Parametric Study and Comparison of Indian Standard Code with British Standard Code for the Design of Light Gauge Cold Formed Flexural Members for Inverted Hat Section

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Abstract - *Cold-formed steel structures are steel structural products that are made by bending flat sheets of steel at room temperature into shapes that will support more than the flat sheets themselves. Cold-formed steel members are governed by local buckling strength this measure differs cold-formed steel from the hot-rolled steel. In this paper, work on Parametric study and comparison of Indian standard code with British standard code for the design of light gauge cold-formed flexural members for inverted hat section has been carried out and presented. The IS: 801-1975 code is based on the working stress method whereas BS 5950-5:1978 code is based on the limit state method. It was shown that both the design concepts as per IS:801-1975 code and BS 5950-5:1978 code gives nearly the same strength.*

Key Words: Cold-formed steel, buckling, Flexural strength, limit state method, working stress method.

1. INTRODUCTION

1.1 Light gauge cold-formed steel

Cold-formed steel sections are also called as light gauge steel sections. Light gauge cold-formed Steel members are widely used in civil works such as buildings, bridge works, drainage works, bodies of cars, railway cabins, transmission towers, and also used in various equipment. Generally Cold-formed steel structural members not thicker than 12·5 mm. Spot, fillet, plug, or slot welds and also by screw, bolts, cold rivets, or any other special devices used to connect the members of light gauge cold-formed steel. The commonly used shapes of light gauge cold-formed Steel are - I-sections, Channel(C) sections, connecting double C-sections back-to-back, Hat sections, sigma sections, T-sections, rectangular hollow sections, circular hollow sections, etc. All these sections are with or without lips. If we say that light gauge cold-formed Steel is stronger than its hot-formed alternative, it means not to say that hot-formed steel is weaker than Light gauge cold-formed Steel. Construction world prefers products that are made using Light gauge cold-formed steel when the need for finishes precise edges and a consonant finish are important.

1.2 Beams –

Beams are laterally supported or laterally un-supported beams.

1.2.1 Laterally Supported Beams -

A Laterally-Supported Beam does not laterally move nor rotate. For example, if the beam is supporting a slab, then this beam will be called a laterally-Supported beam.

1.2.2 Laterally Unsupported Beams -

A point load is acting on the beam, then it will reveal lateral-torsional buckling and therefore such a beam will be called Laterally-Unsupported.

2. OVERVIEW OF THE PROPOSED WORK

The present study is carried out to understand the design of cold-formed light gauge steel flexural members for an inverted hat section with lips and also to compare the strength obtained by IS: 801-1975 and BS 5950-5:1998 codes.

As per Indian standard IS: 801-1975 is a code of practice used for the design of cold-formed light gauge steel structural members and is based on working stress method whereas as per British standard BS 5950-5:1998 used for the design of structural steel-work using cold-formed sections and is based on limit state method.

Results for both IS: 801-1975 & BS 5950-5:1998 codes obtained are then plotted using a graph with Load (kg/m) in the Y-axis & Length (meter) in the X-axis.

3. DESIGN OF FLEXURAL MEMBERS

The codes used for the design of both laterally supported and laterally unsupported beams 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 British standard Code practice for the design of cold-formed thin gauge sections.

3.1 Design Followed for Laterally Supported Beams –

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

Considering an example of an inverted Hat-section of size 60mm x 40mm x 15mm x 2mm.

The design steps involved for laterally supported beams are as follows,

Material Properties:

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

fy= 235 N/mm^2

1.Computation of Sectional Properties:

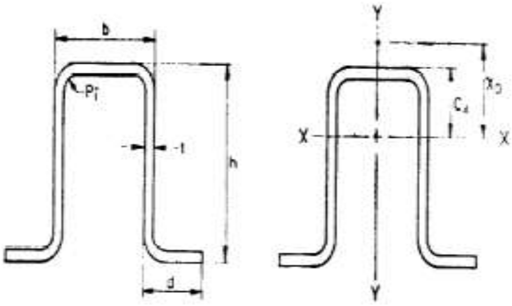
h = 60mm; b = 40mm; d = 15mm;

t = 2 mm; Ri = 3mm

Ixx = 16×10^4 mm^4; Iyy = 12.5×10^4 mm^4

Rxx = 21.5mm; Ryy = 19.0 mm

Zxx = 5.04×10^3 mm^3; Zyy = 3.79×10^3 mm^3



2. Computation of effective widths -

(Refer IS: 801-1975 , cl. 5.2.1.1,pn.6)

Flanges are fully effective (b = w) up to (wlt)lim= $\frac{1435}{\sqrt{f}}$

$(\frac{w}{t}) = (40 - 5 - 5) / (2) = 15$

$(\frac{w}{t})_{lim} = \frac{1435}{\sqrt{f}}$

Where , f = 0.6 x 2350 x (31.7/28.3)

= 158 kg/cm^2

$(w/t)_{lim} = \frac{1435}{\sqrt{1580}} = 36.01 > 15$

Therefore, Flanges are fully effective.

3. Determination of Bending Moment (BM) -

Mmax = fy x Zxx

= 141 x 5.04 x10^3

Mmax = 0.7106 x 10^6 Nmm = 0.71 kNm

4. Load carrying capacity (w) -

Bending Moment,Mmax = 0.71 = $\frac{w \times (l)2}{8}$

$\therefore$ w = 5.68 kN/m

5. Check for web Shear capacity -

Maximum shear force = $\frac{5.68 \times (1)^2}{2}$ = 2.84 kN/m

Maximum average shear stress = 11.83 N/mm^2

Allowable stresses in web of beam

Shear stresses in webs – The maximum average shear stresses (Fv), on the gross area of a flat web shall not exceed (0.4 x fy) = 0.4x2350 = 940 kg/cm2 = 94 N / mm2

For h/t = 56/2 = 28

(h/t)lim. = 4590/$\sqrt{2350}$ = 93.69 > 28

Fv = $\frac{1275 x \sqrt{fy}}{(h/t)}$ = 2207kg/cm^2 = 220.7 N/mm^2

Fv = 94 N/mm^2 > 11.83 N/mm^2

Hence Safe in shear.

6. Check for bending compression in web

fbw = (141 x (31.7 – 2)) / (60-31.7)

= 147.97 N/mm^2

Permissible

Fbw = $\frac{36560000}{28^2}$ = 46632.65 kg/cm^2

= 4663.265 N/mm2 > 147.97 N/mm^2.hence safe.

7. Check for deflection -

$\delta = \frac{5 w l^4}{384 EI}$ = 2.31 mm

Permissible,

L / 325 = 3.076 mm > 2.31 mm. Hence safe.

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

Considering an example of a Hat-section of size 60mm x 40mm x 15mm x 2mm.

The design steps involved for laterally supported beams are as follows -

Material Properties:

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

fy= 235 N/mm^2

1. Computation of Sectional Properties:

h = 60 mm; b = 40mm; d = 15 mm

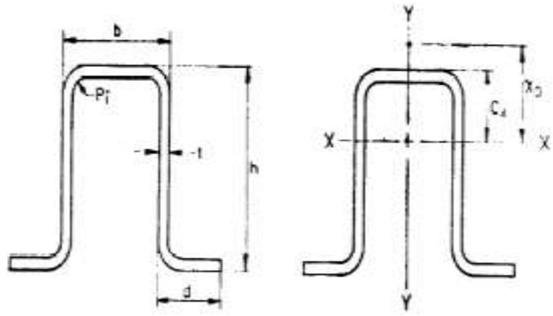
t = 2 mm; Ri = 3 mm

Ixx = 16×10^4 mm^4; Iyy = 12.5×10^4 mm^4

Rxx = 21.5 mm; Ryy = 19.0 mm

Zxx = 5.04×10^3 mm^3; Zyy = 3.79×10^3 mm^3

for length L = 1m,



2. Limiting compressive stress –

(Refer BS 5950-5:1998, cl. 5.2.2.3, pn.17)

The limiting compressive stress, po, may be taken as the lesser of the following values:

$P_o = \{1.13 - 0.0019 \frac{Dw}{t} (\sqrt{\frac{Ys}{280}}) \} p_y$

or $P_o = P_y$

Where,

Po = (1.13 – 0.019 x(60/2)($\sqrt{}$(235/280)) x 204.35

= 220.24 N/mm^2

or

Po = Py = $\frac{235}{1.15}$ = 204.35 N/mm^2

Which is equal to max. stress in compression flange i.e.
fc = 204.35 N/mm^2

3. Effective width of compression flange –

beff/ b = 1
pcr = 18500 x 4 x $(2/30)^2$ = 3288 N/mm^2
fc/pcr = 204.35 / 3288 = 0.062 < 0.123
for fc/pcr < 0.123
beff/ b = 1
Therefore, Flanges are fully effective.

4. Moment of resistance -

Mcr = py x Zxx
= 204.35 x 5.04 x10^3
= 1.03 x 10^6 Nmm
Mcr = 1.03 kNm

5. Load carrying capacity (w)

1.03 = $\frac{w \times (1)^2}{8}$
∴ w = 8.24 kN/m
(Referring from Table2-Load factors and combination BS 5950-5:1998)
Load factor for Imposed load = 1.6
Working load , w = $\frac{8.24}{1.6}$ kN/m = 5.15 kN/m

6. Check for shear resistance –

Shear yeild strength,
Pv = 0.6 x py = 0.6 x 204.35 = 122.61 N/mm^2
Shear buckling strength,
q_{cr} = $(\frac{1000\,t}{D})^2$
= 1275.51 N/mm^2
Maximum shear force,
F_{vmax} = $\frac{5.15 \times 1}{2}$ = 2.58 kN
average shear stress
f_v = (2580)/((2x(56 x 2)) = 11.52 N/mm^2 < q_{cr}
Hence O.K.

7. Check for deflection -

ď = $\frac{5\,w\,l^4}{384\,EI}$
= 2.09 mm
Permissible,
$\frac{L}{325}$ =3.076 mm > 2.09 mm. Hence safe.

3.2 Design followed for Laterally Unsupported Beams-

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

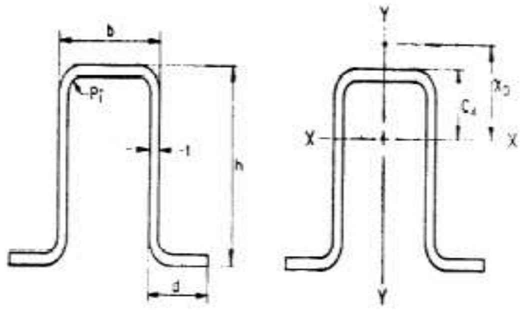
Considering an example of a inverted Hat-section of size 50mm x 40mm x 10mm x 1.60mm.
The design steps involved for laterally unsupported beams are as follows:

Material Properties:

E = 2 x 10^5 N/mm^2
fy= 235 N/mm^2

1.Computation of Sectional Properties:

h = 50 mm; b = 40mm; d = 10 mm
t = 1.60 mm; Ri = 2.40 mm
Ixx = 7.40 x 10^4 mm4; Iyy = 7.72 x 10^4 mm^4
Cy = 21.7 mm;
Rxx = 21.5 mm; Ryy = 19.0 mm
Zxx = 2.61 x 10^3 mm3; Zyy = 2.72 x 10^3 mm^3



For length, L = 1m

2. Computation of Parmeters A and B –

A = (L^2 x Zxc)/dIyc
= (1000^{2}x2.61 x10^3)/ (50x7.72x10^4)
= 676.16
B = $\frac{(\pi^2 \times 2x10^5 \times 1)}{235}$ = 8399.6

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

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

When A is greater than or equal to 1.8B

$f_b = 0.6\,\frac{B}{A}\,fy$

2) Computation of permissible bending stress –

0.36 B = 3023.88 ; 1.8 B = 15119.39
Threfore, $f_b = 0.6\, f_y$ = 0.6 x 235 =141 N/mm^2

3) Compression on unstiffened elements –

$\frac{w}{t} = \frac{(10-2.40-1.6)}{1.60} = 3.75$
$(\frac{w}{t})_{lim} = \frac{530}{\sqrt{2350}}$ = 10.93 > 3.75

$(\frac{w}{t}) < (\frac{w}{t})_{lim}$
Therefore, fc = 0.6 fy = 141 N/mm^2

4) Determination of effective width -

$(\frac{w}{t}) = (\frac{40-4-4}{1.60})$
= 20
$(\frac{w}{t})_{lim} = \frac{1435}{\sqrt{fb}}$
= 38.21 > 20
$(\frac{w}{t}) < (\frac{w}{t})_{lim}$
Hence the flanges are fully effective.

5)Determination of safe load –

Mr = Zxc x fb = 2.61 x 10^3 x 141
= 0.368 KNm
Let, w be the load in kN/m
0.368 = $\frac{w \times (1)^2}{8}$
∴ w = 2.944 kN/m

6) Check for web shear –

Max . Shear force = $\frac{2.944 \times 1}{2}$ = 1.472 kN

Max. average shear stress = $V_{avg} = \frac{1.472 \times 10^3}{50 \times 2 \times 1.60}$ = 9.2 N/mm²

$\frac{h}{t} = \frac{46.8}{1.60}$ = 29.25

$Fv = \frac{1275 \times \sqrt{fy}}{h/t}$ =211.3 kg/cm² = 211.3 N/mm²

< 0.4 fy = 0.4 x 235 = 94 N/mm² > 9.2 N/mm²

Hence safe.

7)Check for deflection -

$ɗ = \frac{5\ w\ l^4}{384\ EI}$

= 2.59 mm

Permissible,

L / 325 =3.076 mm > 2.31 mm.

Hence safe.

3.2.2 Design followed for Laterally Unsupported Beams using BS: 5950-5 : 1998

Considering an example of a Hat-section of size 50mm x 40mm x 10mm x 1.60mm. The design steps involved for laterally unsupported beams are as follows:

Material Properties:

E = 2 x 10^5 N/mm²

fy= 235 N/mm²

1.Computation of Sectional Properties:

h = 50 mm; b = 40mm;

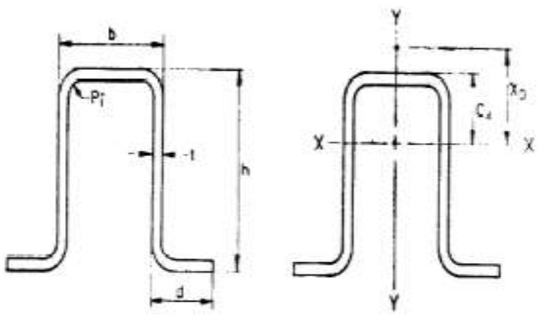
t = 1.60 mm; d = 10 mm

Cy = 21.7 mm; Ri = 2.40 mm

Ixx = 7.40 x 10⁴ mm⁴; Iyy = 7.72 x 10⁴ mm⁴

Rxx = 21.5 mm; Ryy = 19.0 mm

Zxx = 2.61 x 10³ mm³; Zyy = 2.72 x 10³ mm³



2) Limiting stress for stiffened web in bending -

The compressive stress, Po, in a stiffened element which results from bending in its plane, should not exceed the lesser of the following values:

$P_o = \{1.13 - 0.0019 \frac{Dw}{t} (\sqrt{\frac{Ys}{280}}) \} p_y$

or $P_o = P_y$

Where,

Po = (1.13 – 0.019 x(50/2)(√(235/280)) x 204.35

= 219.79 N/mm²

or

$Po = Py = \frac{235}{1.15}$ = 204.35 N/mm²

Which is equal to max. stress in compression flange i.e. fc = 204.35 N/mm²

3) Effective width of compresssion flange –

The ratio of effective width, beff, to full flat width, b, of an element under compression may be determined from the following:

h= B2 /B1 = 42/32 = 1.31

$K1 = 7 - \frac{1.8 \times h}{0.15 + h} - (0.091 \times h^3)$

= 5.18 0r 4 mm minimum

therefore, K1= 4mm

$pcr = 185000 \times K_1 \times (\frac{t}{b})^2$ =185000x4x(1.60/32)²

= 1850 N/mm2

$\frac{fc}{Pcr}$ = (119.76/1850) = 0.065

for $\frac{fc}{Pcr} < 0.123$

$\frac{beff}{b} = 1$

Hence the flanges are fully effective.

4) Moment of Resistance -

Mcr = Zxc x f = 2.61 x10³ x219.79 = 0.53 kNm

5)Buckling Resistance Moment -

Yeild moment of section,

$M_y = P_y$ x Zxc = 204.35 x 2.61 x 10³ = 0.53 kNm

6)Elastic lateral Buckling Resistance Moment -

Effective length = Le = 1m = 100 cm

A =2.34 cm²; E = 2x10⁶ kg/cm²

D=50mm=5cm; ry = 1.82 cm

$M_E = \frac{\pi AED}{2(\frac{LE}{ry})} Cb \{1+\frac{1}{20}(\frac{LE}{ry}\frac{t}{D})^2\}^{1/2}$

M_E = 23.44 kNm

$\frac{Le}{ry}$ = 100/1.82 = 54.94

Determination of M_b

The buckling resistance moment, Mb, may be calculated as follows:

$M_b = \frac{M_E M_Y}{\phi_B + \sqrt{\phi_B 2 - M_E M_Y}} \leq Mc$

Where,

$\phi_B = \frac{M_Y + (1+\eta) M_E}{2}$ = 2.38

Mb = 0.51 kNm < Mc (0.53 kNm)

The ultimate moment is Mb = 0.51 kNm

Let, w be the load in kN/m

$0.51 = \frac{w \times (l)2}{8}$; w = 4.08 kN/m

(Reffering from table-2 ,BS 5950-5:1998, part 5)

Load factor for imposrd load = 1.6

Therefore, $w = \frac{4.08}{1.6}$ = 2.55 kN/m

10) Shear resistance –

Shear yeild strength,

Pv = 0.6 Py = 0.6 x (235/1.15) = 122.6 N/mm²

Shear buckling strength,

$q_{cr} = (\frac{1000 \times t}{D})^2 = (\frac{1000 \times 1.60}{50})^2$

= 1024 N/mm²

Max. shear force, $F_{vmax} = \frac{1.56 \times 1}{2}$

= 1.275 kN

Shear area = 2x50x1.60 = 160 mm²

Average shear stress,$f_v = \frac{0.78 \times 1000}{160}$ = 4.88 N/mm²
< 1024 N/mm² . Hence safe .

11.Check for deflection -

$\delta = \frac{5\ w\ l^4}{384\ EI}$ = 2.09 mm

Permissible,

L / 325 = 3.076 mm > 2.09 mm. Hence safe.

4. RESULTS IN THE FORM OF TABLES AND GRAPHS

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 un-supported beams Load vs Length 60x40x15x2 mm & 50x40x15x2.00 mm.

Length	Laterally supported beams (LSB)		Laterally unsupported beams (LUSB)	
	W (kg/m)		W (kg/m)	
	60x40x15x2 mm		50x40x15x2.00 mm	
L (mts)	IS-801:1975	BS 5950-5:1998	IS 801:1975	BS 5950-5:1998
1.0	579.71	525.15	439.39	386.82
1.5	257.68	233.4	195.28	171.92
2.0	144.90	131.29	109.84	96.70
2.5	92.79	84.02	70.30	74.89
3.0	64.45	58.35	48.82	54.97
3.5	47.31	42.87	35.86	39.57
4.0	36.2	32.82	27.46	32.17
4.5	28.65	25.93	21.69	29.10
5.0	23.15	21	17.57	24.47
5.5	19.17	17.36	14.52	21.78
6.0	16.11	14.58	12.20	17.74
6.5	13.77	12.43	10.39	15.15
7.0	11.83	10.72	8.96	13.89
7.5	10.3	9.33	7.81	11.87
8.0	9.07	8.2	6.86	9.04
8.5	8.06	7.26	6.08	8.35
9.0	7.14	6.48	5.42	6.77
9.5	6.42	5.82	4.86	5.28
10	5.81	5.25	3.39	4.86

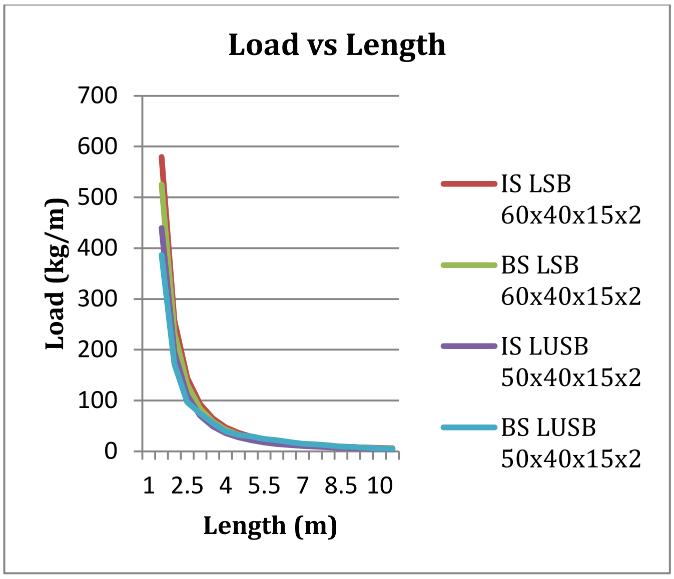


Fig.(a) : Laterally supported beam & laterally unsupported beams Load vs Length 60x40x15x2 mm & 50x40x15x2.00 mm.

Table II: Laterally supported beam & laterally un-supported beams Load vs Length 200x80x15x2 mm.

Length	Laterally supported beams (LSB)		Laterally unsupported beams (LUSB)	
	W (kg/m)		W (kg/m)	
	200x80x15x2 mm		200x80x15x2 mm	
L (mts)	IS-801:1975	BS5950-5:1998	IS-801:1975	BS5950-5:1998
1.0	18603.28	15539.76	18603.28	15539.77
1.5	8082.45	6555.23	8082.45	6188.05
2.0	4750.25	3177.54	4750.25	2966.93
2.5	3207.92	2156.22	3207.92	1837
3.0	2370.11	1577.77	2370.11	1199.54
3.5	1829.05	1069.35	1677.07	798.56
4.0	1467	888.58	1133	555.98
4.5	1219	655.87	850.9	387.32
5.0	1042	598	522.42	244.87
5.5	910.36	437.73	399.29	196
6.0	709.03	377.66	362.42	134.99
6.5	638.61	306.42	278.48	117.33
7.0	582.73	275	206.96	98
7.5	537.66	228.72	166.54	78.11
8.0	450.77	209.09	123	66.86
8.5	370.19	185	78.64	54.78
9.0	344.57	144.97	64.99	43.12
9.5	222.88	127.67	44.78	37.98
10	140.37	97.56	35.66	24.74

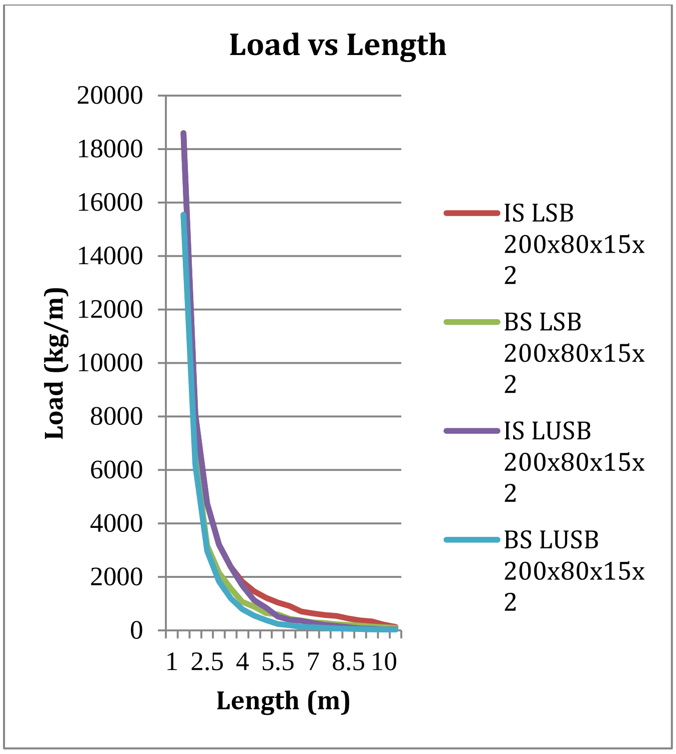


Fig.(b) : Laterally supported beam & laterally unsupported beams Load vs Length 200x80x15x2 mm.

5. CONCLUSIONS

5.1 Laterally supported beams

- The cold-formed light gauge steel inverted hat section 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 have been prepared for various spans.
- Graphs show that curves are reverse parabolic in nature.
- The load-carrying capacity given by IS:801-1975 code is higher than BS 5950-5:1998 code values. But the difference is very small.
- 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.
- Thus we conclude that the IS Code is economical than the BS Code for design laterally supported flexural members.

5.2. Laterally unsupported beams

- The cold-formed light gauge steel inverted hat section 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 have been prepared for various spans.
- Graphs show that the loads worked out by IS code are higher for short span beams, and for long-span beams, BS values are more than IS values. However, for wide flanges IS values are more than BS values except at very long-span beams.
- The load worked out based on IS code is higher than BS code for higher sections with lesser thickness for all spans.
- The load worked out based on BS code is higher than the IS code for wide flanges with higher thickness for longer spans.
- Thus we conclude that for smaller spans laterally unsupported beams, designed by IS code are economical than BS code but the BS code is economical for smaller sections for large spans.
- The IS code is economical for higher sections with smaller thickness and the BS code is economical for higher sections with higher thickness for large spans.

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BIOGRAPHIES –

Prof.R.B.Kulkarni received his Bachelor's degree in Civil Engineering from Karnataka University, Dharwar, India in the year 1984. In the year 1988 received a Postgraduate Diploma in Construction Management from NICMAR, Mumbai, India. I Completed a master's degree in Structural Engineering from Karnataka University, Dharwar, India in the year 1991.

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