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# Compression capacity of short cold-formed steel built-up columns with double-lacing configuration and low sectional compactness

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**Abstract**
Built-up columns composed of multiple cold-formed steel (CFS) sections are often used as structural members to carry the high axial force demand as they are considered as economical and efficient. Though built-up CFS columns with intermediate lacing plates are widely adopted in the practice, very limited studies have been conducted in the past to investigate their performance under various loading conditions. This study presents an experimental investigation on the built-up CFS column specimens with double-lacing configuration and large flat width-to-thickness ratio. Pin-ended support conditions allowing uniaxial bending are adopted in this study. The main parameters varied in this study are the sectional compactness of chord sections, the column slenderness ratio, and the slenderness ratio of lacing elements. The slenderness ratio of the unbraced chord elements is kept constant in all test specimens. The design strengths of built-up CFS columns are computed using North American and Eurocode specifications for CFS structures for the comparison the test results. The design strength predictions of both these standards are found to be conservative.

## 1. Introduction

Cold-formed steel (CFS) members have been used in a wide range of structural and non-structural applications in the construction industries (Hu et al. 2011, Craveiro et al. 2014, Gilbert et al. 2014, Nassif et al. 2014, Koshy and Jayachandran 2018). CFS members have several advantages, such as, high strength-to-weight ratio, easy transportation/handling, faster cost-effective construction, etc. (Landesmann et al. 2016, Javed et al. 2017, Lawson 1992). Advanced cold-rolling processes to produce structural shapes of high-precision and low geometric imperfection have further promoted the scope of utilization of CFS members in the modern constructions (Yu 2010, Sharafi 2018). The main limitation of these members is their proneness to premature buckling instabilities due to thin-walled sections. However, proper design and detailing of CFS sections help in delaying/eliminating the buckling instability, thus substantially enhancing their strength and stiffness characteristics (Dar et al. 2015). A well-designed CFS member can reach its plastic moment capacity (Kumar and Sahoo 2016). Partial-stiffened or

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timber composite CFS sections also improves their load carrying capacity (Dar et al. 2019, 2018a). However, further studies are required to increase the applications of CFS members as structural components in the construction industry.

## 2. Built-up CFS members

Built-up members are often used as columns to satisfy the axial load demand in multi-story structures. By minor adjustments of the spacing between the individual components, built-up columns of high load-resisting capacity can be developed at less cost (Subramanian 2016, Lorkowski & Gosowski 2018, Kalochairetis & Gantes 2011, Kalochairetis et al. 2014). However, there is a lack of extensive research on built-up CFS sections under compression loading, particularly closed built-up sections. The main focus of the past research has been dominant on the performance evaluation of built-up CFS columns connected with batten plates. EI-Aghoury et al. (2010, 2013) conducted tests to study the performance of pin-ended CFS battened columns under concentric and eccentric loading conditions. Overall column slenderness and flat width-to-thickness ratio of the main chord elements were varied. The results revealed that the flat width-to-thickness ratio of the main chord element, the overall slenderness ratio of column and overall slenderness ratio of angle were the main parameters that affect the ultimate strength of the CFS battened built-up columns. Furthermore, torsional and flexural buckling interaction have found to significantly reduce the column strength. Anbarasu et al. (2015) investigated the behavior of CFS web stiffened built-up battened columns. The overall slenderness ratios of the built-up columns were varied in the range of 30 to 120. When the effect of batten plate depth and the number of batten plates used was neglected, the DSM design strengths obtained were found to be conservative. This trend increased with increase in slenderness ratio. When elastic buckling analysis and modified slenderness approach of the NAS (AISI S-100, 2016) Specification was used to calculate the critical elastic column buckling load for the built-up columns, un-conservative design strengths were obtained. By increasing the overall slenderness of the built-up columns, the ultimate strength dropped. Intermediate stiffening of the web helped in eliminating its local buckling completely. Fratamico et al. (2018) conducted test to explore the global buckling and collapse behavior of built-up CFS compression members. The development of composite action was observed through web screw connections, in the occurrence of prominent flexural buckling.

Column end rigidities further reinforces the composite action developed, and significantly enhances the axial resistance of these columns. Roy et al. (2018) performed tests to study the performance of back-to-back gapped built-up CFS columns under axial loading. It was observed that the increase in the vertical spacing between the link-channels reduces the load carrying capacity of the built-up columns. This trend was higher in intermediate and slender columns compared to that of in stub and short columns. Dar et al. (2018b-2018d) conducted a detailed numerical study to explore the performance of closed battened CFS columns comprising of two plain channels with pin ended support conditions. On increasing the toe to toe channel spacing, the load carrying capacity of the built-up columns enhanced. This improvement was more dominant towards slender columns. Mixed buckling modes comprising of flexural bucking and local buckling was observed depending upon the column slenderness. However, the component of local buckling mode was higher in short columns and was observed in terms of flange buckling.

Past studies on CFS built-up columns has witness limited research on columns with lacing connections. Dar et al. (2017, 2018e-2018h) investigated the behavior of CFS laced built-up columns composed of four unstiffened angle sections. Single lacing configuration with pin ended support conditions were considered. The axial capacity of the built-up columns was significantly affected by the lacing slenderness as well as the sectional compactness.

This study presents an experimental investigation has been conducted on short built-up CFS column specimens with double-lacing configuration and large flat to width thickness ratio. Pin ended support conditions allowing uniaxial bending were adopted in this study. The main parameters varied in this study are, sectional compactness of the angle sections, column slenderness ratio and slenderness ratio of lacing elements. The slenderness of the unbraced chord element was kept the same. Furthermore, the design strengths of the built-up columns were computed using North American Specification (AISI S-100, 2016) and Eurocode (EN1993-1-3 2006) for CFS structures for comparison with the test results.

## 3. Experimental Study

The details of the test specimens, material properties, test set-up and test results are given in the following sections.

### *3.1 Test Specimens*

Double-lacing arrangement was adopted to connect the four CFS angles constituting the built-up columns. Angle sections of two different sizes viz. 60×60×1.6 mm, and 100×100×1.6 mm were used to construct the built-up columns. The width of the lacing bars was kept constant at 25 mm. However, its thickness was varied, with 2.5 mm in Model-A and Model-B (angle sections used, 60×60×1.6 mm), and 4mm thick used in Model-C and Model-D (angle sections used, 100×100×1.6 mm). The size of the bolt used to connect the lacing bars to the CFS angle section was 6mm. The cross-sectional details of the test specimens are shown in Fig. 1 and Table 1. To measure the geometric imperfections in all test models, a digital micro-meter with least count 0.01 mm was used. The average value of ratio of overall width/local imperfection in test specimens was computed in the range of 200-600, whereas the ratio of length/global imperfections was in the range of 700-1500.

Table 1: Dimensional details of test specimens

| Specimen | $a$ (mm) | $b$ (mm) | $r$ (mm) | $l$ (mm) | $l_{chord}$ (mm) | $\lambda$ | $b/t$ | $b_e$ (mm) | $L_e$ (mm) | $\lambda_{lacing}$ | Section classification |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Model-A | 30 | 60 | 2.4 | 2325 | 152 | 37 | 37.5 | 75 | 150 | 82.6 | Short |
| Model-B | 30 | 60 | 2.4 | 2025 | 152 | 32.2 | 37.5 | 235 | 150 | 82.6 | Short |
| Model-C | 50 | 100 | 2.4 | 2195 | 245 | 21 | 62.5 | 75 | 250 | 68.8 | Short |
| Model-D | 50 | 100 | 2.4 | 1780 | 245 | 17 | 62.5 | 250 | 250 | 68.8 | Short |

### *3.2 Material Properties*

Three coupon tests were conducted to determine the actual material properties of the steel used in the test specimens. The coupons conforming to Indian Standards (IS 1608, 2005) were prepared for the testing. A computerized universal testing machine was used for conducting the coupon tests. The various details of the coupon tests are given in Table 2. The tensile stress-strain curves of the coupons are shown in Fig. 2.

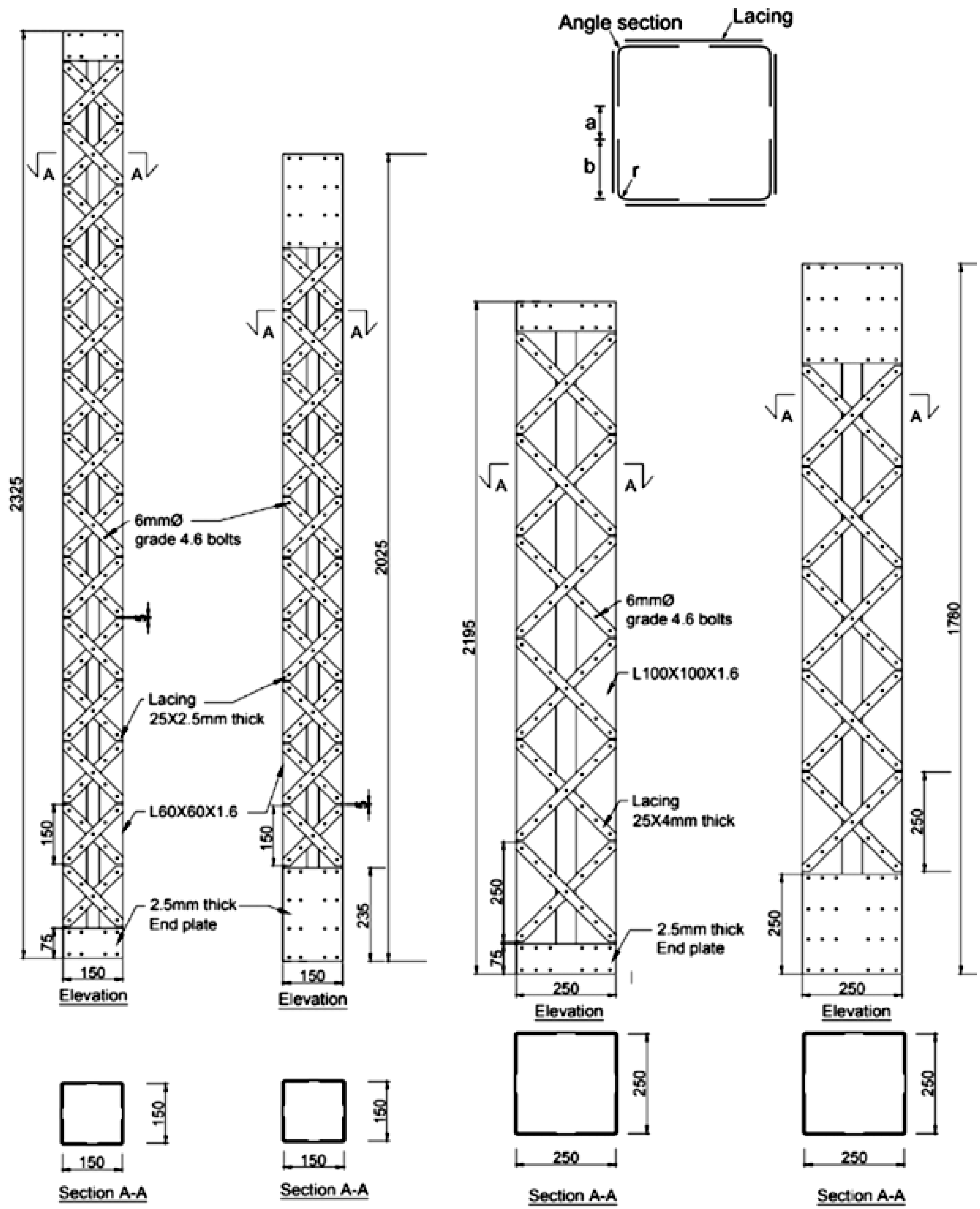



Figure 1: Details of the test specimens

Table 2: Material properties obtained from the coupon tests

| *Coupon* | $f_n$ *(MPa)* | *E (MPa)* | $f_y$ *(MPa)* | $f_u$ *(MPa)* | $\varepsilon$ *(%)* |
|---|---|---|---|---|---|
| Coupon-1 | 350 | 208 | 477.9 | 553.2 | 27.7 |
| Coupon -2 | 350 | 211 | 475.3 | 552.0 | 21.4 |
| Coupon -3 | 350 | 209 | 480.9 | 563.2 | 26.2 |
| *Average* | *350* | *210* | *478.0* | *556.1* | *25.1* |

### *3.3 Test Set-up*

A loading frame of 1000 kN capacity was used to carry out axial testing of CFS built-up columns under concentric loading. The compressive loading on the test specimens was applied using a hydraulic loading jack of 250 kN capacity. The ends of the specimens were connected to the hinge mechanism through an end bearing plate of 25mm thickness. Prior to testing, the hinges were lubricated to minimize the friction. The upper hinge was connected to the hydraulic jack and the lower hinge to the strong floor as shown in Fig. 3. A load cell was used to measure the axial resistance of the test models. A Linear Variable Displacement Transducer (LVDT) was used for measuring the axial shortening in the test models. Two displacement sensors were used to measure the lateral displacement in the two orthogonal directions.

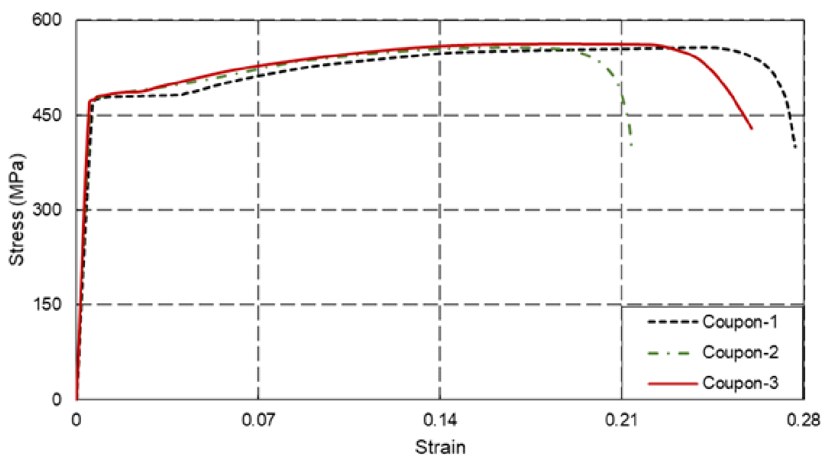



Figure 2: Stress *vs.* strain behavior of the coupons

### *3.4 Test Results*

Fig. 4 shows the load *vs.* displacement response of various models tested. Model-A and Model-B resisted an ultimate axial load of 129.71 kN 113.46 kN respectively, at an axial shortening of 4.1mm and 2.9mm respectively, as shown in Fig. 4(a). The primary reason for the later model to offer lesser axial resistance compared to the former model was the higher magnitude of geometric imperfections. Model-C and Model-D resisted an ultimate axial load of 154.8 kN and 154.9 kN respectively, at an axial shortening of 3.71 mm and 6.1 mm, respectively. All the models displayed higher initial lateral stiffness until their ultimate loads, except Model-B. Model-B developed high lateral stiffness until a load of 60kN. At this stage, the local buckling instability was experienced by one of its chord members, resulting in the drop of its lateral stiffness, as shown in Fig. 4(b).

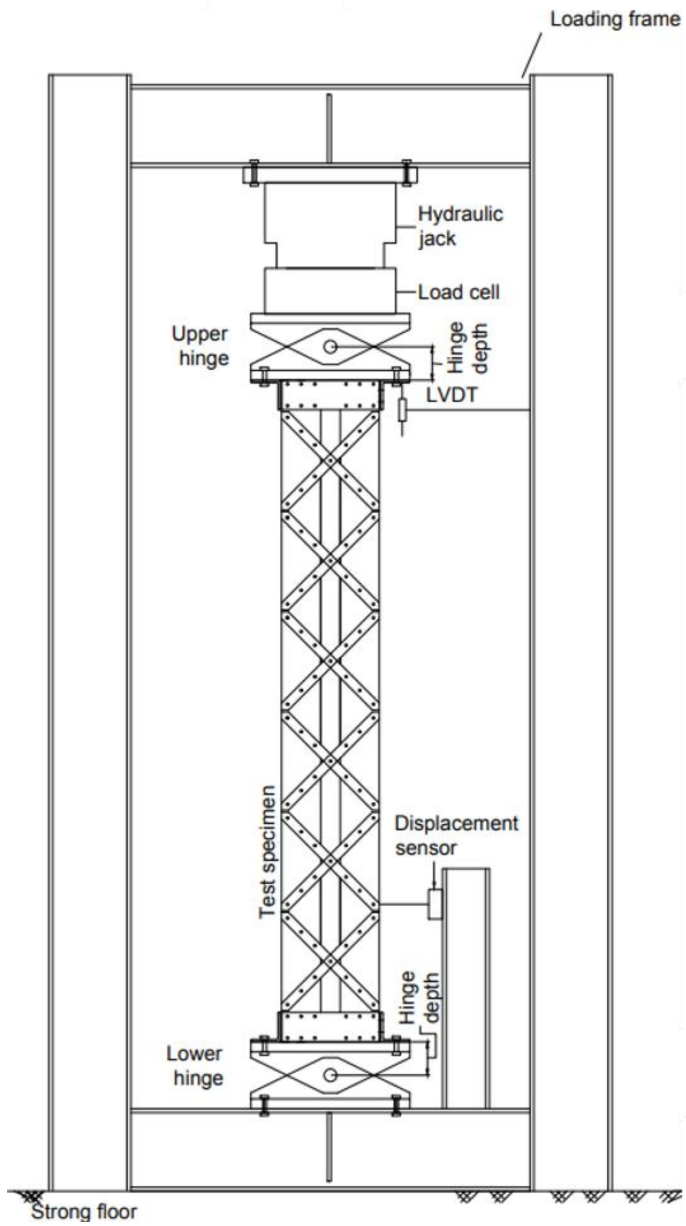


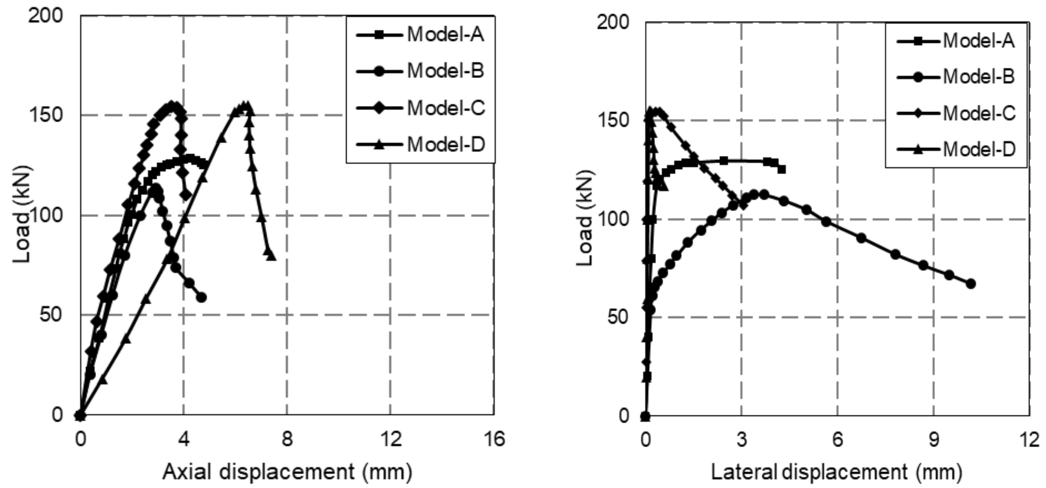

Figure 3: Details of the test set-up

Fig. 5 shows absolute lateral displacement of the models with respect to column height. Due to short column nature, all the models displayed a maximum lateral displacement less than 0.2% of the column height, except Model-B. In Model-B, the initial large geometric imperfections resulted in larger lateral displacement at higher loading, equal to 0.5% of its height. Therefore, the built-up CFS columns fabricated from four unstiffened angles are more responsive to geometric imperfections. Adequate measures must be taken while cold-forming as well as during fabrication, to avoid large imperfections.

A typical strain plot for Model-C is shown in Fig. 6. Like other models, a linear variation in the state of strain was observed in this specimen. The strain reversal clearly indicates that the large lateral displacement in chord members, primarily due to local buckling. However, no cross-sectional yielding was confirmed, which was expected, as the chord members possessed low sectional compactness.

Table 3: Comparison of test results, numerical results are design strengths

| Specimen | Test results | Design strengths | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | NAS | | | | EC3 | | | |
| | $P_{Test}$ (kN) | $\lambda_c$ | $A_{eff}$ ($mm^2$) | $P_{NAS}$ (kN) | $P_{NAS}$/ $P_{Test}$ | $\bar{\lambda}$ | $A_{eff}$ ($mm^2$) | $P_{EC3}$ (kN) | $P_{EC3}$/ $P_{Test}$ |
| Model-A | 129.71 | 0.64 | 288.30 | 117.90 | 0.90 | 0.35 | 273.1 | 110.7 | 0.85 |
| Model-B | 113.46 | 0.57 | 284.65 | 120.04 | 1.05 | 0.31 | 273.1 | 130.5 | 1.15 |
| Model-C | 154.80 | 0.41 | 286.76 | 128.65 | 0.83 | 0.16 | 280.6 | 134.0 | 0.86 |
| Model-D | 154.90 | 0.36 | 285.02 | 129.67 | 0.83 | 0.13 | 280.6 | 134.1 | 0.86 |




(a)Axial shortening (b) Lateral displacement

Figure 4: Load *vs.* displacement response

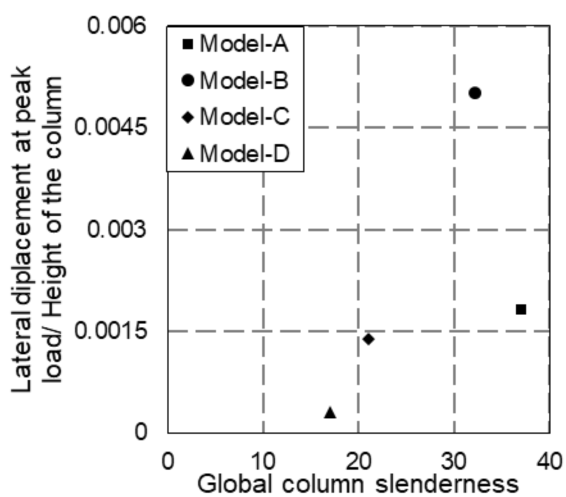



Figure 5: Absolute displacement response

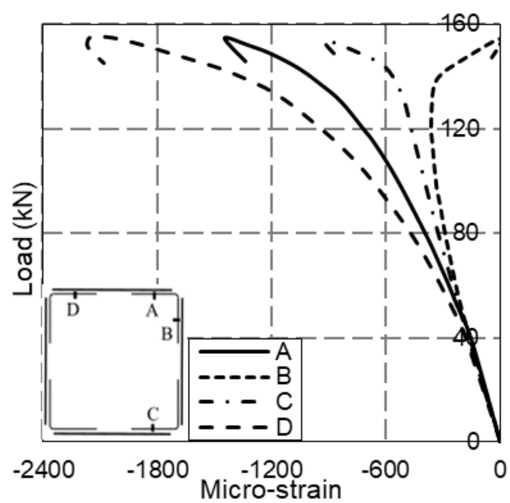
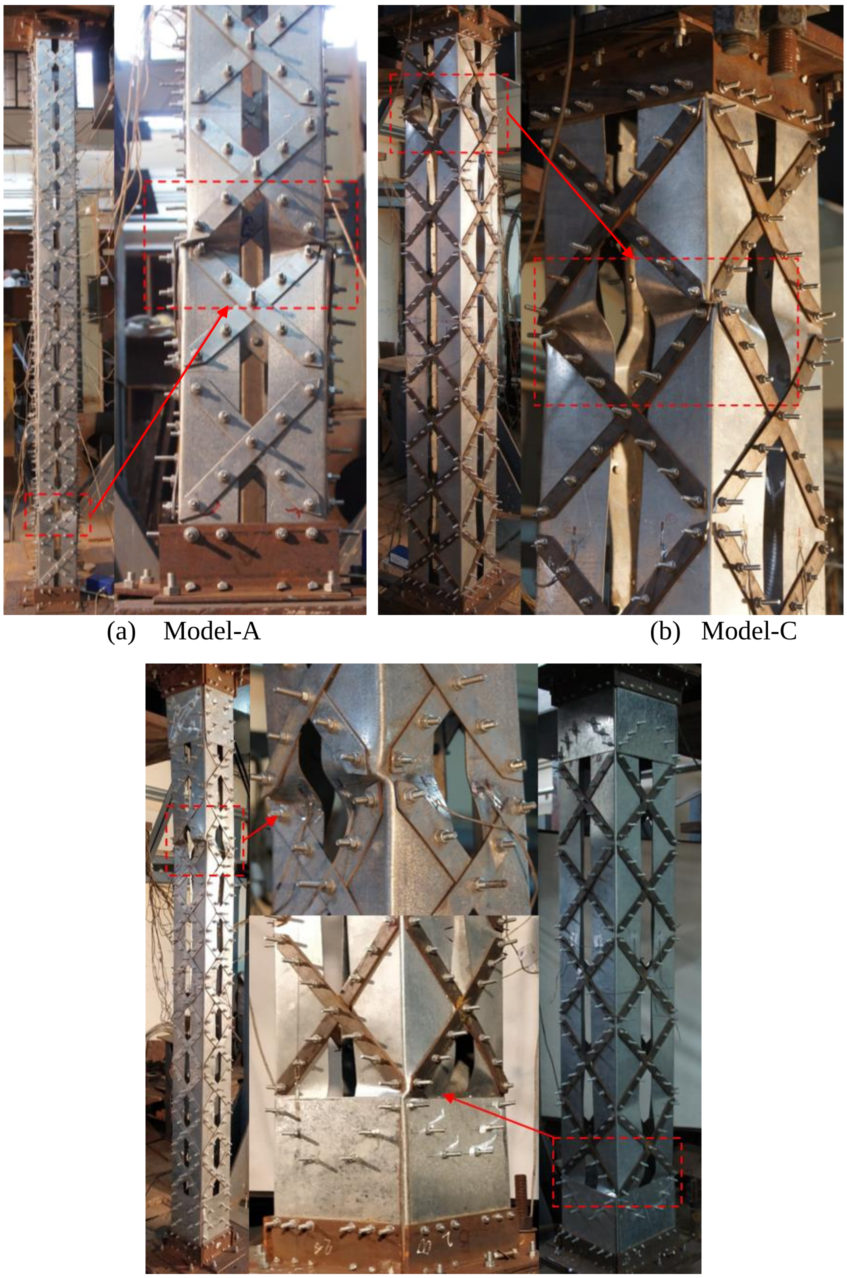



Figure 6: Typical strain plot

Unlike in the previous tests conducted by Dar et al. (2018e) on laced CFS built-up columns with low sectional compactness and single lacing configuration, no visible initial local buckling waves were observed at the early stages of loading. Double-lacing configuration has reduced the unbraced chord length (which reduced the unbraced chord slenderness) which in turn significantly delayed the formation of initial local buckling, and has also substantially enhanced the axial resistance of these columns. Local buckling of the chord members was the primary mode of failure observed in all the models, as shown in Fig. 7. No connection failure in any form was observed in any of the cases. Furthermore, no prominent lacing buckling was observed in any of the cases.



(a) Model-A (b) Model-C

(c) Model-B (d) Model-D

Figure 7: Failure in the test models

There are no specific guidelines in any design specification pertaining to the designing of laced built-up CFS columns. Therefore, design strengths were quantified based on effective width concept as per current North American Specification (AISI S-100, 2016) and Eurocode (EN1993-1-3 2006) for CFS structures and are given in Table 3. The design strength predictions of both North American Specification (AISI S-100, 2016) and Eurocode (EN1993-1-3 2006) were found to be conservative by around 6-13% and 10-17% respectively. Fig. 8 shows the comparison of test results with the design strength predictions of North American Specification (AISI S-100, 2016) and Eurocode (EN1993-1-3 2006).

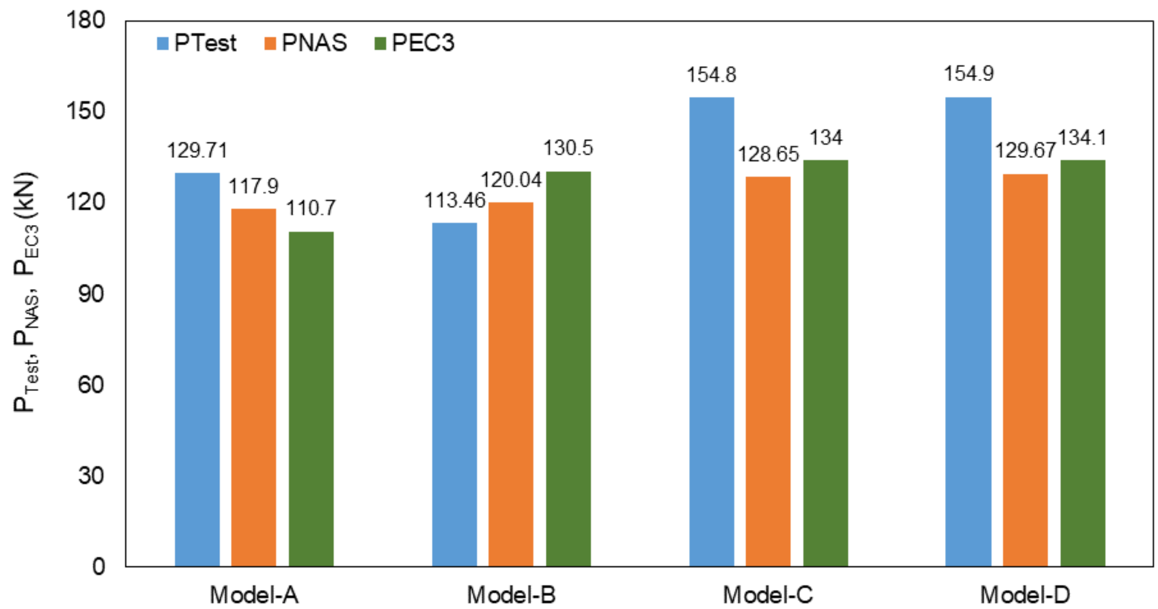



Figure 8: Comparison of test results with the design strength predictions

**4. Summary and Conclusions**

This study presents an experimental investigation conducted on short built-up CFS column specimens with double-lacing configuration and large flat width-to-thickness ratios. Pin-ended support conditions allowing uniaxial bending were adopted. Sectional compactness of the angle sections, column slenderness ratio and slenderness ratio of lacing elements were the main parameters varied in this study. The slenderness of the unbraced chord element was kept the same. Local buckling of the chord elements was the main failure mode observed in all the cases. No connection failure in any form was observed. Furthermore, no prominent lacing buckling was observed in any of the cases. The design strengths of the built-up columns were computed using North American Specification and Eurocode for CFS structures for comparison with the test results. The design strength predictions of both these standards were found to be conservative. However, none of these design standards give clear guidelines for CFS built-up columns with lacing configuration.

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## Notations

| | |
|---|---|
| a | : Spacing between the angle sections |
| $A_{eff}$ | : Effective area of the cross-section |
| b | : Width of angle section |
| CFS | : Cold-formed steel |
| E | : Modulus of elasticity |
| $f_n$ | : Nominal yield strength |
| $f_u$ | : Ultimate strength |
| $f_y$ | : Yield strength |
| l | : Height of the column |
| $l_{chord}$ | : Length of unbraced chord |
| $L_e$ | : Length of end plate |
| LVDT | : Linear variable displacement transducer |
| $P_{EC3}$ | : Design strength predicted by EC-1993-3 |
| $P_{NAS}$ | : Design strength predicted by AISI-S100 |
| $P_{Test}$ | : Ultimate test strength |
| r | : Internal radius of angle section |
| t | : Thickness of chord |
| $\varepsilon$ | : Strain at fracture |
| $\lambda$ | : Overall column slenderness ratio |
| $\lambda c$ | : Critical slenderness |
| $\lambda$ | : Overall non-dimensional slenderness |
| $\lambda_{lacing}$ | : Slenderness of lacing bars |