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Ultimate Capacity of Slender Section Beam-columns with Corrugated Webs

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Abstract

The behavior and strength of slender section beam-columns with corrugated webs under eccentric and concentric loading are affected by many geometric parameters. The effect of these parameters has been studied analytically in the present work. A large number of members having different slenderness ratios, flange width-thickness ratio, web depth-thickness ratio, web panel width-thickness ratio, and corrugation depth-flange width ratio; are investigated. The recommended inclination angle (45°) of corrugated web panel that gives best results for low, intermediate and high slenderness ratio is used in the present study. Complete ultimate strength-member slenderness ratio curves are presented. Failure modes observed are local, interactive local-global and overall buckling. For design purpose; a series of interaction curves for slender I-section beam-columns with corrugated webs were drawn with different normal forces along with either major axis moments or minor axis moments. It is generally considered that the contribution of the web to the ultimate capacity of a beam-column with corrugated web is negligible, and the ultimate capacity will be based on the flanges yield stress. The flange width-thickness ratio is much more effective on the ultimate stress capacity of the member than the web depth-thickness ratio. The member ultimate stress decreases with the increase of web sub panel width-thickness ratio. For members having major axis bending, flange width-thickness ratio has major effect on the member capacity more than that of the web depth-thickness ratio because the flanges have the main role in load carrying capacity.

1. Introduction

Conventional steel plate girders and beam-columns have been used in steel construction for many years. New generation of optimized steel girders is developed using the advances in structural and fabrication technology. One of the developments in structural steel during the past few years has been the production of corrugated web I-beams. In order to increase web buckling strength, traditionally steel beam-columns; construction involves the use of transversely stiffened slender web plates. Engineers have realized that corrugation in webs increase their stability against buckling and can result in an economical design. The web corrugation profile can be viewed as uniformly distributed stiffening in the transverse direction of the beam. When girders with corrugated webs are compared with those with stiffened flat webs, it can be found that trapezoidal corrugation in the web enables the use of thinner webs. Also; corrugated web I-Beams (CWB) eliminate costly web stiffeners and provide a high strength-to-weight ratio. Such beams have been manufactured and used in Sweden, France, Germany, U.S.A. and Japan in buildings and bridges.

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In France, corrugated steel webs were used in a composite girder bridge with concrete flanges along with trapezoidal corrugated steel web 8 mm thick; and web depth-to-thickness ratio in the range of 220 to 375. The beams manufactured and used in Germany have trapezoidal corrugated web with thickness that varies between 2 and 5mm. The web depth-to-thickness ratio was in the range of 150 to 260. Austria, as well, manufactures such beams and uses them. Japan has constructed the Sugitanigawa Bridge, which is a PC6 span connected rigid frame corrugated steel web box girder bridge on the Second Tomei High Way.

The authors will briefly classify previous research approaches considering beams with corrugated web. Most of the researches and studies were to study the shear capacity. The scholars working in this area include Easley (1969), Bergfelt and Aravena (1984), a summary of the research and development in beams and girders with corrugated webs was reported by Elgaaly and Dagher (1990), Scheer et al. (1991), Elgaaly et al. (1996), Johnson and Cafolla (1997).

The main objective of the present research is to study the behavior of slender section beam-columns with corrugated web under concentric and eccentric axial load. It includes the change of different geometrical parameters such as flange width-thickness ratio and other different geometrical configurations of the corrugated web which have effect on the flanges stability. Different cases of slender section columns with corrugated web subjected to different normal forces along with either major axis moments or minor axis moments; are considered. Large number of members having different cross section geometrical parameters as well as member slenderness ratios were selected for each case to draw complete ultimate strength curves. For each case; the ultimate stresses developed in member critical section are drawn and investigated.

2. Finite element model

A nonlinear finite element model was established using the commercial program (COSMOS/M 2.8) to determine the capacity and behavior of slender section beam-columns with corrugated webs. The elements used in the modeling are 4-node quadrilateral thin shell elements "SHELL 4T". This shell element has both membrane and bending capacities for the three dimensional analysis of the structural models. Six degrees of freedom per node; (three translations and three rotations); are considered for structural analysis. The elements are assumed to be isotropic with constant thickness for the problem under consideration. The nodal input pattern for the element both clockwise and counter clockwise node numbering are allowed. For elements coordinate system, the x-axis goes from the first node to the second. The mesh which is used for all models accounts for elements with width 50mm, and the element height is taken so that the aspect ratio of the element does not exceed 2.0. The nodes on the line of intersection between the flanges and the corrugated web have the same degrees of freedom (U_x, U_y, U_z, R_x, R_y, R_z).

A bilinear stress-strain curve which obeying von Mises yield criterion was adopted for material modeling as shown in Figure 1.The material used is mild steel with yield stress of 240 MPa. Automatic Arc-length load control technique is used to control the increment of the external loads. In this technique the loads are incremented according to predefined time curve. Time curve describes the relation between the step number and the value of the load at this step. Each load has its time curve. Moreover, this technique allows for the convergence of the solution in the post-buckling region. Therefore, the descending part of the load-deformation curve can be plotted.

The slender section beam-column with corrugated web is assumed to be simply supported at both ends. Two end head plates are placed at both ends of the beam-column to cover the member cross section. In order to prevent the local buckling and yielding at the end, thick head plates were provided at both ends of the member. The axial concentric force is applied on the

middle of the end plates. The finite element model developed takes into consideration the symmetry of the member, as such, only one half of the column is considered.
Members having sections with various flange width-thickness ratios, web width-thickness ratios and many other geometric parameters are considered in this study. A schematic drawing of finite element half model and the boundary conditions along with the global axis are shown in Figure 1.

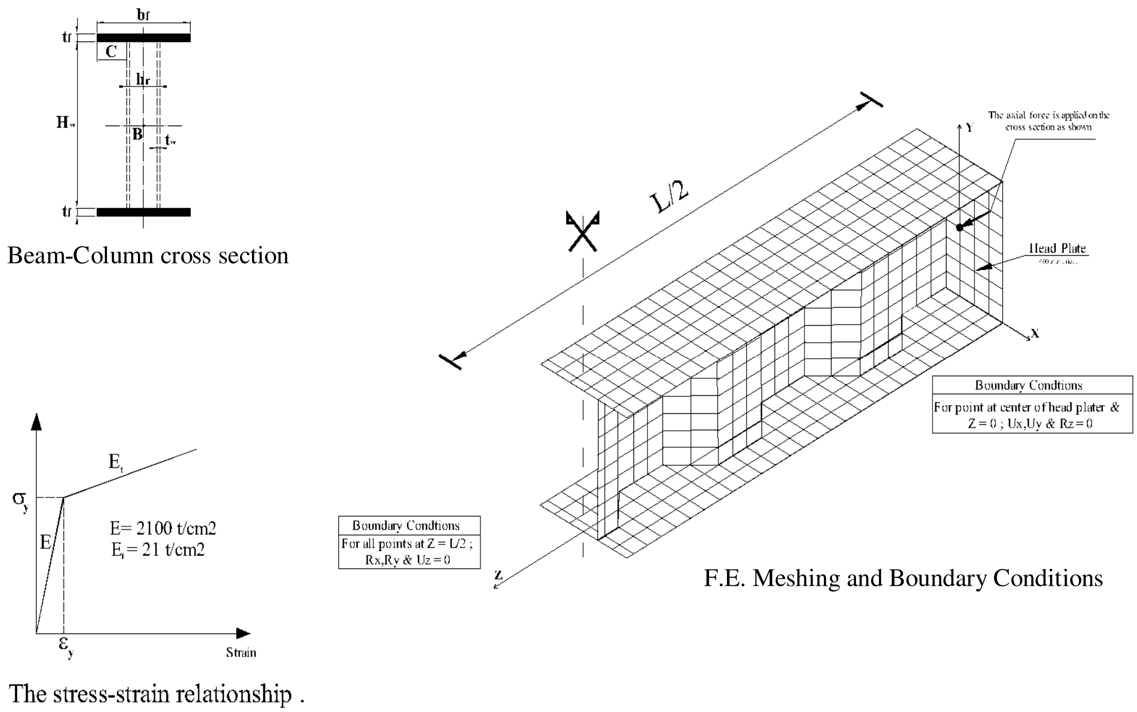


Figure 1: Finite element model

3. Parametric study variables

The main objective of the present study is to investigate the behavior of slender beam-columns with corrugated web under concentric and eccentric loads by considering the effect of some parameters. The web corrugation profile is shown in Figure 2. An introductory study is performed by Marrwa N. (2009) to figure out the most effective ratios of corrugated web geometrical parameters such as; flange outstand to flange thickness ratio C/t_f , web horizontal panel width to thickness ratio b/t_w, corrugation depth to flange width h_f/b_f and inclination angles θ of corrugated web. This study is conducted for members subjected to axial compressive force or under flexural moments applied at member ends. Marrwa concluded that the optimum member axial capacity is corresponding to depth to flange-width ratio $(h_f/b_f) = 0.667$. As well, inclination angles of the corrugated web θ equal to 45^o gives best member capacity for low, intermediate and high slenderness ratio L/r_y of axially or uni-axially loaded members.
These results are implemented in the present study. Also, in the present study; the limits of axially loaded members with slender sections are considered as provided by the American Specification "AISC-1996". These limits; for "web subject to compression"; the minimum width - thickness ratio, H_w/t_w is $586.3/(F_y)^{0.5}$;and $1433/(F_y)^{0.5}$ for web subjected to flexure. For un-stiffened "flange"; the minimum width-thickness ratio, b_f/t_f is $190/(F_y)^{0.5}$. For mild steel with F_y= 240MPa, the previous values are 37.84 for web subjected to compression and 92.5 for web subjected to bending, while it is 12.26 for un-stiffened flanges.

Geometric parameters have been considered in the present study are member slenderness ratio L/r_y, flange width-thickness ratio b_f/t_f, web depth-thickness ratio H_w/t_w, and web sub-panel width-thickness ratio b/t_w,

Various numbers of models are studied in which their dimensions include: web sub-panel width b =150 mm and length l =141mm along with web corrugation angle $\theta = 45^o$. In addition; the flange width-thickness ratio, b_f/t_f, is in the range from 15 to 30; while the web width-thickness ratio H_w/t_w is in the range from 50 to 380. For each cross section; different member lengths are chosen to have wide range of member slenderness ratio L/r_y that ranges from 50 to 300. Values of b_f considered in the present study are 75,100,120 and 150 mm. Also; web depth is varied from 300mm up to1900mm, while web and flange thickness are kept constant with value equal 5 mm.

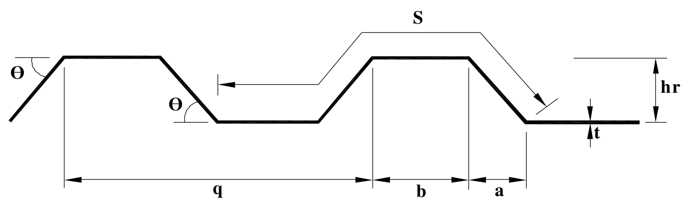


Figure 2: Corrugation profile dimensions

4. Numerical results

4.1 Members subjected to axial compressive forces

The axial load is applied as concentrated force on the center of the end head plates of the beam columns. The end head plates are 40mm thick. Such thickness is sufficient to distribute the stresses uniformly on the whole column cross section.

4.1.1 Load-displacement relationships

For each case; the load-lateral displacement (Ux), of the web midpoint (B) at member mid-height; is monitored. These displacements of the web indicate the post-local buckling strength gained by the member. Figures 3&4 show the load-displacement relationship curves; for two for member sections; versus different slenderness ratios, L/r_y.

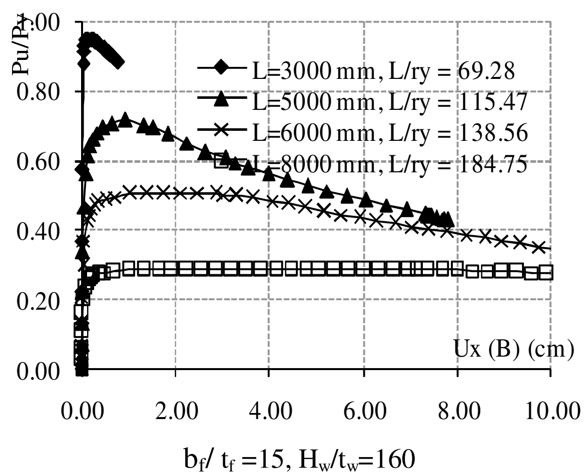
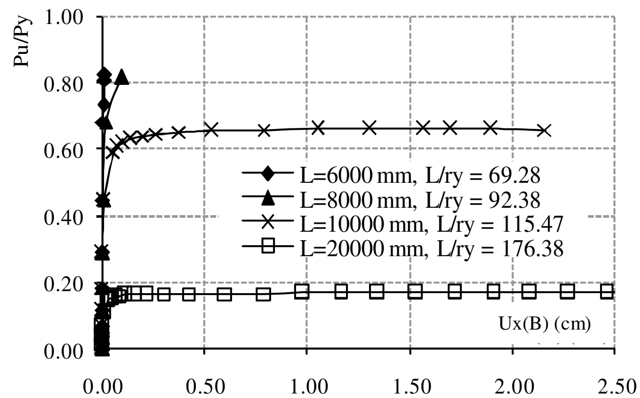


Figure 3: Load-displacements of slender beam-columns with corrugated webs subjected to axial compressive force with b_f/t_f =15

Members with low or intermediate slenderness ratio (L/r_y = 0 to 120), failed by local buckling mode or failed by interactive local-global buckling mode. However; members having high slenderness ratio (L/r_y = 120 to 300), failed by overall buckling. The slope of interaction curve of members that failed by global buckling increases up to global buckling load then remains constant up to failure load showing the post-local buckling strength gained by the section.



$b_f/t_f = 30$, $H_w/t_w = 160$

Figure 4: Load-displacements of slender beam-columns with corrugated webs subjected to axial compressive force with $b_f/t_f = 30$

4.1.2 Ultimate strength curves

Ultimate axial load P_u, is normalized with respect to the squash load P_y of the columns; where ($P_y = A_f * F_y$); and A_f is the area of flanges. The normalized loads are drawn with respect to the member slenderness ratios about the minor axis L/r_y, as shown in Figures 5&6. The figures present the relations for different flange width-thickness ratios, b_f/t_f, and different web depth-thickness ratios, H_w/t_w. The figures indicates that the ultimate loads of members with low slenderness ratio L/r_y decrease with the increase of the section flange width-thickness ratio b_f/t_f, as well as the increase of web depth-thickness ratio H_w/t_w. For members with large slenderness ratio L/r_y, all curves is asymptotic to the theoretical Euler's buckling curve.

Generally, members with low and intermediate slenderness ratios suffer local buckling which depends on the flange width-thickness ratio b_f/t_f, and web depth-thickness ratio H_w/t_w, before

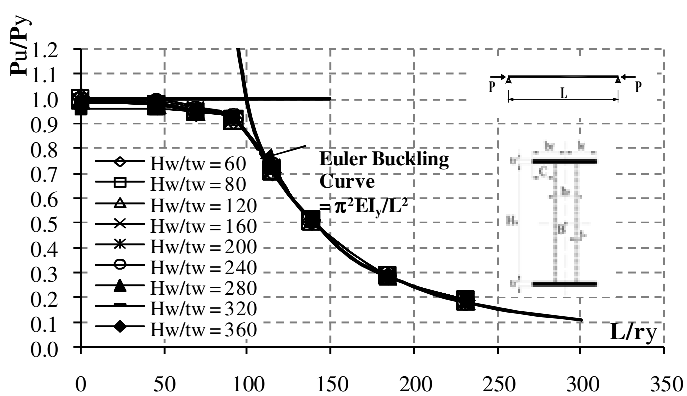


Figure 5: Normalized ultimate axial load versus member slenderness ratio for beam column with corrugated web having different H_w/t_w with $b_f/t_f = 15$

reaching the failure load. However, members with large slenderness ratios fail in the overall buckling similar to members having compact or non-compact sections.

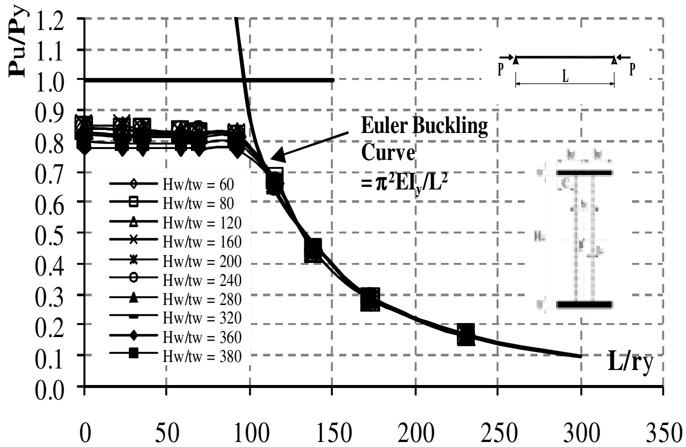


Figure 6:Normalized ultimate axial load versus member slenderness ratio for beam column with corrugated web having different H_w/t_w with b_f/t_f =30

As shown in Figure 7; members with low and intermediate slenderness ratio L/r_y the member ultimate stress decreases with the increase of flange width-thickness ratio b_f/t_f, while for members with high slenderness ratio L/r_y, stress decrease with the increase of flange width-thickness ratio b_f/t_f, but with less amount than case of low and intermediate slenderness ratios. While for different web depth-thickness ratios H_w/t_w ;the ultimate strength is nearly the same for the same slenderness ratio L/r_y.

For different flange outstanding length-thickness ratio C/t_f, it can be seen that the ultimate strength capacity for members increases with the decrease of C/t_f as well as the increase of $H_w/t_{w;}$ as shown in Figure 8.

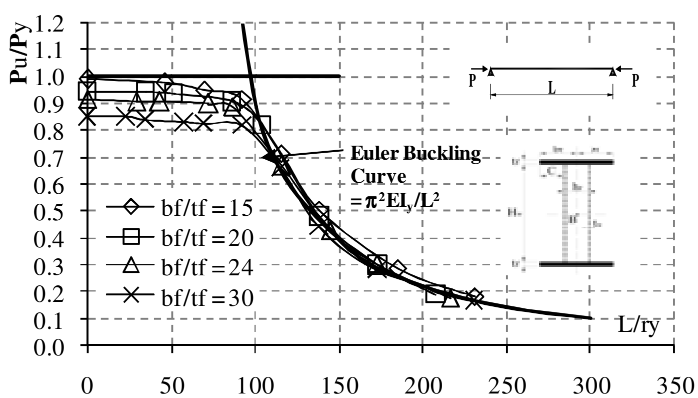


Figure 7:Normalized ultimate axial load versus member slenderness ratio for beam column with corrugated web having different b_f/t_f ratios

Figure 9 shows a comparison between slender beam-columns with corrugated web and slender beam-columns with conventional flat web. Both beam-columns are subjected to axial compressive force. Geometric parameters considered are H_w/t_w =160 and b_f/t_f = 15, 20, 24 and

30. The ultimate axial capacities of members with corrugated webs have higher capacity by 25% to 50% and decreases with the increase of member slenderness ratio L/r_y.

4.1.3 Stress distribution at failure loads

Figures 10 and 11, show the normal stress distribution at failure load of beam-columns having two selected sections, different member slenderness ratios, L/r_y, and different flange width-thickness ratio b_f/t_f. The plotted stress distribution is at member mid span critical section. These sections have b_f/t_f = 15 and 30, and H_w/t_w = 160. It is clear that the corrugated web cannot sustain any axial stresses due to its longitudinal flexibility, whereas the flanges are the ones which carry most of the axial load.

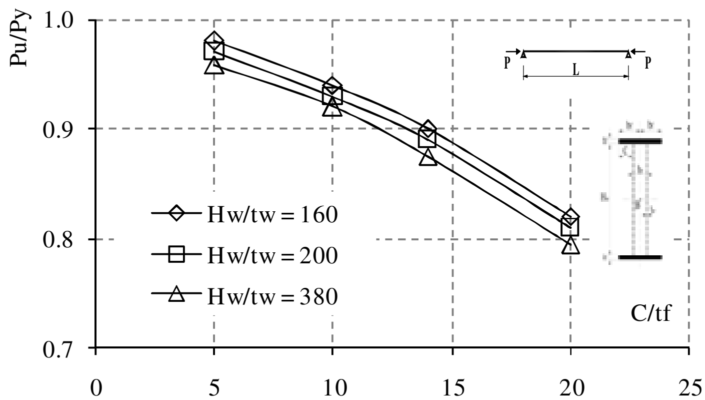


Figure 8: Normalized ultimate axial load versus C/t_f
for beam column with corrugated having different H_w/t_w ratios

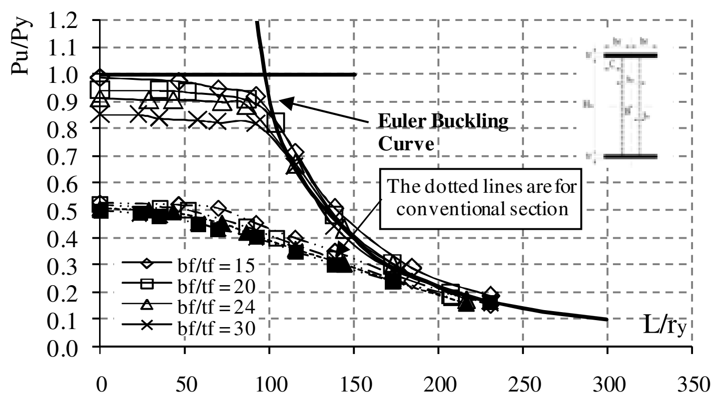


Figure 9: Normalized ultimate axial load versus Normalized ultimate moments
for beam column with corrugated web as well as beam column with flat web
having (H_w/t_w = 160)

In addition; members with b_f/t_f = 15, H_w/t_w =160, low slenderness ratio (L/r_y < 50); failed by local buckling occurred in the web only. Moreover; due to compatibility between flanges and the web the flange rotated with the web. However; members having b_f/t_f 30, H_w/t_w =160, local buckling occurred in both flanges and in the web at the same time. Members with intermediate slenderness ratio (L/r_y = 80 to 120) failed by interactive local-global buckling. For sections with b_f/t_f =15, H_w/t_w =160, the local buckling waves occurred in the web and overall buckling occurred in the whole member. However; for sections with b_f/t_f =30, H_w/t_w =160, the local buckling waves occurred in both web and flanges, also stress distribution across the web is also too small and may be neglected. Members with large slenderness ratio (L/r_y = 120 to 300) failed by overall buckling followed by post buckling for both sections having flange width - thickness ratio b_f/t_f =15 and 30.

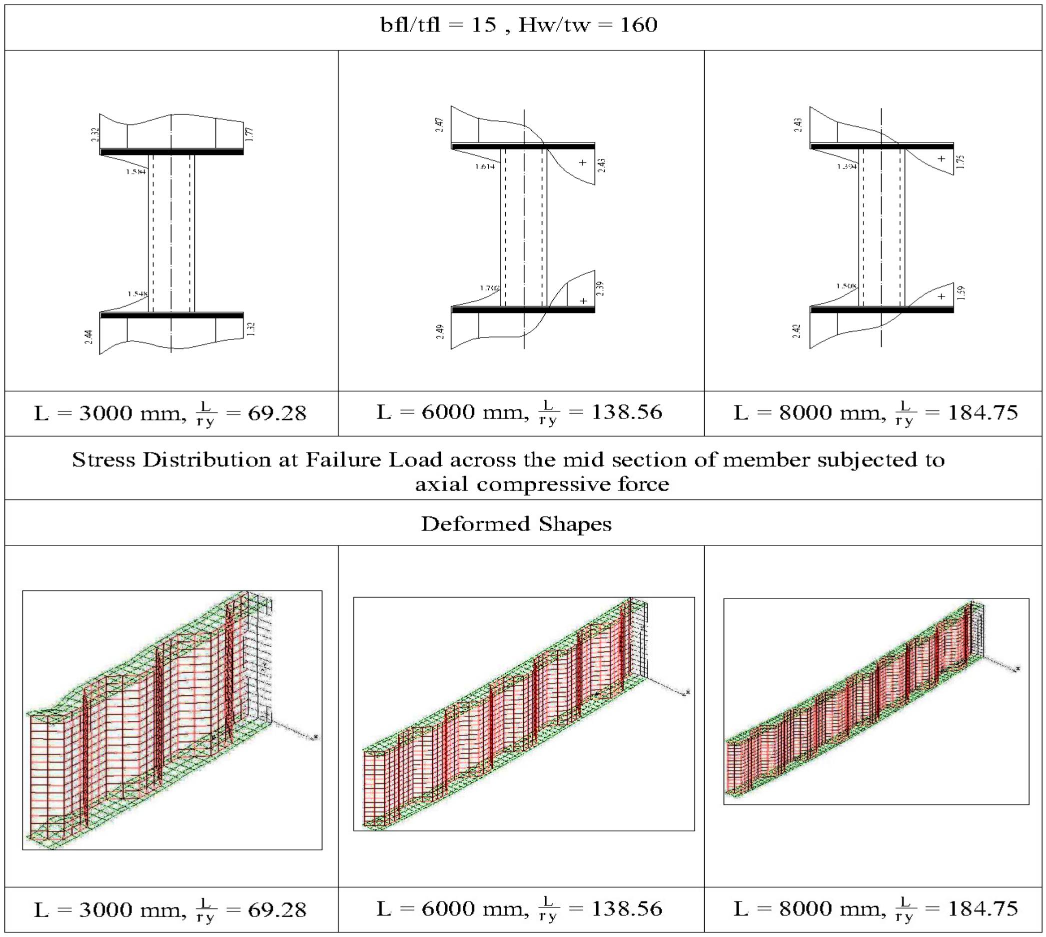


Figure 10: Stress distribution on cross section in members with b_f/t_f = 15 and H_w/t_w = 160

4.2 Members subjected to axial compressive forces together with major axis bending

Beam-columns subjected to axial compression and equal end moments causing single curvature in the members about the major axis X-X are investigated. The loads are applied on the members sequentially. The axial force is applied first from zero to a certain value, this value is kept constant, and then the bending moment is applied from zero to the ultimate value. The axial force is applied at the web midpoint, while the bending moment force is applied in the middle of the upper and bottom flanges.

The ultimate stress capacity of the column section clearly decreases with increasing of the flange width-thickness ratio b_f/t_f, while it is not affected that much by increasing the web depth-thickness ratio H_w/t_w. Members with b_f/t_f = 15, 24 and 30 and low slenderness ratio L/r_y failed by local buckling, while members with intermediate slenderness ratio L/r_y failed by interactive local-global buckling. Also members with large slenderness ratio L/r_y failed by overall buckling mode. The interaction curves between, P_u/P_y and M_x/M_{xy} are drawn for different member slenderness ratios L/r_y in Fig. 12. This figure presents this interaction relationship for members having flange width-thickness ratio b_f/t_f=15.

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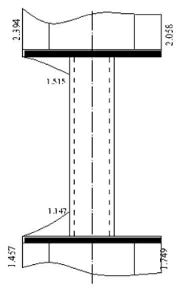
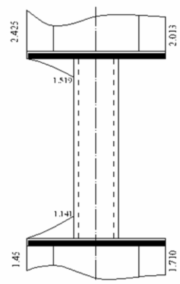
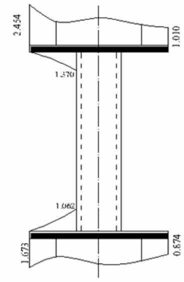
<tr><td>
</td><td>
</td><td>
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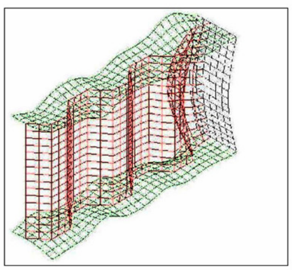
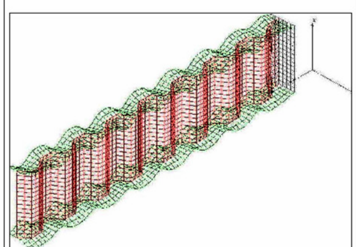
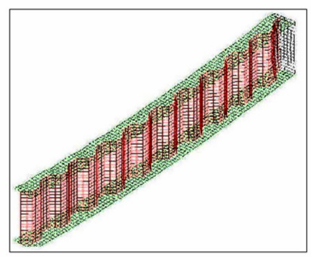
<tr><td>L = 6000 mm, $\frac{L}{r_y}$ = 69.28</td><td>L = 8000 mm, $\frac{L}{r_y}$ = 92.38</td><td>L = 10000 mm, $\frac{L}{r_y}$ = 115.47</td></tr>
<tr><td colspan="3">Stress Distribution at Failure Load across the mid section of member subjected to axial compressive force</td></tr>
<tr><td colspan="3">Deformed Shapes</td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td>L = 6000 mm, $\frac{L}{r_y}$ = 69.28</td><td>L = 8000 mm, $\frac{L}{r_y}$ = 92.38</td><td>L = 10000 mm, $\frac{L}{r_y}$ = 115.47</td></tr>
</table>

Figure 11: Stress distribution on cross section in members with $b_f/t_f = 30$ and $H_w/t_w = 160$

The interaction relationship curves are almost linear. The reduction in the ultimate strength due to increase in the flange and web width-thickness ratio is very clear for members with low slenderness ratio (L/r_y = 50 to 100). The ultimate strength decreases with increasing member slenderness ratio L/r_y. For members with slenderness ratio about L/r_y =150, the reduction in strength is clear for large values of P_u/P, while for the case where P_u/P_y equals zero the reduction is affected by flange width-thickness ratio more than web depth-thickness ratio which is not that much effective. The ultimate strength of members with L/r_y = 200 is not affected by the flange width-thickness ratio and web depth-thickness ratio; since failure is due to overall buckling mode.

4.3 Members subjected to axial compressive forces with minor axis bending

In this section the ultimate capacity of members subjected to axial compressive force P plus minor axis bending moments M_y is determined. These bending moments are acting at the ends of the members causing single curvature about the minor axis Y-Y. The axial force is applied at the web midpoint, while the bending moment equivalent forces are applied on the upper and bottom flanges corners of the member end section.

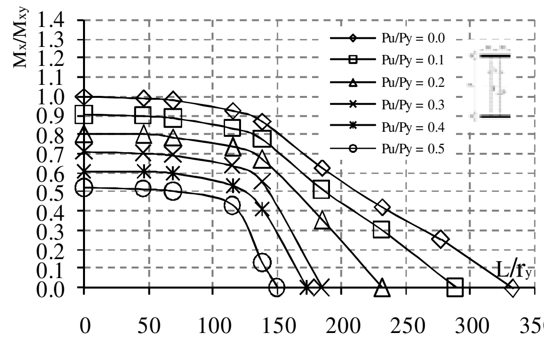


Figure 12:Normalized ultimate moments versus member slenderness ratio for beam column having ($b_f/t_f = 15$& $H_w/t_w = 200$)

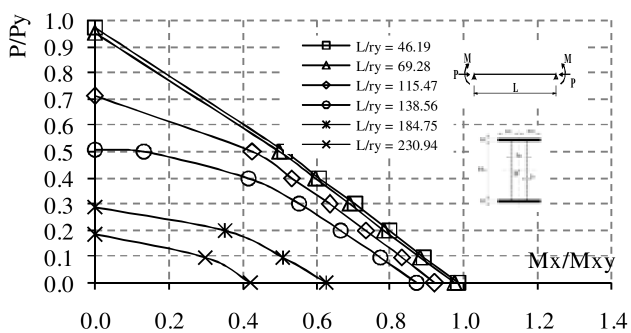


Figure 13: Normalized ultimate axial load versus Normalized ultimate moments for beam column having ($b_f/t_f = 15$& $H_w/t_w = 200$)

4.3.1Ultimate strength capacity

Fig. 14 shows the relation between the normalized M_y with respect to the yield minor axis bending moment M_{yy} versus member slenderness ratio L/r_y. Curves are plotted for P_u/P_y = 0.0, 0.2 & 0.4, along with flange width-thickness ratio b_f/t_f = 15 & 30, and web depth-thickness ratio H_w/t_w =160, 200 & 380. Each curve presents the results obtained for a certain column section with certain flange width-thickness ratio and web depth-thickness ratio. The ultimate stress capacity of the section clearly decreases with increasing of the flange width-thickness ratio b_f/t_f , while it is not affected that much by increasing the web depth-thickness ratio H_w/t_w. Also the percent reduction in ultimate stress capacity due to increasing of P_u/P_y ratio is decreased with increasing of flange width-thickness ratio b_f/t_f. Members with b_f/t_f = 15 and different slenderness ratio L/r_y, suffer no local buckling and failed by global buckling, while members with b_f/t_f= 30 and low or intermediate slenderness ratio L/r_y, failed by interactive local-global buckling mode while members with large slenderness ratio L/r_y; failed by global buckling.

The interactions curves between, P/P_y and M_y/M_{yy} are drawn for different member slenderness ratios. Fig.15 presents this relation for different flange width-thickness ratios and web depth-thickness ratios. The interaction curves are almost linear. The ultimate strength decreases with increasing member slenderness ratio L/r_y. For members with slenderness ratio L/r_y =100 to200, the reduction in strength is clear for large values of P/P_y ratios. The ultimate strength of members with L/r_y greater than 200 is not affected by the flange width-thickness ratio and web depth-thickness ratio because overall buckling failure is controlling the member behavior.

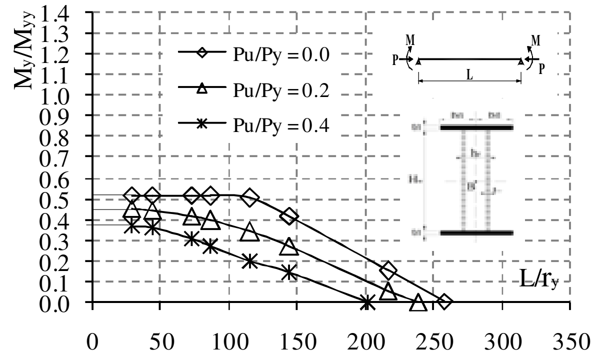


Figure 14:Normalized ultimate moments versus member slenderness ratio
for beam column having ($b_f/t_f = 24$& $H_w/t_w = 200$)

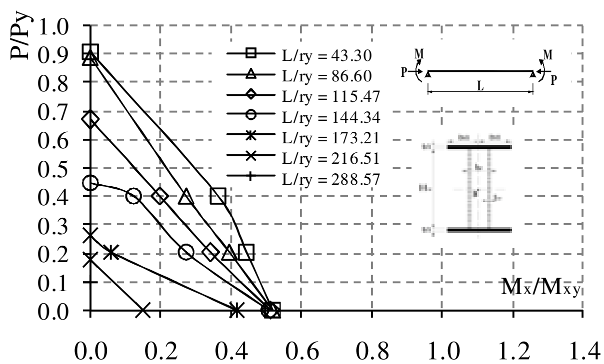


Figure 15: Normalized ultimate axial load versus Normalized ultimate moments
for beam column having ($b_f/t_f = 24$& $H_w/t_w = 200$)

5. Conclusions

From the studies carried out in the present paper, the following results may be drawn out:

It is generally considered that the contribution of the web to the ultimate capacity of a beam-column with corrugated web is negligible, and the ultimate capacity will be based on the flanges yield stress.

The flange width-thickness ratio is much more effective on the ultimate stress capacity of the member than the web depth-thickness ratio.

The ultimate axial capacity of member of corrugated web is greater than member with conventional flat web by 25% to 50% and decreases with the increase of member slenderness ratio.

The ultimate capacity of member of corrugated web subjected to major axis bending is greater than member with conventional flat web by 10% to 15%; and decreases with the increase of member slenderness ratio.

The ultimate load is highly affected by width-to-thickness ratio of the web and flange for members having very low slenderness ratios (below 100). While for members having large slenderness ratios (above 100), the effect of width-to-thickness ratio of the web and flange is negligible.

For members having Major axis Bending, flange width-thickness ratio has major effect on the member capacity more than that of web width-thickness ratio; because the flanges have the main role in load carrying capacity.

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