Latin American Journal of Solids and Structures
www.lajss.org

Study on the Capacity of Cold-formes Steel Built-up Battened Colums Under Axial Compression

Abstract

This paper presents the theoretical and numerical investigation results of the pin ended cold-formed steel built-up battened columns under axial compression. The finite element model was developed using the finite element software ABAQUS. The comparison of finite element analysis results match with the test results available in the literature shows that the analysis model can simulate the buckling behaviour and ultimate capacity of cold-formed steel built-up columns. Three types of sections were selected based on the limitations provided in AISI S100-2007 for prequalified sections for single lipped channel. Spacing between the chords is chosen such that the moment of inertia about major axis equals the moment of inertia about minor axis. After the verification of the finite element model, parametric study has been carried out by varying the slenderness ratio and number of battens. Two types of analytical models were used to obtain the ultimate load by the direct strength method. The ultimate loads from FEA and DSM were compared. At the end, a design recommendation is proposed for DSM to evaluate ultimate strength of the lipped channel built-up battened columns.

Keywords

Direct Strength Method, Distortional buckling, Global buckling, Lipped channel column, Local buckling, Local / Distortional / Global Interaction, etc.

Dr. M. Anbarasu [1] *
P. Bharath Kumar [2]
Dr. S. Sukumar [3]

[1] Assistant Professor,
[2] P.G Student,
[3] Professor,
Department of Civil Engineering,
Government College of Engineering,
Salem - 636 011, Tamilnadu, India

*Author email: gceanbu@gmail.com

Received 17.02.2014
In revised form 22.04.2014
Accepted 06.05.2014
Available online 26.09.2014

1 INTRODUCTION

The use of cold-formed steel members has increased in recent years, especially in light-weight steel construction, such as steel-framed housing, low-rise office buildings, factories, and warehouses due to its high strength to weight ratio. Cold formed steel built-up sections are commonly used as compression members to carry heavier loads and over longer spans when a single individual section is insufficient. The mono symmetric sections have relatively small torsional rigidity that is weak in

twisting compared to doubly symmetric sections. In this study, the cold-formed steel built-up closed sections with web stiffeners are investigated, as shown in Fig. 3(b). The direct strength method is based on open sections, such as the simple lipped channel, lipped channel with web stiffeners, Z-section, hat section, and rack upright section. Therefore, there is a need to investigate the appropriateness of the direct strength method on the type of cold-formed steel built-up battened sections.

The purpose of this paper is to study the behaviour and strength of the cold-formed steel built-up battened columns by Numerical and theoretical analysis. A non-linear finite element model was developed for cold formed steel built up columns. The models account for steel plasticity, geometric non-linearities, and geometrical imperfections. The models were verified using experimental results in the literature (El Aghoury *et al.* (2012)), and used in a parametric study. In the parametric study the slenderness ratio is varied from 30 to 120 for the selected three sections. The strength obtained from the numerical analysis are compared with the design strengths obtained using the direct strength method in the North American Specification (NAS 2007) and Australian/New Zealand Standard (AS/NZS 2005) for cold-formed steel structures. In the calculation of the direct strength method, local buckling stress and distortional buckling stress are required. In this study, two different methods were used to obtain these stresses. The appropriateness of the direct strength method on the cold-formed steel built-up battened columns was investigated. A design recommendation is proposed for DSM to evaluate ultimate strength of the lipped channel built-up battened columns. Furthermore, the advantages and limitations of the proposed design rules are also discussed.

2 LITERARTURE REVIEW

The research performed on hot-rolled built-up members is the foundation of cold-formed steel built-up member research. Rondal and Niazzi (1990) conducted experimental and numerical investigation on battened steel strut composed of cold-formed steel channel profiles connected by batten elements and C-stitches. They proposed a design method for column with C-stitches. Aslani and Goel (1991) verified the modified slenderness ratio analytically and experimentally for hot-rolled members, thereby verifying the AISC built-up member design method. Temple and Elmahdy (1993), carried out an experimental and theoretical study to investigate the behaviour of battened columns made of standard channel hot rolled steel sections. A combination of an equivalent slenderness ratio and limitations to the slenderness ratio of the main members between batten plates (interconnectors) are provided.

Study on bolted double-angle compression members by Sherman and Yura (1998) concluded that preventing shear slip in the end connections of built-up members was a critical factor for strength. Salem *et al.* (2004) numerically studied the strength of battened column built from cold formed lipped channels. They proposed formulae that integrate torsional flange buckling into the unified effective width approach and compared the results with the numerical results. Stone and Laboube (2005) experimented on built-up I-section, studs formed by C-section inter-connected by self-tapping screws in order to verify the modified slenderness approach. They established that the provisions of the North American specification were conservative for thick members. The modified slenderness ratio was recommended for profiles of relatively small in thickness.

Experimental testing, finite element analysis, and theoretical buckling stress calculations conducted by Sukumar *et al.* (2006) were used to develop analytical models for axially and eccentrically loaded cold-formed, open built-up compression members undergoing flexural-torsional buckling. A design curve for the maximum strength of cold-formed built-up columns experiencing distortional buckling or mixed local-distortional buckling was proposed. Lue *et al.* (2006) presents a direct experimental verification of the AISC-LFRD slenderness ratio formula for hot rolled built-up compression members. Young (2008) studied built-up closed sections with intermediate stiffeners, which formed a closed section by screw interconnections in the flanges. The author provided column buckling curves, based on experiments on short, intermediate and slender compression members. Research on capacities of axially loaded, cold-formed, built-up lipped channels by Whittle and Ramseyer (2009) found that the use of the modified slenderness ratio is more conservative for longer and thickened built-up members. Hosseini Hashemi and Jafari's (2009) conducted test to determine the elastic critical buckling load of batten columns about a minor axis. South well plots were used to determine the elastic critical loads of the specimens from the data. The South well theory is modified to ensure the applicability of the method for batten columns. El Aghoury *et al.* (2010 & 2012) studied the effect of the interaction between the leg outstands width–thickness ratio, the angle overall slenderness ratio, and the overall member slenderness ratio on the ultimate capacity of battened columns composed of four equal slender angles. Georgieva *et al.* (2012 a & b), studied the behaviour and strength of cold-formed steel members, and the influence of interconnection pieces between the profiles. The fastener flexibility affects the member stability and strength, especially when bolts are used to interconnect the profiles and clearances are provided to ensure ease of assembly. The additional flexibility due to slip in connections further complicates the development of a suitable design method for built-up CFS members. Anbarasu and Sukumar (2013) studied the effect of lateral ties in the intermediate length rack columns. Anbarasu and Sukumar (2014) investigated the buckling interaction on lipped channel columns.

3 FINITE ELEMENT MODELLING

3.1 General

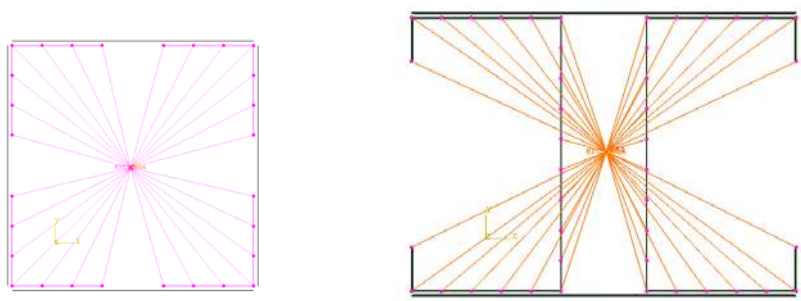
This section of the paper describes the finite element model by using the commercial finite element software package ABAQUS version 6.10 to simulate the cold-formed steel pin ended battened column tests conducted by El Aghoury *et al.* (2012). The models were based on the centre line dimensions of the cross-sections. The residual stresses and the rounded corners of the sections were not included in the model. The effect of residual stresses on the ultimate load is considered to be negligible as shown by Schafer and Pekoz (1998). Prior to analyzing the post-buckling behaviour of the structure, a linear buckling analysis was performed on the specimens to obtain its buckling load and mode shape. Following this a non-linear analysis was carried out by incorporating both geometric and material non-linearities to obtain the load versus end-shortening characteristics and to predict the ultimate load capacity.

3.2 Type of Element and Material Modelling

The specimens were modeled using four noded S4R5 thin shell element with six degrees of freedom at each node. Convergence studies have been carried out on the column in order to determine a suitable finite element mesh for the analysis. The element aspect ratios (length to width) nearly equal to 1.0 for the flange and web elements were used. The size of the element was 10 x 10 mm^2. The material model used in the verification model was identical to the FEM as described in El Aghoury *et al.* (2012). The model included the geometric and material non linearity. Young's modulus of elasticity, E and the yield stress, f_y, of the steel material were considered as 2 X 10^5Mpa and 250Mpa respectively. Elastic perfect plastic stress-strain curve obeying von-misses yield criterion was adopted for material modeling in the parametric study.

3.3 Boundary Conditions and Loading Application

The end conditions of column elastic lines were treated as pinned condition. The loaded end was prevented from both rotation about z-axis and translations in both x and y directions. But in the unloaded end was prevented from translation in three directions x, y and z directions and from rotations about z-axis. These boundary conditions were applied to the independent node of the rigid fixed MPC (Multi Point Constraint) located at the geometric centroid of the section at upper and lower end of the model. Dependent nodes are connected to the independent node using rigid beams and all six structural degrees of freedoms are rigidly attached to each other. This MPC acted as a rigid surface that was rigidly connected to the upper and lower end of the columns as shown in Figure 1.



(a) Validation model (b) Parametric model

Figure 1: Rigid Region.

The axial loads were applied by specifying an axial displacement to the master node at loaded end of the column, which is identical to the pin-ended column tests. For each incremental step of end shortening, the total load or reaction at the unloaded end was obtained. Modified riks arc length method for equilibrium path is used in the analysis.

3.4 Modeling of fastener and Geometrical Imperfections

The connection between the battens and sections were modelled by using mesh independent fastener option available in ABAQUS. A superposition of the minimum local buckling mode and minimum distortional buckling mode were employed for the imperfection shape that were critical and obtained by conducting the eigen buckling analysis. From the detailed considerations from the literature, the local and distortional imperfection was taken equal to the 0.006*w*t and 1*t respectively as recommended by Schafer and Pekoz (1998), in addition the global imperfection magnitude was taken as 1/500 of the full length of the column at the mid-height section for lipped channels were used in the parametric study models to initiate the nonlinear analyses.

3.4 Non-Linear Analysis

Two types of analysis, first the bifurcation buckling analysis to determine the buckling loads and corresponding buckling modes followed by the nonlinear analysis to determine the ultimate loads and deformations, including post buckling reserve strength were carried out using ABAQUS. The non linear analysis was performed considering both geometric and material non linearities.

4 VALIDATION

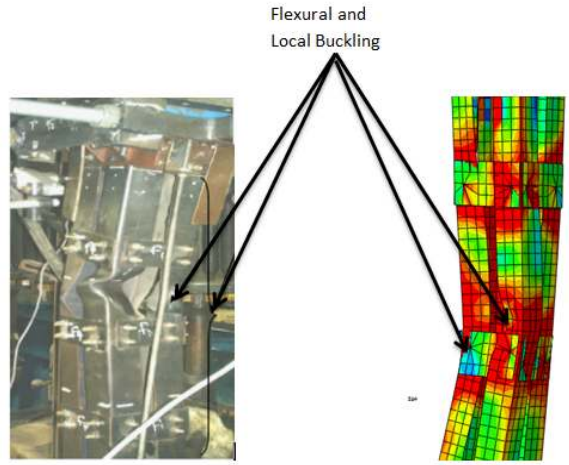
The numerical model was calibrated against the experimental work presented by El Aghoury *et al.* (2012). Four specimens were modelled by using the finite element analysis. The comparison between the experimental ultimate load carrying capacity, and those computed by finite element analysis are presented in table 1 and shows a reasonable agreement between the FEA and the experiments.

Specimen ID (as per literature)	Length	Cross Section	P_{Exp}	P_{Fem}	P_{Exp}/P_{Fem}
1000B80L30B1	1000	80X80	54	57.13	0.945
1000B80L30B2	1000	80X80	48	52.21	0.919
1000B80L30B6	1000	80X80	52	54.8	0.949
1940B80L30B5	1940	80X80	33	35.4	0.932
Mean					0.936
Standard Deviation					0.013

Table 1: Comparison of Finite Element Results with Test Results.

The mean and standard deviation of the Test to FEM ultimate loads for test results from the literature are 0.936 and 0.013 respectively. Some limitations existed with the developed FE model. The FEM results are about 7% higher than the test results. The difference in the numerical model was more likely due to assumed imperfections and simplified modelling of fastener of the sections.

Fig. 2 shows the correlation of the failure deformed shape of the angle battened column. The simulation in finite element model is same as that obtained experimentally. The resemblance of Fig. 2 (a) and 2 (b) demonstrates the reliability of the FEA predictions.



(a) Experimental (b) FEA

Figure 2: Failure Deformed Shape.

5 SECTION DESIGN

The battened columns were made from cold-formed steel lipped channel sections. The lipped channel built-up battened column cross-section dimensions were suitably chosen. The cross-sectional dimension satisfied the limitations given for pre-qualified sections of columns in Direct Strength method (Table 2).

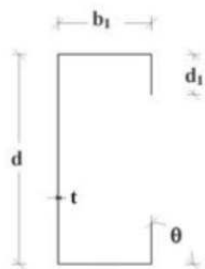
Section	Geometric limitation
	$d/t < 472$
	$b_1/t < 159$
	$4 < d_1/t < 33$
	$0.7 < d/b_f < 5.0$
	$0.05 < d_1/b_1 < 0.41$
	$\theta = 90°$
	$E/f_y > 340$ ($f_y < 593$ MPa)

Table 2: Geometric Limitations of Lipped Channel Sections.

The geometric properties for the three selected specimens are presented in Table 3.

Sl. No	Column designation	Thickness-t (mm)	Web-b (mm)	Flange-d (mm)	Lip-d1 (mm)	Spacing-s (mm)
1	BC 1	1.6	50	90	15	26.2
2	BC 2	1.6	75	150	15	61.4
3	BC 3	2	75	120	15	30.2

Table 3: Section properties and Geometry.

Spacing between the chords was chosen such that moment of inertia about major axis equals the moment of inertia about minor axis. To ensure that the built-up member acts as a unit, AISC E6.2 required that individual components of built-up compression member connector spacing be such that the effective slenderness ratio of each component did not exceed three-fourths of the slenderness ratio of the built-up member. In this study, spacing between the battens was chosen to satisfy this requirement. Fig. 3 shows the detail drawing of a typical specimen. The specimens were labelled to identify their variable parameters. The connection between the battens and sections was made by using self-drilling screw. The width of batten plate used was 40mm. Fig. 4 shows the labeling of the specimen.

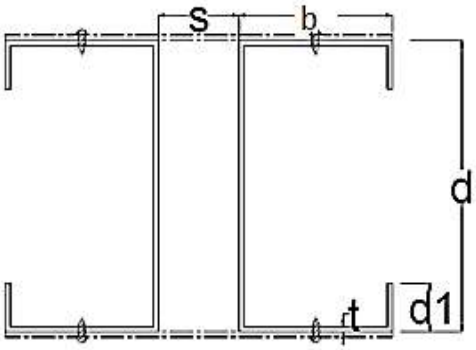


Figure 3: Specimen Details.

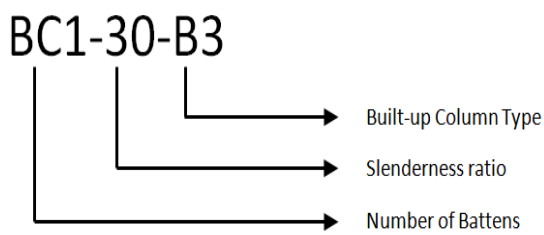


Figure 4: Labelling of specimen.

6 PARAMETRIC STUDY

In the parametric study, theoretical and numerical analysis was carried out for 3 cross-sections as mentioned in Table 3. For each cross-section different member lengths were chosen to obtain a wide range of member slenderness ratios, varying from 30 to 120. In this study, the theoretical axial compressive strengths was determined by using codal provisions OF Direct Strength Method by North American Specifications (AISI S100-2007). Two different approach were used to estimate the ultimate strength of the built-up section specimens.

DSM was formally adopted in AS/NZ, since 2005, as an empirical approach to the Effective Width approach for the design of CFS columns and beams. The method followed from the observation that the buckling strength of a thin-walled member can be directly derived in function of the theoretical elastic buckling loads that trigger local (P_{crl}), distortional (P_{crd}) and overall instability (P_{cre}) in the idealized member (without initial imperfections or residual stresses). DSM is a method that incorporates many of the complicated buckling phenomena in single thin-walled members into a design method as simple and as familiar as possible.

In the first DSM (DSM-1) approach, the ultimate strength of the built-up section was equal to the total of the ultimate strength of two single lipped channel sections (by not considering the depth and number of batten plates). Finite strip analysis software CUFSM were used in this DSM approach.

In the second DSM (DSM-2) approach, the elastic buckling solutions obtained from finite element software ABAQUS were substituted in the DSM equations. The modified slenderness ration

(KL/r)m is used to calculate the critical Euler's buckling stress (P_{cre}). Eqn. 1 refers to the modified slenderness ratio as given in AISI code.

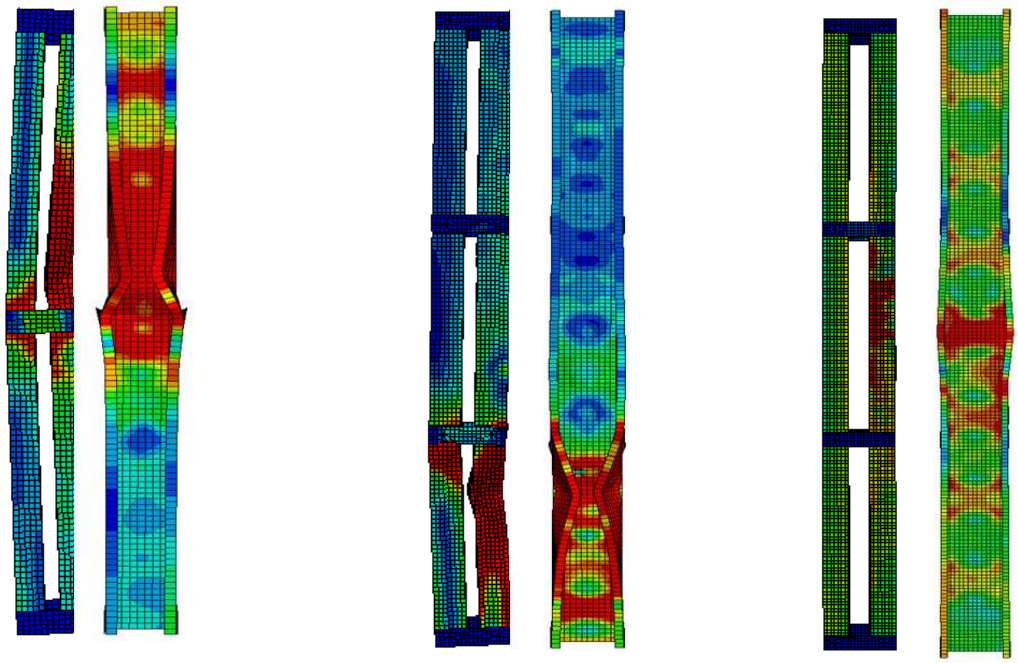
$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2}. \quad (1)$$

Totally 30 number of analysis were carried out by both the methods. The ultimate strength obtained by finite element analysis was compared with the direct strength method shown in Table 4.

Specimen ID	PFEM	Failure Mode	P_{DSM-1}	P_{DSM-2}	P_{FEM}/P_{DSM-1}	P_{FEM}/P_{DSM-2}
BC 1-30-B3	140.89	L+D+F	140.78	154.34	1.001	0.913
BC 1-40-B4	139.64	L+D+F	130.19	151.60	1.073	0.921
BC 1-50-B4	138.14	L+D+F	114.12	148.23	1.210	0.932
BC 1-60-B5	134.71	L+D+F	98.78	142.99	1.364	0.942
BC 1-70-B6	115.24	D+F	84.94	127.82	1.357	0.902
BC 1-80-B7	100.24	D+F	72.93	113.88	1.374	0.880
BC 1-90-B7	85.37	L+D+F	62.62	101.24	1.363	0.843
BC 1-100-B8	71.92	L+D+F	54.67	90.18	1.315	0.797
BC 1-110-B8	61.07	D+F	48.75	80.43	1.253	0.759
BC 1-120-B9	52.42	D+F	44.17	71.74	1.187	0.731
BC 2-30-B4	153.03	L+D+F	143.94	163.23	1.063	0.938
BC 2-40-B5	155.40	L+D+F	128.18	159.11	1.212	0.977
BC 2-50-B6	151.50	L+D+F	111.18	154.03	1.363	0.984
BC 2-60-B8	137.89	L+D+F	94.34	148.06	1.462	0.931
BC 2-70-B9	125.73	D+F	79.04	141.31	1.591	0.890
BC 2-80-B10	111.89	D+F	66.50	133.86	1.683	0.836
BC 2-90-B12	103.14	D+F	55.22	125.80	1.868	0.820
BC 2-100-B13	88.08	D+F	47.12	110.29	1.869	0.799
BC 2-110-B14	75.76	D+F	41.14	96.66	1.842	0.784
BC 2-120-B15	64.89	D+F	36.58	84.60	1.774	0.767
BC 3-30-B4	167.99	L+D	226.98	197.27	0.740	0.852
BC 3-40-B5	169.36	L+D	209.72	191.63	0.808	0.884
BC 3-50-B6	159.21	L+D+F	189.27	184.68	0.841	0.862
BC 3-60-B7	145.98	L+D+F	161.50	176.54	0.904	0.827
BC 3-70-B8	137.70	L+D+F	136.54	167.43	1.008	0.822
BC 3-80-B9	132.91	L+D+F	115.01	155.69	1.156	0.854
BC 3-90-B10	116.10	D+F	97.20	137.49	1.194	0.844
BC 3-100-B11	100.29	D+F	84.40	121.64	1.188	0.825
BC 3-110-B12	87.65	D+F	74.88	106.15	1.170	0.826
BC 3-120-B13	75.25	D+F	67.59	91.18	1.113	0.825
	MEAN				1.28	0.86
	STANDARD DEVIATION				0.30	0.06

Table 4: Comparison of Results.

As a sample, the deformed shape at failure load from finite element meth od is shown in Fig.5. Fig. 6 shows the load versus axial shortening curves for BC1 series columns.



a) For BC1-20-B3 b) For BC2-20-B3 c) For BC3-20-B3

Figure 5: Deformed shape at failure loads from FEM.

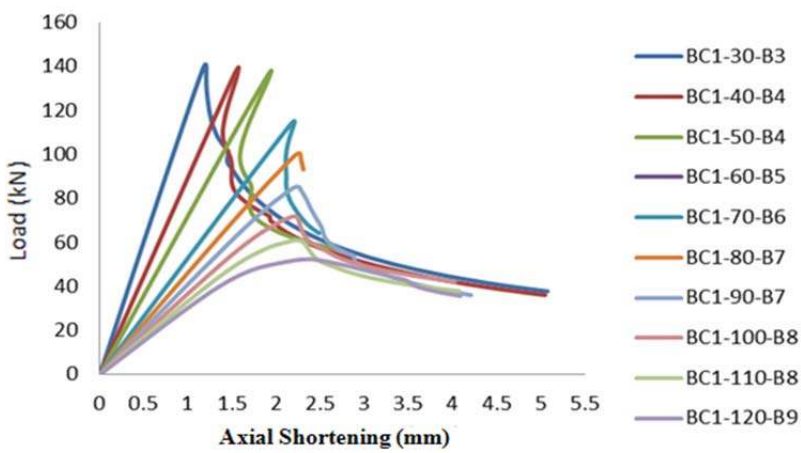


Figure 6: Load Vs Axial Shortening Curve for BC1 series.

7 DISCUSSIONS

The mean and standard deviation of the P_{FEA} to P_{DSM-1} ultimate loads are 1.28 and 0.30 and the mean and standard deviation of the P_{FEA} to P_{DSM-2} ultimate loads are 0.86 and 0.06 respectively. From the comparison of results it is found that, evaluating the resistance of a built-up battened column member as the sum of resistance of individual chords is not representative of the actual structural response of these members.

The DSM-1 approach conservatively predicted the ultimate capacity of the built-up battened column. The conservatism increased with increasing the member slenderness ratio. For the slenderness ratio less than 70 all the specimens failed by combined local (L), distortional (D) and flexural (F) mode. For slenderness ratio greater than 70, the specimens failed by combined distortional (D) and flexural (F) mode. But the predominant mode is distortional buckling and it governs the strength. From the results obtained the following comparisons shall be made for BC-1, BC-2 and BC-3 type columns. Comparison of DSM-1, DSM-2 and FEM Vs. Slenderness Ratio for BC-1, BC-2, BC-3 is shown in 7 (a),(b), &(c).

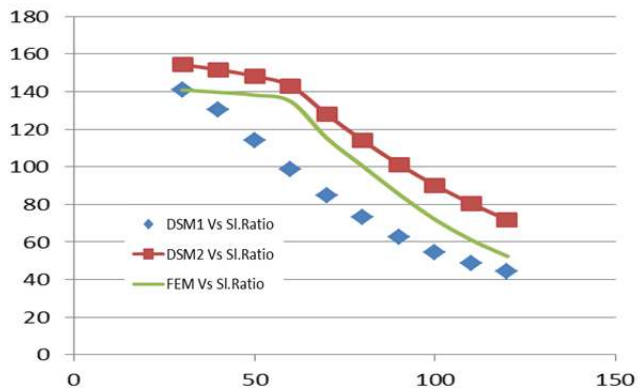


Figure 7(a): Comparison results for BC-1.

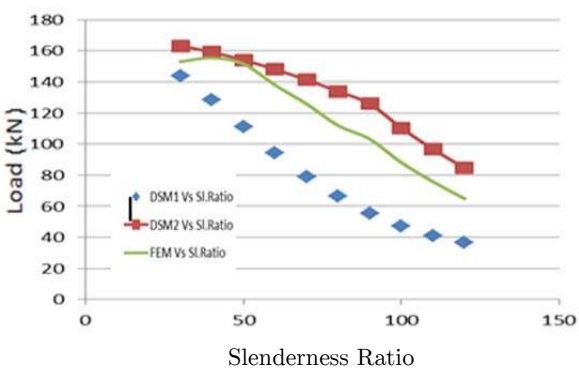


Figure 7(b): Comparison of results for BC-2.

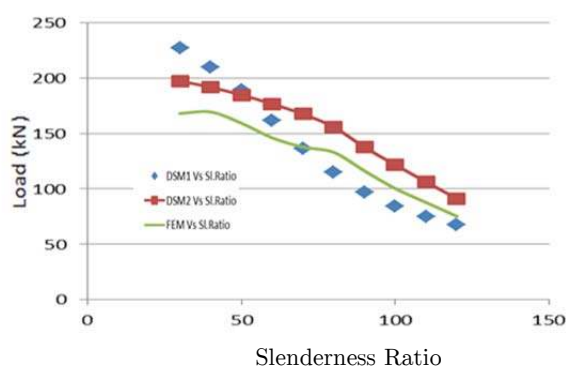


Figure 7(c): Comparison of results for BC-3.

The DSM-2 approach unconservatively predicted the strength of the built-up battened columns. The unconservatism increased with increasing the member slenderness ratio. From the figure 5(a) to 5(c) it can be found that, the FEM and DSM-2 curve follows same trend. For slenderness ratio greater than 70, the rate of unconservatism was more. From the graphs it is shown that DSM-1 underestimates the design strengthwhereDSM-2 overestimates the design strength. From the graph it is observed that the curve follows similar trend. So their behavior will be similar irrespective of the cross section.

8 DESIGN RECOMMENDATION PROPOSED

The comparison of numerical results with the direct strength method (DSM-1) and (DSM-2) results are presented in Fig. 8 and Fig. 9.

In the Fig.8, all the results are on the unsafe side and the variation is almost linear. In the Fig. 9, most of the results are on the safe side and the variation is non-uniform.

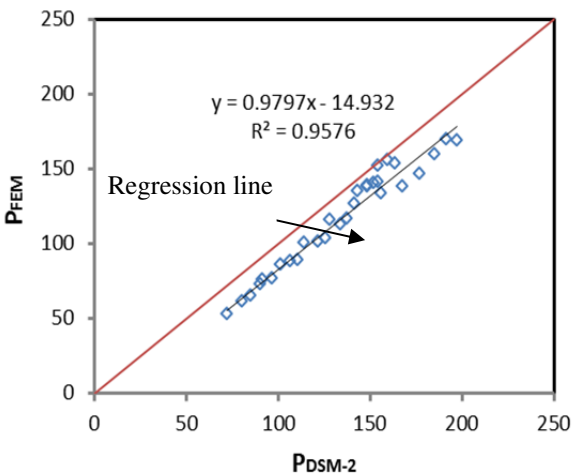


Figure 8: P_{FEA} Versus P_{DSM-2} curve.

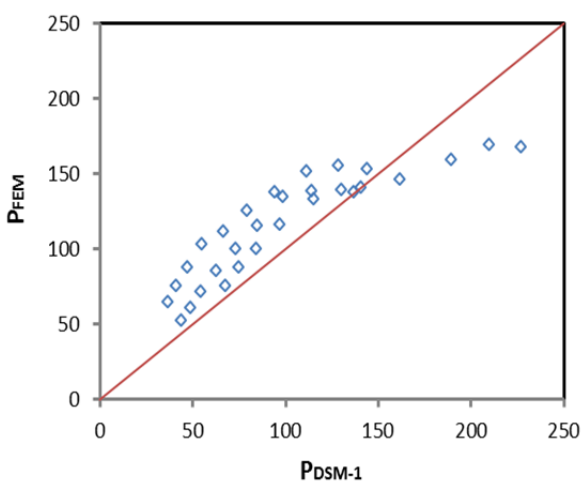


Figure 9: P_{FEA} Versus P_{DSM-1} curve.

For DSM-2 approach, a regression analysis was conducted for the results associated with the 30 analysis. The relationship between the ultimate capacity predicted by finite element analysis (P_{FEA}) and the ultimate resistance evaluated in accordance with the direct strength method (P_{DSM-2}) was almost linear as $P_{FEA} = 0.9774*P_{DSM-2} - 14.932$ with R-squared value (1-(Residual sum of squares/Corrected sum of squares)) was 0.9576 which was more than 0.95 therefore it best fitted with the data. From the analysis a linear equation was proposed to calculate the ultimate strength of cold-formed steel lipped channel built-up battened columns. This equation reasonably predicted the ultimate strength of the section.

9 CONCLUSION

A total of 30 axially loaded built up battened columns were numerically and theoretically studied in this paper. The following conclusions on the axial compressive strength of the built-up battened columns were drawn within the limit of the present investigation.

- The developed finite element models were in reasonable agreement with the test results available in literature, therefore, ABAQUS program can be used to simulate battened columns.
- It was found that the DSM-1 approach was generally conservative and the conservatism increased with increasing the slenderness ratio.
- Use of the modified slenderness ratio in DSM-2 approach unconservatively predicted the strength of the built-up columns.
- Based on the parametric results, a simple design expression was proposed to predict the ultimate strength.
- The ultimate strength of the member decreased with the increase of overall slenderness ratio for irrespective of individual sections.
- For slenderness ratio between 70 and 120, the predominant failure mode was distortional buckling.

Further study is needed for the recalibration of DSM equations, the authors have planned to conduct experiments in near future to predict the ultimate strength of built-u battened columns. Furthermore, the other parameters that effect the ultimate strength of built-up batten columns will be undertaken in future.

References

Rondal J., Niazi M. (1990). Stability of Built-up Beams and Columns with Thin-walled Members, International Coll. Stability of Steel Structures, Budapest, Hungary,

Aslani F, Goel SC. (1991). An analytical criterion for buckling strength of built-up compression members, Engineering Journal 28(4): 159–68 (American Institute of Steel Construction, Inc., Chicago, IL).

Temple MC, Elmahdy G. (1993). An examination of the requirements for the design of built-up compression members in the North American and European standards, Canadian Journal of Civil Engineering 20(6): 895–909.

Schafer BW, Pekoz T. (1998). Computational modelling of cold-formed steel: characterizing geometric imperfections and residual stress, Journal of Constructional Steel Research 47: 193-210.

Sherman DR, Yura JA. (1998). Bolted double angle compression members. Journal Constructional Steel Research 46(1–3): 470–1.

Salem AH, El Aghoury M, Hassan S K and Amin A A , (2004) Post-Buckling Strength of Battened Columns Built from Cold-Formed Lipped Channels, Emirates Journal for Engineering Research 9(2): 117–125.

Stone TA, LaBoube RA. (2005). Behavior of cold-formed steel built-up I-sections, Thin-Walled Structures 43: 1805–1817.

AISC-LRFD. (2005). Load and resistance factor design specifications for structural steel buildings.

AS/NZS 4600(2005). Australian / New Zealand Standard – Cold Formed Steel Structures.

Sukumar S, Parameswaran P, Jayagopal LS. (2006). Local-,distortional-,and Euler buckling of thin walled built-up opencross-sections under compression, Journal of Structural Engineering – Madras 32(6): 447–54.

Dung M. Lue, Tsong Yen, Jui-Ling Liu. (2006). Experimental investigation on built-up columns, Journal of Constructional Steel research 62: 1325–1332.

AISI-S100 (2007). North American Specification for the Design of Cold-Formed Steel Structural members Specifications.

Young B. (2008). Design of cold-formed steel built-up closed sections with intermediate stiffeners, Journal of Structural Engineering 134:727.

Whittle J, Ramseyer C. (2009). Buckling capacities of axially loaded, cold-formed, built-up C-channels, Thin-Walled Structures 47: 190–201.

Hosseini HashemiB., JafariM.A. (2009). Experimental evaluation of elastic critical load in battened columns, Journal of Constructional Steel Research 65: 125-131.

El Aghoury MA, Salem AH, Hanna MT, Amoush EA. (2010). Experimental investigation for the behaviour of battened beam-columns composed of four equal slender angles, Thin-Walled Structures 48(9): 669–83.

El Aghoury MA, Salem AH, Hanna MT, Amoush EA., (2012). Ultimate capacity of battened columns composed of four equal slender angles, Thin-Walled Structures 63:175-185.

Georgieva, I.B., Schueremans, L., Pyl, L. (2012 a). Experimental investigation of built-up double-Z members in bending and compression, Thin-Walled Structures 53(4): 48-57.

Georgieva, I.B., Schueremans, L.,Pyl, L. (2012 b). Composed columns from cold-formed steel Z-profiles. Experiments and code-based predictions of the overall resistance, Engineering Structures 37(4): 125-134.

Anbarasu, M. and Sukumar, S., (2013). Study on the effect of ties in the intermediate length cold formed steel (CFS) columns", Structural Engineering and Mechanics, An International journal 46(3): 323-335.

Anbarasu, M. and Sukumar, S., (2014). Local/Distortional/Global Buckling Mode Interaction on Thin Walled Lipped Channel, Latin American Journal of Solids and Structures 11(8): 1363-1375.