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NUMERICAL INVESTIGATION ON COLD FORM LIPPED CHANNEL STEEL SECTION UNDER ECCENTRIC LOADING

S.Chandravadani[1], Nithya Thomas[2], Hareesh.M[3]

[1,2]*M.Tech (Sructural Engineering), SMBS , VIT University, Vellore, Tamilnadu, (India)*

[3]*Assistant Professor, SMBS, VIT University, Vellore, Tamilnadu , (India)*

ABSTRACT

A numerical investigation on behaviour and nominal strength of cold formed steel lipped channel columns subjected to eccentric loading along major axis is presented in this paper. The strength capacity is arrived for cross section dimension with varying eccentricity load under simply supported condition through numerical analysis FEA package ABAQUS 6.10. The same sections are designed based on effective width as per AISI S-100-2013 and direct strength method specification and the strengths are compared with nominal strength attained through numerical analysis.

I. INTRODUCTION

Structural steel is classified into hot rolled steel and cold form steel based on manufacturing process. Cold formed steels are extensively used in many fields in bridges, buildings, car bodies, storage racks, transmission towers, highway products, railway coaches, etc. These are made by bending a flat sheet at optimum temperature to support more load than the flat sheets using cold rolling machines, press brake or bending brake operation. On comparing with hot rolled steel, cold form steel offers lightness, high strength an stiffness, lower weight due to high strength to weight ratio, economy in transportation, erection and installation. .

A tremendous increase in the use of cold form steel around the globe with regards to many improvements in construction technologies and related standards in recent years. However, when compared to the conventional hot-rolled steel members, the cold-formed steel sections are usually slender and not doubly symmetric. Hence, the thin-walled cold formed steel structures is characterised by buckling behaviours such as local buckling, distortional buckling or flexural torsional buckling due to stress that are lower than yield stress when compression and shear bending forces are applied. These buckling generally leads to difficulties in design considerations while computing strength capacity of members.

Thus the objective of present paper is to develop finite element model for eccentrically loaded column along its major axis.The nominal strength of eccentric column are determined with DSM equations and results are compared with FEM analysis.

II. LITERATURE REVIEW

Mohammad RezaHaidarali et al (2011) have developed on two series of finite element models and examined the buckling behaviour of a laterally restrained CFS. They worked on sections considering material and geometrical nonlinearities: Thus they developed C-section beams. He checked for local/distortional buckling combination and local buckling for their sections.

Schafer et al. (2010) provided an overview of the computational modelling for both elastic buckling and nonlinear collapse analysis, of cold-formed steel members. Software such as CUFSM and ABAQUS were used and methods on how to fully compare finite strip and finite element solutions were elaborated with examples. The importance of residual stresses, boundary conditions , material modelling, element discretization, element choice and solution controls in collapse modelling of cold-formed steel were also highlighted.

EduardodeMirandaBatista (2009) studied on the recent developments of cold-formed column members which are affected by local and global buckling interaction. The formulations of the effective width method (EWM) and direct strength method (DSM) were used. For easier computation of resistance offered by cold formed columns were done by integrating the effective width method and the direct strength method. As such a common procedure is made for both the available methods. They proposed a method by eliminating all problems to access the program used for finding the local buckling values of a CFS sections. For this purpose the original formulas of Effective width method were calibrated by results obtained from experiments

J. Rondal (2005) made a report which highlighted on some recent progresses that took place in the field of CFS members. Importance was given mainly for distortional buckling and in recent development of new types of joints.

B.W. Schafe (2008) compared the Direct Strength Method with the Effective Width Method is provided. This increases the computational stability analysis into the design process. Thus his works enhanced the reliability of direct strength method analysis that's used for beams and columns. Current & on-going research that's aims to extend the DSM is reviewed and complete references provided. DSM was formally adopted in North American cold-formed steel design specifications which are an alternative to the traditional EWM. This paper provides the DSM equations for the design of columns and beams and adopted in the North American Specification.

Put, Pi & Trahair (1999) conducted 34 bending & torsion tests on unbraced simply supported cold formed steel channel beam sections which are loaded eccentrically at mid span. A conclusion on strength of unbraced C CFS was based on the eccentricity of loads. The failure occurred by local buckling. Conclusions made was as the eccentricity increases strength of the sections reduced

III. METHODOLOGY

3.1 Parametric Study

The lipped channel section of dimension 50x50x15 with the thickness of 1.0mm is considered. The section was analysed for 3 effective lengths [750, 1000, 1250 mm] and 4 eccentricities (4.63, 9.27, 13.898, 18.531) along the major axis. The column analysis was carried out by adopting pinned boundary conditions. The material properties of following section are

Youngs modulus,E=210Gpa

Modulus of rupture,G=453885N/mm^2

Yield steel,F_Y=250Mpa

Poissons ratio, μ=0.3

Ultimate stress,f_u=260Mpa with corresponding strain, ε = 0.2%.

3.2 Finite Strip Method

CUFSM, a software program that employs the semi analytical finite strip method to provide the solution for thin walled members. Unlike other materials, CUFSM is widely used for cold formed steel members to find all the buckling mode behaviour and results. The lipped channel section is modelled and analysed .Load factor is ratio between elastic buckling load and first yield load is checked out to determine the nominal strength of the section P_n, M_n.

3.3 Finite Element Modelling

ABAQUS, a finite element analysis software used for modelling the cold formed channel section. A solid plate is also modelled for transfer of load. The eccentric loading is applied as a concentrated load on the top of the plate. The end plates are arranged in such a manner that both the centroids of the end plates and the channel section coincide. For constraints ties are used. The main objective is to determine the Ultimate load. In order to establish the buckling modes eigen values are analysed. Later a mesh size of 10mm is adopted Section analysis was carried out in such a way that except the translational DOF at the top and all other DOF were restrained at both the ends of the section. End plates were provided at the top of the section to introduce eccentric loading to the channel section.

3.4 Effective Width Method

The EWM is a primary design approach for the CFS members in the AISI Specifications including the North American Specification for the Design of CFS Structural Members. However, the Effective Width Method does not have sufficient procedures for predicting the distortional buckling failure. This design method was also carried out to make a comparative study of the section by finding the nominal load and nominal moment and then using them in the interaction equation to determine its safety.

The effective width method is carried out separately for beam design and column design.

FOR COLUMN DESIGN

For column design;

$$P_n = A_e F_n$$

where,

A_e = Effective area of the section.

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right]$$

$$\beta = 1 - \left(\frac{x_0}{r_0}\right)^2$$

$$\sigma_{ex} = \frac{\pi^2 E}{\left(\frac{KL}{r_x}\right)^2}$$

$$\sigma_t = \frac{1}{A r_0^2}\left[GJ + \frac{\pi^2 E C_w}{(KL)^2}\right]$$

$$r_0 = \sqrt{r_x^2 + r_y^2 + x_0^2}$$

$$J = \frac{1}{3}\left((l_1 t_1)^3 + (l_2 t_2)^3 + \cdots + (l_3 t_3)^3\right)$$

x_0 = Distance from the shear center to the centroid of the section.

r_x, r_y = radii of gyration

$$\lambda_c = \sqrt{\frac{F_y}{F_e}}$$

For $\lambda_c > 1.5$;

$$F_n = \left(0.658^{\lambda_c^2}\right) F_y$$

Distortional buckling strength of column;

For $\lambda_d \leq 0.561$,

$$P_n = P_y$$

For $\lambda_d > 0.561$,

$$P_n = \left[\left[1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right]\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y\right]$$

$$\lambda_d = \sqrt{\frac{P_y}{P_{crd}}}$$

$$P_y = A_g F_y$$

$$P_{crd} = A_g F_d$$

A_g = Gross area of the cross-section.

F_d = Elastic distortional buckling stress.

FOR THE BEAM DESIGN

For beam design;

$$M_n = S_c F_c$$

where,

Sc = Elastic section modulus of effective section calculated relative to extreme compression fiber at F_c.

F_c shall be determined as follows;

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For $F_e \geq 2.78F_y$;

$$M_n = M_y$$

For $2.78F_y > F_e > 0.56F_y$;

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right)$$

For $F_e \leq 0.56F_y$;

$$F_c = F_e$$

where,

F_y = Design yield stress

F_e = Elastic critical lateral-distortional buckling stress.

$$F_e = \frac{C_b \pi^2 E d I_{yc}}{S_f (K_y L_y)^2}$$

where,

I_{yc} = Moment of inertia of compression portion of section about centroidal axis of entire section parallel to web, using full unreduced section.

d = Depth of section

S_f = Elastic section modulus of full unreduced section relative to extreme compression fiber in first yield.

C_b shall be permitted to be conservatively taken as unity for all cases.

Distortional buckling strength of beam;

For $\lambda_d \leq 0.673$,

$$M_n = M_y$$

For $\lambda_d > 0.673$,

$$M_n = \left[\left[1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right]\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y\right]$$

$$\lambda_d = \sqrt{\frac{M_y}{M_{crd}}}$$

$$M_y = S_{fy}F_y$$

$$M_{crd} = S_f F_d$$

S_{fy} = Elastic section modulus of full unreduced section relative to extreme fiber in first yield.

S_f = Elastic section modulus of full unreduced section relative to extreme compression fiber.

F_d = Elastic distortional buckling stress.

The minimum value of P_n and M_n from among the flexural-torsional strength as well as the distortional strength for both the column and beam respectively, will be considered as the nominal design strengths of the section.

3.5 DSM Of Beam-Column

Using this method investigation can be done on a cross-section for a given axial load or bending moment. Instead of separately finding P and M values, all the 3 buckling values such as local, distortional and global bucklings are found out for P and M combinations. As a result this anlysis gives a different behaviour compared to the interaction equation .

From the ABAQUS analysis the ultimate load (Pu) and ultimate moment (Mu) are used and the DSM design steps were carried out;

$$x = \frac{C_m M_u}{\alpha M_y}; y = \frac{P_u}{P_y}$$

$$\beta_u = \sqrt{x^2 + y^2}$$

$$\beta_y = \gamma \beta_u$$

where,

C_m = Moment gradient factor.

α = Moment amplification factor.

The minimum of the nominal strengths were taken as the nominal axial strength of the column. The steps carried out are;

Global buckling;

$$\lambda_c = \sqrt{\frac{\beta_y}{\beta_{cre}}}$$

$$\beta_{ne} = \left| \begin{array}{ll} 0.658^{\lambda_c^2} \beta_y, & \lambda_c \leq 1.5 \\ \dfrac{0.877}{\lambda_c^2} \beta_y, & \lambda_c > 1.5 \end{array} \right.$$

Local buckling;

$$\lambda_l = \sqrt{\frac{\beta_{ne}}{\beta_{crl}}}$$

$$\beta_{nl} = \left| \begin{array}{ll} \beta_{ne}, & \lambda_l \leq 0.776 \\ \left[\left[1 - 0.15\left(\dfrac{\beta_{crl}}{\beta_{ne}}\right)^{0.4}\right]\left(\dfrac{\beta_{crl}}{\beta_{ne}}\right)^{0.4} \beta_{ne}\right], & \lambda_l > 0.776 \end{array} \right.$$

Distortional buckling;

$$\lambda_d = \sqrt{\frac{\beta_y}{\beta_{crd}}}$$

$$\beta_{nd} = \left| \begin{array}{ll} \beta_y, & \lambda_d \leq 0.561 \\ \left[\left[1 - 0.25\left(\dfrac{\beta_{crd}}{\beta_y}\right)^{0.6}\right]\left(\dfrac{\beta_{crd}}{\beta_y}\right)^{0.6} \beta_y\right], & \lambda_d > 0.561 \end{array} \right.$$

β_n = min of $\beta_{ne}, \beta_{nl}, \beta_{nd}$

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IV. RESULTS AND DISCUSSIONS

Upon the study of three different length lipped columns with four eccentricities the following conclusions were made:

From ABAQUS-6.10 the ultimate load is determined using maximum LPF (Load Participation Factor) multiply by corresponding Eigen value under non-linear compressive factor. The modes are represented by local (L), distortional (D), and global buckling (G). Thus the result of analysed model is mostly under distortional buckling.

Table-1 DSM design values along with their modes failure

L	P_U	E	M_U	B_U	B_Y	B_{CRE}	B_{CRL}	B_{CRD}	B_{NE}	B_{NL}	B_{ND}	MODE
750mm	37352.1	4.6	172940.2	0.57	0.54	0.82	0.64	0.99	0.41	0.40	0.49	D
	31725.8	9.3	294098.3	0.50	0.46	0.74	0.56	0.89	0.35	0.35	0.43	D
	26245.2	13.9	364756.5	0.44	0.41	0.67	0.50	0.80	0.32	0.31	0.38	D
	22985.9	18.5	425950.9	0.40	0.38	0.65	0.48	0.78	0.30	0.29	0.36	D
1000mm	36631.8	4.6	169605.5	0.56	0.52	0.82	0.64	0.87	0.40	0.40	0.47	D
	30954.7	9.3	286949.7	0.49	0.44	0.74	0.57	0.90	0.35	0.34	0.42	D
	26425.2	13.9	367257.3	0.44	0.40	0.69	0.52	0.83	0.31	0.31	0.38	D
	22709.2	18.5	420823.4	0.41	0.37	0.66	0.46	0.79	0.29	0.29	0.36	D
1250mm	35872.9	4.6	166091.3	0.55	0.51	0.82	0.64	0.99	0.39	0.39	0.47	D
	30219.5	9.3	280135	0.49	0.43	0.75	0.58	0.91	0.34	0.34	0.41	D
	25745.9	13.9	357817.1	0.44	0.39	0.69	0.53	0.84	0.31	0.31	0.37	D
	21887.4	18.5	405594.7	0.40	0.36	0.68	0.51	0.82	0.29	0.30	0.35	D

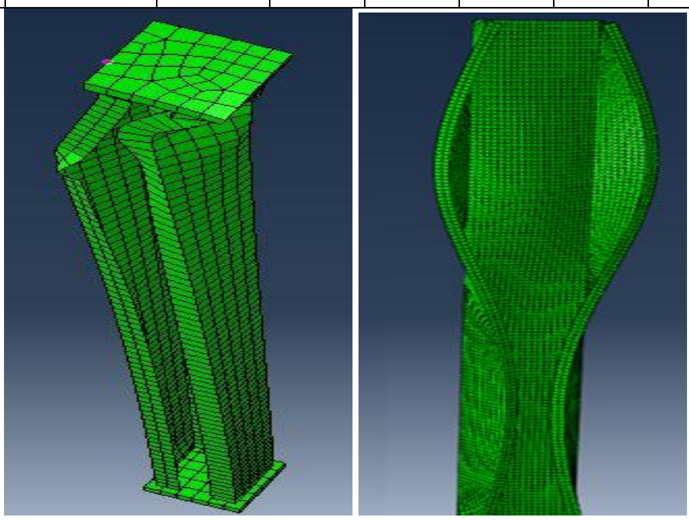

Fig-1 Distortional Buckling

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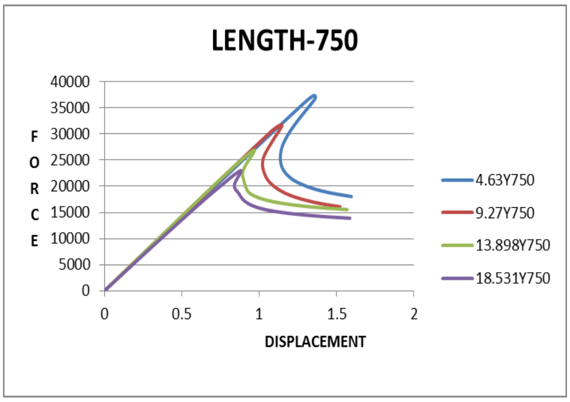
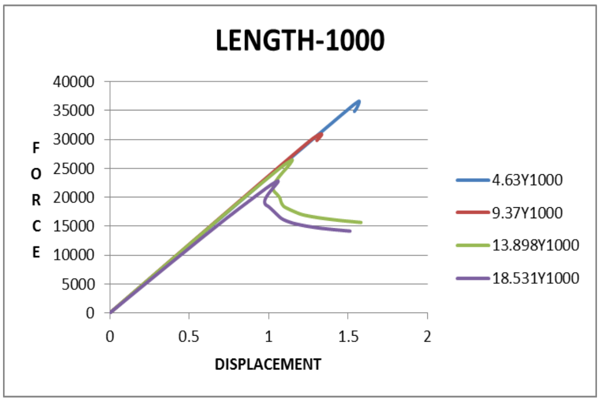
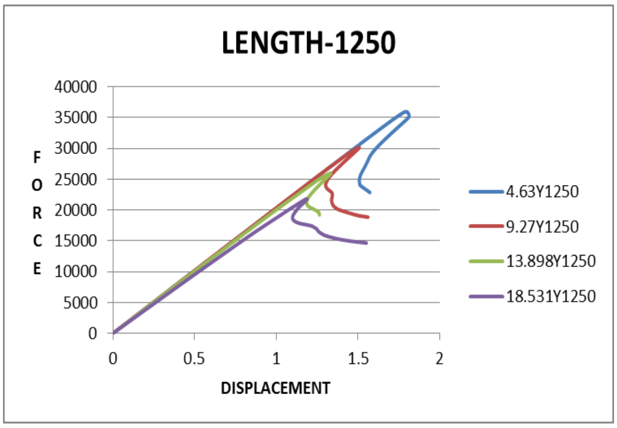
The graph is plotted between force and displacement value that are obtained from FEM analysis for different length with same cross-section to understand the stiffness of sections. It is observed that increase in displacement value increases the force to a point and gradually reduces. For all the different cases, the maximum force is obtained for the eccentricity value of 4.63mm.







From the interaction equation, the effective width method load is determined for various length by eaquating and corresponding percentage variation between ABAQUS and effective width method load is found.

	L=750	L=1000	L=1250
e=4.63	24358	23710	22963.42
e=9.27	20915.8	20199.27	19366.35
e=13.89	18372.17	17750.52	17053.58
e=18.53	16387.32	15858.06	15247.00

Table-2 Effective Width Method

	L=750	L=1000	L=1250
e=4.63	34.79%	35.28%	36.15%
e=9.27	34.08%	34.75%	35.78%
e=13.89	31.72%	32.92%	34.07%
e=18.53	28.72%	30.18%	29.10%

Table-3 Percentage Variation Of Load

V. CONCLUSION

The lipped channel column for various length and considered eccentricity was modelled in ABAQUS and its ultimate load is determined.Using Beam-column procedure the eccentric load column is designed where it proves that increase in length reduces the strength capacity.The results of ABAQUS and Effective Width method were compared where it shows lesser strength capacity in EWM.Therefore it concludes that effective Width method under estimates the ultimate load capacity of column. The code specifications are yet to be revised for design of eccentric loaded cold form steel sections. Studies on accurate prediction of results are to be conducted with the help of more and more researches in this field.

REFERENCES

[1]. B. W. Schafer, "Review: The Direct Strength Method of cold formed steel member design," *Journal of Constructional Steel Research*, vol. 64, no. 7-8, pp. 766–778, 2008.

[2]. Direct Strength Method (DSM) Design Guide, *Committee on Specifications for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, January, 2006.

[3]. I. El-Sheikh, E. M. A. El-Kassas, and R. I. Mackie, "Performance of stiffened and unstiffened cold-formed channel members in axial compression," *Engineering Structures*, vol. 23, no. 10, pp. 1221–1231, 2001.

[4]. T. H. Miller and T. Pekoz, "Load-eccentricity effects on cold formed steel lipped-channel columns," *Journal of Structural Engineering*, vol. 120, no. 3, pp. 805–823, 1994.

[5]. Young and J. Yan, "Finite element analysis and design of fixed ended plain channel columns," *Finite Elements in Analysis and Design*, vol. 38, no. 6, pp. 549–566, 2002. [7] B. Young and K. J. R. Rasmussen, "Behaviour of cold-formed singly symmetric columns," *Thin-Walled Structures*, vol. 33, no. 2, pp. 83–102, 1999.

[6]. Young and J. Yan, "Channel columns undergoing local, distortional, and overall buckling," *Journal of Structural Engineering*, vol. 128, no. 6, pp. 728–736, 2002.

[7]. J. Dejin and C. Shaofan, "Inelastic torsional-flexural buckling of cold-formed open sections under eccentric load," in *Proceedings of the 7th International Specialty Conference on Cold-Formed Steel Structures*, St. Louis, Mo, USA, 1984.

[8]. H. T. Nguyen and S. E. Kim, "Buckling of composite columns of lipped-channel and hat sections with web stiffener," *Thin-Walled Structures*, vol. 47, no. 11, pp. 1149–1160, 2009.