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Buckling Analysis of Cold Formed Steel for Beams

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Abstract- Cold formed steel are nowadays used for building construction especially non-load bearing partition, curved walls, etc due to its flexural strength and good appearance. The cold formed steel enhances the mean yield stess by 15% to 30% as compared to hot rolled steel. In this paper detailed parametric and comparative study of cold formed steel sections by different codes is carried out for prediction of flexural strength of beams. Various codes predict different strength. The flexural strength of cold formed steel beam is carried out and presented using CUFSM software which uses Direct Strength Method for prediction of flexural strength and this flexural strength is compared with IS 801-1975 and experimental results.

Index Terms- Cold formed steel, flexural strength, Direct Strength Method

I. Introduction

Cold formed steel are also called light gauge steel and are cold formed from steel or strips. Cold-formed sections are produced by bending and shaping flat sheet steel at ambient temperatures. 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. 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.

For the determination of member elastic buckling load/moment, CUFSM software (Schafer 2006; Schafer and Ádány 2006) is mainly used which uses finite strip method for calculation and it gives nearer to experimental results as compared to other methods. However, currently, FSA can only handle accurately single-span members (mostly simply supported) subjected to uniform internal force and moment diagrams. Conventional Finite Strip Method (FSM) provides a means to examine all the possible instabilities in a cold-formed steel member under longitudinal stresses (axial, bending, or combinations thereof). Various types of buckling may occur such as local buckling, distortional buckling, flexural-torsional buckling, lateral -torsional buckling as shown in Figure 1.

In Figure 1, the first minimum (Point 1) is a local buckling mode, which involves buckling of the web, compression flange, and lip stiffener. The second minimum (Point 2) is the flange distortional buckling mode and involves the rotation of the compression lip-flange component about the web-flange junction. At longer wavelengths where the purlin is unrestrained, a flexural-torsional or lateral buckling mode occurs (Point 3). However, if the tension flange is torsionally restrained, then a lateral distortional buckling mode may take place, as shown by Point 4 (Hancock 1998). This lateral distortional buckle strength is dependent on the degree of torsional restraint provided to the tension flange (Hancock 1998).

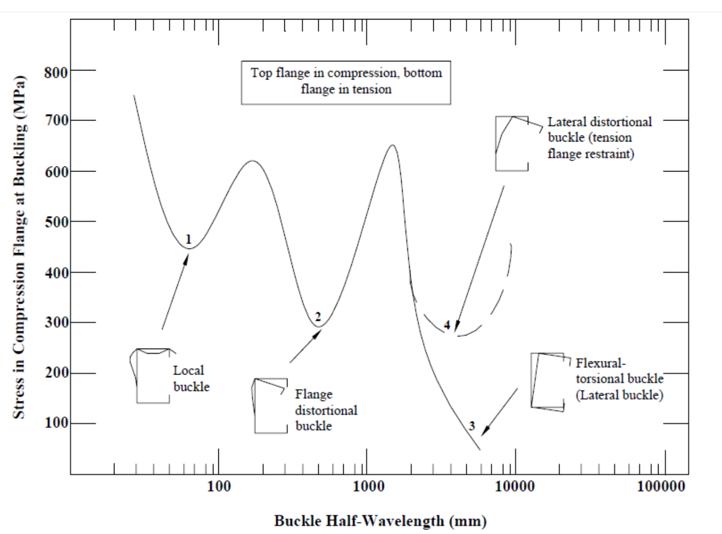

Figure 1: Buckling Modes Subject to a C-Purlin for Major Axis Bending.

II. OBJECTIVES

The objectives of this paper are as follows:

1. To study various modes of buckling occurring in a cold formed steel members when subjected to flexural loading.
2. To calculate finite strip solution for buckling class such as global, distortional or local buckling using CUFSM software.
3. Hence, after calculating the values for loading by using CUFSM software, obtaining the values of flexural strength by Direct Strength Method.
4. To study, design and compare the values of flexural strength by Direct Strength Method as well as by Indian Standard (IS 801-1975) code and by experimental value.

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III. Design for Flexural Strength of Beam

Review of Codal Provisions:

The following codes of practices are studied to know how these limit states are handled :
1) Direct Strength method.
2) IS Code 801-1975 of practice for use of cold formed light guage steel structural members in general building construction.

Different countries use different design methods for the light gauge cold formed steel structures. India uses IS: 801 -1975 which is based on allowable/working stress method of design.

- **Considering an example of a C-lipped section of 21.71446 cm X 6.2966 cm X 0.18034 cm with yield stress of 404.7222 N/mm^2**

A. Computation as per IS code 801-1975 of practice for use of Cold formed light guage steel structural members in general building construction:

Material Properties **:** yield stress fy = 404.7222 N/mm^2

i. Computation of Sectional Properties:

Depth d = 217.1446 mm
Width w = 62.966 mm
Depth of lip D = 24.4942 mm
Thickness t = 1.8034 mm
Area A = 708.64 mm^2
Span of length L = 1000 mm
Centroid: CG of section : Xcg = 18.083 mm
Zcg = 108.572 mm
Moment of inertia : Ixx = 5.0513 X 10^6 mm^4
Izz = 0.4250 X 10^6 mm^4

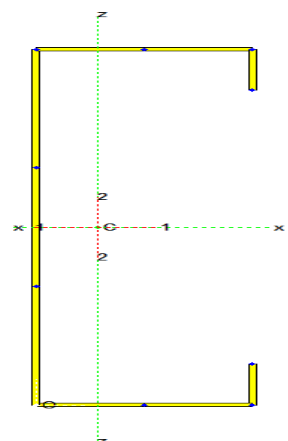

Figure 2: C lipped section

ii. Computation of effective width:

Checking of above section as per clause 5.2.2.1 IS 801-1975 (Page No: 6):
Effective width calculation of compression elements :
Flange is fully effective if $\frac{w}{t} \leq \left(\frac{w}{t}\right) lim$
Hence $\frac{w}{t} = \frac{6.2966}{0.18034} = 34.9151$
$\left(\frac{w}{t}\right)$ lim $= \frac{1435}{\sqrt{\text{fy}}} = \frac{1435}{\sqrt{404.7222}} = 71.330$
Hence $\frac{w}{t} < \left(\frac{w}{t}\right) lim$.
Therefore Entire area is effective.

iii. Determination of safe load:

Section modulus Se = $\frac{Ixx}{Zcg} = \frac{5.0513\ X\ 10^6}{108.572} = 46525.7$ mm^3
Allowable resisting moment = Se X fy
= 46525.7 X 404.7222
M = 18.8 x 10^6 Nmm
Let w be the load in N/mm

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$$\frac{w\ X\ 1000^2}{8} = 18.8\ X\ 10^6$$

w = 150.4 N/mm

iv. Check for web shear :

Maximum Shear force = V = $\frac{150.4\ X\ 1000}{2} = 75.2\ X\ 10^3$ N

Maximum average shear stress Fmax = $\frac{V}{A} = \frac{75.2\ X\ 10^3}{708.64} = 106.118$ N/mm^2

$$\frac{h}{t} = \frac{217.1446}{1.8034} = 120.4084$$

$$\frac{4590}{\sqrt{fy}} = \frac{4590}{\sqrt{404.7222}} = 228.1571$$

As per clause 6.4.1 IS 801-1975 (Page No: 15) :

Since $\frac{h}{t} < \frac{4590}{\sqrt{fy}}$

Therefore the gross area of a flat web = Fv = $\frac{1275\sqrt{fy}}{\frac{h}{t}} = \frac{1275\sqrt{404.7222}}{120.4084}$

Fv = 213.025N/mm^2

Fv must not be greater than Fv_{max} = 0.4fy = 0.4 X 404.7222

Fv_{max} = 161.88 N/mm^2

Hence Fv = Fv_{max} =161.88 N/mm^2.

Thus, Fv = Fv_{max} = 161.88 N/mm^2 this is greater than the maximum Average shear stress of Fmax =106.118 N/mm^2. Thus the beam is therefore safe in shear.

v. Check for bending compression in web :

As per clause 6.4.2 IS 801-1975 (Page No: 16) :

Actual compression stress at junction of flange and web :

$f_{bw} = f_c\ X\ \frac{62.966-1.8034}{62.966}$

$= 0.4\ X\ fy\ X\ \frac{62.966-1.8034}{62.966}$

= 235.878 N/mm^2

Permissible:

$F_{bw} = \frac{36560000}{(\frac{h}{t})^2}$ kg/cm^2

$= \frac{3585311.24}{(\frac{h}{t})^2}$ N/mm^2

= 247.29 N/mm^2

Since $F_{bw} > f_{bw}$. Hence Safe in bending.

vi. Combined Bending and Shear Stresses in Webs :

As per clause 6.4.2.3 IS 801-1975 (Page No: 16) :

$$\sqrt{\left(\frac{fbw}{Fbw}\right)^2 + \left(\frac{Fmax}{Fv}\right)^2} \leq 1$$

where, f_{bw}= actual compression stress at junction of flange and web;

$F_{bw} = \frac{3585311.24}{(\frac{h}{t})^2}$ N/mm^2

Fmax = actual average shear stress, that is, shear force per web divided bv webs area;

Fv = allowable shear stress, except that the limit of 0.4fy, shall not apply.

$$\sqrt{\left(\frac{235.875}{247.29}\right)^2 + \left(\frac{106.118}{213.025}\right)^2} = 0.9934$$

Since Combined Bending and Shear Stresses in Webs is less than unity. Hence the section is safe.

vii. Determination of deflection :

Deflection $\delta = \frac{5wL^4}{384EI} < \frac{L}{325}$

where w = 150.4 kN/m = 150.4 N/mm

L = 1000 mm

E = $2.033 \text{ X } 10^5$ N/mm^2

Ixx = $505.1343 \text{ X } 10^4$ mm^4

Hence $\delta = \frac{5 \; X \; 150.4 \; X \; (10^3)^4}{384 \; X \; 2.033 \; X \; 10^5 \; X \; 505.1343 \; X \; 10^4} = 1.9096$ mm.

Permissible :

$\frac{L}{325} = \frac{1000}{325} = 3.076$ mm.

Hence safe.

B. Computation as per Direct Strength method (DSM) :

i. Check for Section as per AISI-S100-07:

Material properties is same as in IS 801-1975 calculations.

The following Checks must be satisfied for the C section as per AISI-S100-07 Section 1.1.1.2 :

$\frac{ho}{t} = \frac{217.446}{1.8034} = 120.5755 < 321.$ OK.

$\frac{bo}{t} = \frac{62.966}{1.8034} = 34.9151 < 75.$ OK.

$\frac{D}{t} = \frac{24.4942}{1.8034} = 13.5822 < 34.$ OK.

$\frac{ho}{bo} = \frac{217.1446}{62.966} = 3.4486 > 1.5$ and < 17 OK.

$\frac{D}{bo} = \frac{24.4942}{62.966} = 0.3890 < 0.70$ OK.

$\frac{E}{fy} = \frac{2.033 \; X \; 10^5}{404.7222}$ $502.3198 > 421$ OK.

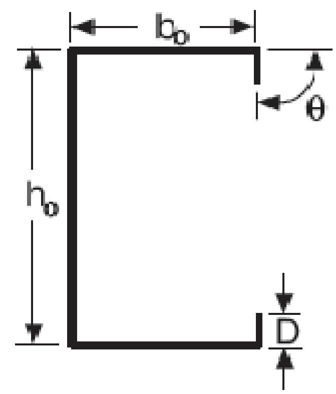

Figure 3: C lipped section notations

ii. Calculation of Yield moment and Critical Elastic Buckling Moment:

From CUFSM software assigning the value for fy = 404.7222 N/mm^2,we get,

Yield Moment My = $18.82987 \text{ X } 10^6$ Nmm.

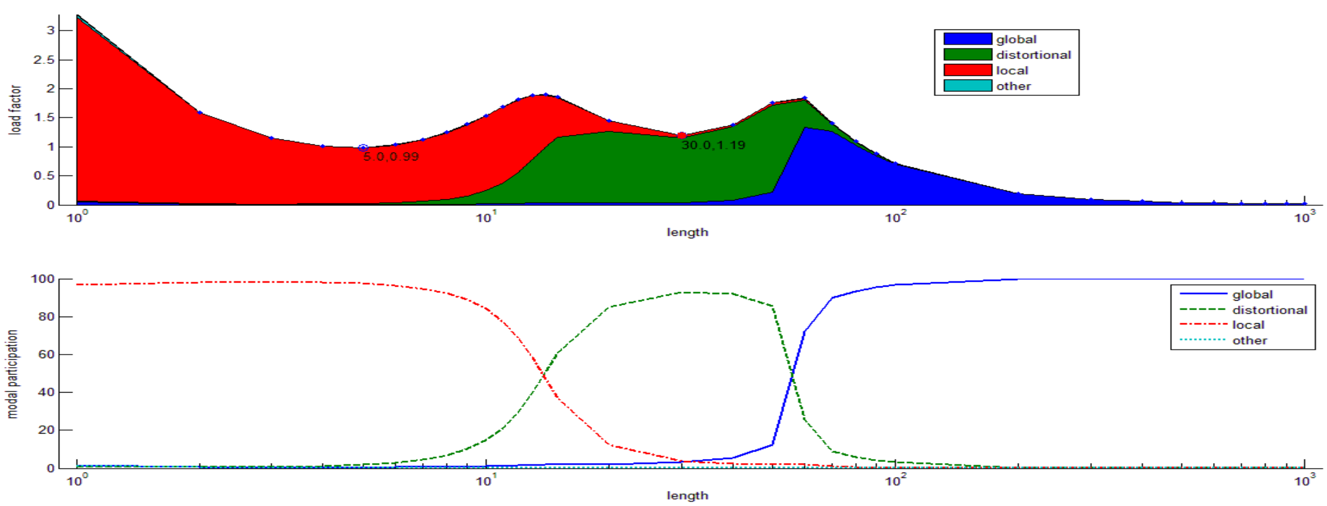

Figure 4: Graph of load factor vs length.

From Figure 4 we obtain the load factors as:

Local Buckling = $\frac{Mcrl}{My} = 0.98606$

Distortional Buckling = $\frac{Mcrd}{My} = 1.1922$

Global Buckling = $\frac{Mcre}{My} = 0.71685$

Hence Critical Elastic Local Buckling Moment Mcrl = 0.98606 X My

$= 0.98606 \text{ X } 18.82987 \text{ X } 10^6$
$= 18.56738 \text{ X } 10^6$ Nmm.

Critical Elastic Distortional Buckling Moment Mcrd = 1.1922 X My
$= 1.1922 \text{ X } 18.82987 \text{ X } 10^6$
$= 22.44897 \text{ X } 10^6$ Nmm.

Critical Elastic lateral torsional Buckling Moment Mcre = 0.71685 X My
$= 0.71685 \text{ X } 18.82987 \text{ X } 10^6$
$=13.49819 \text{ X } 10^6$ Nmm.

iii. Calculation of Nominal Flexural Strength :

As per AISI-S100-07 Section 1.2.2 Nominal Flexural Strength of beam is minimum of local, distortional and lateral torsional buckling and is calculated as follows:

a. Nominal flexural strength for Lateral-torsional buckling per AISI-S100-07 Section 1.2.2.1 :

The nominal flexural strength, Mne, for lateral-torsional buckling shall be calculated in accordance with the following:

for $M_{cre} < 0.56M_y$
$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$
for $2.78M_y \geq M_{cre} \geq 0.56M_y$
$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$
for $M_{cre} > 2.78M_y$
$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

where, Mcre = Critical elastic lateral torsional buckling moment.
My = Yield Moment.

Here equation 1.2.2-2 satisfies the following condition:
2.78My > Mcre > 0.56My
$2.78 \text{ X } 18.82987 \text{ X } 10^6 > 13.49819 \text{ X } 10^6 > 0.56 \text{ X } 18.82987 \text{ X } 10^6$
$52.347 \text{ X } 10^6 > 13.49819 \text{ X } 10^6 > 10.544 \text{ X } 10^6$

Hence,
$$\text{Mne} = \frac{10}{9}My\left(1-\frac{10My}{36Mcre}\right)$$
$$\text{Mne} = \frac{10}{9}X\ 18.82987\ X\ 10^6\left(1-\frac{10\ X\ 18.82987\ X\ 10^6}{36\ X\ 13.49819\ X\ 10^6}\right)$$
$\text{Mne} = 12.81482 \text{ X } 10^6$ Nmm

Hence Nominal flexural strength for Lateral-torsional buckling is:

$\text{Mne} = 12.81482 \text{ X } 10^6$ Nmm

b. Nominal flexural strength for Local buckling as per AISI-S100-07 Section 1.2.2.2 :

The nominal flexural strength, Mnl, for local buckling shall be calculated in accordance with the following:
Mcre = Critical elastic local buckling moment.

for $\lambda_\lambda \leq 0.776$
$$M_{n\lambda} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$
for $\lambda_\lambda > 0.776$
$$M_{n\lambda} = \left(1-0.15\left(\frac{M_{cr\lambda}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\lambda}}{M_{ne}}\right)^{0.4}M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$
where $\lambda_\lambda = \sqrt{M_{ne}/M_{cr\lambda}}$ (Eq. 1.2.2-7)

My = Yield Moment.

Here equation 1.2.2-6 satisfies the following condition:

Local-global slenderness ratio λ_ℓ is given as:
$$\lambda_\ell = \sqrt{\frac{Mne}{Mcrl}} = \sqrt{\frac{12.81482\ X\ 10^6}{18.56738\ X\ 10^6}} = 0.83 > 0.776$$
Since $\lambda_\ell > 0.673$, nominal flexural strength, Mnl is given by Eq. 1.2.2-6 as follow:
$$\text{Mnl} = \left(1-0.15\left(\frac{Mcrl}{Mne}\right)^{0.4}\right)\left(\frac{Mcrl}{Mne}\right)^{0.4}Mne$$

$$= \left(1 - 0.15\left(\frac{18.56738\ X\ 10^6}{12.81482\ X\ 10^6}\right)^{0.4}\right)\left(\frac{18.56738\ X\ 10^6}{12.81482\ X\ 10^6}\right)^{0.4} X\ 12.81482\ X\ 10^6$$

Mnl = 12.27769 X 10^6 Nmm (local-global interaction reduction)

Hence Nominal flexural strength for local buckling is:

Mnl = 12.27769 X 10^6 Nmm.

c. Nominal flexural strength for Distortional buckling as per AISI-S100-07 Section 1.2.2.3:

The nominal flexural strength, Mnl, for local buckling shall be calculated in accordance with the following:

Mcrd = Critical elastic distortional buckling moment.

for $\lambda_d \le 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y / M_{crd}}$ (*Eq.* 1.2.2-10)

My = Yield Moment

Distortional slenderness ratio λ_d :

$$\lambda_d = \sqrt{\frac{My}{Mcrd}} = \sqrt{\frac{18.82987\ X\ 10^6}{22.44897\ X\ 10^6}} = 0.92 > 0.673$$

Since $\lambda_d > 0.673$, nominal flexural strength, Mnl is given by Eq. 1.2.2-9 as follow:

$$\text{Mnd} = \left(1 - 0.22\left(\frac{Mcrd}{My}\right)^{0.5}\right)\left(\frac{Mcrd}{My}\right)^{0.5} My$$

$$\text{Mnd} = \left(1 - 0.22\left(\frac{22.44897\ X\ 10^6}{18.82987\ X\ 10^6}\right)^{0.5}\right)\left(\frac{22.44897\ X\ 10^6}{18.82987\ X\ 10^6}\right)^{0.5} X\ 18.82987\ X\ 10^6$$

Mnd = 15.62116 X 10^6 Nmm.

Hence Nominal flexural strength for distortional buckling is:

Mnd = 15.62116 X 10^6 Nmm

iv. Nominal flexural strength of the beam as per AISI-S100-07 Section 1.2.2:

Nominal flexural strength of the beam is minimum of Mne, Mnl, Mnd.

Hence Mn = 12.27769 X 10^6 Nmm

= 12.27769 kNm.

For beams meeting geometric and material criteria of section Ω and ϕ shall be as follow:

$\Omega = 1.67$, $\phi = 0.9$

Hence, design strength ϕ Mn = 11.04992 kNm.

Allowable design strength Mn/Ω= 7.351911 kNm.

IV. RESULTS

As per IS 801-1975 Flexural strength of beam = 18.8 kNm.

As per Direct Strength Method Flexural strength of beam = 11.04992 kNm.

Experimental value of flexural strength for cold formed steel beam is M = 11.72445 kNm.

V. CONCLUSION

- A comparative study on the flexural strength of lipped channel sections based on different code provisions and the values are compared with respective experimental values. With the comparative study, parametric study has been conducted by varying the lip depth for selected sections through CUFSM analysis, which is the background analysis for DSM.
- The load factor corresponds to distortional buckling for each cross sectional shape has been calculated.
- IS: 801 provisions are not accounting for distortional buckling and hence it over predicts the strength.
- Direct strength method predicts the section strength closer to the experimental results. Load factor corresponds to distortional buckling increases up-to the ratio of lip depth to flange width and later it decreases.

VI. ACKNOWLEDGMENT

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