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Lacing and unsupported chord stability characteristics in cold-formed steel built-up columns: Tests and design aspects

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Abstract

Columns are the principal structural members of the super-structure directly connected to the foundations. Both their capacity and stiffness response need to be adequate for satisfactory performance. Generally, in cold-formed steel (CFS) framing systems, two channels are fastened in the back-to-back arrangement to form a built-up column (I-type) and are the primary compression members adopted in low rise CFS framed structures. This approach is very convenient, particularly from the fabrication consideration. Still, it limits the load-carrying potential that the channels (chords of the built-up column) can develop when a suitable transverse gap is provided between the chords. The previous research on gapped CFS built-up columns shows that such columns perform better than the conventional I-type built-up columns, particularly from axial capacity and stability considerations (mostly the torsional one). The past research on transversely gapped built-up columns has mostly focused on the ones with battened connections, and very little research on CFS laced columns has been reported. The limited past research on CFS laced columns has indicated that when plain angle sections are adopted as chords, they experience early local buckling during the initial stages of loading, thereby resulting in an early failure at a lower load. The use of plain channel as chords in CFS battened columns has shown superior performance than those with plain angles as chord. This justifies the need to explore the performance of CFS laced columns composed of plain channels, which has been addressed in the current study. Two plain channels were arranged in a toe-to-toe configuration to form a closed built-up section, with lacing plates being adopted as lateral connectors. In a single lacing configuration, a single self-drilling screw was used to fasten each end of the lacing plate to the chords. The transverse spacing of the chords and the lacing slenderness was varied. The effect of these variations on the axial compression capacity and stability response of CFS laced columns was assessed, with particular consideration to the lacing and unsupported chord stability characteristics. The structural performance of the built- columns were evaluated in terms of their peak loads resisted, load-displacement response and the failure modes. Lastly, the North American Specification and Eurocode for CFS structures were used to determine the design strengths for comparison. Inconsistencies in the accuracy were noted in the predictions made by both these standards.

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1. Introduction

Cold-formed steel (CFS) members have become increasingly popular in constructing mid-rise residential and commercial structures, owing to their desirable structural features, the most important of which are lightweight and faster construction due to ease of manufacturing and erection. These features make it more suitable for structures when the construction site is remote, and the transportation/handling of the structural elements can be carried out conveniently. It also promotes timely completion of building assignments, giving it an advantage over building materials because it simply requires assembling/connecting different structural elements to complete the project. Although CFS sections have several major advantages that encourage their utility in structural construction, the thin-walled nature of the CFS section still limits their acceptance due to the local buckling instability of the various cross-sectional thin plate elements (Yu 2010; Zeimian 2010). This has encouraged researchers, particularly those working on structural steel, to conduct research in this area and develop a suitable solution to these issues of uncertainty. As a result, numerous research studies have brought out efficient, reliable and affordable solutions to improve the buckling performance of various types of improved cross-sections (Zhou et al. 2022; 2021a-b; Nie et al. 2020a-b; Selvaraj & Madhavan 2022; Li and Young 2022; Li et al. 2021; Rasmussen et al. 2020; Landesmann et al. 2016; Camotim et al. 2018; Kumar & Sahoo 2016; Bian et al. 2016; Kesawan et al. 2017; Paratesh et al. 2019; Joorabchian et al. 2021; Derveni, et al. 2020; Maderia et al. 2015; Gatheeshgar et al. 2020).

2. CFS built-up columns

A typical conventional CFS built-up column (I-type) is made by fastening two channel sections in a back-to-back configuration through the webs at standard longitudinal spacing. Such types of built-up columns are widely used in building CFS framing systems. Several attempts have been made to enhance the buckling performance of such I-type built-up columns by specifying the limits to the flat width-to-thickness ratios for the various cross-sectional elements and also by recommending suitable patterns of the screwed connections at various locations (Selvaraj & Madhavan 2021; Mahar et al. 2021a;b; Mahar and Jayachandran 2021; Roy et al. 2018; Fratamico et al. 2018a-b). The introduction of a suitable transverse gap between the channels can further improve the structural performance of these built-up columns (Subramanian 2016). Various lateral connecting systems can achieve adequate structural integrity between the chord members. The transverse gap also controls the stability characteristics as well as the torsional resistance in such columns (Dabaon et al. 2015; Anbarasu & Dar 2020a-b Zhang & Young 2015; Anbarasu 2020; Vijayanand & Anbarasu 2021;2020; Anbarasu et al. 2015; Anbarasu and Venkatesh, 2019; Ghannam, 2017). However, the toe-to-toe configuration between the chord members has indicated an improved performance over the ones with the chords arranged in the back-to-back orientation (Meza et al. 2020a-b; Kherbouche & Megnounif 2019), and owes that improvement to the closed sectional configuration of these built-up cross-sections, which has been confirmed through more studies (Zhang & Young 2018; Liao et al. 2017; Dar et al. 2018; 2019a-b; 2020a-c;2021a-f; 2022 Roy et al. 2019). From the limited studies on CFS closed section battened columns (EI Aghoury et al. 2010; 2013; Dar et al. 2021c-d; 2020a-b; Anbarasu & Dar 2020; Anbarasu 2020; Rahnavard et al. 2021), the unsupported chord slenderness (slenderness of the chord between the intermediate battens) governs the structural performance of such columns, particularly the short and intermediate columns. Also, the lateral connectivity in battened columns is discontinuous. With these shortcomings, there is a need to explore the performance of CFS laced columns with plain channels adopted as chords members.

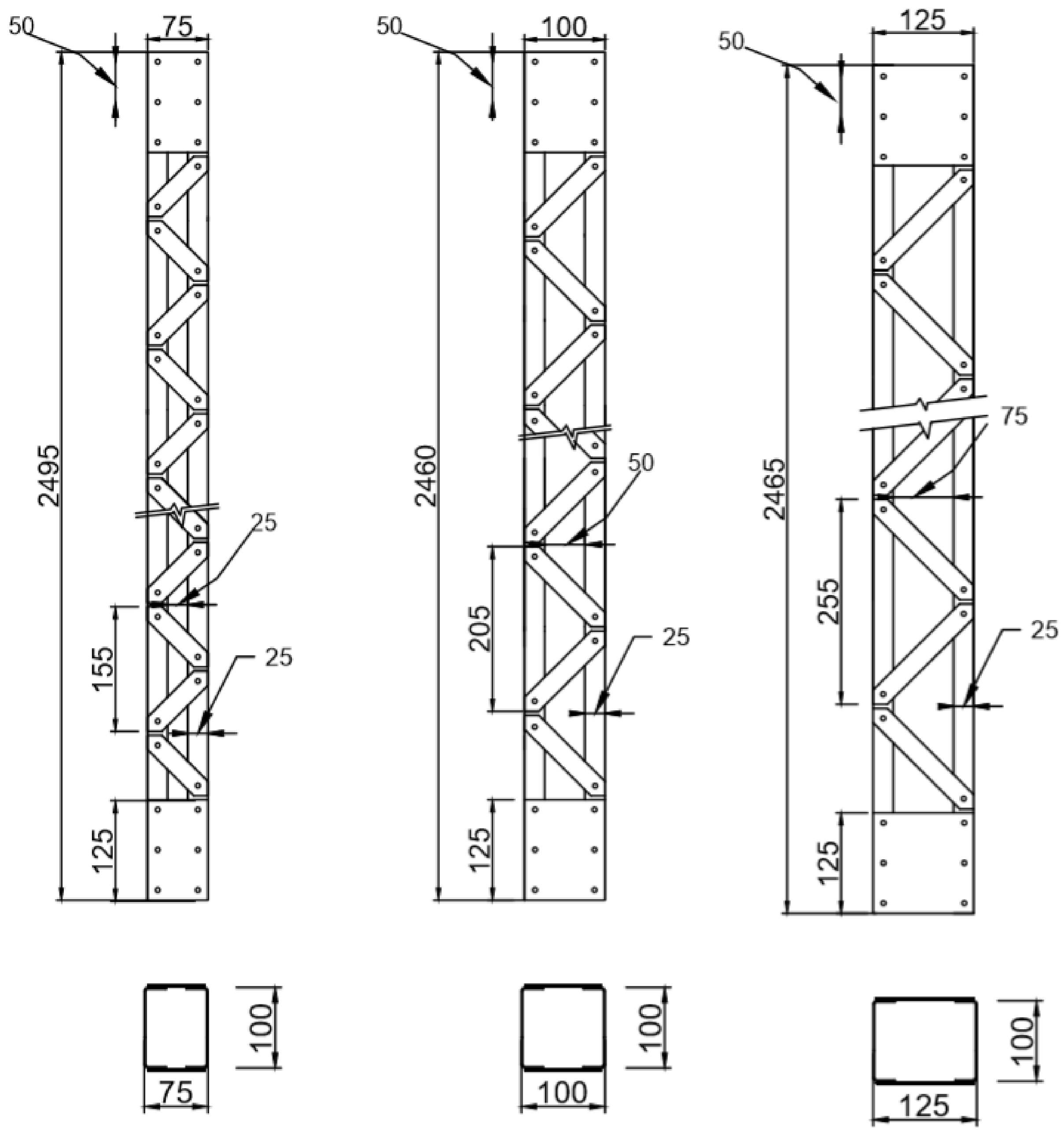
3. CFS laced built-up columns

The major part of the previous research work carried out on CFS laced columns comprised of plain angles being adopted as chord members to facilitate the flexibility of adopting the transverse gap in two orthogonal directions for more structural efficiency. However, such laced columns suffered from early local buckling during the primary stages of loading. This early instability in the chords resulted in early failure at a lower axial load. Therefore, promoting the adoption of plain channels as chords in CFS built-up columns. A study carried out on CFS laced built-up columns with the chord itself being adopted as a built-up section (comprising of two lipped channel sections to form a built-up angle profile) indicated a satisfactory performance (Bastos & Batista 2019). However, the complex fabrication process for such built-up columns does not promote their adoption in real practice. Also, the built-up columns composed on many mono-sectional profiles result in a complex behaviour with a weak post-peak behaviour, as shown by previous studies. This justifies the need for exploring the behaviour of CFS laced built-up columns composed of two plain channel sections to form a closed built-up section.

In this experimental study, two plain channels were arranged in a toe-to-toe configuration to form a closed built-up section, with lacing plates being adopted as lateral connectors. In a single lacing configuration, a single self-drilling screw was used to fasten each end of the lacing plate to the chords. The transverse spacing of the chords and the lacing slenderness was varied. The effect of these variations on the axial compression capacity and stability response of CFS laced columns was assessed, with special consideration to the lacing and unsupported chord stability characteristics. In addition, the structural performance of the built- columns were evaluated in terms of their peak loads resisted, load-displacement response and the failure modes. Lastly, the North American Specification and Eurocode for CFS structures were used to determine the design strengths for comparison.

4. Test specimens

Six test specimens (as shown in Fig.1(a-c)) were fabricated to complete the intended research objectives of the current study. Two plain channel sections were used as chord members and were oriented in toe-to-toe arrangement to form a closed built-up section (as shown in Fig.2) with lacing elements used as lateral connectors. The channel sections were formed using press braking operation from a steel strip 150mm wide and 2mm thick. Each channel had a web depth of 100 mm and a flange width of 25mm. The radius of curvature at the flange web junction was 2.4mm. The length of each channel supplied by the supplier was 2500mm. Three different values of the transverse spacing between the tips of the flanges, viz., 25mm, 50mm and 75mm were adopted, as shown in Fig.1. The width of each lacing plate was 25mm. Steel plates of three different thicknesses viz., 2.5mm, 4mm and 6mm were used to prepare the lacing elements. The lacing plates were adopted on the single lacing configuration with the lacing inclination of 45^0. The height of the specimens varied from 2460mm to 2495mm. The depth of the end plates adopted in all the six specimens was 125mm. A single self-drilling screw (5mm diameter) was used to connect each end of the lacing plate to the flanges of the chords. The end plates were connected to the chords using the same size of screws in three rows. The labelling of the specimens was carried out such that important details get reflected, e.g., in the label “SL-25-2.5”, SL reflects the single lacing arrangement being adopted, 25 indicates the transverse spacing (in mm) between the tips of the flanges of the chords, placed in the toe-to-toe orientation, and finally the last number 2.5 represents the thickness (in mm) of the lacing element.



(a) (b) (c)

Figure 1: Longitudinal details of the test specimens, (a) SL-25-2.5 & SL-25-6; (b) SL-50-2.5 & SL-50-6; SL-65-4 & SL-75-6.

5. Material Properties

The actual material properties of the steel used to form the chord members need to be determined. Therefore, tensile tests were performed on the tensile coupons prepared according to the Indian Standards (IS 1608, 2005) and were derived from the channels. An MTS universal testing machine (UTM) was used to perform these tensile checks. Totally three coupon tests conducted. The average yield strength (f_y in MPa), ultimate strength (f_u in MPa), elasticity modulus (E in GPa), and elongation (e in percent) values were recorded as 423.2, 501.7, 203 and 25.2, respectively. Fig.3 presents typical stress *vs.* strain curve obtained from the material tests.

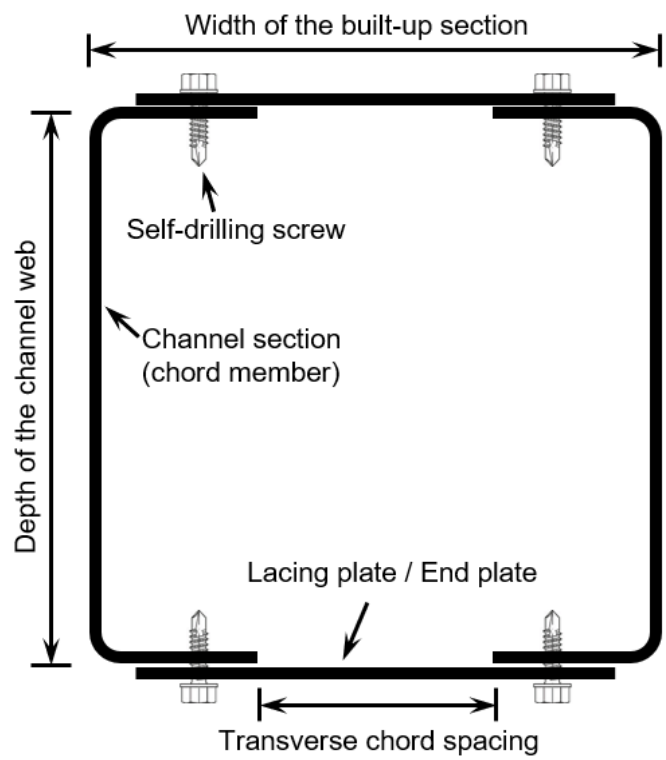


Figure 2: Cross-sectional details of the built-up column specimens

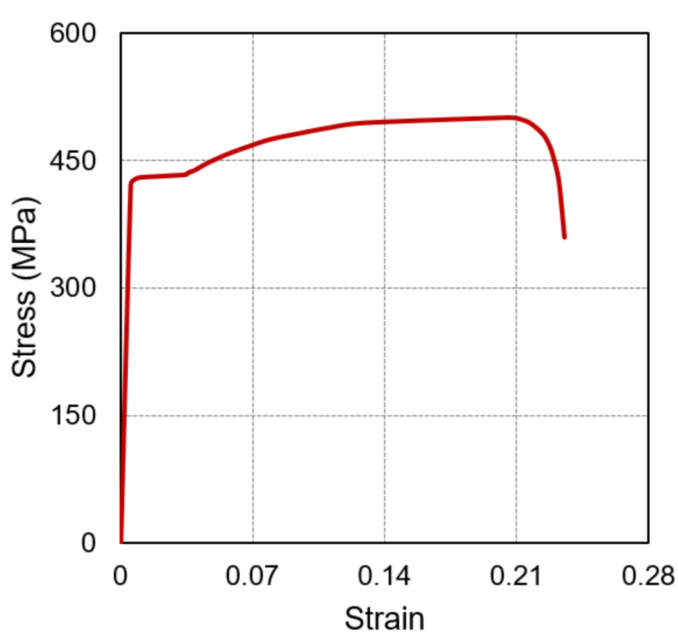


Figure 3: Typical stress-strain plot of the coupons

6. Test Set-up

A strong loading frame of 300 kN capacity (as shown in Fig. 4) was used to perform the concentric axial compression tests on the laced built-up column specimens. A hydraulic jack (500kN) was used to apply the compression loading axially. A load cell of the same capacity was used to monitor the axial loading component during the testing of the specimens. Two displacement sensors (LVDTs) were adopted to note the axial shortening and lateral deflections (at the mid-height). The load cell and the LVDTs were connected to a computerized data acquisition system.

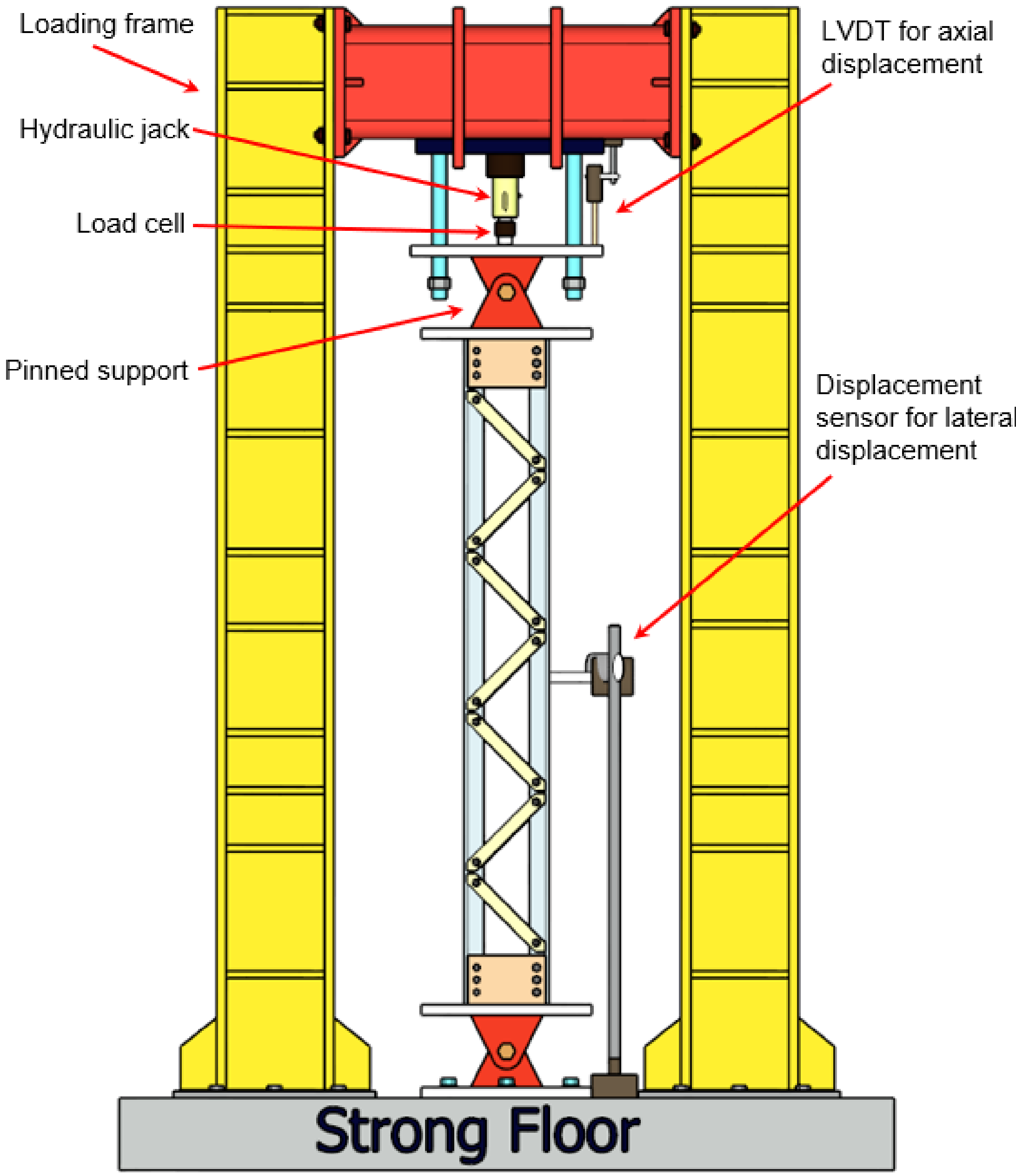
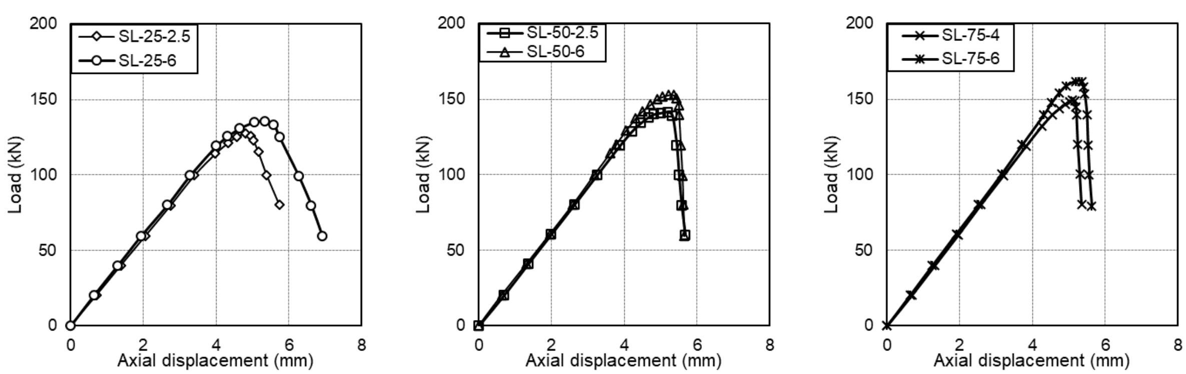


Figure 4: Details of the test set-up

7. Test Results

Fig.5(a-c) presents the axial load *vs.* axial shortening behavior of all the specimens. The trend of axial load *vs.* axial shortening curves of specimens SL-25-2.5 and SL-25-6 is nearly similar and is shown in Fig.5(a). The peak axial resistance developed by SL-25-2.5 was axial 128.6kN with a corresponding axial shortening of 4.64mm. The same for specimen SL-25-6 was 135.3kN and 5.34mm, in the same order. It can be noted that the axial strength improvement of 6.7kN (5.2%) was observed due to the change in the lacing thickness from 2.5mm to 6mm. However, there was no noticeable variation in the axial stiffness. The axial load *vs.* axial shortening plots of specimens SL-50-2.5 and SL-50-6 are very similar and shown in Fig.5(b). The ultimate axial strength produced by SL-50-2.5 was axial 143.3kN with a corresponding axial shortening of 5.5mm. The same for specimen SL-50-6 was 154.1kN and 5.22mm, in the same order. It can be noted that the axial strength improvement of 10.8kN (7. 35%) was observed due to the change in the lacing thickness from 2.5mm to 6mm. Again, there was a meagre variation in the axial stiffness. The axial load *vs.* axial shortening plots of specimens SL-75-4 and SL-75-6 are very similar and shown in Fig.5(c). The ultimate axial strength produced by SL-50-2.5 was axial 149.1kN with a corresponding axial shortening of 5.01mm. The same for specimen SL-50-6 was 162kN and 5.35mm, in the same order. It can be noted that the axial strength improvement of 12.9kN (8.7%) was observed due to the change in the lacing thickness from 2.5mm to 6mm. Again, a small variation in the axial stiffness was noted. It was also noted that the post-peak behavior of specimens SL-25-2.5 and SL-25-6 was different from that of the remaining four specimens. The stiffness of the falling branch of the post-peak curve in SL-25-2.5 and SL-25-6 was lower than the other specimens. The primary reason behind this variation is the higher overall slenderness in these two specimens than the remaining four specimens.



(a) (b) (c)

Figure 5: axial load *vs.* axial shortening behaviour, (a) SL-25-2.5 & SL-25-6; (b) SL-50-2.5 & SL-50-6; SL-65-4 & SL-75-6.

All the six specimens experienced local buckling in the chord, located nearly at the mid-height of the columns, as shown in Fig.6(a-f). In the chord, the web element mostly suffered the local buckling instability. The main reason for this behavior is the web's vulnerability towards local buckling due to high sectional slenderness compared to the flange. The variation in the lacing slenderness did not influence the mode of failure or the location of the failure (along the column height). Furthermore, the variation in the transverse spacing also did not alter the mode of failure in the various specimens. It was also noted that none of the specimens suffered from any type of connection failure, clearly indicating that the self-drilling screws were adequately designed. In some cases, even though the local buckling of the chord around the column mid-height was close to the connection, the structural integrity between the lacing plate and the chord was not affected, again reflecting the adequacy of a single screw adopted to connect each end of the lacing to the chord.

(a) (b) (c) (d) (e) (f)

Figure 6: Failure in the test specimens, (a) SL-25-2.5; (b) SL-25-6; (c) SL-50-2.5; (d) SL-50-6; (e) SL-65-4, and (f) SL-75-6.

8. Design strengths

Currently, no design guidelines predict the axial strengths of CFS laced columns composed of channel sections as chords. Therefore, North American Specification (AISI S100:2020) and European Standards EN1993-1-3 (2006), meant for designing CFS structures were used to quantify the design strengths of the various specimens. Both these standards give basic design steps for conventional I-type CFS built-up column made by fastening two channel sections in a back-to-back configuration, through the webs at standard longitudinal spacing. Fig.7(a&b) and Table 1 compare the design strength prediction of these standards against the test strengths. It was noted that both the North American Specification (AISI S100:2020) and European Standards EN1993-1-3 (2006) mostly predicted the strengths of CFS laced columns composed of plain channels unconservatively, particularly when the overall slenderness of the built-up columns was low. The degree of unconservativeness increased with the increase in the overall slenderness of the laced columns. The indiscrpancies observed between the design predicted strength and the test strengths calls for more research to be carried out on such built-up columns that will generate a large pool of data points which will be helpful for the development of reliable design rules for CFS laced columns composed of plain channel sections.

Table 1: Comparison of test results and design strengths

Specimen	P_{Test} (kN)	P_{NAS}/P_{Test}	P_{EC3}/P_{Test}
SL-25-2.5	128.57	1.14	1.02
SL-25-6	135.27	1.09	0.97
SL-50-2.5	143.30	1.21	1.17
SL-50-6	154.10	1.13	1.09
SL-75-4	149.10	1.22	1.24
SL-75-6	161.97	1.12	1.14

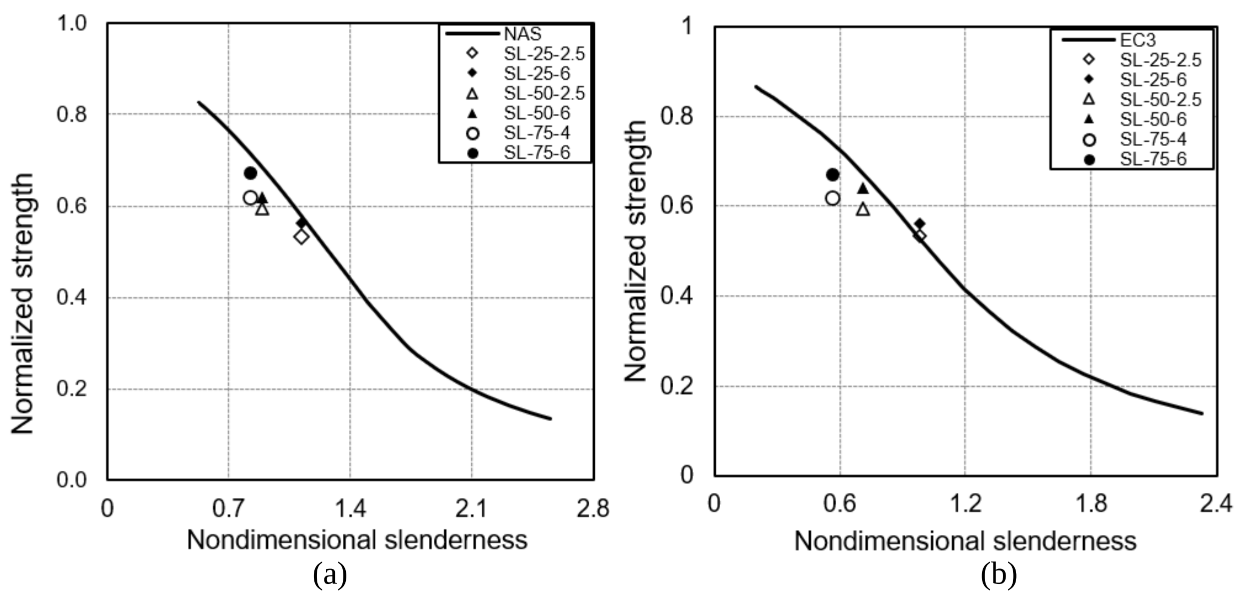


Figure 7: Comparison of test strengths and the predicted strengths, (a) North American Specification (AISI S100:2020) ; (b) European Standards EN1993-1-3 (2006)

5. Summary and Conclusions

An experimental study on CFS laced columns composed of plain channel sections was attempted in the current investigation. Pin ended support conditions were adopted for all the laced column specimens. The impact of various important parameters like transverse spacing between the tip of the channel flanges and lacing thickness was examined. The influence of these variations on CFS laced columns' axial compression resistance and stability response was evaluated. The variation in the structural behavior of the built-up columns was monitored in terms of their resisted peak loads, load-displacement responsiveness and modes of failure. Finally, the North American Specification (AISI S100:2020) and European Standards EN1993-1-3 (2006) for CFS structures were used to assess the design strengths for comparison. In their accuracy, the forecasts of each of these standards demonstrated inconsistency. Both the North American Specification (AISI S100:2020) and European Standards EN1993-1-3 (2006) mostly predicted the strengths of CFS laced columns composed of plain channels unconservatively, particularly when the overall slenderness of the built-up columns was low. The degree of unconservativeness increased with the increase in the overall slenderness of the laced columns. All the specimens failed by local buckling of the web located near the mid-height region of the built-up columns. No failure at the connection levels were observed, which reflected adequacy of the connection design adopted. The variation in the lacing thickness resulted in the peak strength variation ranging from 5.2%-8.7%.

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Notations

CFS	: Cold-formed steel
E	: Modulus of elasticity
f_u	: Ultimate strength
f_y	: Yield strength
P_{NAS}	: Design strength predicted by North American Specification (AISI S100:2020)
P_{EC3}	: Design strength predicted by and European Standards EN1993-1-3 (2006)
P_{Test}	: Peak test strength
ε	: Strain at fracture