Behaviour of Cold Formed Built - Up Steel Compression Member

K.Thiyagu, P.Kanaka, R.Elanchezhiyan

Abstract: The use of cold-formed thin-walled steel structural members has increased in recent years. Especially, Cold-formed steel columns are widely used in the construction industry due to their lightweight, easy installation, erection and economy. The strength and efficiency of cold-formed steel profiles depends on the cross-sectional shape, which controls the three fundamental buckling modes: local, distortional and global. As most of their sections are open with only one symmetrical axis, they would likely fail by twisting and interacted with the other buckling modes such as local and distortional buckling. In order to improve the ultimate strength of columns, a built-up column section with distinct shape was selected from the detailed study of Literatures and three specimens of thickness 1.6mm were fabricated with different lengths 500mm, 600mm and 700mm. Consequently, buckling behaviour of built up steel members was investigated theoretically with Direct Strength Method (with the help of CuFSM) as well as experimentally and the results were compared with the buckling modes obtained numerically using ANSYS software and it is found that the ultimate load carrying capacity of the column increases with the decrease of slenderness ratio and finally a new innovative and economical column element was presented.
Keywords: Thinwalled steel, Buckling, Ultimate Strength.

I. INTRODUCTION

General

Steel products are widely used in building industries, such as bridges, roof trusses, transmission line towers, multi – storied buildings, etc., because of its higher strength resulting in the reduction in dead weight. There are two main families of steel structural members. One is the familiar group of hot - rolled member and the other is less familiar but of growing importance, i.e., cold - formed steel depends on the manufacturing. The thickness of cold formed steel sheets or strip are generally ranges from 0.4 mm to 6.4 mm. Much thicker material up to 8 mm can be produced if pre-galvanized material is not required for the particular application. Normally, the yield strength of steel sheets used in cold-formed sections is at least 280 N/mm^2, although there is a trend to use steels of higher strengths, and sometimes as low as 230 N/mm^2.

The use of hot – rolled steel section become uneconomical for the steel structures subjected to light and moderate loads and for the structural members of short span lengths (Eg : Joists, Purlins, Girts, Roof trusses, Complete framing of one and two storey residential, Commercial and Industrial structures).

So, cold formed steel sections have gained considerable prominence over hot – rolled sections. The cold formed steel sections can be produced to any shape according to the requirements by the following three methods.

    i)   Cold roll forming
    ii)   Press brake operation
    iii)   Bending brake operation

The cold formed steel sections are widely used in the carbodies, railway coaches, air crafts, various types of equipments, storage racks, grain bins, highway products, transmission line towers, transmission poles, drainage facilities and bridge construction.

Definition

Thin sheet steel products are extensively used in building industry, and range from purlins to roof sheeting and floor decking. Generally these are available for use as basic building elements for assembly at site or as prefabricated frames or panels. These thin steel sections are cold-formed, (i.e.) their manufacturing process involves forming steel sections in a cold state (i.e. without application of heat) from steel sheets of uniform thickness. These are given the generic title Cold Formed Steel Sections. Sometimes they are also called Light Gauge Steel Sectionsor Cold Rolled Steel Sections. The method of manufacturing is important as it differentiates these products from hot rolled steel sections.

Necessity of Cold Formed Steel

The use of hot – rolled steel sections become uneconomical for the steel structures subjected to light and moderate loads and for the structural members of short span lengths (e.g: joists, purlins, girts, roof trusses, complete framing of one and two storey residential, commercial and industrial structures). So the study on behaviour of cold formed steel framing members is unavoidable to reduce the cost of a building made up of steel structures.

Built - up members

Members fabricated from industry such as two or more structural steel elements which are connected together by means of welding or riveting to form a single element are called as built-up members.The advantages of built-up members are as follows:

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Built-up CFS members usually have symmetric cross-sections, higher strength and better resistance against out-of-plane movement. Because the production method remains unchanged, composed CFS members are a relatively cheap alternative to single profiles, which easily fail in overall buckling, if not laterally supported. Built-up solutions are adopted in practice, regardless of the lack of design rules to predict the member strength.

Different modes of failure in cold formed steel sections.

The major factor, which arises in the design of cold formed steel members, is the susceptibility of these members to a wide variety of buckling modes. The thin walls of such members are often liable to suffer local buckling in compression, and this must be taken into account in the design of almost any cold formed steel structural member. The Different modes of failure in cold formed steel sections are

- Local buckling
- Distortional buckling
- Overall flexural buckling
- Overall flexural – torsional buckling.

Self-tapping screws

Self-tapping screws provide an economical means of assembling components, especially where dissimilar materials must be joined together. They offer a particular advantage where occasional disassembly may be necessary for maintenance or repairs.

Objective of the Study

Specific objectives of this research are as follows:
- ✓ To understand the buckling and ultimate strength behaviour of cold formed built-up compression member under axial compression load.
- ✓ To compare the theoretical calculation by direct strength method (with help of CuFSM) with numerical analysis done using ANSYS 12.0.
- ✓ To investigate experimentally the buckling strength of the builtup column after fabrication.
- ✓ To Comparethe Numerical, Experimental and Theoretical results.

II. METHODOLOGY

Selection of Section

The specimens were modeled to form a lipped angle section with intermediate stiffeners on both legs, and then two of the open sections were connected at their lips using self tapping screws to form a built – up closed octagonal box section. The self tapping screws are normally placed at 20mm distance from both the end of the columns and after that, the connection is made at 100 – 150 mm equal interval throughout the length of column. The cross section of the specimen and arrangement of screw spacing are shown in figure 3.1. And the cross sectional dimensions are illustrated in table 3.1.
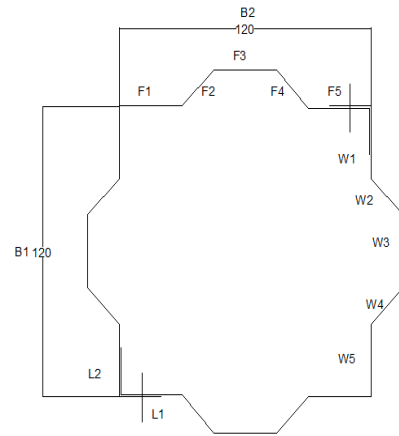

Fig 1 Cross section of the specimen

Where,			
F 1 = 30	F 4 = 21.21	W 1 = 30	W 4 = 21.21
F 2 = 21.21	F 5 = 30	W 2 = 21.21	W 5 = 30
F 3 = 30		W 3 = 30	
All dimensions are in 'mm'			

TABLE 1 Details of specimen

SI. No	Description of specimen	Width	Depth	Size of lips	Thickness	Length or Span
1	COL 1	120	120	25	1.6	500
2	COL 2	120	120	25	1.6	600
3	COL 3	120	120	25	1.6	700
All dimensions are in 'mm'						

III. ANALYSIS OF THE BEAM

Theoritical Analysis

The theoretical part of analysis aims at determining the ultimate load carrying capacity of the section chosen using Direct Strength method which is familiar nowadays and a software naming CuFSM to find load factor values.
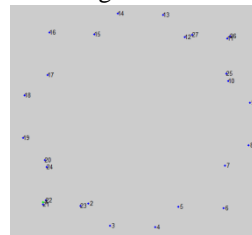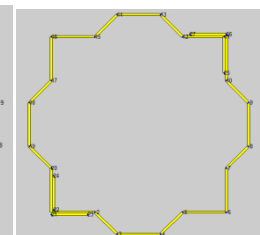

Fig.2 Node generation in CuFSM   **Fig.3 Element generation in CuFSM**

TABLE 2 Sectional properties obtained from CuFSM

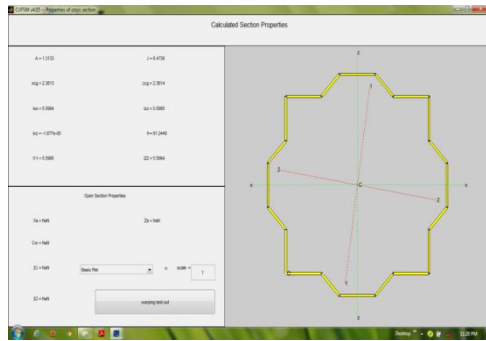| Sectional properties	Specimen thickness 1.6 mm	
	In inches	In mm
Area	1.3133	847.288
J	8.4736	$3.53 \times E^{06}$
X_{cg}	2.3613	59.977
Z_{cg}	2.3613	59.977
I_{xx}	5.5995	$2.33 \times E^{06}$
I_{zz}	5.5995	$2.33 \times E^{06}$
I_{xz}	$-1.677 \times E^{-05}$	$-4.5 \times E^{03}$

Fig 4 Geometric properties of specimen Load factor obtained from CuFSM
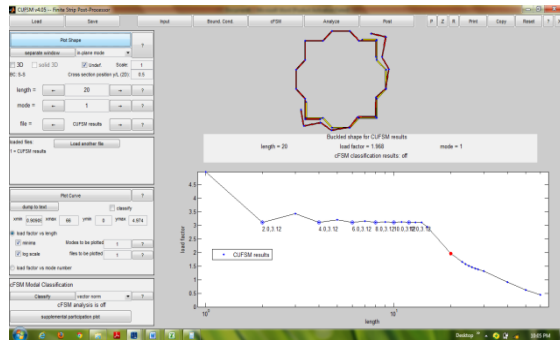

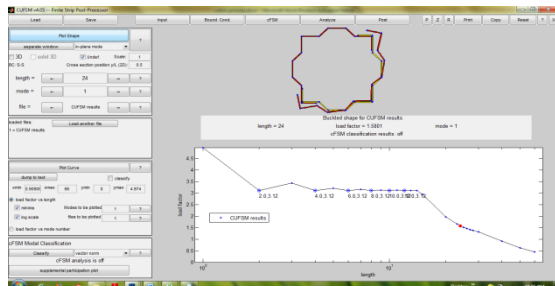**Fig 5 Load Factor for Buckling behaviour of 500mm length specimen**

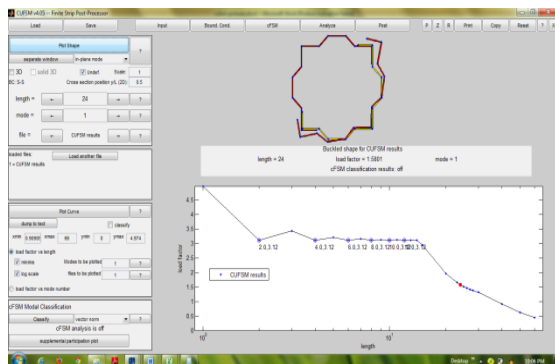**FIG 6 Load factor for buckling behaviour of 600mm length specimen**

FIG 7 Load factor for buckling of 700mm length specimen

The load factors obtained for the different buckling modes of the specimen are described in table.3.

TABLE 3 Load factor obtained from CuFSM

S. No	Thickness of specimen	Type of buckling	Load factor
1	1.6	Distortional	1.968
2	1.6	Distortional	1.5801
3	1.6	Distortional	1.3796

Experiment Analysis

The experimental program of the project involves the fabrication of the specimens followed by tests to investigate the behaviour of columns under various buckling modes and to determine the ultimate load carrying capacity of the columns.

Test setup   Column 3

Column 2   Column 1

Fig 8   Buckling modes of column

TABLE 4 Test Details

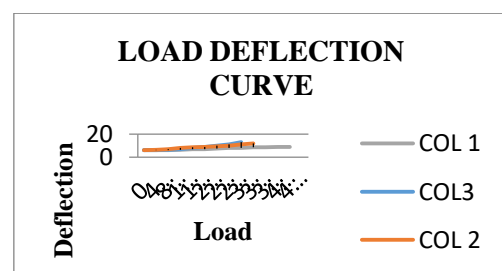| Sl.No	Load (kN)	Deflection (mm)		
		Column 1	Column 2	Column 3
1.	0	6	6	6
2.	40	6	6.5	6
3.	80	6	7	6.5
4.	120	6.5	8	7
5.	160	7	8.5	8
6.	200	7	9	9
7.	240	7.5	9.5	10
8.	280	8	10	11
9.	320	8	11	13
10.	350	8.5	12	-
11.	360	8.5	-	-
12.	400	9	-	-
13.	430	9	-	-

Fig 9 Load vs. Deflection Curve

Numerical analysis

The Finite Element Method (FEM) is deemed the best possible method when conducting complicated engineering analysis and computational issues. The ANSYS, widely applied engineering analysis package software, is large non-linear finite element analysis software with analytic functions of structure, fluid, electric field, and magnetic field. In this study, FEM was carried out through the ANSYS software to understand the mechanical behavior of columns under compression.
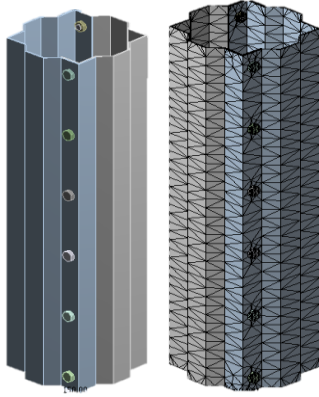

Fig 10. Element generation and Finite element mesh Boundary conditions

The finite element modeling was simulated the built – up box columns which are connecting two angle sections compressed between hinged ends. Thehinged – ended boundary condition was modeled by restraining moment and rotation only at both ends of the column. This is due to the load applied at the top end of the column. The nodes other than the two ends were free to translate and rotate in any directions.
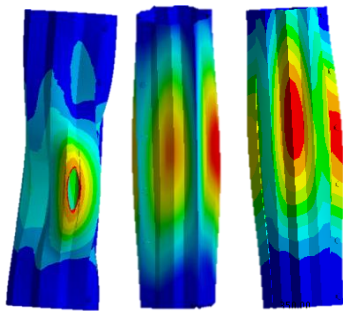

Fig 11. Typical buckling modes of columns

IV.   RESULTS AND DISCUSSION

Theoretical analysis results

According to the North American Specification (NAS) – 2001 of direct strength method (DSM), the nominal axial strength of the thin walled elements was calculated based on the load factor and type of buckling produced. The design equations and recommendations are previously explained.

Ultimate load prediction by DSM

For every specimen length, the ultimate load carrying capacity were calculated and tabulated in table 5.

TABLE 5 Ultimate loads predicted from Direct Strength Method

S. No	Description of specimen	Ultimate load (kN)	Failure mode
1	COL 1	416.866	Distortional
2	COL 2	334.700	Distortional
3	COL 3	292.230	Distortional

Experimental analysis results

From the load vs. deflection chart, the ultimate load carrying capacity of cold rolled steel columns are derived and summarized as follows:

TABLE 6 Ultimate loads predicted from Experiment

S.No	Specimen	Ultimate load carrying capacity
1	COL 1	430 KN
2	COL 2	350 KN
3	COL 3	320 KN

Comparison results

For the different length specimens of same thickness, the ultimate load carrying capacity of the specimen under theoretical and experimental analysis was tabulated.

TABLE 7 Comparison of results

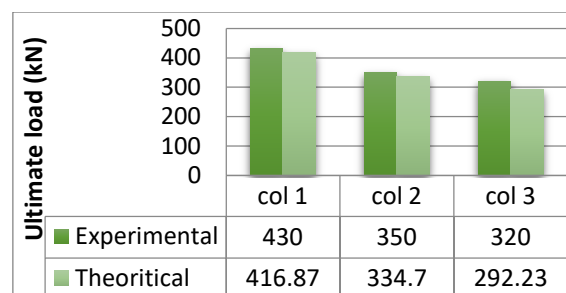| Description of specimen	Slenderness ratio	Ultimate load (kN)		Ratio of Ultimate load
		Experimental results	Theoretical results	P_{The} / P_{Exp}
COL 1	9.535	430	416.866	0.971
COL 2	11.44	350	334.700	0.956
COL 3	13.33	320	292.230	0.913

Fig 12. Comparison of DSM and Experimental results

Discussion on results

For COL 1, the failure is due to distortional buckling. Critical load derived from theoretical analysis is 416.866 kN and load from experiment is 430 kN. The ratio of theoretical value to experimental value is 0.971.

For COL 2, the failure is due to distortional buckling. Critical load derived from theoretical analysis is 334.7 kN and load from experimentis 350 kN. The ratio of theoretical value to experimental value is 0.956.

For COL 3, the failure is due to distortional buckling. Critical load derived from theoretical analysis is 292.23 kN and critical load from numerical analysis is 320 kN. The ratio of theoretical value to experimental value is 0.913.

The investigation shows that experimental and theoretical results are in good agreement. In all the specimens, the initial mode of failure is by distortional buckling and final failure by local buckling and the distortional buckling controls the specimen. Also the buckling mode obtained through ANSYS resembles the actual failure mode of specimen under loading.

V. CONCLUSION

Cold formed built – up octagonal shaped closed box sections were tested under axial compression and the ends of columns were simulated as hinged ends with varying lengths and the test strengths were compared with strength values obtained from theoretical analysis.

Following are the decisions have been made.

- ✓ Buckling mode prediction from numerical analysis (ANSYS 12.0) was compared with experimental buckling modes.
- ✓ Even though the failure modes of the columns involved local buckling, distortional buckling of the webs and flexural buckling, the significant mode of failure is controlled by distortional buckling.
- ✓ Ultimate load carrying capacity is inversely proportional to slenderness ratio (L/r ratio).
- ✓ When the slenderness ratio (L/r ratio) decreases the ultimate load carrying capacity of the specimens have been increased.
- ✓ With reference to COL 3, the ultimate load carrying capacity of COL 2 and COL 1 increased by 9.37%, 34.37% respectively.

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