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An Experimental and Analytical Study on Cold Formed Steel Floor Trusses with Bare and Concrete Filled Compression Chords

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Abstract

Floor trusses fabricated from cold-formed steel (CFS) sections are commonly preferred for floor system construction in CFS buildings in certain parts of the world. An integrated experimental and numerical study has been conducted to investigate the behavior of such trusses. The investigation included both bare CFS trusses, as well as trusses with the top chord members filled with concrete in an attempt to eliminate compression buckling. Bare steel truss tests included seventeen full-scale trusses with the major test variables of CFS section thickness, type of diagonal-to-chord connection, and number of connection fasteners. Test results have shown that the analytically calculated and measured responses differ in terms of truss stiffness. Measured stiffness of the trusses within the service load level came out to be lower than the stiffness obtained using two dimensional analytical models. Hence, additional experiments on screw connections were conducted and by these results analytical expressions were developed to simulate connection flexibility. An additional five truss specimens were tested as part of the investigation on steel-concrete composite trusses. Test results indicate that the presence of concrete infill inside the compression chord member prevents the initiation of chord local/distortional buckling and provides significant improvement in truss response in terms of increase in stiffness and load capacity.

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1. Introduction

Floor systems used in cold-formed steel (CFS) buildings can be in various forms. Solid-web cold-formed C-shaped joists can be utilized together with oriented strand board (OSB) or concrete deck to form a composite floor system [1–4]. Another alternative is to fabricate trusses from CFS lipped channel sections as shown in Fig. 1. Similar to the joists, these trusses can be combined with OSB or concrete deck to arrive at a composite floor system. Even though C-shaped steel joists require less workmanship when compared to the cold-formed trusses, the truss system becomes the preferred option in cases where low cost roll formers with limited capacity are used. Although the truss system requires more workmanship, these trusses can be built by combining relatively smaller sections together, therefore regaining the economic advantage overall. Floor trusses also let pipes and other utilities through, and preferred by designers to overcome multidisciplinary clash problems such as mechanical pipes, HVAC systems, etc.



Figure 1: CFS floor truss system building example

In a typical floor truss the top and bottom chord members are constructed from a continuous lipped channel section. Due to the ease in fabrication, sections with similar thickness are generally employed for both chords. The diagonals members are attached to the chords by making use of rivets or self-tapping screws. Fabricators usually prefer to use at least one rivet in each connection. These holes are overlapped and a rivet is driven through during the assembly phase. The ends of the diagonals are usually placed inside the top and bottom chords so that the fasteners can be driven through the flanges of the chords and diagonals. The lips of the chord members require a special treatment at the connection regions in order to provide an easy placement of the diagonals. As shown in Fig. 2, typical connection methods include removing or straightening the chord flanges by unfolding the lips or using gusset plates to connect members externally. A case with straightened flanges is shown in Fig. 2a. As depicted in this figure the straightening operation introduces a discontinuity in the chord cross section at the locations where the diagonals are connected. In addition, eccentricities in the connections are inevitable in these types of systems as shown in Fig. 2a.

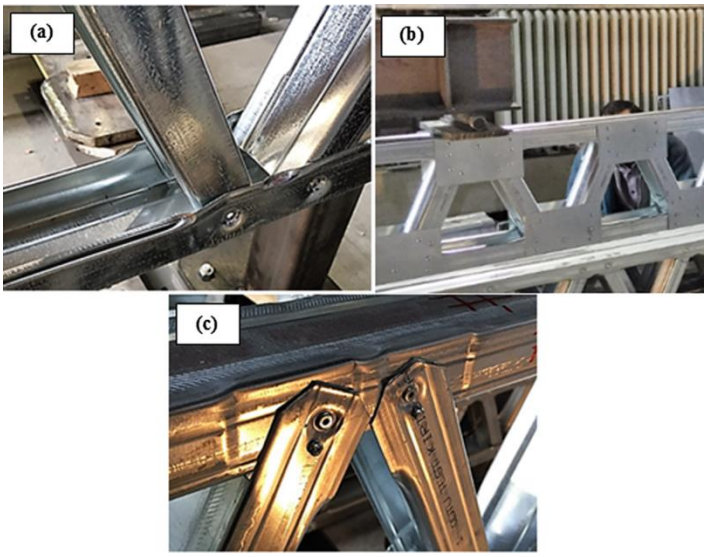


Figure 2: Connection details used in truss specimens: (a) diagonal end inserted inside chord; (b) diagonal end placed inside chord and cover plates attached; (c) diagonal flanges attached to exterior faces of chord

Analysis and design of these systems present a variety of challenges due to their complex geometry and difficulties in assessing cold-formed member behavior. In general, design is greatly simplified by using two dimensional truss models and assuming pin-ended members. In other words, chord members and diagonals are designed based on pure axial forces where the compression chord governs the capacity of the system. CFS design specifications such as AISI S100-16 [5] are utilized to determine member capacities. Local, distortional, and global buckling modes of the chord and diagonal members are considered according to the design specification being followed.

2. Experimental Study on CFS Trusses

An experimental study was undertaken where 22 full-scale CFS floor trusses were investigated under four point bending. Effects of parameters, such as connection detail at diagonal-to-chord joints, number of fasteners used at each connection, CFS member thickness and the presence of concrete infill inside top

chord were studied under the scope of the experimental program.

Full scale truss specimens in 6 meters length were subjected to four-point bending tests under monotonically increased vertical displacement loading using the test setup shown in Fig. 3. Loading was applied at two points 555 mm spaced out from the mid-span. A clear span between two roller supports of 5810 mm was considered. Locations of the loading points were selected to coincide with the joints where the diagonal members meet with the top chord. Similarly, the supports at two ends of the trusses were located at the joints where the diagonal members meet with the bottom chord.

Instrumentation that was used in truss loading tests included a load cell to measure the force applied by hydraulic piston, linear variable displacement transducers (LVDTs) to measure the vertical displacement at mid-span and strain gauges to monitor strains at various locations in top and bottom chord members. Fig. 3 shows the loading points and the LVDTs used on the specimens.

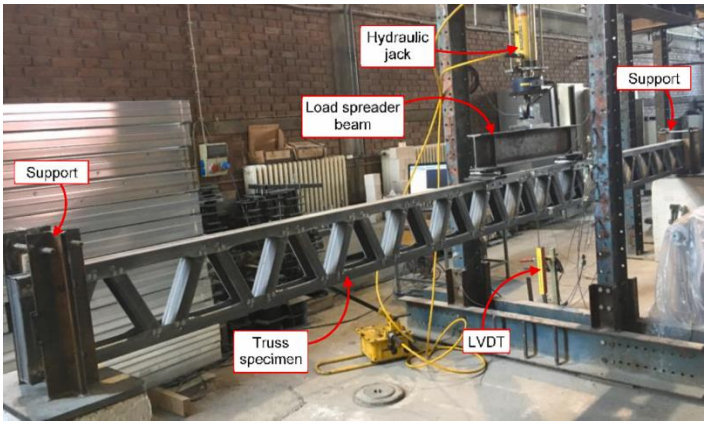


Figure 3: Test setup

2.1 Test Variables and Test Specimens

The details of specimens are given in Table 1. All specimens employed CFS lipped channel sections for chord and diagonal members. The geometric details of truss specimens and the size of the channel sections are given in Fig. 4. As shown in this figure, all trusses had an overall depth of 300 mm measured from the outer faces of the chords. The longest unsupported length of the chord members in between the points where the diagonals connect to these members was also 300 mm. Connection eccentricities are unavoidable in these systems as mentioned before. Due to the orientation of members, eccentricities of 60 mm and 80 mm were present between the diagonal connection points in the upper compression chord and the lower tension chord, respectively. Lipped channel sections that were used to fabricate the truss specimens had an overall depth of 90 mm and flange width of 45 mm, as shown in Fig. 4b.

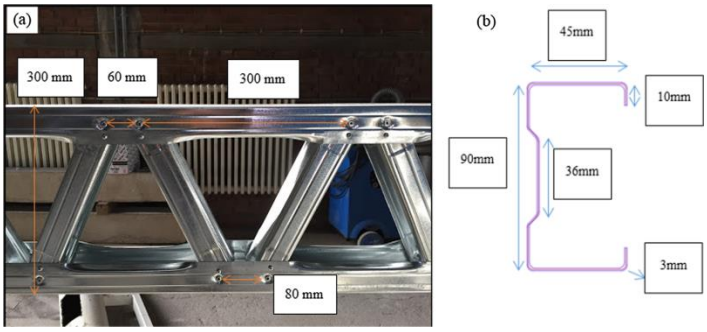


Figure 4: (a) Geometric details of truss specimens; (b) lipped channel section dimensions

Nominal lipped channel thicknesses of 0.8 mm, 1.2 mm, and 1.5 mm were considered in the study. The same CFS thicknesses were employed in all specimens, except for Specimens 14 and 15. In general, due to the ease in fabrication, all members of a truss are rolled from steel coils having similar thickness and this leads the compression chord strength to govern the design of a CFS truss. The use of two different CFS section thicknesses was investigated in Specimens 14 and 15. Tension chord and diagonal members made from 0.8 mm thick CFS section were used together with 1.2 mm and 1.5 mm thick compression chords in Specimens 14 and 15, respectively. The idea here was to promote a ductile system by having tension yielding of the bottom chord prior to compression chord buckling.

Three different diagonal-to-chord connection details shown in Fig. 2 were investigated. Most of the specimens employed the standard connection detail shown in Fig. 2a, where ends of diagonals are driven into the chord members. This is accomplished by unfolding the lips to straighten the flanges of the chords. Specimens 3 and 11 employed a connection detail with gusset plates (Fig. 2b). In this connection detail, ends of diagonals are driven into the chord members and then 188 x 100 x 0.8 mm gusset plates are connected to the chords and the diagonals in order to reinforce the joint region. The third type of connection detail shown in Fig. 2c was employed in Specimens 12 and 13. The discontinuity in the chord member's cross section is eliminated in this type of joint detail. No trimming or straightening operations are required for chord member flanges. Instead, the webs of the diagonals are removed at their ends so that the flanges of

the diagonals can be connected to the flanges of the chords from the outside. It should be emphasized that this connection detail is generally not preferred by fabricators as it requires relatively more effort and adversely affects the fabrication cost. Nevertheless, two specimens with this type of connection detail were tested to investigate the case with a continuous chord cross section without straightened flanges. It should be noted that in connection types shown in Figs. 2a and 2c, the depth of either the diagonals or the chords have to be modified so that the members can be driven into each other. For the case where the diagonals are driven into chords with unfolded lips (Fig. 2a), the diagonal member ends are pressed and the overall depth is reduced by at least twice the member thickness. Conversely, for the cases where the diagonals are attached to the chords from outside (Fig. 2c), the chord members are pressed along the joint region to reduce its overall depth. While this type of a connection employs a continuous chord, a significant amount of imperfection, as shown in Fig. 2c, is introduced into the chord member at the joint locations.

The number of connection fasteners was considered as a variable to investigate its influence on truss behavior. Connection between the diagonal and the chord members was provided with rivets and screws, as shown in Fig. 2. During the fabrication stage, rivet holes were punched by roll-forming machine and 4.8 mm diameter blind rivets were used in these holes. In the majority of specimens, screws were used to provide connection between the diagonal and chord members, in addition to rivets. These screws were driven with electric hand drills. The screws used in trusses are #6 (3.5 mm diameter) standard screws with a length of 25 mm. Connections that employ a rivet and one, two, or three screws between each flange of diagonal and chord members were considered in the experimental program. Specimen 7 was configured with a single #12 (5.5 mm diameter) screw to connect each flange of diagonal and chord members at each diagonal-to-chord connection. The rivets and screws used in this study comply with the DIN 7337 [6] and DIN 7504N [7] specifications.

Two orientations of top chord CFS section were utilized in truss specimens, as indicated in Fig. 5a. In specimens with concrete infill, the CFS lipped channel section forming the truss top chord was filled with concrete and two additional specimens were fabricated with additional shear transferring screws. In the specimens without the shear transferring screws, shear force transfer between the CFS section and concrete infill occurred through the protruding part of the screws and rivets used for connecting the diagonal members to the chord member. In two of the specimens, additional screws were placed with 5.5 mm diameter and 50 mm length along the top chord at 100 mm spacing in an attempt to increase the horizontal shear force transfer and to improve the composite action between the top chord CFS section and concrete infill. A single 10 mm diameter reinforcing steel bar was placed inside the top chord CFS section along the entire length of the concrete infilled specimens prior to the placement of the concrete filling in order to prevent concrete cracking due to temperature and shrinkage effects.

The specimen with a reinforced concrete slab had 80x600 mm concrete slab over the top chord of the truss. As indicated in Figure 5d, the top chord CFS member in this specimen remained embedded inside the slab. The reinforcement used inside the slab included four 10 mm diameter reinforcing steel bars placed along the truss length and 10 mm diameter reinforcing steel bars placed at 250 mm spacing in the transverse direction.

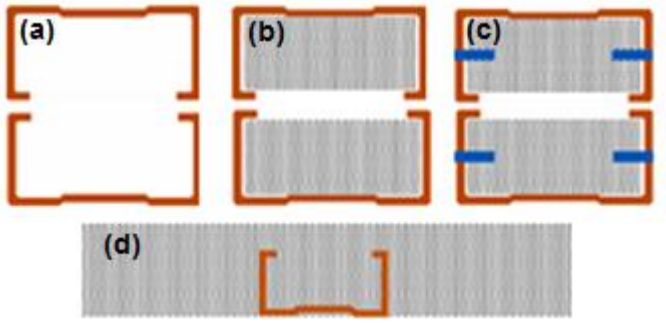


Figure 5: (a) Inverted top chord; (b) concrete infill; (c) concrete infill with shear transfer screws; (d) concrete slab

The experimental program was conducted in several stages and in each stage different steel sheets were used to roll the lipped channels. Coupon samples were extracted from steel sheets that were used to rollform the lipped channels used in truss specimens. Results from tension testing of these coupon samples are presented in Table 2 alongside the measured thicknesses for the heats of steel used in the experimental program. Yield stress (F_y) and ultimate strength (F_u) values presented in Table 2 indicate that the measured yield strengths are generally higher than the specified values for European grade S350 according to EN 10025 [16]. Two mixes of concrete with a target 28-day strength of 30 MPa were used for the specimens including a concrete filled top chord or a concrete slab.

Table 1: Test results

Specimen	Material Heat	Truss/Connection Type	Failure Load (kN)	Failure Type*
1	1	Rivet + 1 Screw	5.7	T1
2	1	Rivet + 3 Screws	5.9	T1
3	1	Gusset Plate	7.0	T1
4	2	Rivet + 1 Screw	5.1	T1
5	2	Rivet + 3 Screws	5.4	T1
6	3	Rivet Only	6.8	T2
7	3	Single #12 Screw	11.9	T1
8	3	Rivet + 1 Screw	11.5	T1
9	3	Rivet + 2 Screws	13.6	T1
10	3	Rivet + 3 Screws	12.6	T1
11	3	Gusset Plate	13.1	T1
12	4	Rivet + 1 Screw/ Flanges with Continuous Lips	11.9	T1
13	4	Rivet + 3 Screws/ Flanges with Continuous Lips	12.6	T1
14	3,1	Rivet + 2 Screws/ Thicker Upper Chord	10.7	T1
15	5,2	Rivet + 3 Screws/ Thicker Upper Chord	12.8	T3
16	6	Rivet + 2 Screws	19.7	T1
17	6	Rivet + 2 Screws/ Inverted Upper Chord	18.8	T1
18	6	Rivet + 2 Screws/ Concrete Infill	27.9	T3
19	6	Rivet + 2 Screws/ Inverted Upper Chord/ Concrete Infill	27.6	T3
20	6	Rivet + 2 Screws/ Concrete Infill/ Shear Transfer Screws	28.3	T3
21	6	Rivet + 2 Screws/ Inverted Upper Chord/ Concrete Infill/ Shear Transfer Screws	28.0	T3
22	6	Rivet + 2 Screws/ Inverted Upper Chord/ Concrete Slab	35.1	T3

*T1: Compression chord buckling; T2: Rivet shear-off; T3: Tension chord yielding

Table 2: Material properties

Material Heat	Nominal Thickness (mm)	Measured Thickness (mm)	F_y (MPa)	F_u (MPa)
1	0.8	0.84	396	468
2	0.8	0.78	353	409
3	1.2	1.26	409	470
4	1.2	1.16	452	513
5	1.5	1.43	421	541
6	1.5	1.60	350	510

2.2 Experimental Results

Measured response of all truss specimens in terms of load capacity is summarized in Table 1. The reported load capacity values represent the maximum value of the applied vertical load during experiments. The observed failure mode for each specimen is also indicated in this table. Three distinct failure modes were observed during the experimental program. Most of the truss specimens tested with no concrete infill failed due to buckling of the compression chord either inside the eccentric chord connection (Fig. 6a) or outside the eccentric chord connection (Fig. 6b). Specimens with connections reinforced with gusset plates exhibited a distortional buckling type of failure mode away from connection region (Fig. 6c). Specimen 6, which employed a single rivet for diagonal-to-chord connections, failed through rivet shear-off at a very low load level. Tension chord yielding was observed in Specimens 15. All specimens tested with concrete infilled compression chord or with a concrete slab (Specimens 18-22) also experienced tension chord yielding with no visual damage on compression chord.

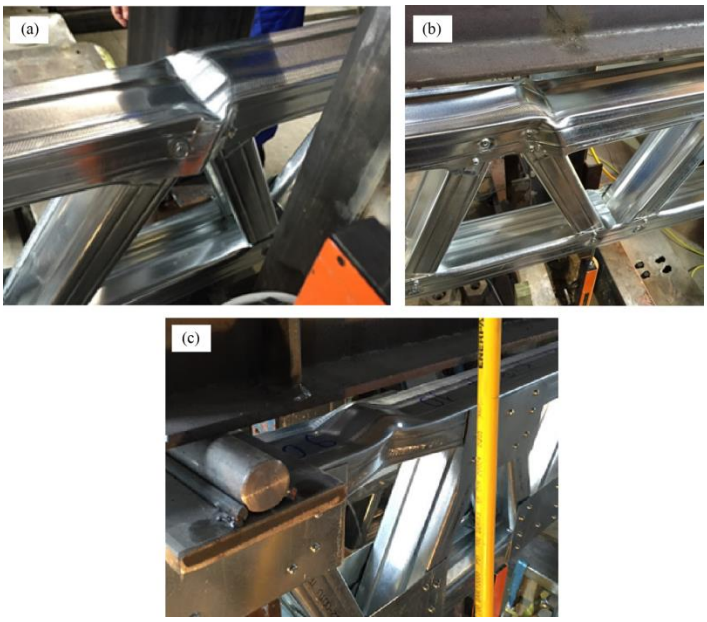


Figure 6: Failure modes: *(a) buckling inside eccentric connection; (b) buckling outside eccentric connection; (c) buckling within unsupported length*

Experimental results indicate that the number of fasteners used in the connection regions has a significant influence on overall truss behavior. The responses of trusses made from 1.2 mm thick lipped channels are compared in Fig. 7. In this figure the applied load versus mid-span deflection response of Specimens 6-10 are compared. Specimen 6 failed at a relatively low load level due to premature failure of the single rivet connection close to one of the support points. In contrast, Specimen 7 which employed a single #12 screw failed at almost twice the load. Regardless of the type of connection, service load stiffness of Specimens 6 and 7, which employed only one fastener, are close to each other. The load capacity changes slightly for Specimens 7-10 regardless of the number of fasteners used. On the other hand, there is a direct correlation between the number of

connection fasteners and the service load stiffness. The truss stiffness increases as the number of fasteners used in diagonal-to-chord connections is increased. The truss stiffness however, tends to saturate as the total number of fasteners used to connect each flange of diagonal and chord members reaches to three. In other words, the stiffness values for trusses with rivet + 2 screws and rivet + 3 screws are very close to each other.

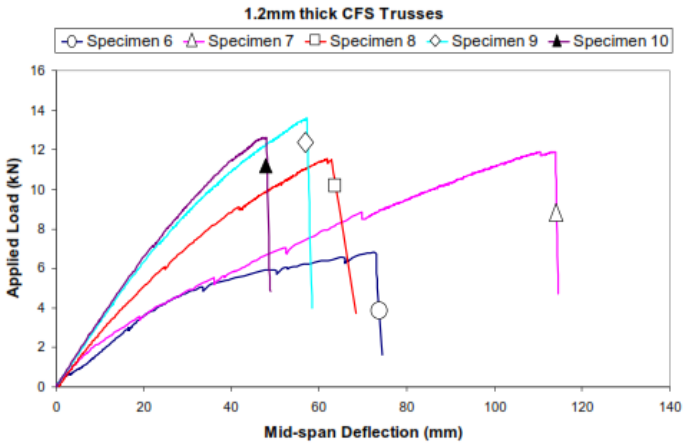


Figure 7: Load-deflection response of 1.2 mm thick CFS trusses

The load-displacement response of Specimens 10, 11, and 13 are compared in Fig. 8. Specimen 10 employs the standard connection detail where the diagonal ends are inserted inside the chords, Specimen 11 employs an additional gusset plate in the connection and Specimen 13 utilizes a continuous chord where the diagonals are connected to the chords from the outside. Presence of the additional gusset plates resulted in the compression chord failure to be located away from the diagonal-to-chord connection regions. Data presented in Fig. 8 suggests that there are only slight differences between the load capacities of truss specimens with three different connection details. The truss with additional gusset plates has a higher stiffness when compared with the other two specimens. The same observations are valid for both cases of 0.8 mm and 1.2 mm CFS section thicknesses. Test results indicate that costly measures such as providing additional gusset plates in the connection regions or connecting the diagonals to the outer face of the chords do not result in significant improvements in load carrying capacity.

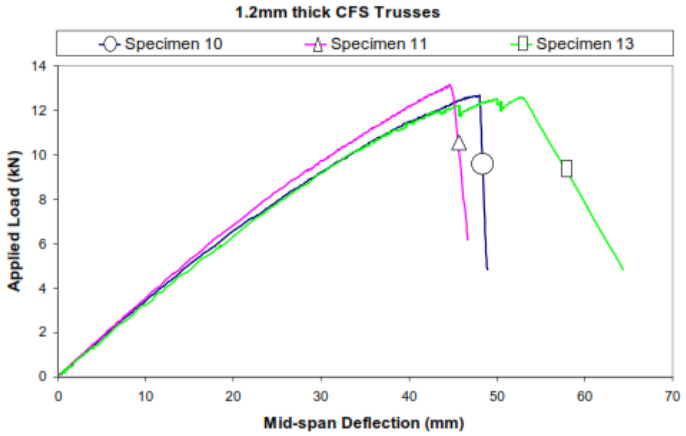


Figure 8: Load-deflection response of 1.2 mm thick CFS trusses with different connection types

In specimens tested with concrete infilled compression chord or with a concrete slab, no CFS chord buckling was observed during load testing. Testing of these specimens was terminated when mid-span deflection of approximately L/50 (120 mm) was reached. A comparison of the load versus mid-span responses of the inverted top chords trusses are provided in Fig. 9. The beneficial effect of concrete infill is clearly observed in load-deflection responses. Truss specimens with concrete infilled compression chord exhibited higher load capacity and stiffness compared to the companion trusses tested with no concrete infill. Concrete infill increased the load capacity by 43%. Presence of a concrete slab over CFS truss resulted in further improvement in flexural response in the form of increased stiffness and load capacity. The concrete slab increased the truss stiffness by approximately twice and the load capacity by 27% compared to the specimen with concrete infilled top chord.

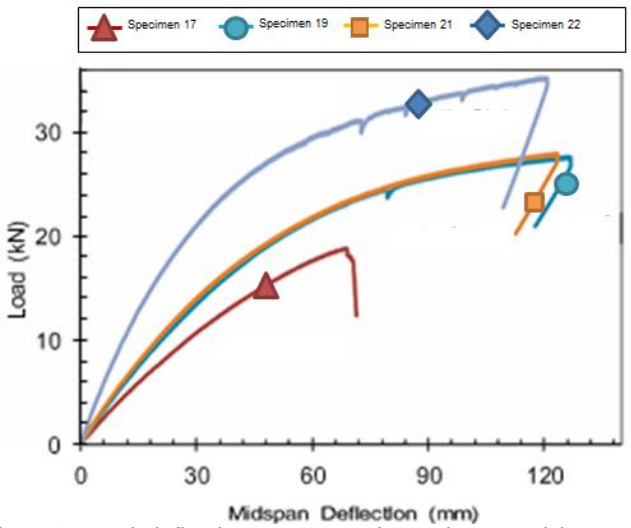


Figure 9: Load-deflection response of specimens with concrete infilled compression chord and concrete slab

3. Analysis of Truss Stiffness

3.1. Stiffness Calculations

Trusses in the current study were modeled according to the Structural Analysis Section of AISI S214-12 [9] and using the computer program SAP2000 v18 [10]. Beam elements were used to model the chord and diagonal members. Moment releases were assigned to the ends of the diagonal members. Based on AISI S214-12 [9] full member cross section properties were used instead of the effective section properties. For each truss specimen, two models were analyzed: single-node connection case and two-node connection case. In the two-node connection models, the location of the nodes coincided with the physical location of rivets connecting each end of diagonal members to chord members.

Comparison of the measured and predicted responses has shown that the measured stiffness is systematically less than the stiffness predicted by the computer model. The stiffness differences are more pronounced when using the single-node analysis as opposed to two-node analysis. The average measured-to-predicted stiffness values are 0.66 and 0.74 for trusses modeled using single- and two-node analyses, respectively [8]. The difference between the predicted and calculated stiffnesses are observed to increase when fewer fasteners are used at diagonal-to-chord connections. The over-prediction of truss stiffness is significant for Specimens 6 and 7, where either a single rivet or a single #12 screw is used for each connection. The differences between the predicted and experimentally determined stiffnesses were attributed to the neglect of connection deformations in analysis models. A separate study has been undertaken to address this issue and details of this study are reported in the following section.

3.2. Assessment of Diagonal-Chord Connection Flexibility

Strength and stiffness of cold-formed steel connections made using self-tapping screws have been the subject of previous research [11–18]. The majority of these investigations focus solely on the strength of such connections with the aim of developing analytical expressions to predict the connection load capacity corresponding to different failure modes. In an attempt to resolve the discrepancy between the truss stiffnesses measured from the experiments and determined by the computer analysis model, stiffness of the diagonal-to-chord connections used in truss specimens were investigated further. A set of experiments was conducted on CFS single lap joint specimens. The experimental data was used to quantify the flexibility present at diagonal-to-chord connections. The connection flexibility determined this way was then incorporated in the computer analysis model to predict the truss stiffness.

3.2.1. Connection Tests

The connection test specimens were arranged with different combinations of rivets and self-tapping screws in order to reflect the connection details used in truss specimens. The fastener configurations used in test specimens are shown in Fig. 10 together with the pictures of the test layout. Each specimen consisted of two 1.2 mm thick steel strips connected by a combination of rivets and screws. The rivets and screws used in the connection specimens were of the same type as those in truss specimens (i.e., 4.8 mm diameter blind rivets and 3.5 mm diameter self-tapping screws). The steel strips were taken from steel sheets that were used to rollform the CFS lipped channels used to fabricate truss specimens.

Each specimen was named based on the number of fasteners (e.g., Specimen 2R + 4S included two rivets and four screws). The specimens were loaded with monotonically increasing displacement loading and the relative displacement between two steel plies was monitored with an extensometer (Fig. 10a).

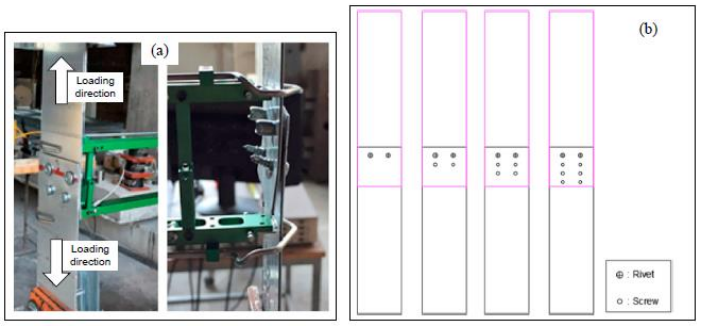


Figure 10: Connection tests: (a) test layout; (b) fastener configurations used in specimens

The force-deformation response of connection specimens with different number of fasteners is shown in Fig. 11. The connection response is generally characterized by an initial linear region followed by a stiffness degradation region and then a secondary linear region. Each of these regions corresponds to a combination of screw tilting and plate bearing deformation modes. The measured load capacities corresponding to a deformation of 1.5 mm were 4.7, 11.2, 15.0, and 16.9 kN, respectively with zero, two, four, and six screws in addition to two rivets. These values indicate that the major part of the connection load capacity is provided by screws rather than rivets.

The connection force-deformation response can be represented by an analytical expression as follows:

$$F = A \times \Delta \times (1 - \exp\left(\frac{-1}{B \times \Delta}\right)) \quad (1)$$

where *F*: total force on the connection (kN), Δ: connection deformation (mm), *A* and *B*: constants for different number of connectors. The values of *A* were determined by curve fitting to test data as 49.05, 34.34, 44.15, and 41.20 for specimens 2R, 2R + 2S, 2R + 4S, and 2R + 6S, respectively. Similarly the values of *B* were determined as 10.0, 3.0, 2.8, and 2.2 for specimens 2R, 2R+2S, 2R+4S, and 2R+6S respectively. A comparison of the measured force–deformation responses with those obtained from the proposed expressions is presented in Fig.11.

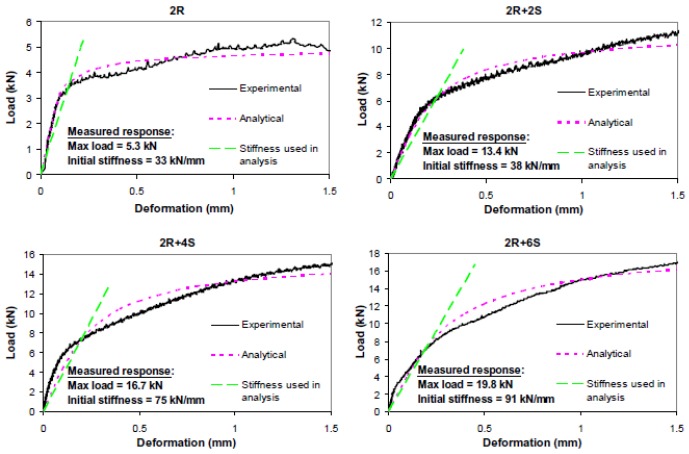


Figure 11: Force deformation response of connection specimens

3.2.2. Truss Stiffness with the Effect of Connection Flexibility

Truss analysis models described earlier have been modified in order to incorporate the additional axial flexibility present at diagonal-to-chord joints due to fastener deformation. For this purpose, linear axial spring elements were used at both ends of the diagonal members. In the model, diagonal member ends were connected to chord members with moment releases, hence any rotational restraint that may have been present at the connection was neglected. The analyses were repeated for the 1.2 mm thick truss specimens with different number of fasteners used at diagonal-to-chord connections, namely Specimens 6, 8, 9, and 10. The stiffness values used for axial spring elements in each model were based on the analytical expressions derived using the measured load-deformation response of connection specimens. For each model, secant stiffness of the corresponding analytical expression was calculated considering a specific point. This point was taken as the one that corresponds to 60% of the maximum axial force in diagonal members when the truss model was loaded with the experimentally determined maximum vertical load.

The relation between truss stiffness determined considering the additional axial flexibility present at diagonal-to-chord joints with the experimentally determined load-deflection behavior of the four truss specimens is presented in Fig. 12. Superimposed on the same plots are the responses determined from linear analysis with the absence of axial springs at diagonal member ends. The truss mid-span deflection corresponding to length/240 limit, which is usually used for serviceability check of beams, is also indicated on the plots. The load carrying capacity of trusses at this level of mid-span deflection (length/240) is very close to 60% of their ultimate load carrying capacity. It should be mentioned that all analysis models employed the two-node connection modeling approach. Incorporating the axial flexibility of diagonal-to-chord joints in analysis models resulted in improved accuracy of truss stiffness prediction. The measured-to-predicted ratios of truss stiffness are 0.44, 0.63, 0.78 and 0.84 for Specimens 6, 8, 9, and 10 respectively when rigid connections are used in modeling. For the flexible connection case, the ratios modify to 0.64, 0.86, 0.96, and 1.01 for Specimens 6, 8, 9, and 10 respectively. Truss stiffness predicted using flexible connections agreed well with the experimentally determined response especially for trusses where three (rivet+2screws) and four (rivet+3screws) fasteners were used to connect each flange of diagonal and chord members.

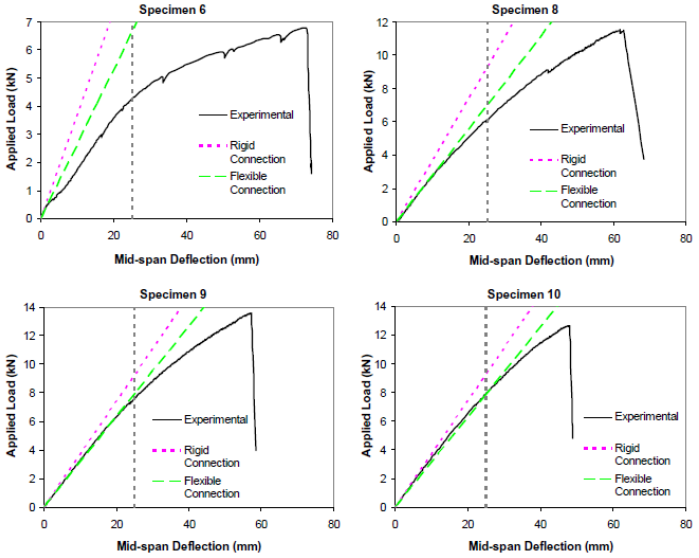


Figure 12: Prediction of truss stiffness using linear analysis

The connection flexibility has a significant influence on the stiffness of diagonal members. The axial stiffness of diagonals considering the flexibility of its connections (K_{SYS}) can be estimated as follows:

$$\frac{1}{K_{sys}} = \frac{1}{K_d} + \frac{2}{K_{conn}} \quad (2)$$

where K_d: axial stiffness of the diagonal, K_{conn}: stiffness of the connection. The 2 term in the numerator represents the presence of flexible connections at both ends of the member. For the sizes of the channel section diagonals studied in this experimental program the axial stiffness of diagonals (K_d) vary between 103 kN/mm and 193 kN/mm depending on the thickness of the diagonal. The secant stiffness of connections corresponding to 60% of the ultimate applied load on the diagonals, shown in Fig. 12, varies between 24 kN/mm and 37 kN/mm depending on the number of connectors. It is clear from these values that the connections are much more flexible compared to the diagonal member itself. Considering Eq. (2), the ratio of axial stiffnesses of the diagonals with and without flexible connections vary between 0.06 and 0.15 for the cases studied herein. This implies that in cases where rigid connection models are used, the axial stiffness of the diagonals should be reduced by multiplying the original stiffnesses by these ratios at the design stage to accurately predict the truss deflections.

4. Analysis of Chord Strain Profiles

A common assumption used in the design of CFS trusses is that the members of the truss are considered to be under pure axial forces that produce uniform amount of axial stress on the cross-section. Global bending of the chord members can also be an issue when the truss is relatively shallow.

4.1. Analytical Studies of Chord Strains

An analytical approach was developed in an attempt to predict the response of CFS trusses without the need of detailed FE analysis or physical testing. The approach is based on determining the cross-sectional strains in CFS chord members due to axial force and bending moment effects. External loading acting on trusses induces both axial strain and bending strain on chord members. The total chord strains at the most critical cross section can be estimated by calculating the sum of strains due to these two effects. The flowchart of the chord strain computations is given in Fig. 13.

Axial forces in chord members were calculated by dividing the moment exerted on the truss by the distance between the centroids of top and bottom chord members. The chord axial force was then converted into the corresponding axial strain by dividing it with the cross-sectional area and the modulus of elasticity (Eqn. (3)).

$$\varepsilon_{axial} = \frac{M_{truss}}{d*E*A_{chord}} \quad (3)$$

where, d is the distance between the centroids of top and bottom chords, M_{truss} is the external moment acting on the truss at the mid-span section, E is the modulus of elasticity for steel and A_{chord} is the cross-sectional area of each chord member.

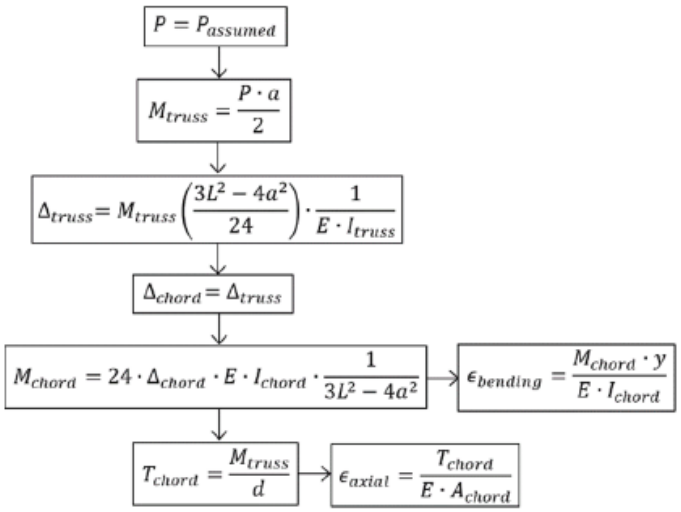


Figure 13: Flowchart for chord strains

Chord bending strains were determined by making use of the vertical deflections at the truss mid-span cross section. Bending moments developing in chord members (M_{chord}) were calculated based on the assumption that the mid-span deflection (Δ) of the truss as a result of the applied external loading is equal to the mid-span deflection of the chord member itself resulting from the bending moment (M_{chord}) (Eqn. (4)). It should be noted that any additional mid-span deflection due to shear deformation of the open-web truss cross section is neglected in Eqn. (4).

$$\frac{M_{chord}}{EI_{chord}} = \frac{24\Delta}{3L^2 - 4a^2} \quad (4)$$

where, EI_{chord} and L represent the flexural rigidity and the length of CFS chord members, respectively, while a represents the length of shear span. The chord bending moment determined this way was then utilized to calculate the chord bending strain at the mid-span cross section at a location y distance away from the

centroid of chord CFS section (Eqn. (5)). The computed axial and bending strains were then summed together to determine the total chord strains (Eqn. (6)).

$$\varepsilon_{\text{bending}} = \frac{M_{chord}}{EI_{chord}} y \quad (5)$$

$$\varepsilon_{\text{total}} = \varepsilon_{\text{bending}} + \varepsilon_{\text{axial}} \quad (6)$$

4.2. Experimental Studies on Chord Strains

Chord members in four of the truss specimens (Specimens 4, 5, 15, and 16) were instrumented with strain gauges in order to investigate the variation of stresses in typical cross-sections. Attachment of strain gauges on compression chord in one of the truss specimens is shown in Fig. 14. The strain gauges used for this purpose were TML FLA-5-11-3L type. The instrumented specimens were subjected to several elastic loading-unloading cycles in order to verify the readings of strain gauges and make sure that consistent strain readings are obtained in all loading cycles. During these loading cycles, special attention was paid not to load the specimens beyond half of their expected load capacities.



Figure 14: Strain gauges attached on truss chord member

Readings from the strain gauges were used to determine the cross-sectional strain distribution in chord members, as shown in Fig. 15. For Specimens 4, 5, and 16 only the compression chord members were instrumented with strain gauges, while both chords were instrumented in Specimen 15. Presence of flexural effect on both compression and tension chords in truss specimens is clearly evident in the experimentally determined strain profiles. Non-uniform strain distribution across the depth of chord members, as shown in Fig. 15, indicates that truss behavior based on the assumption of pure axially loaded members was not valid for the truss specimens. In addition, there are deviations from linearity across the depth of the sections. In Specimens 4 and 5 the lips were subjected to more compressive strains than would be expected from a linear profile while the opposite is true for Specimens 15 and 16. The strains across the web plate are expected to be fairly uniform. Considering the top two points in the strain plots in Fig. 15, the strain profiles show that the stiffened webs are subjected to non-uniform strains, where the edges of the web plate are subjected to higher strains than the stiffened center portion for Specimens 4, 5, and 16.

The chord strain profiles obtained from the analytical method are compared in Fig. 15 with those obtained from the experimental part of the study. The given strain profiles for each specimen were obtained under a load corresponding to half of the experimentally measured load capacity. Therefore, these strain profiles can be considered to represent the service condition strains. Accuracy of the proposed analytical procedure seems to be well for all trusses that have been tested.

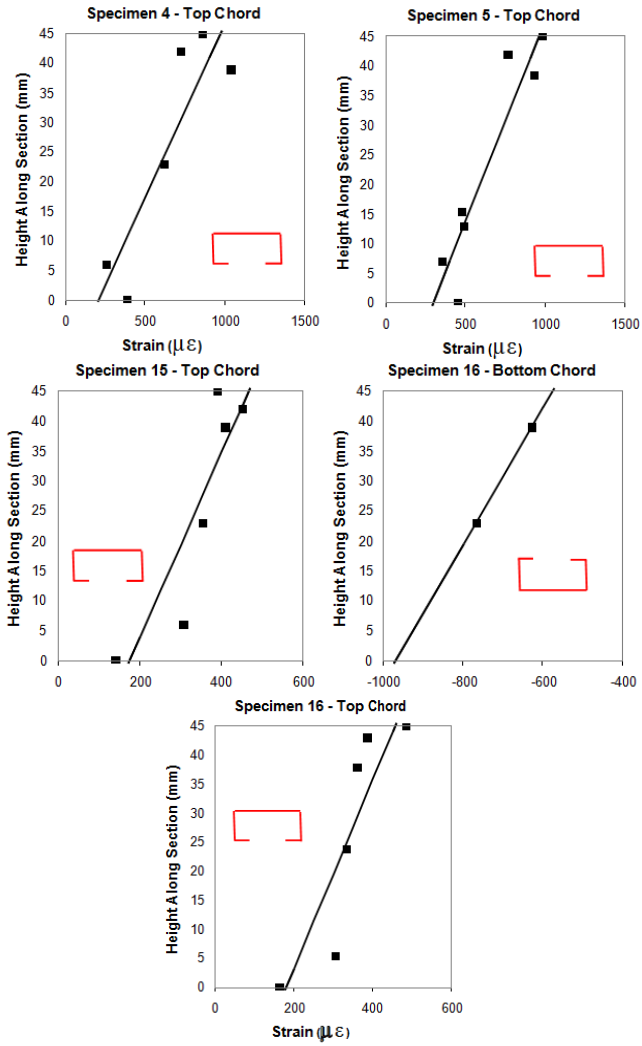


Figure 15: Experimentally determined strain profiles in truss chord sections

7. Conclusions

An experimental study on 22 full-scale floor trusses fabricated from cold-formed steel lipped channel sections tested under four point bending has been presented. Thickness of lipped channels, number of connection fasteners, type of diagonal connection, and presence of concrete infill inside compression chords were considered as the variables of the testing program. Complementary experimental studies were also conducted to investigate the stiffness of typical connections used in diagonal-to-chord joints of these trusses. Major findings from the study are as follows:

- Three types of diagonal-to-chord connections were considered in the experimental study. The first one employed the standard detail where the flanges of the channel section are straightened by un-folding the lips. The second one had diagonals with webs removed such that the chord member is continuous without any discontinuity due to unfolding of lips. The third one is similar to the first detail but with gusset plates. The impact of using different connections did not result in discernable differences. Therefore, the standard detail, which is usually considered to be more economical, can be employed in these systems.
- The number of fastener used at diagonal-to-chord connections was determined to have a significant influence on truss response. The stiffness of trusses was found to be directly proportional to the number of connectors used.
- Presence of concrete infill inside the compression chord member prevented the initiation of chord local/distortional buckling. In these specimens the failure mode was tension yielding of the bottom chord member. With the introduction of concrete infill within compression chord, significant improvement in truss response in terms of increase in stiffness and load capacity was obtained. Trusses with concrete infilled compression chord were also observed to exhibit relatively ductile response. It should be emphasized that these results may not be valid for other truss layouts than the one tested in the current study or for trusses with weaker web members than chords.
- The service load stiffness of the trusses was found to be over-predicted by classical 2-Dimensional structural analysis models. The reason for this overprediction was determined to be the neglect of connection flexibility.
- The complementary experimental study on connections has shown that the stiffness of the connections are much lower than the axial stiffness of the members. For the cases studied herein the overall stiffness of the diagonal members with connection flexibility effects is between 6 and 15 % of the original stiffness.
- An analytical procedure based on a simple linear-elastic mechanics approach was developed in order to predict the response of steel-concrete composite CFS trusses without the need of detailed FE analysis or physical testing. Chord strain profiles determined using the proposed analytical procedure at service load levels agree well with the experimental results

8. Acknowledgments

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