness was increased from 2 mm to 3 mm as shown in Table 2. All the chord members for the conventional roof truss specimen had higher deflection compared to all of the heel plate truss specimens. However, the ultimate load for the conventional truss specimen was lower than that of the heel plate truss specimens.
4. Conclusion
The experimental results were analysed and discussed on three small-scale CFS roof truss specimens were fabricated with different thicknesses of the heel plate. The major failure mode for all of the heel plate specimens was local buckling of top chords adjacent to the heel plate. The failure mode for conventional roof truss was distortional buckling of top chords at the ridge and local buckling of bottom chords at the support. The ultimate load capacity and maximum deflection slightly increased with the increase of the thickness of the heel plate. All the deflection mode for the heel plate truss specimens were sagging deflection at top chords and sagging hogging sagging deflection at bottom chords. As for conventional truss specimen, all of the chord members were in sagging deflection.

Appendix A

![Figure A1. Load versus deflection of conventional roof truss.](image1)

![Figure A2. Load versus deflection of T2L150 specimen.](image2)
Figure A3. Load versus deflection of T2.5L150 specimen.

Figure A4. Load versus deflection of T3L150 specimen.
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