OPTIMIZATION OF COLD FORMED STEEL SECTION

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ABSTRACT - Upto date HRS members have been recognised as most popular and widely used in construction, but from few decades use CFS members is flourishing. However, behaviour of CFS section is not completely understood. Simple sections like C-section and Z-section are used commonly due to ease of its formation and connection. These C-section and Z-section suffers from buckling modes. Hence, this leads to reduction in load carrying capacity of member. Therefore, research is based on study of various local modes of failure.

KEYWORDS* *Hot rolled steel (HRS), cold formed steel (CFS), elastic buckling, global buckling, local buckling, distortional buckling (DB), flexural behaviour, lateral torsional buckling (LTB), web crippling, web buckling, critical stress, effective width, stress gradient, End two Flange loading (ETF), Interior two Flange loading (ETF), Interior One Flange loading (IOF), End One Flange loading (EOF) .

I. INTRODUCTION

In steel construction, there are two types of structural steel sections. One is hot rolled steel section and other is cold formed steel sections. Hot rolled steel is formed at high temperatures which is just below the melting point. Cold formed steel sections are formed by moulding steel sheet into desired shape. Moulding or Formation of section is done either by of cold roll forming, press brake, or bending brake. The thickness of steel sheet or strip generally used in cold-formed steel members ranges from 0.0393 in. (1 mm) to about 0.118 in. (3 mm).Minimum yield strength is 280 MPa. These sections are coated with zinc or aluminium –Zinc of minimum thickness of 0.4mm. As we are aware that, hot rolled sections have been used from passed decades whereas cold formed steel sections are used from mid of 18th century in United States and Great Britian. However, such steel members are not widely used upto 1940. The early development of steel buildings has been reviewd by Winter. Since, 1946 use of cold formed steel as thin-walled accelerated.

Cold formed steel sections provides following advantages over other building material:

- With respect to HRS:

i. High strength to weight ratio
ii. Light weight
iii. Pre-coated metals have high resistance to corrosion
iv. Economical
v. Carbon content less than 0.1%

- With Respect to R.C.C

i. Any desired shape of any length can be produced

ii. Dimensional accuracy
iii. Uniform strength
iv. Also provide encased cells for electrical and other conduits
v. Quick production
vi. Light weight, easy to transport and erect
vii. Also provide encased cells for electrical and other conduits
viii. Recyclable material

As we all know that every mterial has some advantanges and some disadvantages with respect to other materials. Similarly, CFS sections has following disadvantages over other building material

- With respect to HRS

i. Difficult to connect if weld is used
ii. In thin sections, if there is hole it leads to corrosive failure of section due to very small thickness

- With respect to R.C.C

i. Low fire resistant
ii. More careful treatment is required against corrosion (e.g. Painting)

We all know that every material has its own properties i.e. some advantages and some disadvantages and some failure problems. HRS section has instability problem like lateral buckling, torsional buckling, lateral-torsional buckling, web crippling, web buckling whereas CFS section has instability problems similar to HRS sections but some additional problems like DB and double curve (LTB) for long span beams. Hence, it is important to eliminate these modes of failure to increase ultimate capacity of member.

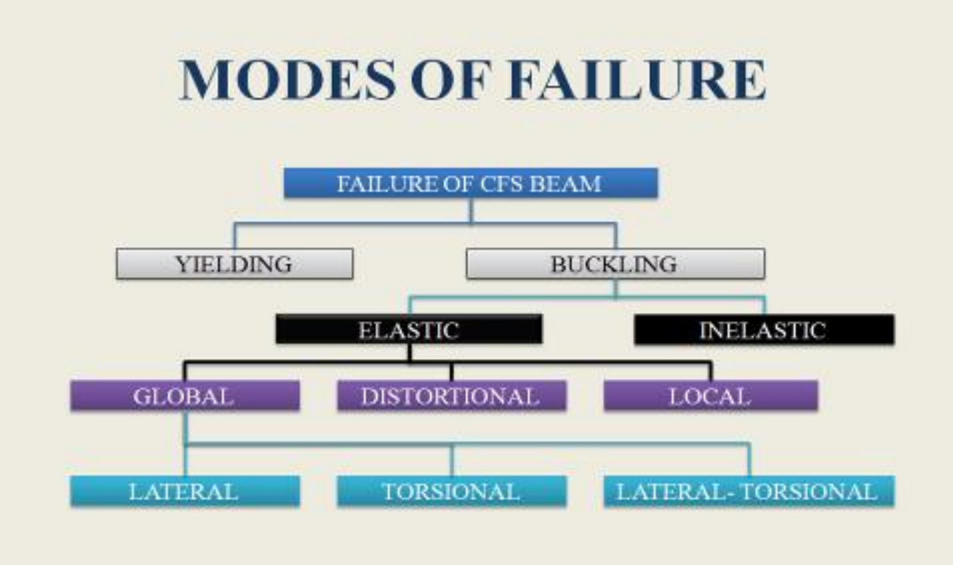

Figure 1 Thomas H, K.kang

II. OBJECTIVES, SCOPE OF WORK & LIMITATIONS

CFS section are widely used as structural elements.The study considers various boundary conditions in experiment. The objectives of this study are as following

- To study the various local modes of failure for CFS beam
- To study effective section properties for CFS beams.

Scope of the work for this study

- Experimental study
- Static loading condition is considered (Pure crippling load)

The limitation of this work is that beams fastened to support and fastened to roof condition is not been considered.

III. METHODOLOGY

First failure that occurs in CFS beams is local failure i.e. either of bearing failure of flange or web crippling. These failure occurs below concentrated point load or at support.

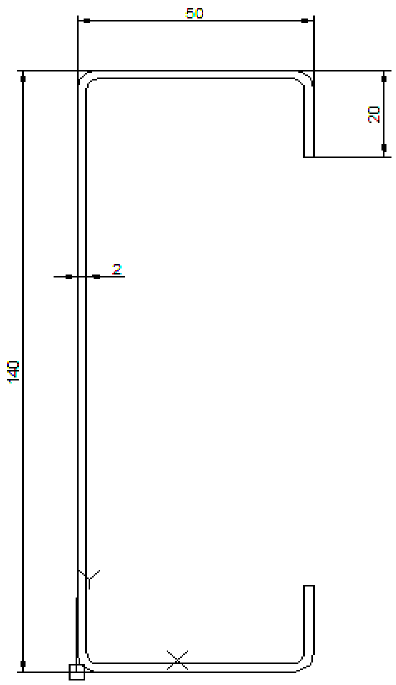

Figure 2 C- 140 x 50 x 20 x 2 mm

Web crippling under various boundary conditions (ETF,EOF,ITF,IOF) have been conducted by Lyse and Godfrey, Rocky, Bagchi, and El-gaaly, Roberts and Neware, Bergfelt, Edlund,and others. Above people concluded that web crippling occurs due to following factors:

1. Non-uniform stress distribution under applied load and adjacent portions of the web.
2. Elastic and inelastic stability of the web element.
3. Local yielding in the immediate region of load application.
4. Bending produced by eccentric load (or reaction) when it is applied on the bearing flange at a distance beyond the curved transition of the web.
5. Initial out-of-plane imperfection of plate elements.
6. Edge restraints provided by beam flanges based on the fastened condition to the support and interaction between flange and web elements.
7. Inclined webs for decks and panels.

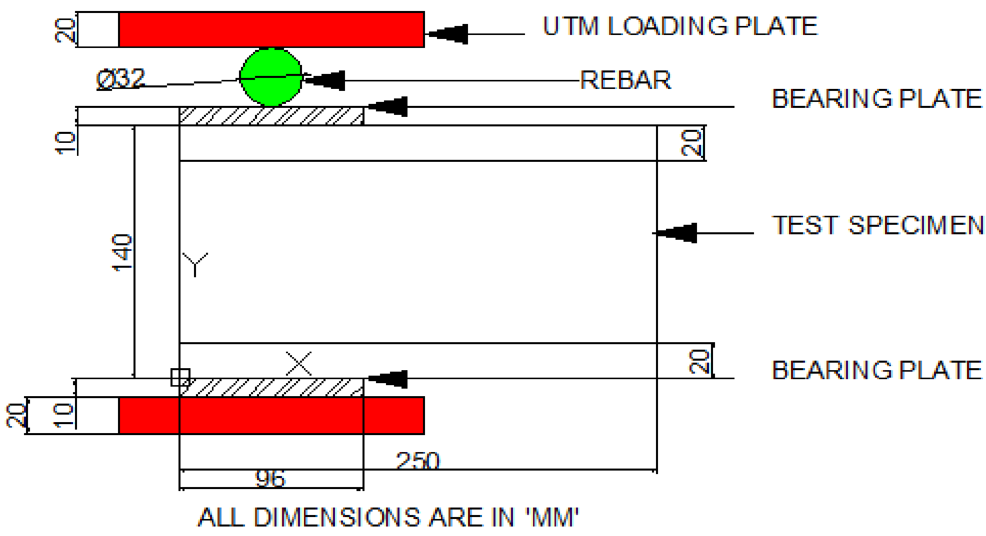

Figure 3 ETF

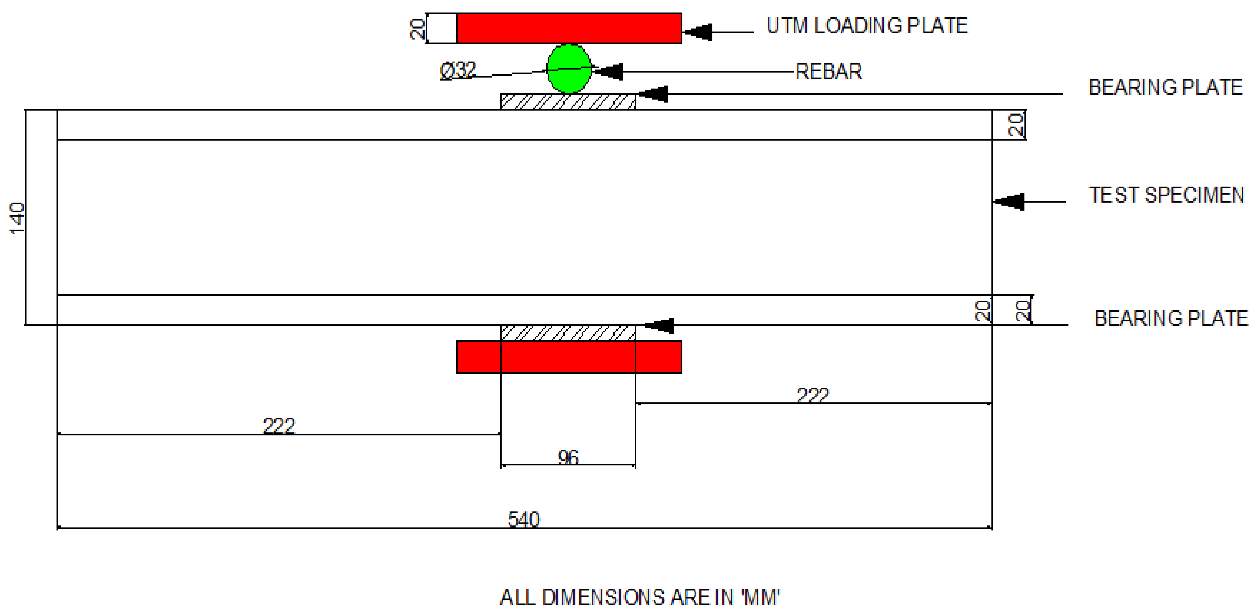

Figure 4 ITF

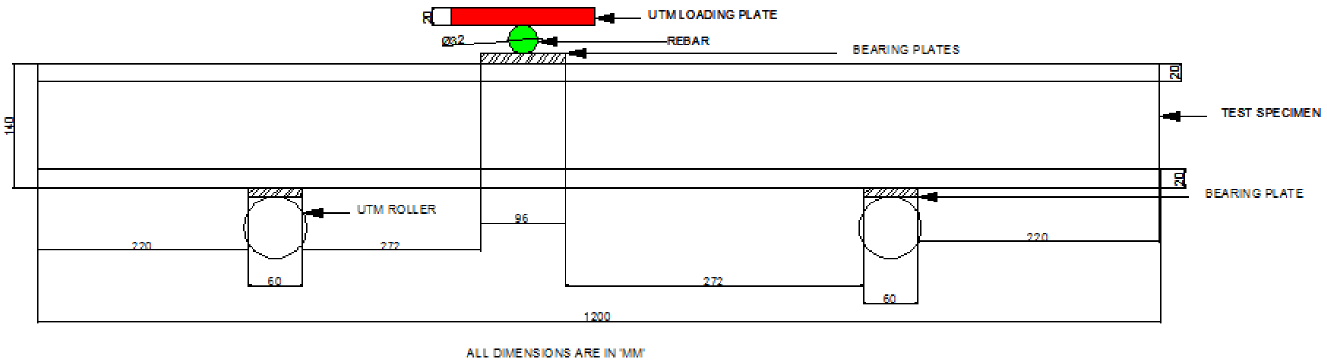

Figure 5 IOF

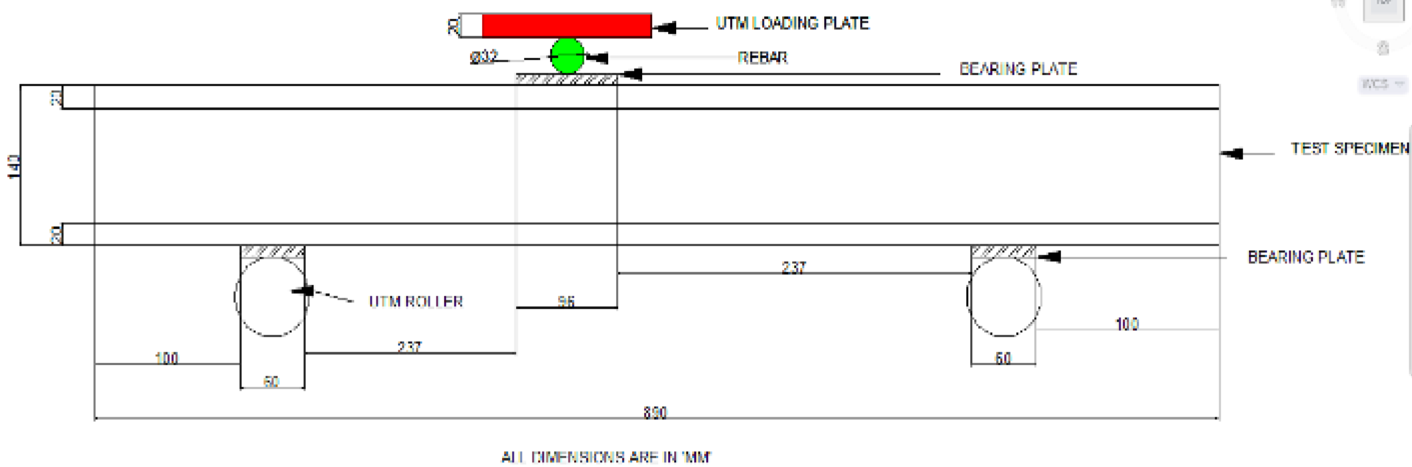

Figure 6 EOF

Basically,when to say One flange loading and when to say two flange loading is very complex to identify, when distance between two opposite loading is less than 1.5 times of depth of web excluding inner bent radius then we can say as "Two Flange Loading" and when distance between two opposite loading is greater than 1.5 times of depth of web excluding inner bent radius then we can say as "One Flange Loading". Now, the question starts how to identify Exterior Flange loading and Interior Flange loading, when distance between end of bearing to end to end member is less than 1.5 times of depth of web excluding inner bent radius then we can say as" Exterior Flange loading" and when distance between end of bearing to end to end member is greather than 1.5 times of depth of web excluding inner bent radius then we can say as" Exterior Flange loading". The above fig. shows ETF (fig. 3), ITF (fig. 4), IOF (fig. 5), EOF(fig. 6). Analytical result is been calculated using North American Specification (S100-16),

$$P_n=Ct^2F_y(\sin\theta)*[1-C_R\sqrt{(R/t)}]*[1+C_N\sqrt{(N/t)}]*[1-C_H\sqrt{(H/t)}]$$

Where,

P_n - web crippling capacity (Newton)
C - coefficient from table (G5-1,G5-2,G5-3,G5-4, G5-5)
F_y - design yield stress (MPa)
θ - angle between plane of web and plane of bearing(45°-90°)
H - depth of web excluding inner bent radius
t - thickness of member (mm)
R - inner bend radius (mm)
C_R - inner bend radius coeff. from table (G5-1,G5-2,G5-3,G5-4, G5-5)
C_N - bearing length coeff. from table (G5-1,G5-2,G5-3,G5-4, G5-5)
C_H - web slenderness coeff. from table (G5-1,G5-2,G5-3,G5-4, G5-5)
N - length of bearing (min. 19mm)

Figure 7 Experimental test setup

Figure 8 Experimental test setup

IV. RESULT AND DISCUSSION

Capacity of member is calculated analytically and compared with test results. For, Analyitcal results, above formula is used. Pure web crippling mode is considered and then test is conducted.Result are as shown below. Only for case A i.e. C- 140 x 50 x 20 x 2 mm above figure are drawn. Support conditions are unfastened to support for both cases.

Case A] C- 140 x 50 x 20 x 2 mm

F_y = 280 MPa

Inner bend radius = 4.45 mm

Table 1.Web crippling test results

Sr No.	Loading Condition	Length (mm)	Pa (kN)	Pt (kN)	Pt/Pa	Failure Type
1	ETF	250	6.97	4	0.573	WC
2	ITF	540	12.19	12.4	1.017	WC
3	IOF	1200	17.34	15.4	0.888	BF
4	EOF	890	10.2	15.2	1.49	BF
5	ETF	710	6.97	7.4	1.06	WC.

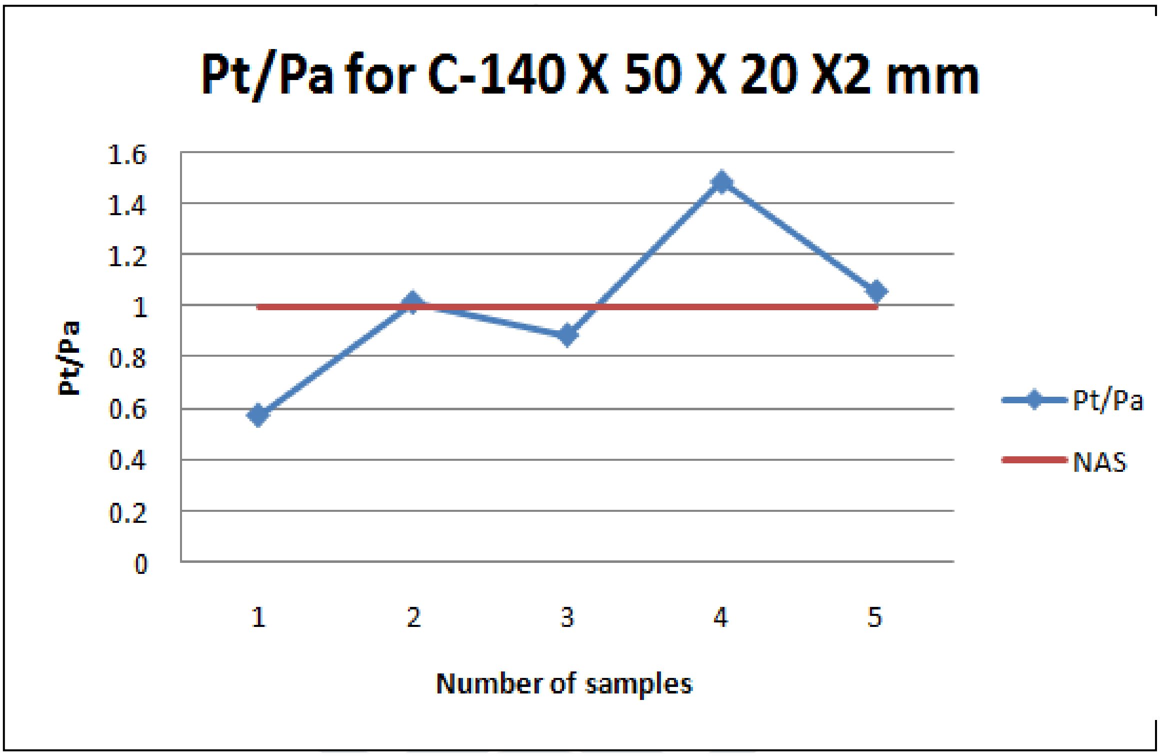

Figure 10 Graphical represetation of result

- For sample 1,ETF with length 250 mm, pure web crippling mode of failure was observed i.e. only web crippling.
- For sample 2, ITF with length 540 mm, pure web crippling mode of failure was observed i.e. only web crippling.
- For sample 3, IOF with length 1200 mm, bearing failure was critical mode along with it small amount of bending of section was observed.
- For sample 4, EOF with length 890 mm, bearing failure was critical mode along with it small amount of bending of section was observed.
- For sample 5, ETF with length 710 mm, web crippling was critical mode along with it small amount of bending of section was observed.

Case B] C- 100 x 50 x 20 x 2 mm
F_y = 280 MPa
Inner bend radius = 6 mm
Bearing length for all specimen was same i.e. 96 mm.

Table 2. Web crippling test results

Sr No.	Loading Condition	Length (mm)	Pa (kN)	Pt (kN)	Pt/Pa	Failure Type
1	ITF	420	5.40	8.2	1.518	W_C
2	ETF	100	6.47	3	0.463	W_C
3	ETF	310	6.47	10	1.499	W_C

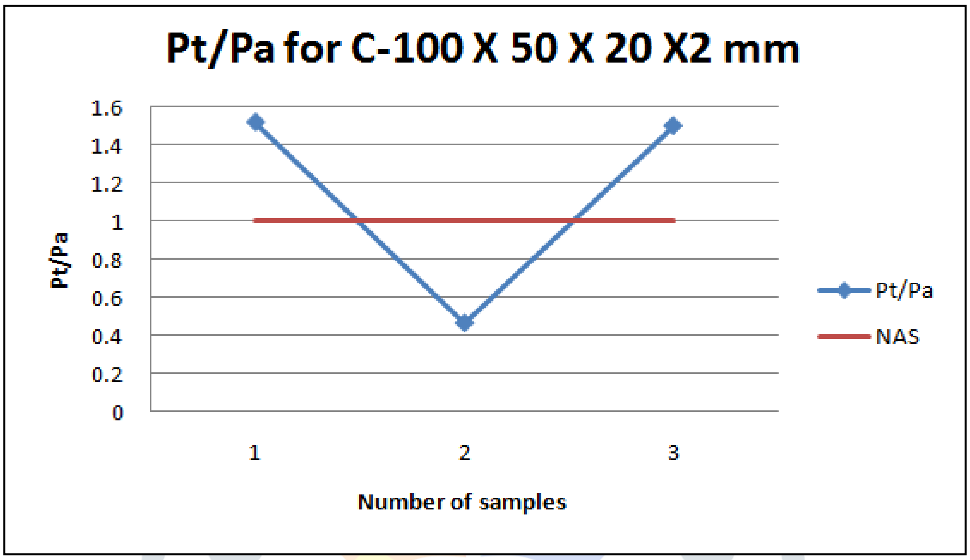

Figure 11 Graphical representation of result

- For sample 1, ITF with length 420 mm, pure web crippling mode of failure was observed i.e. only web crippling.
- For sample 2, ETF with length 100 mm, pure web crippling mode of failure was observed i.e. only web crippling.
- For sample 3,ETF with length 310 mm, pure web crippling mode of failure was observed i.e. only web crippling.

V. CONCLUSION

Pure web crippling modes were considered and results were obtained. Hence, these results are far greater than unit value one. For combined modes i.e when length of member increases mode of failure changes i.e. bearing failure, with some amount of bending occurs and then test values comes closer to analytical and hence ratios of test and analytical is near one. From above experiments performed for pure web crippling shows that AISI S100-16, considers combined modes of failure in which web crippling is critical mode of failure. Also we can, conclude that, as span of beam increases, it leds to fail in bearing.

VI. ACKNOWLEDGEMENT

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VII. NOTATIONS

Pa- Analytical load
Pt- Test load
F_y - Design yield stress
IS - Indian Standard
AISI- American Iron and Steel Institute
Wc- Web crippling
Bf- Bearing failure

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