International Journal of Engineering and Technical Research (IJETR)
ISSN: 2321-0869, Volume-2, Issue-11, November 2014

Parametric study and comparison of Indian standard code with British standard code for the Design of Light gauge cold formed flexural members

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***Abstract*—In the construction industry both structural and non-structural elements are created from thin gauges of sheet steel. These building materials encompass columns, beams, joists, studs, floor decking, built-up sections and other components. Cold-formed steel construction materials differ from other steel construction materials known as hot-rolled steel (structural steel). The strength of elements used for design is usually governed by buckling. The effect of cold working is thus to enhance the mean yield stress by 15% to 30%. In this dissertation, work on parametric study and comparison of flexural design strength of cold-formed light gauge steel section based on IS: 801-1975 (Indian standard) and BS 5950-5:1998 (British standard) codes has been carried out and presented. The IS: 801-1975 code is based on working stress method and BS 5950-5:1978 code is based on limit state method. It was observed that both the design concepts give nearly the same strength.**

***Index terms* -Cold-formed steel, buckling, Flexural strength, limit state method, working stress method.**

I. Introduction

A. Light gauge cold formed steel

Light gauge steel structural members are cold formed from steel or strips. Various cold formed light gauge members can be divided into three heads

i. Framing members, such as beams, joists, studs etc.
ii. Floor and wall panels and long span roof deck.
iii. Wall claddings and standard roof deck.

The thickness for the framing members generally ranges from 1.2mm to 4.0mm. The thickness of floor and wall panel sections and for long span roof deck varies from 1.2mm to 2.5mm. The thickness of wall claddings and standard roof deck varies from 0.8mm to 1.2mm. In India, light gauge members are widely used in bus body construction, railway coaches etc., and the thickness of these members may vary from 1.0mm to 3.2mm. There are mainly two processes for manufacturing or forming the light gauge sections:
(i) Cold-rolling
(ii) Pressing press-brakes
Light gauge members can be either cold-formed in rolls or by press brakes from flat steel generally not thicker than 12.5mm.

Manuscript received November 22, 2014.
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For repetitive mass production, these are formed most economically by cold-rolling, while small quantities of special shapes are most economically produced on press brakes. The latter process, with its great versatility of shape variation makes this type of construction as adaptable to special requirements as reinforced concrete is in its field use. Presently, light gauge members are produced in India both by press brake system (for use in small quantities) and by cold -rolling (for use in large quantities). These members are connected together by spot, fillet, plug or slot welds, bolts, cold rivets, or by special fasteners.

B. Beams

In the design of cold-formed steel flexural members, consideration should first be given to the moment-resisting capacity and the stiffness of the member. It may be found that in many cases, the moment of inertia of the section is not a constant value but varies along the span length due to the non-compactness of the thin-walled section and the variation of the moment diagram.

In addition to the design features discussed above beams are laterally supported or laterally unsupported beams.

1. Laterally Supported Beams

A beam which does not laterally move nor rotate is known as Laterally-Supported Beam. It depends upon the kind of restraint provided at the supports as well as on the loading. If, for example, beam is supporting a slab, then this beam will be laterally-Supported beam.

2. Laterally Unsupported Beams

If a point load is acting on the beam, then it will exhibit lateral-torsional buckling and therefore such a beam will be called Laterally-Unsupported. If the beam is restrained at intervals, then lateral torsional buckling will take place between the restraints and intermediate supports.

II. Related work

M S Deepak [1]evaluated structural behavior of cold-formed steel lipped 'C' channel beams due to lateral buckling of beams and load carrying capacity. It was seen that the decrease in d1/t ratio shows increase in load carrying capacity. And also there is increase in deflection of beams. The failure of the beam is at the critical sections where the stresses are maximum, at points of loading. Warping reduces the lateral buckling effect.

Nirosha Dolamune Kankanamge[2] proposedStructural Behavior and Design of Cold Formed Steel Beams at Elevated Temperatures presents the details and results of the experimental and numerical studies conducted in this research including a comparison of results with the predictions using available design rules. It also presents the recommendations made regarding the accuracy of current design rules as well as the new developed design rules for cold formed steel beams both at ambient and elevated temperatures.

Somadasa Wanniarachchi [3]proposed a detailed Investigation into the Structural Behavior of Rectangular hollow flanges beams (RHFB) subjected to Flexural action. Research was carried out in Two Phases

- First Phase included Experimental Investigation
- Second Phase involved a methodical and comprehensive investigation aimed at widening the scope of FEA to investigate the buckling and ultimate failure behaviors of RHFBs subjected to flexural actions.

Overall, the innovative RHFB sections can performs well as economically and structurally efficient flexural members. Investigations into different combinations of thickness in the flange and web indicate the increasing the flange thickness is more effective than web thickness in enhancing the flexural capacity of RHFBs. The current steel design standards, AS 4100 (1998) AND AS/NZS 4600 (1996) are sufficient to predict the section moment capacity of RHFBs. However, the results indicate that the AS/NZS 4600 is more accurate for slender sections whereas AS 4100 is more accurate for compact section.

Benjamin W.Schafer [4]The objective of this paper is to provide a review of the Direct Strength Method for cold formed steel design**.** The Direct Strength Method is a new design methodology for cold-formed steel members. The method has been formally adopted as an alternative design procedure in Appendix 1 of the North American Specifications for the Design of Cold-Formed Steel Structural Members, as well as in the Australian/New Zealand Standard for cold-formed steel design. The conclusions drawn from the study were,

The Direct Strength Method employs gross cross-section properties, but requires an accurate calculation of member elastic buckling behavior. Numerical methods, such as the finite strip method or generalized beam theory, are the best choice for the required stability calculations.

The reliability of the Direct Strength Method equals or betters the traditional Effective Width Method for a large database of tested beams and columns. Extensive design aids are now available for engineers who want to apply the Direct Strength Method in design. Expansion of the Direct Strength Method to cover, shear, inelastic reserve, and members with holes are all underway. In addition, development of a Direct Strength Method for beam-columns continues and will provide cross-section specific interaction with far greater accuracy than the simple (essentially linear) interaction equations in current use. Much work remains for the continued development of the Direct Strength Method.

Gustavo Monteiro de Barros Chodraui, Jorge MunaiarNeto, Roberto Martins Gonçalves, and Maximiliano Malit [5]discussed about distortional buckling of cold formed members. In this study, an analysis was made about lipped channels under compression and bending, comparing the results obtained through the Brazilian Standard simplified model against those of the elastic analysis via the finite strip method, evaluating the conformity and the range of validity of the simplified model application. Distortional mode is part of the most recent codes, including the new Brazilian Standard NBR 14762Brazilian Association 2001, and can constitute the critical mode in open section thin-walled members, being most sensitive to distortion members with wide flanges and edge stiffeners, such as lipped channels and Z sections, hat, and rack sections. The analysis of elastic buckling via the finite strip method has been widely employed to evaluate the parameters of interest in distortional buckling, allowing one to easily determine the elastic buckling stress for various half wavelengths and thus build general elastic buckling curves corresponding to local, distortional, and overall modes. The results have indicated a good agreement with those obtained by the finite element method. The Brazilian Standard NBR 14762(Brazilian Association 2001) procedure is based on the Australian/New Zealand Standard AS/NZS 4600 (Australian/New Zealand Standard 1996) and corresponds to the simplified model proposed by Lau and Hancock1987. When compared with results obtained via the finite strip method, this procedure has led to satisfactory results for compression, provided the limits established by the code are respected. In the case of bending, differences obtained were greater; including those for members contained within the range of application indicated by NBR 14762, indicating that the model requires revision and adjustments.

MarkkuHeinisuo, JuhaKukkonen [6] The paper resistance of cold-formed steel members by new euro standard deals with the new European standard EN 1993-1-3 for cold-formed steel structures. The main scope of the study was to consider the cases when designing the cold-formed members which are not supported between supports, e.g. members of the truss, or is supported continuously at both sides along the members, e.g. members of the wall or floor elements. Finally, some conclusions are given based on the results of the study. Mean of all results in the study gives some picture for the safety margin of the new Eurocode compared to test results. All cases considered in the references Heinisuo, Kukkonen (2005) and Kukkonen (2005) are taken into account in the following. Total amount of 272 cases were considered, 178 compressions, 64 bending and 30 bending and compression cases. It was seen, that the new Eurocode is 10-30% conservative for compression members. For pure bending the safety margin is less than 10%. The interaction for compression and bending the safety margin varies from 6% to over 40% in these cases. For the bending of U beam around weak axis and with compression at flanges the code gives typically about 70% safety margin to tests and for these cases more refined method is recommended to be used when designing these structures.

III. OBJECTIVE & OVERVIEW OF PROPOSED WORK

A. Objectives

In this paper, we propose a Parametric study and comparison of Indian standard code and British standard code for the design Light gauge cold formed flexural members. 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. Hence in this work the objective of our study is

- To conduct a comparative study of light gauge steel sections using different country codes for different sections.
- To undertake a detailed parametric study of light gauge steel sections by different codes.

B. Overview of the proposed work

The present study is carried out to understand the design of cold formed light gauge steel flexural members using IS: 801-1975 & BS 5950-5:1998. Different countries use different codes as per Indian standard IS: 801-1975 is a code of practice for use of cold-formed light gauge steel structural members in general building construction the design of members is carried out by working stress method whereas the BS: 5950-5:1998 structural use of steelwork in building – Part5. Code practice for design of cold formed thin gauge sections, here the design of members is carried by limit state method. Thus, results for both IS code & BS codes obtained are then plotted using graph with Load (kg/m) in the Y-axis & Length (m) in the axis.

IV. DESIGN OF FLEXURAL MEMBERS

The design of cold formed light gauge steel sections here is carried out using different codes for both laterally supported and laterally unsupported beams. The codes used for the design study are

- **A. IS: 801-1975** Indian standard Code practice for use of cold-formed light gauge steel structural member's in general building construction.
- **B. BS 5950-5:1998** Structural use of steelwork in building-Part 5. Code practice for design of cold formed thin gauge sections.

A. Design Followed for Laterally Supported Beams using IS: 801-1975

Considering an example of a C-section (back to back) of size 200mm x 100mm x 30mm x 2.5mm. The design steps involved for laterally supported beams are as follows

Material Properties:

$$E = 2 \times 10^5 N/mm^2$$

$fy = 2400\ kg/cm^2$

1. Computation of Sectional Properties:

$$t = 2.5mm;\ R_i = 3mm$$

$w = 200mm = 200 - 2 \times (2.5 + 3) = 189mm$

$b = 100mm = 100 - 2 \times (2.5 + 3) = 89mm$

$Lip = 30mm = 30 - (2.5 + 3) = 24.75mm$

$R = 4.25\text{mm}; \text{C} = 2.71\text{mm}; \text{L} = 6.6725\text{m}$

$$Ixx = 1442.331 \times 10^4 mm^4$$

$$Zxx = 144.233 \times 10^3 mm^3$$

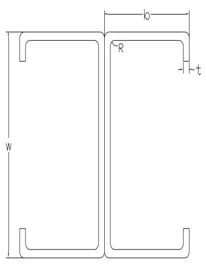

Fig.(a): Typical I-Section

For Length, $L = 1m$

2. Computation of effective widths

For unstiffened element,

$$\frac{w}{t} = \frac{(30 - 5.5)}{2.5} = 9.8 < 10.82 \quad [\frac{530}{\sqrt{2400}} = 10.82]$$

$$f_c = f = 0.6 \times f_y = 0.6 \times 2400 = 1400\ kg/cm^2$$

For Stiffened flanges:

Radius of curve = 6 + 4 = 10mm

Therefore,

$w = 100 - 2 \times 5.5 = 89mm$

$$\frac{w}{t} = \frac{89}{2.5} = 35.6$$

$$\left(\frac{w}{t}\right) lim = \frac{1435}{\sqrt{f}} = \frac{1435}{\sqrt{2400}} = 37.81$$

$$\frac{w}{t} < \left(\frac{w}{t}\right) lim$$

∴ Entire area is effective

3. Determination of safe load

$$M = fxZ$$

$$M = 1440 \times 1442.331$$

$$= 207695.66 \text{ kgcm} \quad (20769.56\ Nm)$$

Let w be the load in kg/m

$$\frac{w(1)^2}{8} = 207695.66 = 16615.56\ kg/m$$

4. Check for web shear

$$\text{Max. shear force} = \frac{166.15 \times 1}{2} = 83.075\ kN$$

$$\text{Max. average shear stress} = 85.21\ N/mm^2$$

$$\frac{h}{t} = \frac{195}{2.5} = 78;\ \frac{4590}{\sqrt{2400}} = 93.69$$

$$\therefore Fv = \frac{1275\sqrt{fy}}{h/t} = 800.79\ kg/cm^2 < 0.4 \times 2400$$

$$= 960\ kg/cm^2$$

Thus $Fv = 80.079\ N/mm^2$, this is smaller than the max.Average shear stress of $88.28\ N/mm^2$. The beam is therefore unsafe in shear.

5. Check for bending compression in web

$$f_{bw}{}' = 144 \times \frac{100 - 2.5}{100} = 140.4\ N/mm^2$$

Permissible

$$F_{bw} = \frac{3656000}{(h/t)^2} = 600.92 > 140.4\ N/mm^2. \text{Hence safe}$$

6. Determination of deflection.

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

$$ɗ = 0.737mm$$

Permissible

$$ɗ = \frac{1000}{325} = 3.076mm. \text{Hence safe.}$$

A. Design followed for Laterally Supported Beams using BS 5950-5:1998

Considering an example of a C-section (back to back) of size 200mm x 100mm x 30mm x 2.5mm.The partial safety factor 1.6 for live load is used to get working load to compare with ASD designs. The design steps involved for laterally supported beams are as follows:

Material Properties:

$E = 2 \times 10^5 N/mm^2$

$fy = 2400\ kg/cm^2$

1. Computation of Sectional Properties:

$$t = 2.5mm;\ R_i = 3mm$$

$$w = 200mm$$

$b = 100mm$

$Lip = 30mm$

$R = 4.25\text{mm}; \text{C} = 2.71\text{mm}; \text{L} = 6.6725\text{m}$

$Ixx = 1442.331 \times 10^4 mm^4$

$Zxx = 144.233 \times 10^3 mm^3$

For Length, $L = 1m$

Only the compression flange is subjected to local buckling

Limiting stress for stiffened web in bending

$$p_o = \left\{1.13 - 0.0019\frac{D_W}{t}\left(\sqrt{\frac{Y_s}{280}}\right)\right\}p_y$$

or

$$p_o = p_y$$

where

$p_o = p_y = \frac{240}{1.15} = 208.69\ N/mm^2$

$p_o = \left\{1.13 - 0.0019\frac{200}{2.5}\left(\sqrt{\frac{240}{280}}\right)\right\}\frac{240}{1.15}$

$$= 206.45\ N/mm^2$$

which is equal to the maximum stress in the compression flange, i.e.

$f_c = 206.45\ N/mm^2$

Effective width of compression flange

$h = B2/B1 = 189 / 89 = 2.123$

$K_1 = 5.4 - \frac{1.4h}{0.6+h} - 0.02h^3 = 4.12\ or\ 4\ minimum$

$p_{cr} = 185000 \times K_1 \times \left(\frac{t}{b}\right)^2 = 583.89\ N/mm^2$

$\frac{f_c}{p_{cr}} = \frac{206.45}{583.89} = 0.353 > 0.123$

$\frac{b_{eff}}{b} = \left[1 + 14\left\{\left(\sqrt{\frac{f_c}{p_{cr}}}\right) - 0.35\right\}^4\right]^{-0.2} = 0.9902$

$\frac{b_{eff}}{89} = 0.9902 = 88.13mm$

2. **Moment of Resistance**

$M_{cr} = Z_{xr} \times p_y = 29.78\ kNm$

Let w be the load in kg/m

$$\frac{w(1)^2}{8} = 29.78 = 238.24\ kN/m = 23824\ kg/m$$

Referring from Table 2-Load factors and combination BS 5950-5:1998

γ_f forImposed load= 1.6

$$\therefore \text{working load}, w = \frac{238.24}{1.6} = 148.889\ kN/m$$
$$= 14888.9\ kg/m$$

3. **Shear Resistance**

Shear yield strength,

$p_v = 0.6 \times p_y = 125.21 N/mm^2$

Shear buckling strength,

$q_{cr} = \left(\frac{1000t}{D}\right)^2 = 156.25 N/mm^2$

Maximum shear force,

$$F_{vmax} = 75.245 kN$$

Shear area $= 200 \times 2.5 = 500 mm^2$

Average shear stress $f_v = \frac{75.245 \times 10^3}{500}$

$$= 150.49 N/mm^2 < q_{cr} \therefore OK$$

4. **Check for deflection**

$$\delta = \frac{5wL^4}{384EI} < \frac{L}{325}$$
$$\delta = 0.667mm$$

Permissible

$$\delta = \frac{1000}{325} = 3.076mm. \text{ Hence safe.}$$

A. Design followed for Laterally Unsupported Beams using IS: 801-1975

Considering an example of a C-section (back to back) of size 180mm x 50mm x 25mm x 2mm. The design steps involved for laterally supported beams are as follows:

Material Properties:

$E = 2 \times 10^5 N/mm^2$

$f_y = 2400\ kg/cm^2$

1. **Computation of Sectional Properties:**

$$t = 2mm;\ R_i = 3mm$$
$$w = 180mm$$

$b = 50mm$

$Lip = 25mm$

$R = 4\text{mm}; \text{C} = 2.548\text{mm}; \text{L} = 6.28mm$

$Ixc = 584.812 \times 10^4 mm^4$

$$Iyc = 33.178 \times 10^4 mm^4$$
$$Zxc = 64.979 \times 10^3 mm^3$$

For Length, $L = 1m$

2. **Computation of parameters A and B**

$A = \frac{L^2 Z_{xc}}{dI_{yc}} = \frac{(1000)^2 \times 64.979 \times 10^3}{180 \times 33.178 \times 10^4} = 1088.05$

$B = \frac{\pi^2 E C_b}{f_y} = \frac{\pi^2 \times 2 \times 10^5 \times 1}{240} = 8224.67$

When A is greater than 0.36B but less than 1.8B

$f_b = \left(\frac{2}{3} - \frac{A}{5.4B}\right)f_y$

When $A \geq 1.8B$

$f_b = 0.6\frac{B}{A}f_y$

3. **Computation of permissible bending stress**

$$1.8B = 14804.41\ ; 0.36B = 2960.88$$
$$\therefore f_b = 0.6 \times 2400 = 1440\ kg/cm^2$$

4. **Compression on Unstiffened Elements**

$$\frac{w}{t} = \frac{25-5}{2} = 10 < \frac{530}{\sqrt{2400}} = 10.18$$
$$\therefore fc = 0.6 \times 2400 = 1440\ kg/cm^2$$

5. **Determination of effective width**

$$\frac{w}{t} = \frac{50 - 2 \times 5}{2} = 20$$
$$\left(\frac{w}{t}\right)_{lim} = \frac{1435}{\sqrt{f_b}} = 37.81$$

Hence the full section is effective.

6. **Determination of safe load**

$M_r = Z_{xc} \times f_b = 108581.39\ kgcm\ (10858.139\ Nm)$

Let w be the load in kg/m

$$\frac{w(1)^2}{8} = 108581.39 = 8686.511\ kg/m$$

7. **Check for web shear**

$$\text{Max. shear force} = 43.43\ kN$$
$$\text{Max. average shear stress} = 63.86\ N/mm^2$$
$$\frac{h}{t} = \frac{170}{2} = 85;\ \frac{4590}{\sqrt{2400}} = 93.69$$
$$\therefore Fv = \frac{1275\sqrt{fy}}{h/t} = 734.85\ kg/cm^2 < 0.4 \times 2400$$
$$= 960\ kg/cm^2$$

Thus, $Fv = 73.485\ N/mm^2$, which is much greater than the max. Average shear stress of $63.86\ N/mm^2$. The beam is therefore safe in shear.

8. **Check for bending compression in web**

$$f_{bw}{}' = 1440 \times \frac{90-2}{90} = 1408\ kg/cm^2$$

Permissible

$$F_{bw} = \frac{36560000}{(h/t)^2} = 5060.21\ kg/cm^2$$
$$> 1408\ kg/cm^2. \text{Hence safe}$$

9. Determination of deflection.

$$d = \frac{5wL^4}{384EI} < \frac{L}{325}$$
$$d = 0.819mm$$

Permissible

$$d = \frac{1000}{325} = 3.076mm. \text{Hence safe.}$$

B. Design followed for Laterally Unsupported Beams using Bs 5950-5:1998

Considering an example of a C-section (back to back) of size 180mm x 50mm x 25mm x 2mm. The design steps involved for laterally supported beams are as follows:

Material Properties:

$E = 2 \times 10^5 N/mm^2$
$f_y = 2400\ kg/cm^2$

1. Computation of Sectional Properties:

$$t = 2mm;\ R_i = 3mm$$
$$w = 180mm$$

$b = 50mm$
$Lip = 25mm$
$R = 4\text{mm}; C = 2.548\text{mm}; L = 6.28\text{mm}$
$Ixc = 584.812 \times 10^4 mm^4$

$$Iyc = 33.178 \times 10^4 mm^4$$
$$Zxc = 64.979 \times 10^3 mm^3$$

For Length, $L = 1m$
Only the compression flange is subjected to local buckling

2. Limiting stress for stiffened web in bending

$$p_o = \left\{1.13 - 0.0019\frac{D_W}{t}\left(\sqrt{\frac{Y_s}{280}}\right)\right\}p_y$$

or

$$p_o = p_y$$
$$p_o = p_y = \frac{240}{1.15} = 208.69\ N/mm^2$$
$$p_o = \left\{1.13 - 0.0019 \times \frac{180}{2}\left(\sqrt{\frac{240}{280}}\right)\right\}\frac{240}{1.15}$$
$$= 202.79\ N/mm^2$$

which is equal to the maximum stress in the compression flange, i.e.
$f_c = 202.79\ N/mm^2$

3. Effective width of compression flange

$$h = B2/B1 = 170/40 = 4.25$$
$$K_1 = 5.4 - \frac{1.4h}{0.6 + h} - 0.02h^3 = 2.638\ or\ 4\ minimum$$
$$p_{cr} = 185000 \times K_1 \times \left(\frac{t}{b}\right)^2 = 1850\ N/mm^2$$
$$\frac{f_c}{p_{cr}} = \frac{206.45}{1850} = 0.111 < 0.123$$
$$\frac{b_{eff}}{b} = 1 = 40mm$$

4. Moment of Resistance

$$M_{cr} = Z_{xr} \times f_c = 13.177kNm$$

5. Buckling resistance moment(M_b)

Yield moment of the section,

$$M_y = p_y * Z_c = 13.561kNm$$

6. Elastic lateral buckling resistance moment

Effective length $L_E = L$
$A = 16.72cm^2; E = 2000000\ kg/cm^2;$
$D = 180\ mm = 18\ cm;\ L_E = 100 = 100\ cm; r_y$
$= 2.25\ cm$

$$M_E = \frac{\pi^2 AED}{2\left(\frac{L_E}{r_y}\right)^2} * C_b\left\{1 + \frac{1}{20}\left(\frac{L_E}{ry} * \frac{t}{D}\right)^2\right\}^{\frac{1}{2}}$$
$$= \frac{\pi^2 \times 16.72 \times 2 \times 10^6 \times 18}{2 \times (100|2.25)^2} \times 1$$
$$\times \left\{1 + \frac{1}{20}\left(\frac{100}{2.25} * \frac{2}{180}\right)^2\right\}^{\frac{1}{2}}$$
$$M_E = 168.99kNm$$
$$\frac{L_E}{r_y} = \frac{100}{2.258} = 39.34$$

Perry coefficient η when $\frac{L_E}{r_y} < 40C_b, \therefore \eta = 0$

$$\phi_b = \frac{M_y + (1+\eta)M_E}{2} = 99.57$$
$$M_b = \frac{M_E M_y}{\phi_b + \sqrt{(\phi_b)^2 + M_E M_y}} \le M_c$$
$$M_b = 11.634\ kNm \le 13.17kNm$$

∴The ultimate moment is $M_b = 11.634\ kNm$
Let w be the load in kg/m

$$\frac{w(1)^2}{8} = 11.634 = 93.072\ kN/m$$

Referring from Table 2-Load factors and combination BS 5950-5:1998

$$\gamma_f \text{ for Imposed load} = 1.6$$
$$w = \frac{93.072}{1.6} = 58.17\ kN/m = 5817.30\ kg/m$$

7. Shear Resistance

Shear yield strength,
$p_v = 0.6 \times p_y = 125.21N/mm^2$
Shear buckling strength,

$$q_{cr} = \left(\frac{1000t}{D}\right)^2 = 123.45N/mm^2$$

Maximum shear force,

$$F_{vmax} = 29.85\ kN$$

Shear area $= 180 \times 2 = 360mm^2$
Average shear stress$f_v = 82.94N/mm^2 < q_{cr} \therefore OK$

8. Determination of deflection.

$$d = \frac{5wL^4}{384EI} < \frac{L}{325}$$
$$d = 0.653mm$$

Permissible

$$d = \frac{1000}{325} = 3.076mm. \text{Hence safe.}$$

V. Results in the Form of Tables and Graphs

A. Comparative Results Obtained for Laterally Supported Beams as well laterally unsupported beams for Both IS: 801-1975 & BS 5950-5:1998

Table I: Laterally supported beam & laterally unsupported beams Load vs Length 30x15x10x1.15mm & 30x15x10x1.6mm.

Length	Laterally supported beams (LSB)		Laterally unsupported beams (LUSB)	
	W (kg/m)		W (kg/m)	
	30 X 15 X10 X 1.15mm		30 X 15 X 10 X 1.6mm	
L (mts)	IS-801:1975	BS 5950-5:1998	IS-801:1975	BS 5950-5:1998
1	154.71	151.93	145.3	78.39
1.5	68.76	67.52	28.7	26.6
2	38.68	37.98	9.08	12.19
2.5	24.75	24.31	3.72	6.63
3	17.19	16.88	1.79	4.01
3.5	12.63	12.4	0.97	2.62
4	9.67	9.5	0.57	1.81
4.5	7.64	7.5	0.35	1.31
5	6.19	6.08	0.23	0.97
5.5	5.11	5.02	0.16	0.74
6	4.3	4.22	0.11	0.58
6.5	3.66	3.6	0.08	0.47
7	3.16	3.1	0.06	0.38
7.5	2.75	2.7	0.05	0.31
8	2.42	2.37	0.04	0.26
8.5	2.14	2.1	0.03	0.22
9	1.91	1.88	0.02	0.18
9.5	1.71	1.68	0.02	0.16
10	1.55	1.52	0.01	0.14

Table II: Laterally supported beam & laterally unsupported beams Load vs Length 100x50x15x2mm.

Length	Laterally supported beams (LSB)		Laterally unsupported beams (LUSB)	
	W (kg/m)		W (kg/m)	
	100 X 50 X 15 X2mm		100 X 50 X 15 X 2mm	
L (mts)	IS-801:1975	BS 5950-5:1998	IS-801:1975	BS 5950-5:1998
1	3150.52	2973.72	3150.52	2461.85
1.5	1400.23	1321.65	1400.23	935.7
2	787.63	743.43	787.63	446.41
2.5	504.08	475.8	504.08	244.3
3	350.06	330.41	280.29	146.71
3.5	257.19	242.75	151.29	94.31
4	196.91	185.86	88.68	63.89
4.5	155.58	146.85	55.37	45.14
5	126.02	118.95	36.33	32.99
5.5	104.15	98.3	24.81	24.82
6	87.51	82.6	17.52	19.12
6.5	74.57	70.38	12.72	15.03
7	64.3	60.69	9.46	12.03
7.5	56.01	52.87	7.18	9.78
8	49.23	46.46	5.54	8.06
8.5	43.61	41.16	4.35	6.72
9	38.9	36.71	3.46	5.66
9.5	34.91	32.95	2.79	4.81
10	31.51	29.74	2.27	4.13

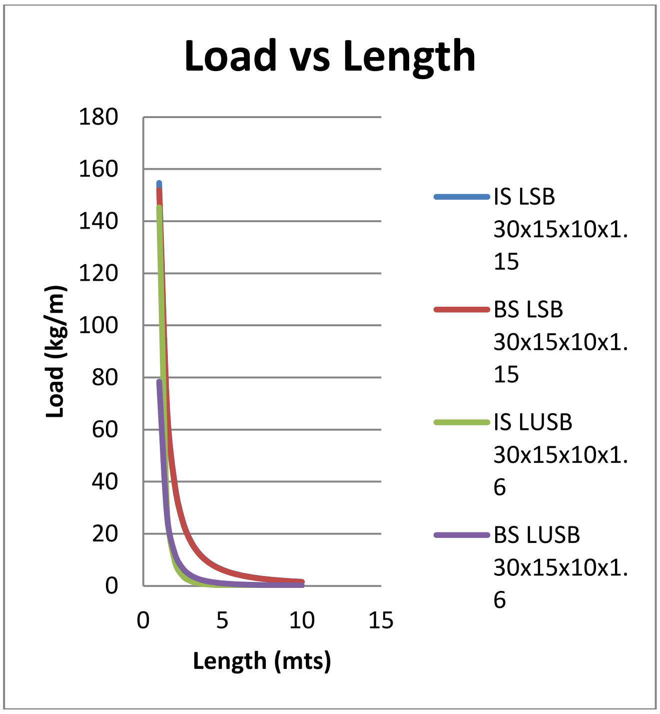

Fig (b): Laterally supported beam & laterally unsupported beams Load vs Length 30x15x10x1.15mm & 30x15x10x1.6mm

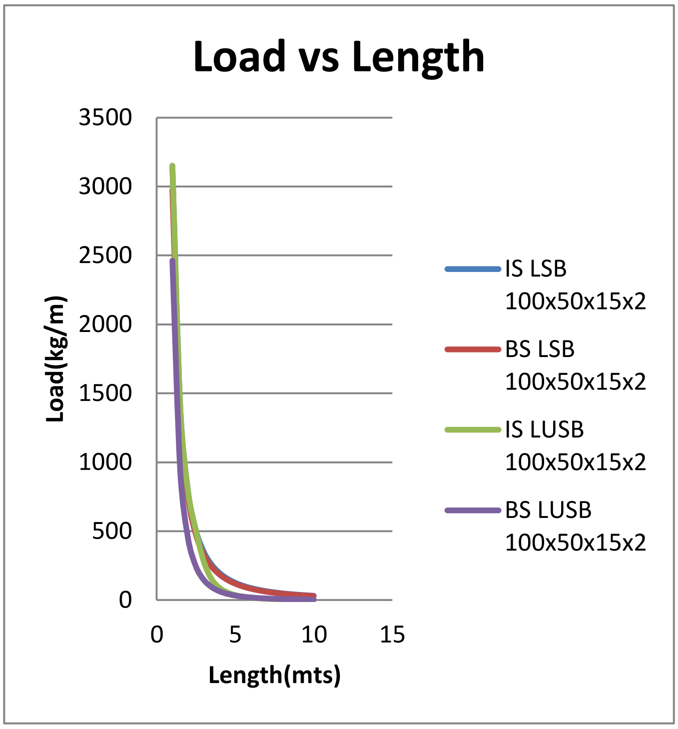

Fig (c): Laterally supported beam & laterally unsupported beams Load vs Length 100x50x15x2mm.

Table III: Laterally supported beam & laterally unsupported beams Load vs Length 250x100x25x2mm.

Length	Laterally supported beams (LSB)		Laterally unsupported beams (LUSB)	
	W (kg/m)		W (kg/m)	
	250 X 100 X25X2mm		250 X 100 X25X2mm	
L (mts)	IS-801:1975	BS 5950-5:1998	IS-801:1975	BS 5950-5:1998
1	17137.0	14533.61	17137.01	14533.61
1.5	7616.45	6459.38	7616.45	6361.47
2	4284.25	3633.4	4284.25	3329.82
2.5	2741.92	2325.38	2741.92	1939.15
3	1904.11	1614.85	1904.11	1223
3.5	1398.94	1186.42	1363.05	816.24
4	1071.06	908.35	1001.6	568.55
4.5	846.27	717.71	753.61	409.6
5	685.48	581.34	576	303.31
5.5	566.51	480.45	444.36	229.8
6	476.03	403.71	343.96	177.53
6.5	405.61	343.99	263.47	139.46
7	349.73	296.6	199.6	111.18
7.5	304.66	258.38	151.47	89.78
8	267.77	227.09	117	73.34
8.5	237.19	201.16	91.81	60.54
9	211.57	179.43	73.04	50.43
9.5	189.88	161.04	58.84	42.37
10	171.37	145.34	47.92	35.88

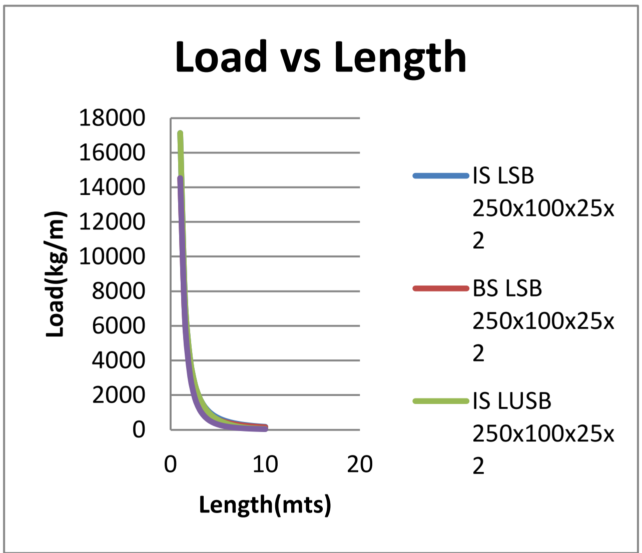

Fig (d): Laterally supported beam & laterally unsupported beams Load vs Length 250x100x25x2mm

B. Observations, Discussions and Conclusions

1. Laterally supported beams

a) The cold-formed light gauge steel C-sections (back to back) from 30x15x10x1.15mm to 250x100x25x2mm for various thickness the comparative graphs (span along X-axis and load along Y-axis) for uniformly distributed load carrying capacity based on IS:801-1975 and BS 5950-5:1998 for various spans have been prepared. From the graphs it is observed that the curves are reverse parabolic in nature. The loads worked out by IS code are higher than BS code values. But the difference is very small.

b) For higher sections if the flange width is more, the difference in the values worked out by IS code and BS code is also more which can be seen from the section $180 \times 100 \times 25 \times 2$ mmand $250 \times 100 \times 25 \times 2$mm.

c) Thus we conclude that the laterally supported flexural members designed by IS Code are economical than BS Code.

2. Laterally unsupported beams

a) The cold-formed light gauge steel channel sections from $30 \times 15 \times 10 \times 1.15$ mm to $250 \times 100 \times 25 \times 2$mm for various thickness the comparative graphs (span along X-axis and load along Y-axis) for uniformly distributed load carrying capacity by IS code and BS code for various spans have been prepared.

b) From the graphs it is observed that the loads worked out by IS code are higher for short span beams, and for long span beams BS values are more thanISvalues. For small sections the BS and IS curves cross at 1m to 1.5m span of beams and for higher size sections it is at 4m to 5m span of beams. However for wide flanges IS values are more than BS values except at very long span beams.

c) It is observed for the sections $100 \times 100 \times 15 \times 2\ mmand\ 180 \times 50 \times 25 \times 3.15mm$ that at the point of change of equations for the computation of permissible bending stress some distortion in the values arises which is seen in the pattern of the curve.

d) For higher sections with lesser thickness the load worked out based on IS code are higher than BS code for all spans.

e) For wide flanges with higher thickness the load worked out based on BS code are higher than IS code for longer spans.

f) Thus we conclude that laterally unsupported beams for smaller spans, designed by IS code are economical than BS code but for large spans the BS code is economical for smaller sections. For higher sections and with smaller thickness IS code is economical and for higher thickness the BS code is economical for large spans.

ACKNOWLEDGMENT

I would like to add a few heartfelt words for the people who have been part of this dissertation by supporting and encouraging me. At the onset, I would like to thank **ALMIGHTY**. I would also like to express my Deepest Gratitude to my guide **Prof.R.B.Kulkarni,** Civil Department, KLS Gogte Institute of Technology, Belgaum, Karnataka, India **for** supporting me during dissertation work and guiding me with his valuable suggestions. Last, but not the least, I would like to thank my peers and friends who provided me with valuable suggestions to improve my dissertation. Finally I attribute all my success in life to **My Parents** for their moral and intellectual support. It is my greatest pleasure to dedicate this achievement to **My Parents.**

REFERENCES

[1] M S Deepak, "Investigation on lateral-torsional buckling performances of cold formed steel 'C' channel sections" ,International

journal of Emerging trends in Engineering and Development, Issue 2, Vol 4, May 2012.
[2] Nirosha Dolamune Kankanamge, "Structural behavior and Design of Cold formed steel beams at elevated temperatures", May 2010.
[3] Somadasa Wanniarachchi, "Flexural behavior and Design of Cold formed steel beams with rectangular hollow flanges", December 2006.
[4] Benjamin W .Schafer, "Review: The direct strength method for cold formed steel design", September 2006.
[5] Gustavo Monteiro de Barros Chodraui, Jorge MunaiarNeto, Roberto Martins Goncalves, and Maximiliano Malit, "Distortional buckling of cold formed steel members" Journal of Structural Engineering, ASCE 2006, 132:636-639.
[6] MarkkuHeinisuo, Juhakukkonen Structural Mechanics, "Resistance of cold formed steel members by new Euro Standard", Vol 39, 2006,pp 3-21.
[7] *Dr.* B.C. Punmia, Ashok Kumar Jain, Arun Kumar. Jain, "Design of Steel Structures", Jan 1998,pp 561-600.
[8] Wie-wenYu, PH.D., P.E., "Cold-formed steel design," Third edition, 1924.
[9] Light-gauge steel engineers association: http://www.Igsea.org/en/wiki/coldwork.
[10] Light-gauge steel engineers association: http://www.Igsea.org/en/wiki/coldwork.
[11] http://www.tkk.fi./units/civil/steel/publicatisons/seminaron steel structures: design of cold-formed steel structures.
[12] http://en.wikipedia.org/wiki/coldwork.
[13] J.Rhodes and R.M.Lawson, "Design of structures using cold formed steel sections", SCI publication 089, the steel construction Institute, U.K.1992Indian Standard IS: 801-1975 "Code of practice for use of cold-formed light gauge steel structural members in general building construction", Bureau of Indian Standards, New Delhi (1976).
[14] British Standard BS5950-5:1998, "Structural use of steel work in building-Part 5. Code practice for design of cold-formed thin gauge sections".

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