



Experimental investigation on the buckling stability of cold-formed steel laced stub columns: Influence of critical parameters

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Abstract

Cold-formed steel (CFS) members are used more often in building construction as they are light in weight, cost-effective, faster and easy to assemble, making them highly suitable for modular construction. As the main structural components, columns need to acquire considerable axial strength and stiffness to meet the various structural requirements. In a CFS low-rise framed structure, built-up columns made up of two web-fastened channels configured in the back-to-back order are used as column elements. Even though this arrangement is easy and convenient to fabricate, the I-type built-up configuration is highly inefficient and restricts its load-carrying capacity. A simple transverse gap between the chords can improve both the strength as well as stiffness characteristics of such built-up systems. Previous research on gapped CFS built-up columns found that they outperformed typical I-type built-up columns in terms of axial capacity and stability. There have been limited research outcomes reported on CFS-laced columns, as a large part of the previous studies on transversely gapped built-up columns has focused mainly on battened columns. Plain angle sections are more prone to early local buckling during the early phases of loading, which leads to an early failure at a lower load, according to past studies, which do not recommend their adoption as chord members. Additionally, plain channel sections have proven to function better than plain angle sections when used as chords in CFS laced and battened built-up columns. However, limited research has been conducted on CFS laced columns, with none on CFS stub columns. Therefore, further study CFS laced stub columns composed of plain channel sections is needed, which has been addressed in the current study. The study assessed the effect of critical parameters on the strength and stability of CFS laced stub columns, with particular consideration given to lacing stability characteristics and end plate width. The column behavior was studied in terms of failure modes and load-displacement plots. Lastly, the design strengths for comparison were determined using the North American Specification and Eurocode for CFS structures.

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1. Introduction

In recent years, the popularity of utilizing cold-formed steel (CFS) sections in the construction of mid-rise residential and commercial structures has increased significantly. The desirable qualities of these structures, such as their lightweight members, faster construction times, and easy fabrication and assembly, have contributed to this trend. Additionally, when the construction site is remote, the advantages of CFS become even more apparent, as transportation and handling of structural members becomes more convenient. This approach also promotes timely completion of building assignments, as it only requires assembling and connecting different structural elements to finish the project. Despite the many benefits of CFS sections, their thin-walled nature still poses a challenge due to local buckling instabilities experienced by various thin cross-sectional plate elements (Yu 2010; Zeimian 2010). As a result, researchers in the field of structural steel have focused on finding effective and cost-efficient solutions to improve the buckling performance of various types of modified cross-sections. (Li and Young 2022; Selvaraj & Madhavan 2022a; Zhou et al. 2022;2021a-b; Li et al. 2021; Derveni et al. 2020;Nie et al. 2020a-b; Rasmussen et al. 2020; Camotim et al. 2018; Paratesh et al. 2019; Landesmann et al. 2016; Kumar & Sahoo 2016; Bian et al. 2016; Kesawan et al. 2017; Gatheeshgar et al. 2020;Joorabchian et al. 2021; Maderia et al. 2015).

2. CFS built-up columns

CFS (Cold-Formed Steel) built-up columns of I-type are widely used in constructing CFS framing systems due to their simple fabrication process. These columns are constructed by fastening two back-to-back channel sections through their webs at standard longitudinal spacing. Several studies have been conducted to improve the buckling strength of such columns by suggesting limits to flat width-to-thickness ratios and specifying appropriate patterns for the screwed connections at various locations(Mahar et al. 2023;2021a;b; Selvaraj & Madhavan 2022b;2021; Mahar and Jayachandran 2021; Fratamico et al. 2018a-b; Kechidi et al. 2020;Roy et al. 2018). Incorporating an appropriate transverse gap between the channels can significantly improve the structural performance of these built-up columns (Subramanian 2016). Various lateral connection systems can provide proper structural rigidity and integrity between chord members. The transverse gap also controls the stability characteristics as well as the torsional resistance in such columns (Dabaon et al. 2015; Zhang & Young 2015; Vijayanand & Anbarasu 2021;2020; Anbarasu 2020; Anbarasu & Dar 2020a-b; Anbarasu et al. 2015; Ghannam 2017; Anbarasu and Venkatesh, 2019). However, the toe-to-toe configuration between the chord members has shown to have improved performance over the ones with the chords arranged in the back-to-back orientation (Kherbouche & Megnounif 2019; Meza et al. 2020a-b). Closed sectional configuration of these built-up cross-sections with superior performance has been confirmed through more studies (Liao et al. 2017; Zhang & Young 2018; Roy et al. 2019; Dar et al. 2018; 2019a-b; 2020a-c;2021a-f;2022a-d;). The structural performance of CFS closed section battened columns is governed by the unsupported chord slenderness, particularly the short and intermediate columns (Dar et al. 2022a;c;e;2021c-e; 2020a-c; EI Aghoury et al. 2010; 2013; Anbarasu & Dar 2020; Rahnavard et al. 2023;2021; Anbarasu 2020;). The lateral connectivity in battened columns is discontinuous and justifies the need to explore the performance of CFS laced columns composed of plain channels as chord members

3. CFS laced built-up columns

Previous research on CFS laced columns has focused on using plain angles as chord members to allow for greater flexibility in adopting the transverse gap in two orthogonal directions, which can lead to higher structural efficiency (Dar et al. 2018;2019a-b;2021a-b). However, these laced columns have often experienced early local buckling during the primary stages of loading, resulting in early failure at a lower axial load. To address this issue, researchers have suggested using plain channels as chords in CFS built-up columns. While studies have shown that built-up columns composed of built-up chords made up of two-lipped channel sections can provide satisfactory performance (Bastos & Batista 2019, the complex fabrication process for such columns limits their adoption in real-world practice. Additionally, built-up columns made up of multiple profile elements can result in complex behavior with weak post-peak performance. A recent study on CFS laced columns composed of plain channel sections as chord members has shown superior axial resistance and buckling performance, but it was limited to low to moderate slenderness. Therefore, there is a need to explore the behavior of CFS laced stub built-up columns composed of two plain channel sections.

In current study, two plain channels were set in a toe-to-toe orientation to obtain a closed-type built-up profile. Lacing plates used as lateral connecting system. Single-lacing format was used, with a one self-drilling screw at each end to fasten the lacing element to the channel chords. The lacing slenderness and the end plate width parameters were varied and their impact on the axial stability and capacity of CFS laced built-up stub columns was investigated. The column performance of the laced systems was analyzed based on the peak column capacity, load-displacement plots and the type of failure exhibited. Finally, the North American Specification and Eurocode for designing CFS members were followed to obtain the design strengths for comparison against the test strengths.

4. Test specimens

Four stub column specimens were prepared to investigate the axial behavior of CFS laced built-up stub columns, as shown in Fig.1. The closed built-up section was formed by arranging two plain channels in a toe-to-toe configuration, as shown in Fig.2, with lacing elements used as lateral connectors. The channel sections were made by press braking a 150mm wide and 2mm thick steel strip, resulting in a web depth of 100 mm and a flange width of 25mm. The chords were spaced transversely at a distance of 75mm between the flange tips, and each channel was 900mm long. Lacing plates with a width of 25mm and two thicknesses, 4 mm and 6 mm, were used, adopting a single-lacing configuration with a lacing inclination of 45° . The specimens had a height of 900 mm, and two different depths of end plates were used: 125 mm and 250 mm. Each end of the lacing plate was connected to the flanges of the chords using a single self-drilling screw (5mm diameter), while the end plates were connected to the chords using the same size of screws in three rows for 125 mm wide end plates and six rows for 250 mm wide ones. To reflect important details, the specimens were labeled with a code, such as "SL-4-125," where "SL" indicates the single-lacing arrangement, "4" represents the lacing thickness (in mm), and "125" denotes the width of the end plate (in mm).

All dimensions are in mm.

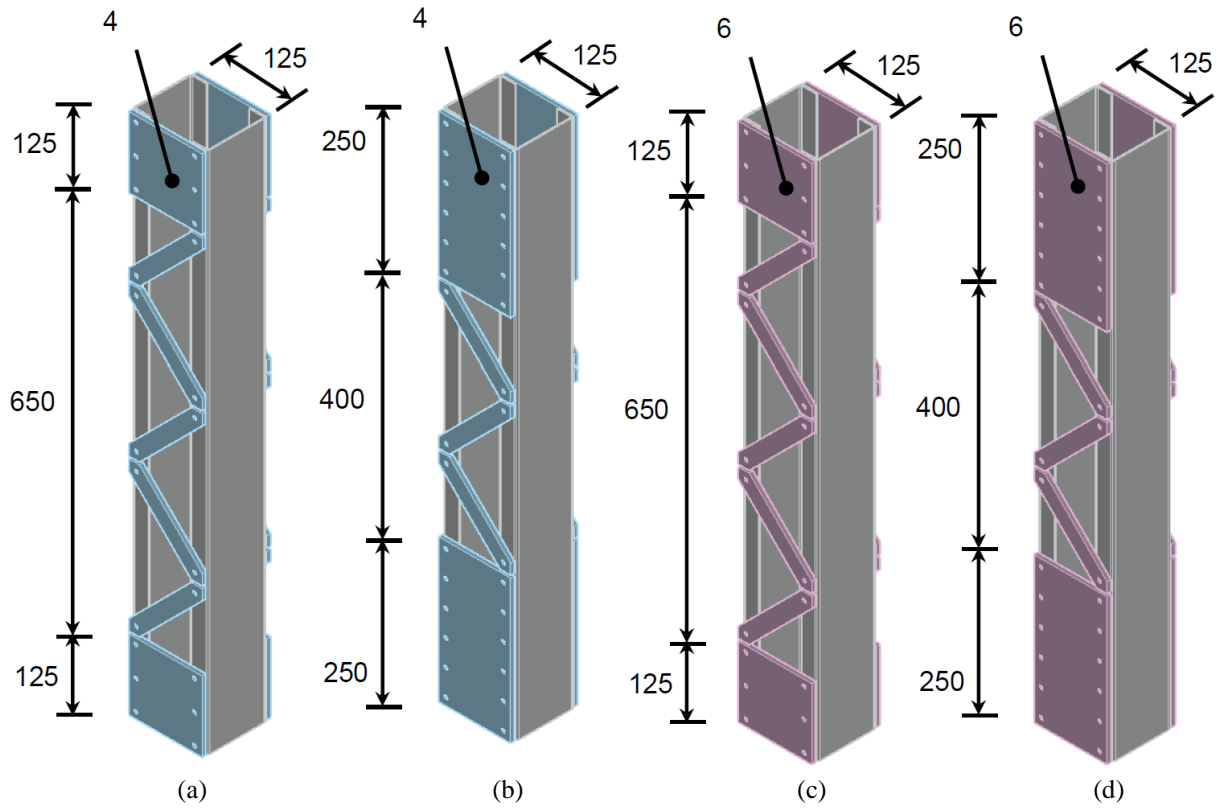


Figure 1: Longitudinal details of the test specimens, (a) SL-4-125; (b) SL-4-250; (c) SL-6-125; and (d) SL-6-250

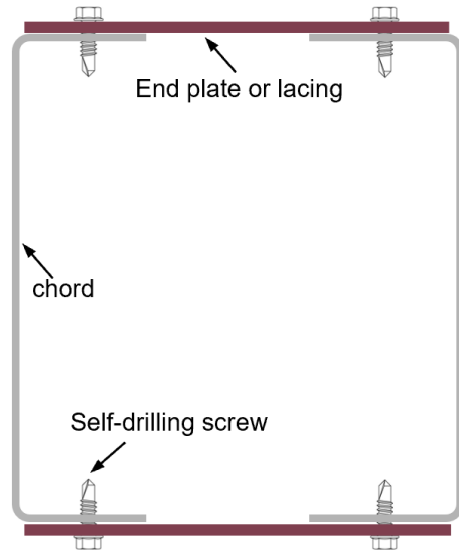


Figure 2: Cross-sectional details of the built-up column specimens

5. Material Properties

To determine the actual material properties of the steel used to form the chord members, tensile tests were carried out on the coupons prepared according to Indian Standards (IS 1608, 2005). The channel sections were used to extract the tensile coupons. For this purpose, an MTS Universal Testing Machine (UTM) was used to perform the tensile tests, and three coupon tests were carried out in total. The recorded values for average yield strength (f_y in MPa), ultimate strength (f_u in MPa), elasticity modulus (E in GPa), and elongation (e in percent) were 423.2, 501.7, 203, and 25.2, respectively. The material tests exhibited a sharp yielding behavior as shown in the stress vs. strain curve, which is presented in Fig.3.

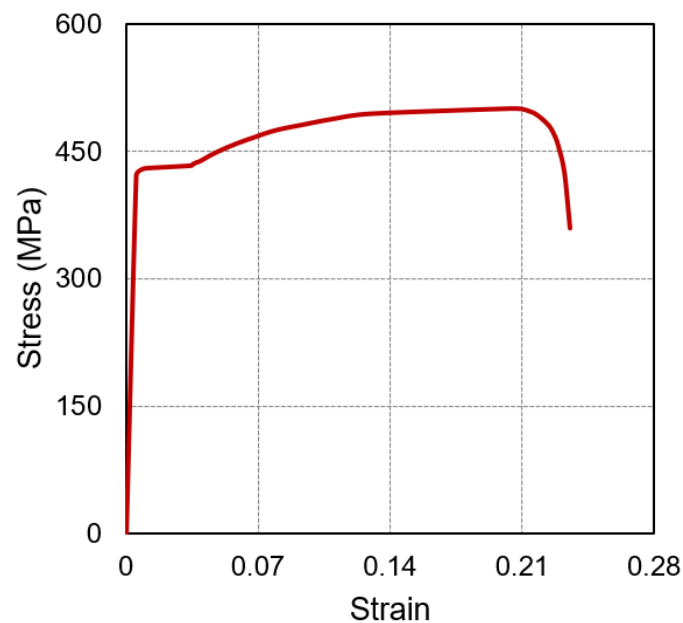


Figure 3: Typical stress-strain plot of the coupons

6. Test Set-up

The axial compression tests were performed on |CFS laced built-up stub column specimens using a loading frame with a capacity of 300 kN, as displayed in Fig.4. The axial compression load was applied using a hydraulic jack with a capacity of 500 kN, and a load cell of the same capacity was used to monitor the axial loading of the specimens. In order to record mid-height lateral displacement and axial shortening, two displacement sensors (LVDTs) were employed. To gather the data, the load cell and LVDTs were connected to a computerized data acquisition system. Further information regarding the test set-up can be found in a study by Dar et al. (2022a).

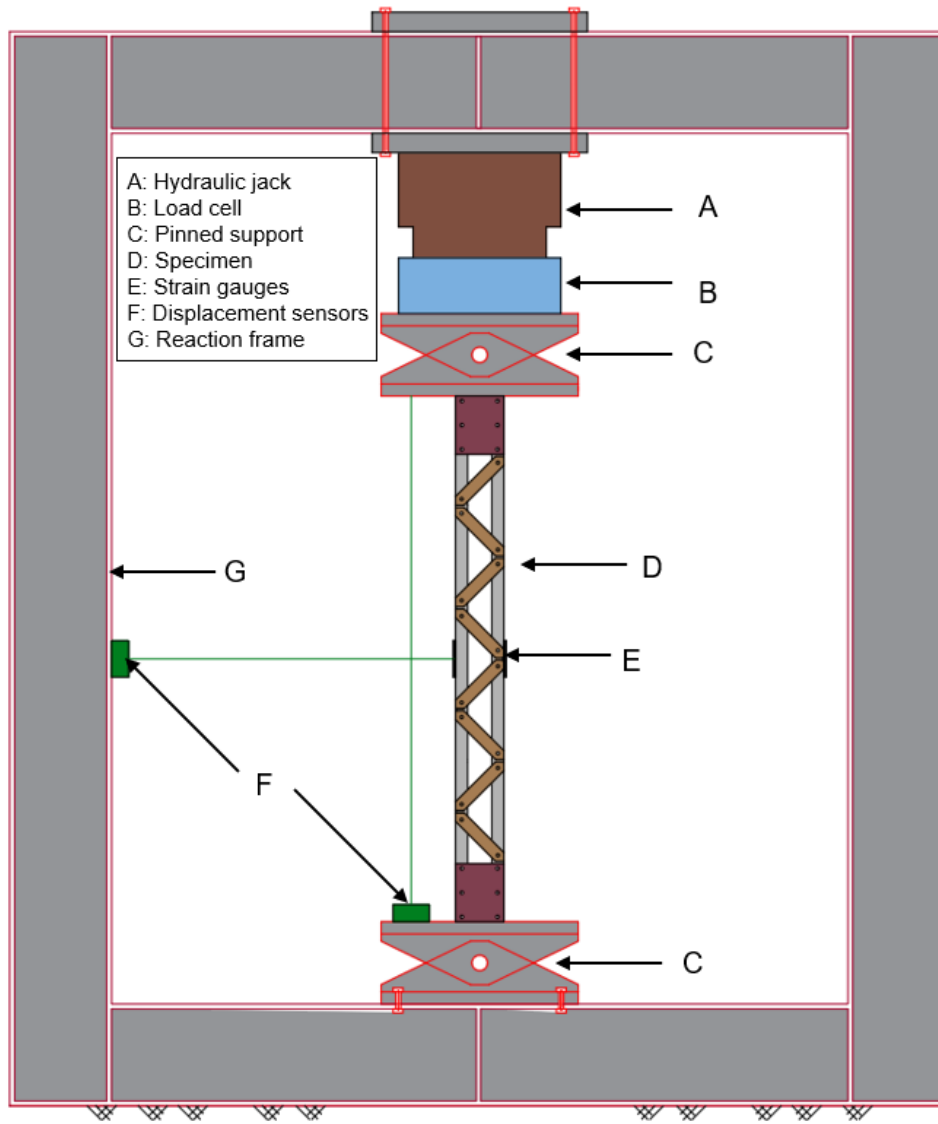


Figure 4: Details of the test set-up

7. Test Results

Fig.5(a-b) compares the axial load vs. axial shortening curves of specimens with same lacing thickness but different end plate widths. All the specimens exhibited a typical stub column trend with a nearly linear rising branch till their peak loads, followed by a sudden fall afterwards. The peak loads carried by SL-4-125 and SL-4-250 were 195.08 kN and 198.10 kN, respectively. A meager capacity improvement of 3.01 kN or 1.5% was achieved. For specimens SL-6-125 and SL-6-250, the peak loads attained were 198.73 kN and 203.78 kN, respectively, resulting in a capacity enhancement of 5.04 kN or 2.53%. In both these case where the end plate width was increased from 125 mm to 250 mm against the lacing thickness of 4 mm and 6 mm, the improvement in the column strength as well as stiffness was very small, as shown in Fig.5(a-b). Fig.5(c-d) draws the axial load vs. axial shortening comparison in specimens with same end plate widths but different lacing thicknesses. The ultimate loads carried by SL-4-125 and SL-6-125 were 195.08 kN and 198.73 kN, respectively. As a result, the capacity increases by 3.65 kN or

1.87%. The maximum loads for SL-4-250 and SL-6-250 were 198.01 kN and 203.78 kN, respectively. A minimal capacity improvement of 5.68 kN or 2.86% was achieved. Again, by varying the lacing slenderness a small increase in the column capacity was achieved. However, in comparison with the end plate width variation, the lacing thickness variation resulted in a relatively larger stiffness improvement, as shown in Fig.5(c-d). Since all the specimens were stub columns, the axial shortening was very small, less than 1.4 mm, as shown in Fig.5(a-d).

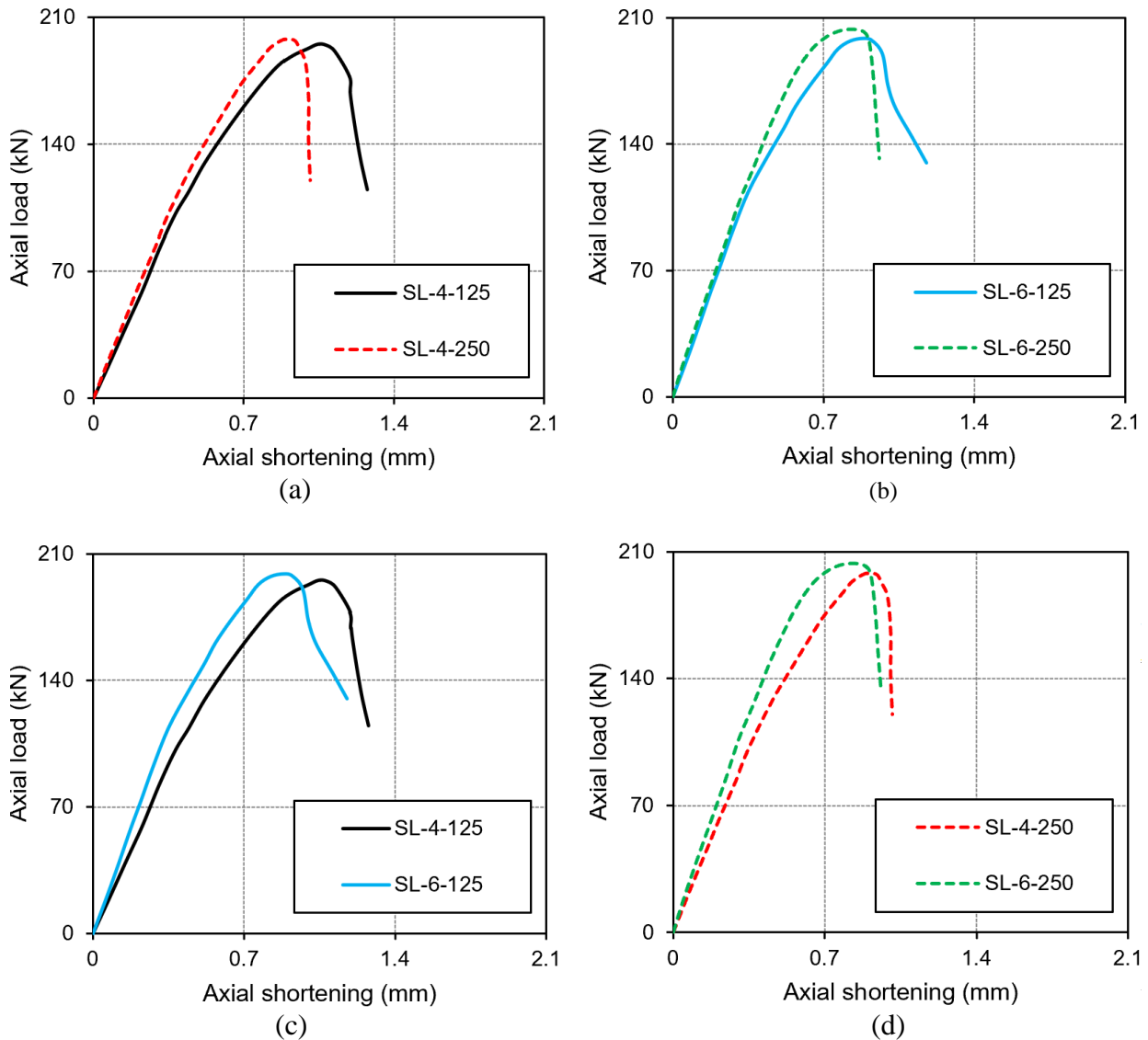
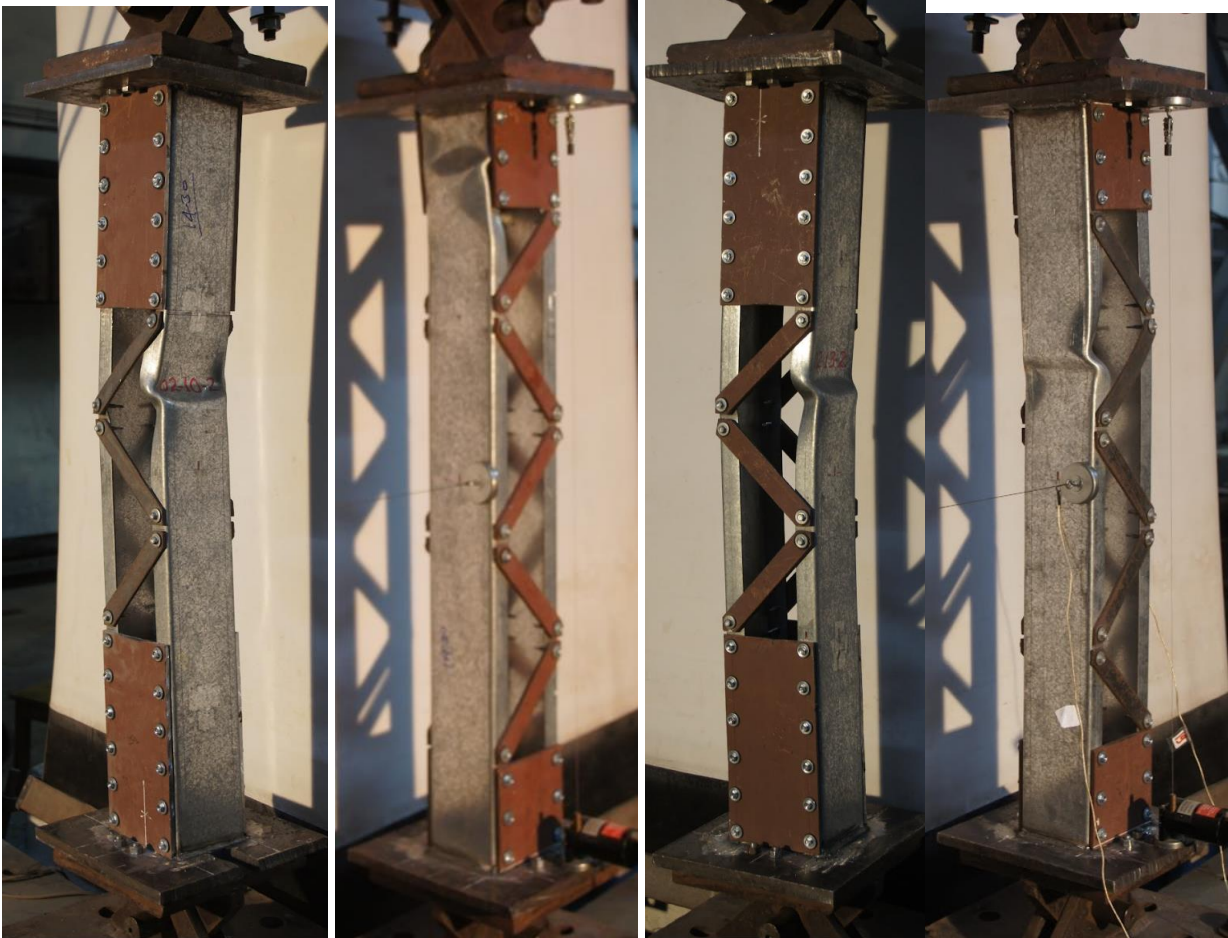


Figure 5: Comparison of axial load vs. axial shortening response, (a) SL-4-125 & SL-4-250; (b) SL-6-125 & SL-6-250; SL-4-125 & SL-6-125; SL-4-250 & SL-6-250.

All the specimens being stub columns failed by local buckling of the chord. Since the slenderness of the columns was very low, the magnitude of local buckling deformations was very large. Despite the chord flange plate element having a free end (on the flange tip side) compared to the web (which has both ends fixed), the local buckling deformation started in the web of the chord. The large plate slenderness in the web element compared to the flange was responsible for

the local buckling initiation there. Upon further loading, the web buckling magnified and the flange element also started to experience local buckling. Finally the specimens collapsed upon the attainment of the peak loads by the respective specimens. Specimens with thinner lacing elements (4 mm) also suffered from lacing buckling due to large lacing slenderness. Specimen SL-6-125 had a stockier lacing element and a smaller end plate. In this case the local buckling on the chord occurred close to the end plate. It must be noted that in none of the specimens experienced any type of connection failure, furthering verifying the adequacy of the screws in terms of size and number adopted.



(a) (b) (c) (d)
Figure 6: Failure modes in the test column specimens, (a) SL-4-125; (b) SL-4-250; (c) SL-6-125; (d) SL-6-250

8. Design strengths

At present there are no guidelines available to predict the axial capacity of laced built-up columns made up of CFS sections. So, the current North American Specification (AISI S100:2020) and European Code EN1993-1-3 (2006), used to design CFS members, were adopted to obtain the design strengths of all the specimens. Both of these standards include fundamental design steps for traditionally used I-type CFS built-up columns which are formed by connecting

two channels in a back-to-back arrangement through the webs. Fig.6 and Table 1 show a comparison of the design strength predictions to their test strengths. It was noted that the North American Specification (AISI S100:2020) predicted the strengths of CFS laced stub columns composed of plain channels conservatively while European Code EN1993-1-3 (2006) predictions were unconservative. However, the deviations shown by the design strengths were small as the ratio of design strength to test strength for North American Specification (AISI S100:2020) was 0.95 with a standard deviation of 0.02. The same for European Code EN1993-1-3 (2006) were 1.02 and 0.02, respectively. Compared to CFS stub columns CFS laced columns with larger slenderness the codal predications show a greater deviation with regards to the test strengths. Further investigations are required across a wide range of parameters to ascertain if the same trend continues or not.

Table 1: Comparison of test results and design strengths

Specimen	P_{Test} (kN)	P_{NAS} (kN)	P_{EC3} (kN)	P_{NAS} / P_{Test}	P_{EC3} / P_{Test}
SL-4-125	195.08	187.95	203.78	0.96	1.04
SL-4-250	198.10	187.95	203.78	0.95	1.03
SL-6-125	198.73	187.95	203.78	0.95	1.03
SL-6-250	203.78	187.95	203.78	0.92	1.00
Average				0.95	1.02
Standard deviation				0.02	0.02

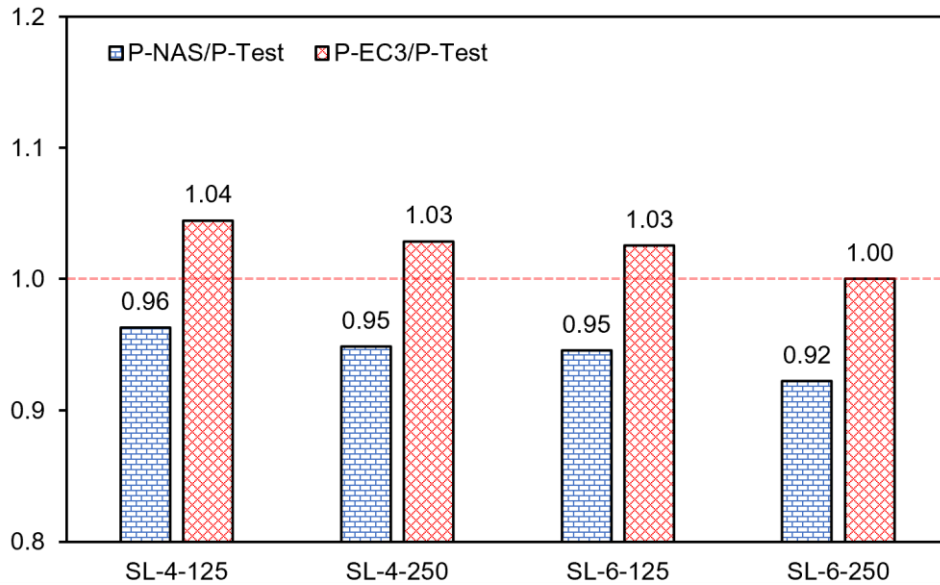


Figure 7: Comparison of test strengths and the predicted strengths

5. Summary and Conclusions

In the present study an experimental investigation on CFS laced built-up stub columns composed of plain channel sections was performed. Pin-ended supports were used for all the specimens. The influence of critical parameters like end plate width and lacing thickness was assessed. The impact of these variations on the built-up columns' axial resistance and stability response was studied. The variation in the structural behavior of the built-up columns was monitored in terms of their peak loads, load-displacement curves and failure modes. All the stub column specimens

failed by prominent local buckling in the chord. Lacing buckling was noted in specimens with slenderer lacing plates. None of the specimens exhibited any type of connection failure even when the chord local buckling was close to the screwed connection. Finally, the North American Specification (AISI S100:2020) and European Code EN1993-1-3 (2006) for CFS structures were used to check their design adequacy. Comparing the design strengths brought out by both these codes against the test strengths reflected inconsistency, although minor in magnitude. The North American Specification (AISI S100:2020) predicted the column strengths conservatively while the European Standards EN1993-1-3 (2006) predicted unconservatively.

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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