Asian Journal of Science and Applied Technology (AJSAT)
Vol.3.No.1 2015 pp.59-65
available at: www.goniv.com
Paper Received :08-03-2015
Paper Accepted:20-03-2015
Paper Reviewed by: 1. R. Venkatakrishnan 2. R. Marimuthu
Editor : Prof. P.Muthukumar

STUDY ON BUILT-UP COLUMN ON COLD FORMED STEEL CHANNEL PROFILES

DR. SANKARAN.S, SHANMUGAM.L M.E.,AND KARTHIKEYAN N.M
PG Student, Structural Engineering
Skp Engineering College, Tiruvannamalai-606611

ABSTRACT

Two types of structural steel members are being used, namely hot rolled steel and cold formed steel. Use of cold formed steel for buildings and structures is gaining popularity in India for a decade. Hot rolled steel member design and behavior are well developed, whereas the cold formed steel member behavior and design is not developed fully. Cold-formed steel members usually display local–global buckling interaction which strongly affects the structural strength of columns and beams. The strength of steel cold-formed members is usually computed with the various design methods and the design standards. The present study is addressed to the analysis of the strength of steel cold-formed beams and columns with the different design methods. The design provisions developed in the various design methods of practices have been reviewed and a comparative study has been carried out on design flexural strength and compression capacity of various CFS sections. For this purpose design method and compared with Analysis results. This study presents the details of the studies carried out and the conclusions arrived.

Keywords: Cold formed steel, Built-up Lipped channel section, Direct strength method.

I.INTRODUCTION

Thin sheet steel products are extensively used in building industry, and range from purlins to roof sheeting and floor decking. 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 Sections or Cold Rolled Steel Sections. The thickness of steel sheet used in cold formed construction is usually 1 to 3 mm. Much thicker material up to 8 mm can be formed if pre-galvanized material is not required for the particular application. Yielding of the material, local buckling, distortional buckling, overall buckling and any possible interaction among these modes affect the strength of the cold-formed steel compression members. The study evaluates only accuracy of DSM in evaluating the strength of such members. The reliability analysis to arrive at the partial safety factor for the design is outside the scope of this paper. In addition, the applicability of DSM for hinged channel compression member, which suffer shifting of the effective section centroid axis in the post local buckling range. Because of cold-formed steel members are thin; the width-to-thickness ratios are large, especiallycompared to hot-rolled steel shapes. The thinmember's elements (flange, web, lip, etc...) may buckle locally before reaching yield stresses when the member is subject to compressive, flexural, shear, or bearing loads. Local buckling is a major consideration in design of cold-formed steel members.

2.FINITE ELEMENT METHOD

The finite element method (FEM) is a numerical procedure for analyzing structures and continua. FEM originated in a structural engineering and has been used in the fields of heat transfer, fluid flow, electric and magnetic fields and many others. FEM is the principle applicable to any boundary condition, geometrical and material variation. The finite element method has been proven as efficient and powerful approach to calculate the elastic buckling load and ultimate strength of cold-formed steel (CFS) structural members. A successful static analysis of the unstable collapse and post buckling behavior of CFS member requires the non- linear solution method consider geometric non linearity, material non linearity, boundary nonlinearity and residual stresses of the physical objects, as well as have the capability to deal with convergence, locking and other difficulties related to implementing the numerical algorithm. There are numerous commercial programs are ANSYS.

Fig 1 Finite Element Discretization

3. THEDIRECT STRENGTH METHOD, (DSM)

TheDirect Strength Design Method is initially proposed in 1988 and has been adopted by the North American Cold-Formed Steel Specifications in 2004 as an alternative to the traditional EWM to estimate the compression and the flexural member strength, the conventional effective width method, the need to divide the cross-section into many plate elements makes the calculations a complex process. Local buckling behavior and buckling interaction with global buckling modes of the member flexural, torsional, flexural–torsional and lateral-flexural buckling can also be included in design rules by means of the so-called direct methods. The DSM represents an important improvement of the design rules of cold-formed members because: (i) it is a muchmore understandable process of design, allowing to visualize the computational results of the buckling modes and its minimum critical loads and/or bending moments; (ii) it is based on less complex strength equations and strategies if compared with the EWM; (iii) and finally converges to more accurate theoretical strength resultsthan theEWM, when compared with experimental results.

DSM is based on calibrated strength curves for columns and beams which must be applied with the help of the results of plate buckling analysis of the member and taking into account its complete folded cross-section. To use this method, two quantities should be calculated first: the cross-sectional plastic resistance ($P_y=A_gF_y$) and the critical elastic buckling load for the buckling mode under consideration.

3.1DSM EQUATIONS NORTH AMERICAN SPECIFICATION AISI-S 100: 2007.

Capacity of the column is calculated as per DSM based upon three limit states. Global buckling, local buckling and distortional buckling. The Global, Local, Distortional slenderness ratios are considered correspondingly to calculated the section capacity as per DSM equations.

3.2 COLUMN DESIGN

The nominal axial strength, P_n, is the minimum of P_{ne}, P_{nl} and P_{nd} as given below. Forcolumns meeting the geometric and material criteria of Section 1.1.1.1, Ωc and φc are as follows:

USA and Mexico		Canada
Ωc(ASD)	φc (LRFD)	φc (LSD)
1.80	0.85	0.80

3.2.1 FLEXURAL, TORSIONAL, OR TORSIONAL-FLEXURAL BUCKLING.

The nominal axial strength, P_{ne}, for flexural, or torsional-flexural buckling is

For $\lambda c \le 1.5$ $P_{ne} = (0.658^{\lambda c^2})\,Py$ (Eq 3.2.1.1)

For $\lambda_c > 1.5$, $P_{ne} = (\frac{0.877}{\lambda c^2})\,P_y$ (Eq 3.2.1.2)

Where $\lambda_c = \sqrt{\frac{P_y}{P_{cre}}}$ (Eq 3.2.1.3)

$P_y = A_gF_y$ (Eq 3.2.1.4)

P_{cre}= Minimum of the critical elastic column buckling load in flexural, torsional,

3.2.2 LOCAL BUCKLING

The nominal axial strength, Pnl, for local buckling is

For $\lambda_l \le 0.776$, $Pnl = P_{ne}$ (Eq 3.2.1.5)

For $\lambda_l > 0.776$, (Eq 3.2.1.6)

$P_{nl} = (1\text{-}0.15(\frac{P_{crl}}{P_{ne}})^{0.4})(\frac{P_{crl}}{P_{ne}})^{0.4}P_{ne}$ (Eq 3.2.1.7)

Where $\lambda_l = \sqrt{\frac{p_{ne}}{P_{crl}}}$ (Eq 3.2.1.8)

P_{crl} = Critical elastic local column buckling load

P_{ne} is defined in Section3.2.1.1

3.2.3 DISTORTIONAL BUCKLING

The nominal axial strength, Pnd, for distortional buckling is

For $\lambda_d \le 0.561$, $P_{nd} = P_y$ (Eq 3.2.1.9)

For $\lambda_d > 0.561$, (Eq 3.2.1.10)

$P_{nd} = (1\text{-}0.25(\frac{P_{crd}}{P_y})^{0.6})(\frac{P_{crd}}{P_y})^{0.6}P_y$ (Eq 3.2.1.11)

Where $\lambda_d = \sqrt{\frac{p_y}{P_{crd}}}$ (Eq 3.2.1.12)

P_{crd} = Critical elastic distortional column buckling load

P_y is given inEq. 3.2.1.4

In the above equations: P_y is the squash load (axial resistance) of the cross-section; P_{cre} is the critical elastic global buckling load; P_{crd} is the critical elastic distortional buckling load; P_{crl} is the critical elastic local buckling load; λ_c, λ_d and λ_l are the slenderness for global, distortional and local buckling modes, respectively P_{ne}, P_{nl} and P_{nd}. are the column axial strength for global, distortional and local buckling modes, respectively. The axial strength of the column is the minimum of P_{ne}, P_{nl} and P_{nd}.

4. SPECIMEN DETAILS

The single section dimensions were selected based on North American Specification for the design of cold-formed steel structures member-2007

Fig2 Section Geometric Details

Channel Section	Dimension mm					Battened Size mm			Length mm
	h_o	bo	d	t	R_i	b	d	t	L
150CS50x25x4	150	50	25	4	6	170	100	4	3000
150CS50x25x5	150	50	25	5	7.50	170	100	5	3000
180CS50x25x4	180	50	25	4	6	190	100	4	3000
180CS50x25x5	180	50	25	5	7.50	190	100	5	3000
200CS50x25x4	200	50	25	4	6	210	100	4	3000
200CS50x25x5	200	50	25	5	7.50	210	100	5	3000

5. DESIGN OF PROPOSED SECTION

Capacity of the column is calculated as per DSM based upon three limit states. Global buckling, local buckling and distortional buckling. Design of proposed section 150CS50X25X4 [North American Specification AISI-S 100: 2007]

5.1 Section Properties.

$A = 2400mm^2$	$R_{xx} = 55.8mm$
$C_y = 16.3mm$	$R_{yy} = 18.2mm$
$I_{xx} = 6.62X10^6mm^4$	$Xo = 42.7mm$
$I_{yy} = 9.73X10^6mm^4$	$J = 0.522cm^4$
$R_i = 6mm$	$C_w = 1750cm^6$

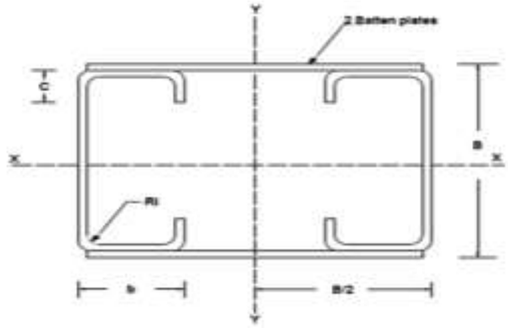

Fig 3Cross section details of battened column

5.2 FLEXURAL, TORSIONAL, OR TORSIONAL-FLEXURAL BUCKLING

The nominal axial strength, P_{ne}, for flexural, or torsional-flexural buckling is

$$F_e = \left(\frac{\sigma t \sigma ex}{\sigma t + \sigma ex}\right) \qquad \text{(Eq.5.2.1)}$$

$$\sigma_t = \frac{1}{Aro^2}\left[GJ + \frac{\pi^2 ECw}{KtLt}\right] \qquad \text{(Eq.5.2.2)}$$

$$r_o = (r_x)^2+(r_y)^2+(2X_o)^2 \qquad \text{(Eq.5.2.3)}$$

$$r_o = (55.8)^2+(63.65)^2+(42.7)^2$$

$$r_o = 120.24$$

$$\sigma_t = 35.64 \text{ N/mm}^2$$

Where

F_e = Elastic buckling stress

r_o= Polar radius of gyration of cross section about shear centre

σ_t = Torsional buckling stress

G = Shear modulus of steel

J = Saint-Venant torsion constant

K_t= Effective length factors for twisting

L_t = Unbraced length of member for twisting

$$\sigma_{ex} = \frac{\pi^2 E}{\left(\frac{KxLx}{rx}\right)^2} = 709.22 \text{ N/mm}^2 \qquad \text{(Eq.5.2.4)}$$

$$F_e = \frac{(35.64)(709.22)}{(35.64)+(709.22)} = 34.04 \text{N/mm}^2$$

$$P_{cre} = A_g F_e \qquad \text{(Eq.5.2.5)}$$

$$P_{cre} = (3000)(34.04) = 81.44 \text{ KN}$$

$$\lambda_c = \sqrt{\frac{P_y}{P_{cre}}} \leq 1.5 \quad P_y = A_g F_y \qquad \text{(Eq.5.2.6)}$$

$$P_y = (2400X232) = 556.8 \text{ KN}$$

$$\lambda_c = \sqrt{\frac{P_y}{P_{cre}}} = \sqrt{\frac{556.8}{81.44}} = 2.61>1.5$$

$$P_{ne} = \left(\frac{0.877}{\lambda c^2}\right) P_y \qquad \text{(Eq.5.2.7)}$$

$$P_{ne} = 71.42 \text{ KN}$$

Where

P_{cre}= Minimum of the critical elastic column buckling load in flexural, torsional.

5.3 LOCAL BUCKLING

The nominal axial strength, Pnl, for local buckling

$$F_{cr} = \frac{K\pi^2 E}{12(1-\mu 2)}\left(\frac{t}{w}\right)^2 \qquad \text{(Eq.5.3.1)}$$

Where

F_{cr} = Plate elastic buckling stress
F_y = Yield stress
K = Plate buckling coefficient
Stiffened elements K value is 4
Unstiffened elements K value is 0.43

For Web K = 4

$$F_{cr} = \frac{4\pi^2 2.074X105}{12(1-(0.3)2)}\left(\frac{4}{150}\right)^2$$

F_{cr} = 534.08 N/mm^2

$$\lambda = \sqrt{\frac{Fy}{F_{cr}}} \leq 0.673 b = w \text{ when } \lambda \leq 0.673 \qquad \text{(Eq.5.3.2)}$$

$$\lambda = \sqrt{\frac{232}{534.08}} = 0.659 \leq 0.673$$

Web= 150 mm

For Flange K = 4

$$F_{cr} = \frac{4\pi^2 2.074X105}{12(1-(0.3)2)}\left(\frac{4}{50}\right)^2$$

F_{cr} = 4805.88 N/mm^2

$$\lambda = \sqrt{\frac{232}{4805.88}} = 0.221 \leq 0.673$$

Flange = 50mm

For Lip K = 0.43

$$F_{cr} = \frac{0.43\pi^2 2.074X105}{12(1-(0.3)2)}\left(\frac{4}{25}\right)^2$$

F_{cr} = 2066.43 N/mm^2

$$\lambda = \sqrt{\frac{232}{2066.43}} = 0.335 \leq 0.673$$

Lip = 25 mm

$$A_e = 2[t(h+2b+2d)] \qquad \text{(Eq.5.3.3)}$$

A_e = 2400 mm^2

$$P_{crl} = A_e F_{cr}(min) \qquad \text{(Eq.5.3.4)}$$

P_{crl} = (2400)(534.08)
P_{crl} = 1281.80 KN

Where
P_{crl} = Critical elastic local column buckling load

$$\lambda_l = \sqrt{\frac{p_{ne}}{P_{crl}}} \leq 0.776 \qquad \text{(Eq.5.3.5)}$$

$$\lambda_l = \sqrt{\frac{71.42}{1281.80}} = 0.236 \leq 0.776$$

$P_{nl} = P_{ne}$ =71.42 KN

Where
P_{nl} = Nominal axial strength for local buckling
P_{ne}=Nominal axial strength for Flexural – Torsional buckling

5.4 DISTORTIONAL BUCKLING

The nominal axial strength, P_{nd}, for distortional buckling is

$$F_d = \frac{\alpha K d\pi^2 E}{12(1-\mu 2)}\left(\frac{t}{w}\right)^2 \qquad \text{(Eq.5.4.1)}$$

$$K_d = 0.1\left[\frac{bc sin\theta}{ht}\right]^{1.4} \leq 8.0 \qquad \text{(Eq.5.4.2)}$$

$$K_d = 0.1\left[\frac{(50)(25)sin90}{150x4}\right]^{1.4} \leq 8.0$$

K_d = 0.28

$$F_d = \frac{1(0.28)\pi^2 E}{12(1-(0.3)2)}\left(\frac{4}{50}\right)^2$$

F_d= 336.41 N/mm^2

Where
F_d =Distortional buckling stress
K_d =Plate buckling coefficient for distortional buckling

$$P_{crd} = A_g F_d \qquad \text{(Eq.5.4.3)}$$

P_{crd} = (2400)(336.41)
P_{crd} = 807.38 KN

Where
P_{crd} = Critical elastic distortional column buckling load

$$\lambda_d = \sqrt{\frac{p_y}{P_{crd}}} \leq 0.561 \qquad \text{(Eq.5.4.4)}$$

$$\lambda_d = \sqrt{\frac{556.8}{807.38}} = 0.830 \leq 0.561$$

$$P_{nd} = (1-0.25(\frac{P_{crd}}{P_y})^{0.6})(\frac{P_{crd}}{P_y})^{0.6} P_y \qquad \text{(Eq.5.4.5)}$$

$$P_{nd} = [\ (1-0.25(\frac{807.38}{556.8})^{0.6}](\frac{807.38}{556.8})^{0.6}(556.8)$$

P_{nd} = 478.41 KN

Where
P_{nd} = Nominal axial strength for distortional buckling

Predicate column capacity:
P_n = [P_{ne}, P_{nl}, P_{nd}]KN
P_n= [71.42, 71.42, 478.41]KN
$P_{n(min)}$ = 71.42 KN$P_{n(max)}$ = 478.41KN

Table1.Predicate Column Capacityof Sections

Sections	ColumnCapacity P_n=[P_{ne}, P_{nl}, P_{nd}] KN	P_n(min) KN	P_n(max) KN
150CS50x25x4	[71.42, 71.42, 478.41]	71.42	478.41
150CS50x25x5	[107.76, 107.76, 618.99]	107.76	618.99
180CS50x25x4	[90.18, 90.18, 483.51]	90.18	483.51
180CS50x25x5	116.56, 116.56, 634.68	116.56	634.68
200CS50x25x4	[100.05, 100.05, 474.27]	100.05	474.27
200CS50x25x5	[130.14, 130.14, 638.40]	130.14	638.40

6. NUMERICAL INVESTIGATION

Most of the engineering problems today make it necessary to obtain approximate numerical solutions to problems rather than exact closed form solutions. The basic concept behind the finite element analysis is that structure is divided into a finite number of elements having finite dimensions and reducing the structure having infinite degrees of freedom to finite degrees of freedom. Then original structure is assemblage of these elements connected at a finite number of joints called nodal points (Nodes). For the finite element analysis advanced software ANSYS V 13.0 were used.

6.1 ELEMENT USED FOR FEM

SOLID185 is used for 3-D modeling of solid structures. It is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. The element has plasticity, hyper elasticity, stress stiffening, creep, large deflection, and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyper elastic material**.**The proposed section is **150CS50X25X4**

*6.2 Modeling***:**

Mechanical APDL method is used to built-up FE model. Above figure shows FE model created in ANSYS

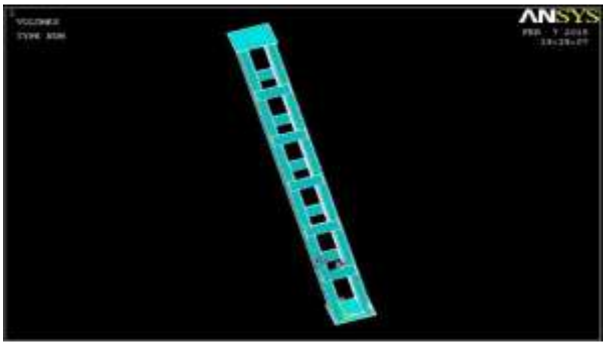

Fig 4 Finite Element model in ANSYS

6.3 Meshing

The important step of FE method is meshing, size of mesh depends upon parent material to be modeled also the degree of accuracy required. Results can be significantly changed if proper meshing size is not decided. For this FE model meshing size is kept 1.0following figure shows meshed model in ANSYS.

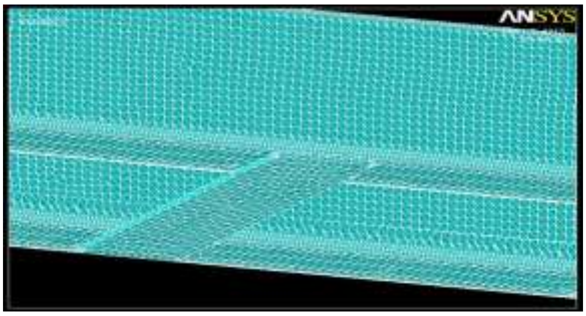

Fig 5Meshing of FE model

7. ANALYSIS AND RESULT PREDICTION

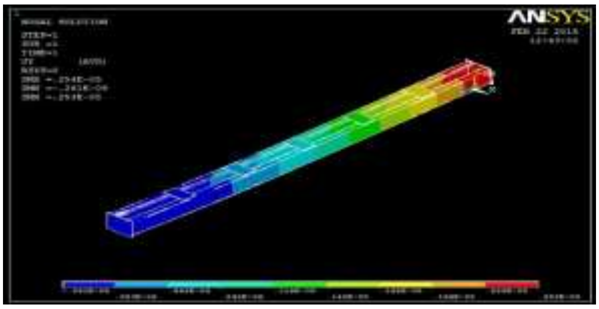

Fig 6 Deformed shape of FE model

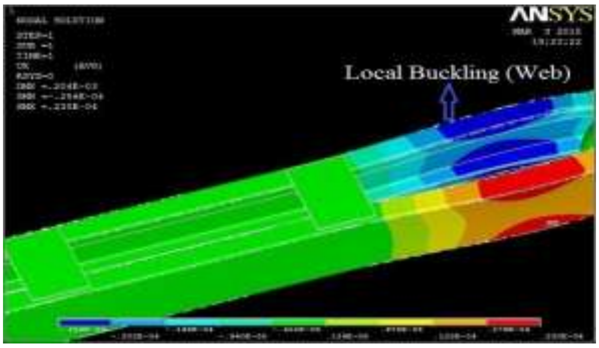

Fig 7Local Buckling of FE model

With the use of static analysis strength have been predicted of cold formed steel sections. Figure above shows deformed shape of FE Model in ANSYS V 13.0

8. RESULTS AND DISCUSSION

Finite element method result for Cold Formed steel sections

Table2Load and deflection Cold formed steel channel section 150CS50x25x4

Theoretical Load (KN)	Deflection (mm)	Analysis Load(KN)	Deflection (mm)	P_{THE}/P_{NUM}
478.41	0.08035	490.63	0.08242	0.97

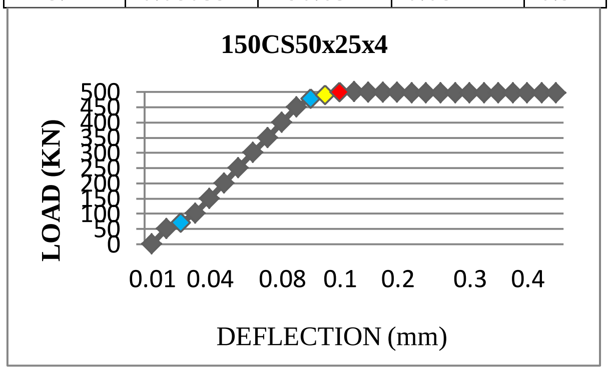

Fig8 Load v/s Deflection curve for 150CS50x25x4 by ANSYS

Table3Load and deflection Cold formed steel channel section 150CS50x25x5

Theoretical Load (KN)	Deflection (mm)	Analysis Load(KN)	Deflection (mm)	P_{THE}/P_{NUM}
618.99	0.08484	636.75	0.08730	0.96

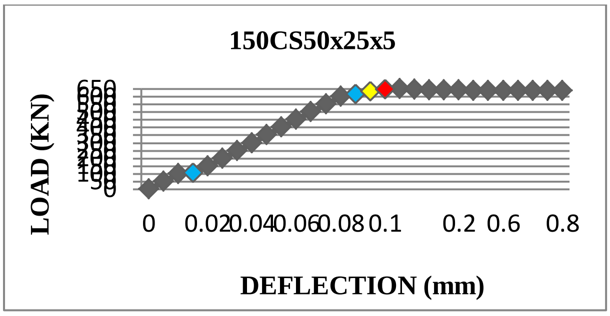

Fig 9 Load v/s Deflection curve for 150CS50x25x5 by ANSYS

Table 4 Load and deflection Cold formed steel channel section 180CS50x25x4

Theoretical Load (KN)	Deflection (mm)	Analysis Load(KN)	Deflection (mm)	P_{THE}/P_{NUM}
483.51	0.05441	512.38	0.05765	0.94

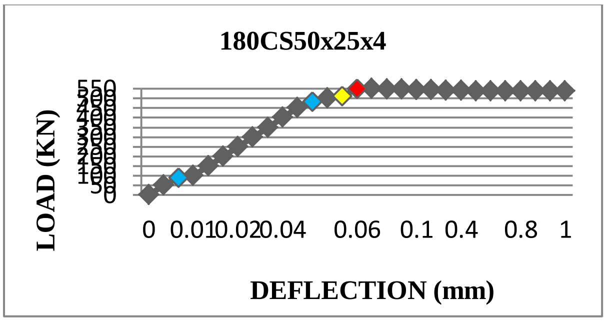

Fig10 Load v/s Deflection curve for 180CS50x25x4 by ANSYS

Table 5 Load and deflection Cold formed steel channel section 180CS50x25x4

Theoretical Load (KN)	Deflection (mm)	Analysis Load(KN)	Deflection (mm)	P_{THE}/P_{NUM}
634.68	0.05712	656.25	0.06063	0.96

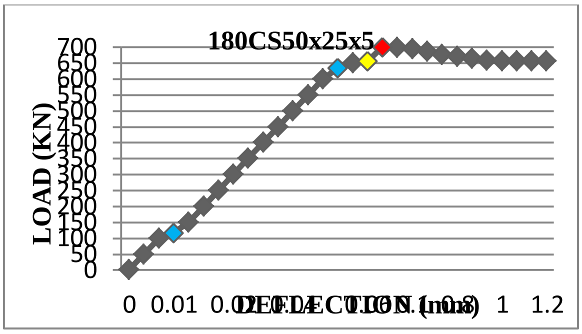

Fig 11Load v/s Deflection curve for 180CS50x25x5 by ANSYS

9.CONCLUSION

The Direct Strength Method is a new design methodology for cold formed steel members. The behavior of Cold Formed Steel battened columns with lip were studied analytically and results were compared with the predicated nominal compressive strength using AISI S100-2007 code. The Local buckling failure was observed web of channel column with lip. AISI standard provided a good agreement for the column of low plate width to thickness ratio and high plate width to thickness ratio.

REFERENCES

1. *Beshara.B and LaBoube.R.A (2001) "Lateral braced C-sections under bending'', Journal of thin walled sections vol.39, pp.827-839.*
2. *Cheng Yu and Weiming Yan (2011) Effective Width Method for determining distortional buckling strength of cold-formed steel flexural C and Z sections. Elsevier Thin-Walled Structures 49 (2011) 233–238.*
3. *Eduardo de Miranda Batista (2010) Effective section method: A general direct method for the design of steel cold-formed members under local–global buckling interaction. Elsevier Thin-Walled Structures 48 (2010) 345–356.*
4. *I.Georgieva, L. Schueremans, L. Vandewalle and L. Pyl (2012) Design of built-up cold-formed steel columns according to the direct strength method. Procedia Engineering 40 (2012) 119 – 124*
5. *Haiming Wang, YaochunZhung (2009) 'Bending strength and behaviour of cold-formed steel C-section flexural members with upright, inclined and complex edge stiffeners'.*
6. *Hong-GuangLuo, Yao-JieGuo and Shi-Cheng Ma (2006) "Distortional buckling of thin-walled inclined lipped channel beams". Journal of Structural Engineering vol.189, pp. 104-112.*
7. *IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE) ISSN: 2278-1684 PP: 27-36.Sarvade*

S.M. (2010) "Load carrying capacity of innovative Cold Formed Steel column"
8. *A.Landesmann and D. Camotim (2013) On the Direct Strength Method (DSM) design of cold-formed steel columns against distortional failure. Elsevier Thin-Walled Structures 67 (2013) 168–187.*
9. *Luis Laim, Joaao Paulo C. Rodrigues and Luis Simoes da Silva (2013) Experimental and numerical analysis on the structural behavior of cold-formed steel beams. Elsevier Thin-Walled Structures 72 (2013) 1–13.*
10. *M. Macdonalda and M.A. Heiyantuduwa (2012) A design rule for web crippling of cold-formed steel lipped channel beams based on nonlinear FEA. Elsevier Thin Walled Structures 53 (2012) 123–130.*
11. *E. Magnucka-Blandzi and K. Magnucki (2011) Buckling and optimal design of cold-formed thin-walled beams: Review of selected problems. Elsevier Thin-Walled Structures 49 (2011) 554–561.*
12. *Olli Kaitila (2006) "Steel structures at high temperatures and to develop finite element analysis procedures that realistically model the behavior of cold-formed thin-walled steel members in fire conditions". Journal of thin walled steel sections vol14.pp456-487.*
13. *B.W. Schafer., 2008 "The Direct Strength Method of cold-formed steel member design", Journal of Constructional Steel Research 64 (2008) 766–778.*
14. *Vijayasimhan M, Marimuthu V., Palani G.S. and Rama Mohan Rao P (2013) "Comparative Study on Distortional Buckling Strength of Cold-Formed Steel Lipped Channel Sections"Research Journal of Engineering Sciences Vol. 2(4),10-15, April (2013).*
15. *AISI-S 100:2007 – North American Specification for the design of Cold –formed steel structural members Specifications.*
16. *AS/NZ 4600:2005 – Australian / New Zealand Standard – Cold Formed Steel Structures.*
17. *IS: 801-1975 – Design of Cold formed Steel structures.*
18. *IS: 811-1987, "Indian Standard Specification for Cold-Formed Light Gauge Structural Steel Sections" Bureau of Indian Standards, New Delhi. India.*