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Testing of CFS double-laced built-up columns: Stability and design considerations

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Abstract
Cold-formed steel (CFS) members are becoming more popular in building construction owing to their unique features that provide lightweight, cost-efficiency, and fast assembly, ideal for modular construction. As the primary structural members, columns must develop sufficient axial strength and stiffness for adequate functioning. In a typical CFS framing system (used for low-rise structures), built-up columns comprising of two web-fastened channel sections oriented in the back-to-back arrangement (I-type), are used as main compression members. This approach for constructing built-up columns is quite convenient, especially in terms of fabrication simplicity, making it widely adoptable. However, the inefficient sectional configuration in I-type built-up columns restricts the load-carrying capacity that the channels (column chords) can develop if a suitable transverse gap is provided between the same. Previous studies on gapped CFS built-up columns demonstrated that they outperform the traditional I-type built-up columns in terms of axial capacity and stability considerations. Most of the past research on transversely gapped built-up columns has mostly focused on battened columns, and limited research on CFS laced columns has been reported. The limited findings indicated that plain angle sections should not be preferably used as chord members as they are more vulnerable to early local buckling during the initial stages of loading, which results in an early failure at a lower load. Also, the adoption of plain channel sections over plain angle sections as chords in CFS laced and battened built-up columns have displayed a superior performance. However, only single and N-type lacing configurations have been reported. This justifies the need to study further CFS double-laced columns composed of plain channel sections, 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 adopted as lateral connectors. The transverse spacing between the chords and the lacing slenderness were varied. The effect of these variations on the axial strength and stability characteristics of CFS double-laced columns was assessed, with particular consideration to the lacing and unsupported chord stability characteristics. Lastly, the North American Specification and Eurocode for CFS structures were used to determine the design strengths for comparison. The predictions made by both these standards showed inconsistencies in their accuracy.

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1. Introduction

The adoption of cold-formed steel (CFS) members in the construction of mid-rise residential and commercial structures have grown extensively in the recent decade. Desirable structural features like lightweight members, faster construction, easy fabrication and assembly are the primary reasons behind this development. These features become more beneficial when the construction site is remote and transportation/handling structural members becomes convenient. It also promotes the completion of building assignments on time, providing it an advantage over building materials because it only requires assembling and connecting different structural elements to finish the project. Although CFS sections have several major advantages that encourage their use in structural construction, the thin-walled nature of CFS sections still limits their widescale adoption due to the local buckling instabilities experienced by various thin cross-sectional plate elements (Yu 2010; Zeimian 2010). This has prompted researchers, particularly those working on structural steel, to do research in this field and devise an appropriate way to handle these uncertainty problems. As a result, many research investigations have produced effective, reliable, and cost-effective solutions for improving the buckling performance of various types of modified cross-sections. (Zhou et al. 2022; 2021a-b; Nie et al. 2020a-b; Selvaraj & Madhavan 2022a; 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 traditional CFS built-up column (I-type) is constructed by fastening two back-to-back channel sections through their webs at standard longitudinal spacing. Such type of built-up columns are widely used in building CFS framing systems, primarily due to their simple fabrication process. Many studies were carried out to improve the buckling strength of such I-type built-up columns by recommending limits to the flat width-to-thickness ratios for the various cross-sectional elements and by specifying appropriate patterns for the screwed connections at various locations (Selvaraj & Madhavan 2022b;2021; Mahar et al. 2023;2021a;b; Mahar and Jayachandran 2021; Kechidi et al. 2020;Roy et al. 2018; Fratamico et al. 2018a-b). Incorporating an appropriate transverse gap between the channels can significantly improve the structural performance of these built-up columns (Subramanian 2016). Several lateral connecting systems can be adopted to provide adequate structural 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; 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;2022a-d; Roy et al. 2019). From the limited studies on CFS closed section battened columns (EI Aghoury et al. 2010; 2013; Dar et al. 2022a;c;e;2021c-e; 2020a-c; Anbarasu & Dar 2020; Anbarasu 2020; Rahnavard et al. 2023;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 and justifies the need to explore the performance of CFS laced columns composed of plain channels as chords members.

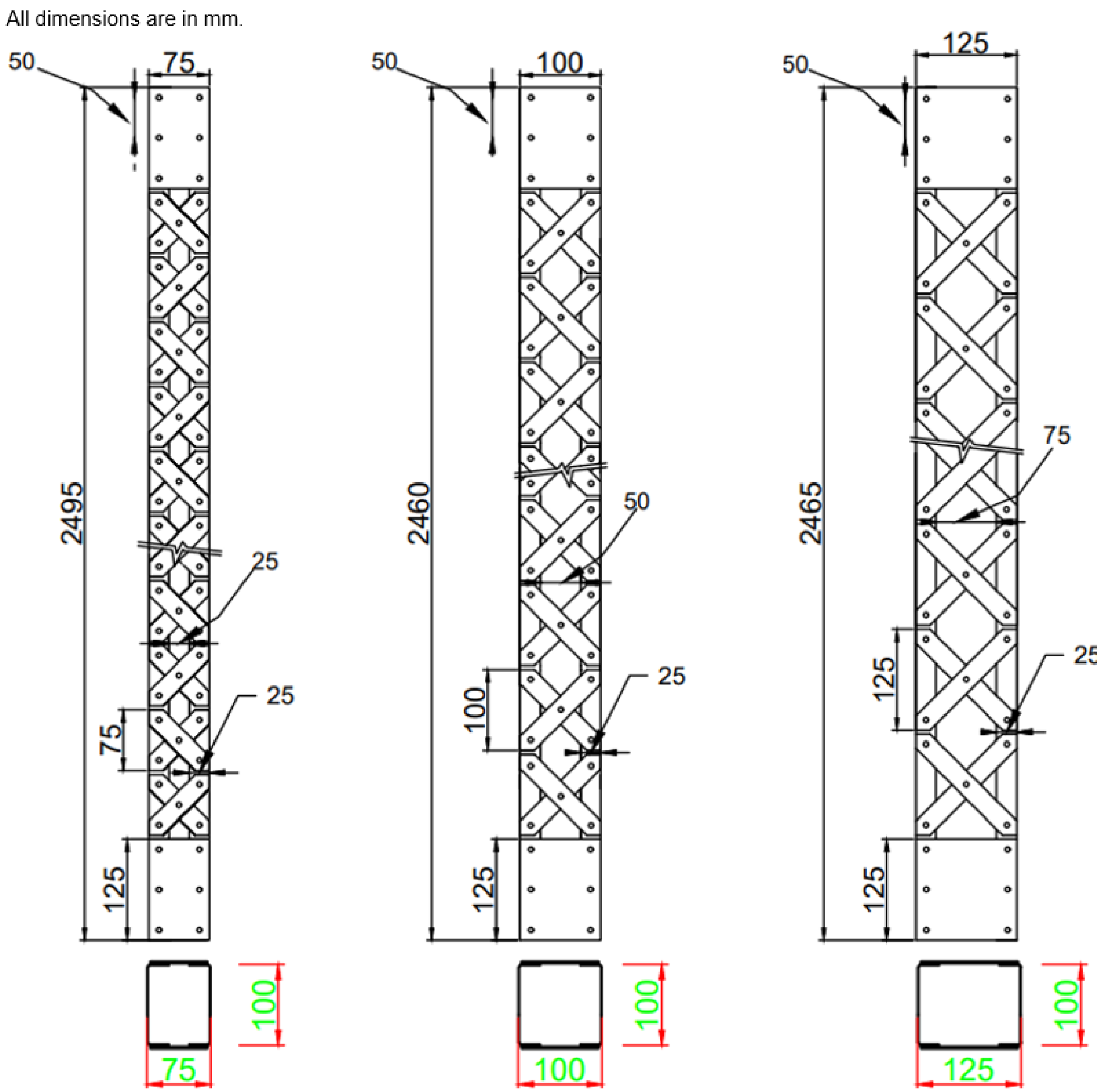
3. CFS laced built-up columns

A significant part of the previous research on CFS laced columns focused on plain angles being used as chord members to facilitate the flexibility of adopting the transverse gap in two orthogonal directions for higher structural efficiency (Dar et al. 2018;2019a-b;2021a-b). 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). Though, 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. A recent study on CFS laced columns composed of plain channel sections as chord members indicated a superior axial resistance and buckling performance, but was limited to single and N-type lacing configurations only (Dar et al. 2022a). This justifies the need for exploring the behaviour of CFS double-laced built-up columns composed of two plain channel sections.

In this experimental investigation, two plain channels were arranged in a toe-to-toe configuration to form a closed built-up section, with lacing plates used as lateral connectors. Double-lacing configuration was adopted, with a single self-drilling screw was used to fasten each end of the lacing plate to the chords, and one screw to connect the two lacing plates at the intersection. The transverse spacing of the chords and the lacing slenderness was varied. The effect of these variations on the axial strength and stability characteristics of CFS double-laced columns was assessed, with particular 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. The predictions made by both these standards showed inconsistencies in their accuracy.

4. Test specimens

A total of six specimens (see Fig.1(a-c)) were fabricated to study the axial behavior of CFS double-laced columns. Two plain channel sections (used as chord members) were arranged in toe-to-toe configuration to form a closed built-up section (as shown in Fig.2). Lacing elements were 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, i.e., 25mm, 50mm and 75mm were adopted, as shown in Fig.1. The width of each lacing plate was 25mm and was constant in all the specimens. Steel plates of two thicknesses i.e., 1.7mm and 2.5mm were used to prepare the lacing elements. The lacing plates were adopted on the double-lacing configuration with a 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 lacing plates were also connected at their intersection using a screw of same size. 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 "DL-25-2.5", DL reflects the double-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) DL-25-1.7 & DL-25-2.5; (b) DL-50-1.7 & DL-50-2.5; DL-75-1.7 & DL-75-2.5.

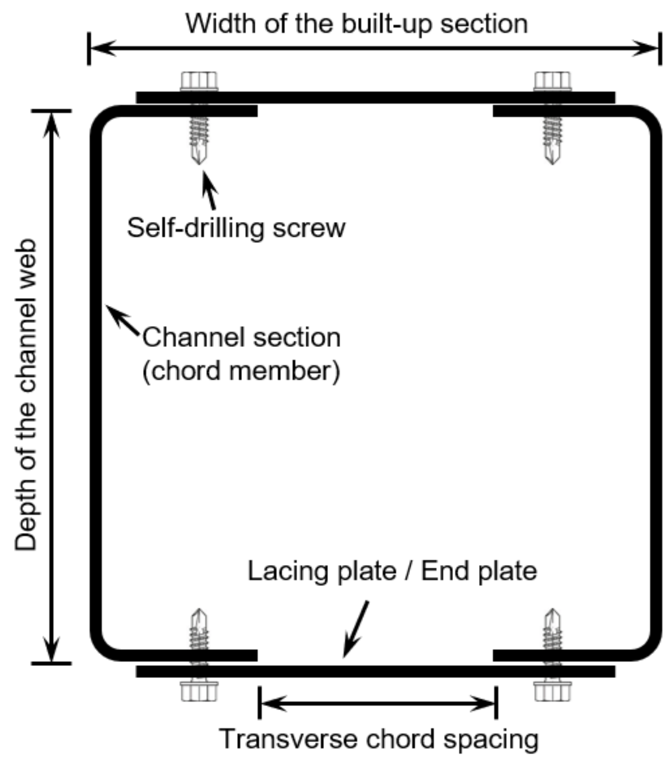


Figure 2: Cross-sectional details of the built-up column specimens

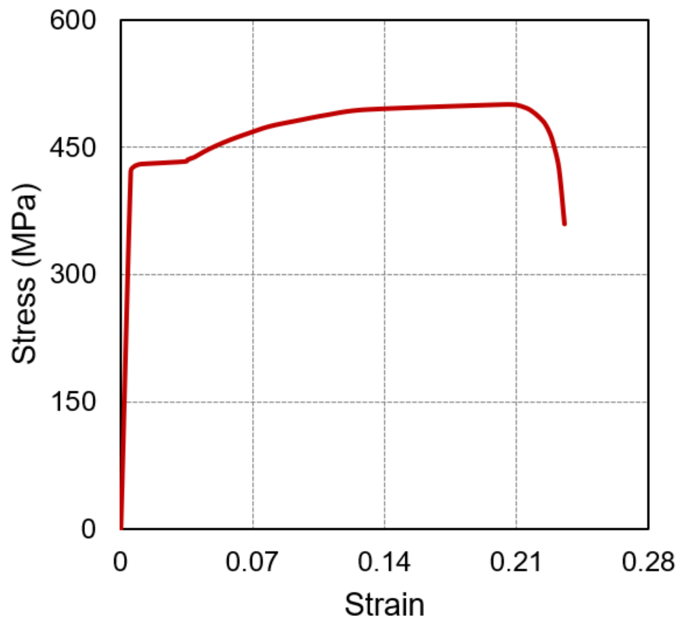


Figure 3: Typical stress-strain plot of the coupons

5. Material Properties

Tensile tests were performed on the coupons prepared according to the Indian Standards (IS 1608, 2005) to determine the actual material properties of the steel used to form the chord members. The tensile coupons were extracted from the channel sections. An MTS universal testing machine (UTM) was used to perform these tensile tests. In total three coupon tests were performed. 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.

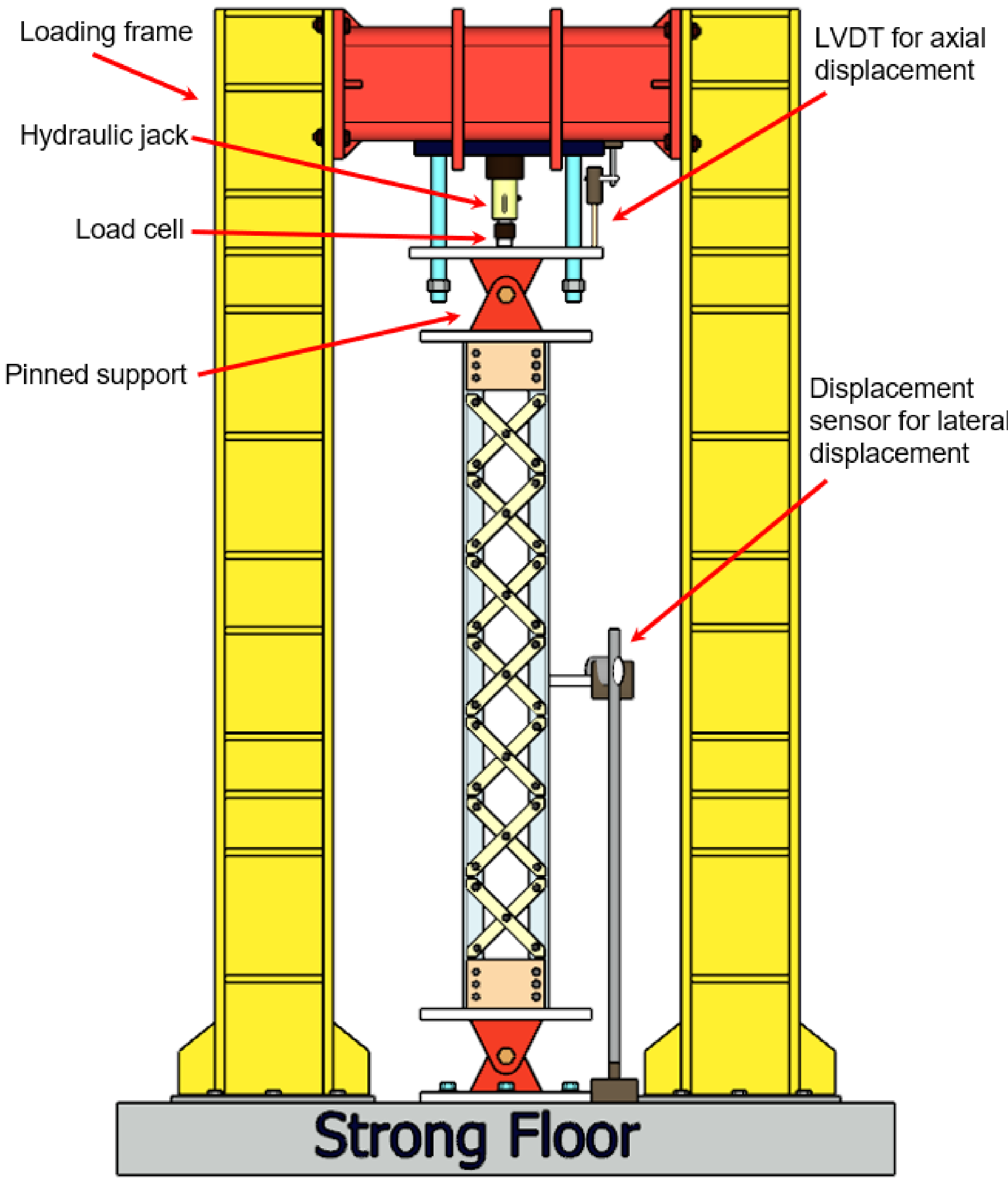


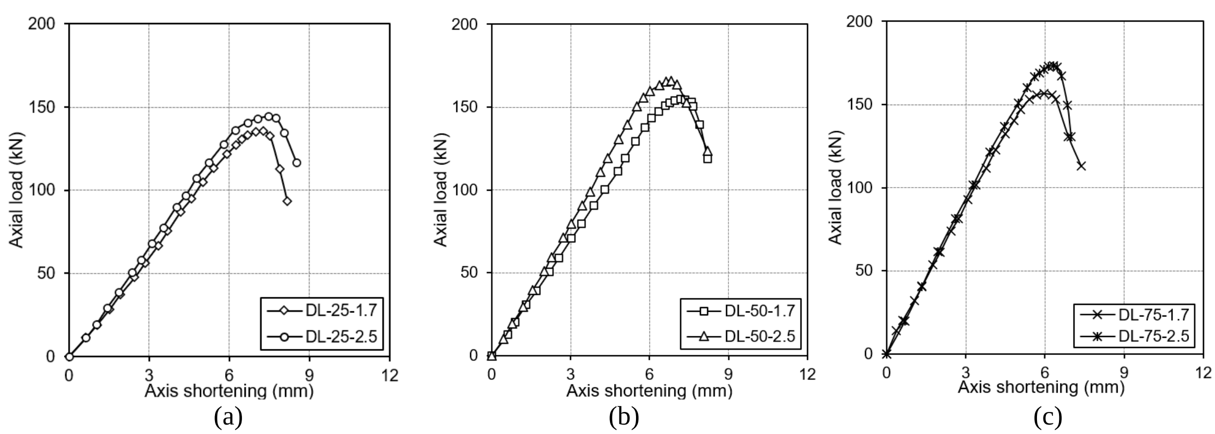
Figure 4: Details of the test set-up

6. Test Set-up

A 300 kN strong loading frame was used to perform the axial compression tests on the double-laced built-up column specimens, as shown in Fig. 4. A 500 kN hydraulic jack was used to apply the axial compression load. A load cell of the same capacity was used to monitor the axial loading during the testing of specimens. Two displacement sensors (LVDTs) were adopted to record the axial shortening and mid-height lateral displacement. The load cell and the LVDTs were connected to a computerized data acquisition system.The other details pertaining to the test set-up can be found elsewhere (Dar et al.2022a).

7. Test Results

Fig.5(a-c) presents the axial load *vs.* axial shortening characteristics of all the specimens. DL-25-1.7 and DL-25-2.5 exhibit a nearly similar load-displacement trend, as shown in Fig.5(a). The peak axial resistance developed by DL-25-1.7 was axial 135.38 kN with a corresponding axial shortening of 7.27 mm. The same for specimen DL-25-2.5 was 144.33 kN and 7.47 mm respectively, in the same order. It is evident that an axial strength improvement of 8.95 kN (6.61%) was noted by changing the lacing thickness from 1.7 mm to 2.5 mm. However, a small variation in the axial stiffness was observed due to this variation. The axial load *vs.* axial shortening plots of specimens DL-50-1.7 and DL-50-2.5 look identical, as shown in Fig.5(b). The ultimate axial strength produced by DL-50-1.7 was axial 154.64 kN with a corresponding axial shortening of 7.18 mm. The same for specimen DL-50-2.5 was 165.92 kN and 6.82 mm respectivelly, in the same order. In this case an axial strength improvement of 11.27 kN (7. 29%) was observed due to the same variation in the lacing thickness (1.7 mm to 2.5 mm). However, in this case the variation in the axial stiffness was higher than the previous one. The axial load *vs.* axial shortening curves of specimens DL-75-1.7 and DL-75-2.5 are identical, as shown in Fig.5(c). The ultimate axial strength produced by DL-75-1.7 was 156.72 kN with a corresponding axial shortening of 5.95 mm. The same for specimen DL-75-2.5 was 173 kN and 6.33 mm respectively, in the same order. It can be noted that the axial strength improvement of 16.54 kN (10.56%) was observed due to the lacing thickness variation. Again, a small variation



(a) (b) (c)

Figure 5: axial load *vs.* axial shortening behaviour, (a) DL-25-1.5 & DL-25-2.5; (b) DL-50-1.7 & DL-50-2.5; DL-75-1.7 & DL-75-2.5.

in the axial stiffness was noted in this case. The post-peak response of all the six specimens was identical, i.e., a sudden drop after the peak load was attained.

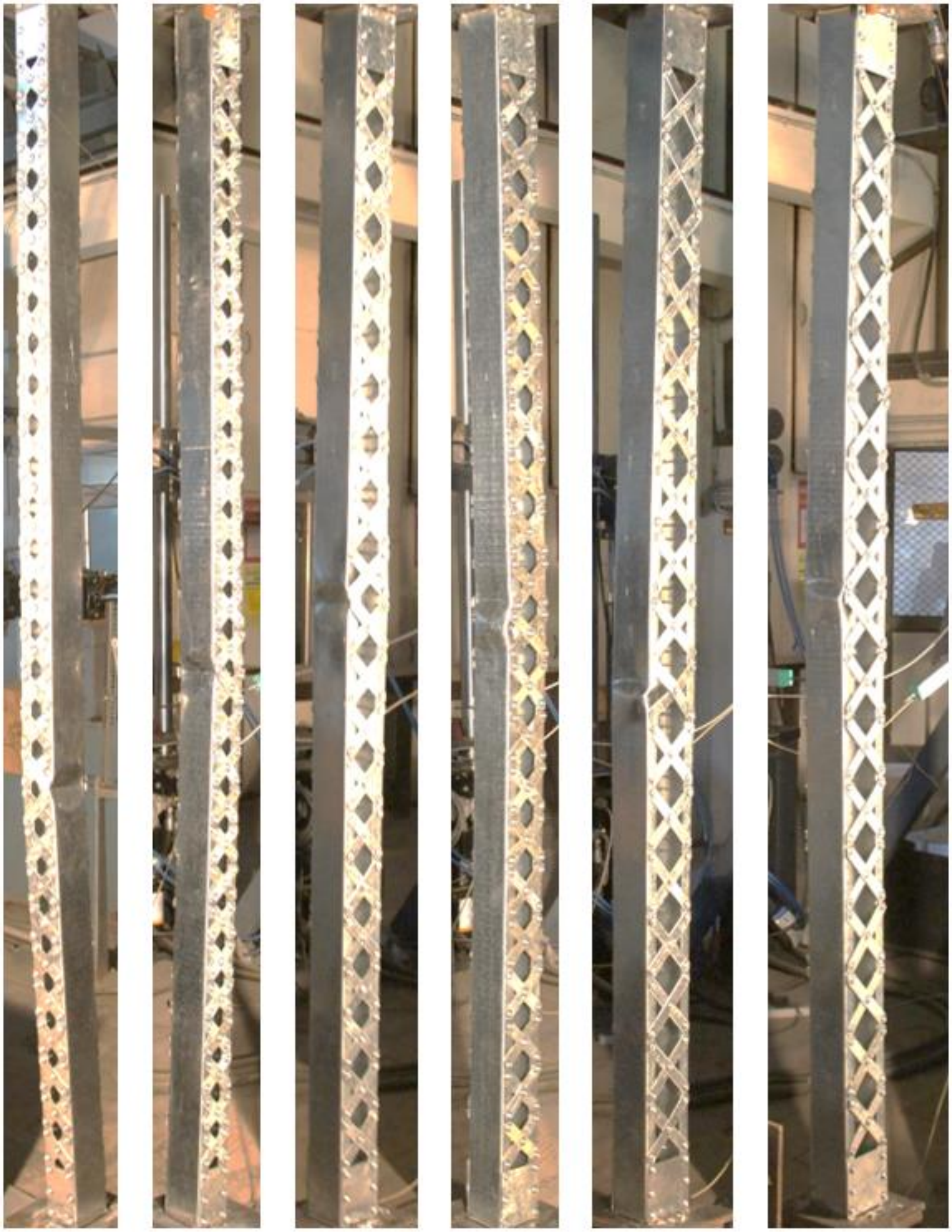
All the six specimens failed by local buckling of the chord, located nearly at the mid-height of the columns, as shown in Fig.6(a-f). In the chord, the web element mostly experienced the local buckling instability. The fundamental cause of this behaviour is the web's vulnerability to local buckling due to its high sectional slenderness compared to the flange. The lacing slenderness neither altered the failure mode nor its location along the column height. Besides, the variation in the transverse spacing also did not influence the failure mode in the various specimens. Furthermore, none of the specimens suffered from any type of connection failure, clearly indicating that the self-drilling screws adopted sufficed. It was noted 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.

8. Design strengths

Currently, no guidelines are available in any design code for predicting the axial strengths of CFS laced columns composed of channel sections. Therefore, the North American Specification (AISI S100:2020) and European Code EN1993-1-3 (2006), brought out 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 columns made by fastening two channel sections in a back-to-back configuration, through the webs at standard longitudinal spacing. Fig.7 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 Code 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 and the lacing slenderness was high. The degree of unconservativeness decreased with the increase in the overall slenderness of the laced columns. The indiscrpancies observed between the design predicted strength and the test strengths call 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 double-laced columns composed of plain channel sections.

Table 1: Comparison of test results and design strengths

Specimen	P_{Test} (kN)	P_{NAS} (kN)	P_{EC3} (kN)
DL-25-1.7	135.38	142.98	139.32
DL-25-2.5	144.33	142.98	139.32
DL-50-1.7	154.64	170.62	168.46
DL-50-2.5	165.92	170.62	168.46
DL-75-1.7	156.72	182.45	182.06
DL-75-2.5	173.26	182.45	182.06



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

Figure 6: Failure in the test specimens, (a) DL-25-1.7; (b) DL-25-2.5; (c) DL-50-1.7; (d) DL-50-2.5; (e) DL-75-1.7, and (f) DL-75-2.5.

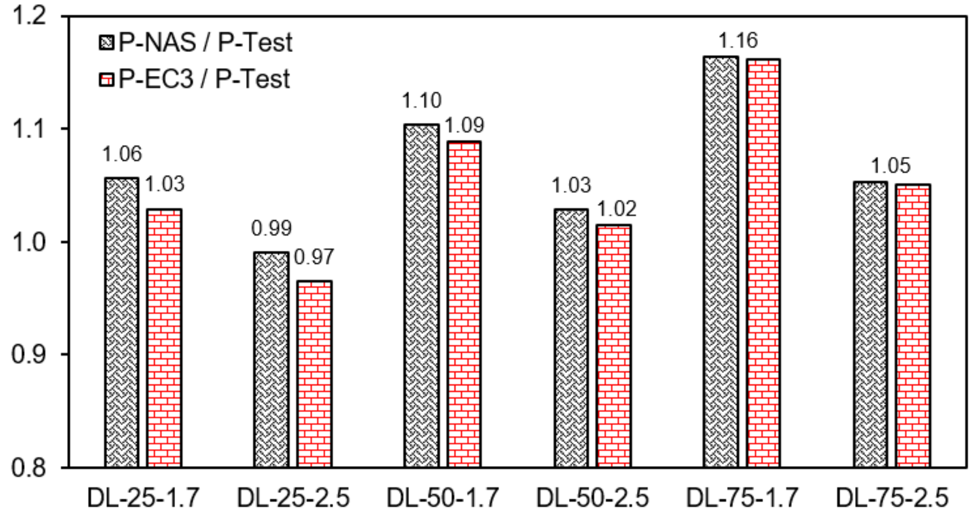


Figure 7: Comparison of test strengths and the predicted strengths

5. Summary and Conclusions

The current study presented an experimental investigation on CFS double-laced columns composed of plain channel sections. Pin-ended supports were used for all the specimens. The influence of critical parameters like transverse spacing between the chords 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. 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. Both the North American Specification (AISI S100:2020) and European Standards EN1993-1-3 (2006) mostly over-predicted the strengths of CFS double-laced columns composed of plain channels, particularly when the overall slenderness of the built-up columns was low and the lacing slenderness was high. The degree of unconservativeness decreased 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 6.61%-10.56%.

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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