CIVE 3205

Example C3

Built-up Column Sections

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N.M. Holtz

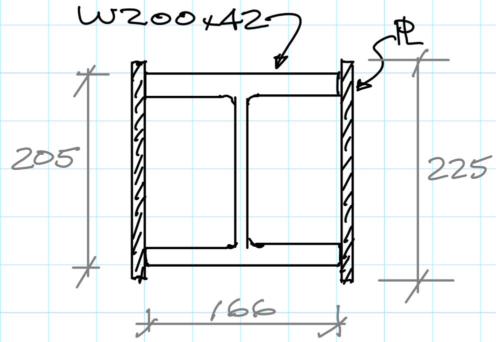
Revisions:

- Mar 1, 2012 - original posting

An existing column is a W200x42 of 350W steel. Due to building renovation and change of use, the column will have an unbraced length of 4500 mm and must support a factored axial load of 950 kN.

From p. 4-41, $C_r \approx 620$ kN $\therefore$ N.G.

Solution:
- reinforce column by welding plates to flange tips:



provide $\approx$ 10mm for welding ea. side
$\therefore$ width $\approx 205 + 10 + 10 = 225$

W200 x 42

$A = 5310$ mm^2 $\qquad b = 166 \quad t = 11.8$

$I_x = 40.9 \times 10^6$ mm^4 $\qquad d - 2t = 181$

$I_y = 9.00 \times 10^6$ mm^4 $\qquad w = 7.2$

To estimate plate size:

$$A_{req} = \frac{950}{620} \times 5310 = 8136 \text{ mm}^2$$

$$A_{PL} = 8136 - 5310 = 2826$$

$$t_{req} = \frac{2826}{225 \times 2} = 6.3 \text{ mm.}$$

try 7mm PL (a preferred size, see p 6-145)

i) local buckling
all components are supported on 2 edges
so appropriate limit is $\frac{670}{\sqrt{F_y}} = \frac{670}{\sqrt{350}} = 35.8$

flange: $\frac{b_{el}}{t} = \frac{166}{2 \times 11.8} = 7.0 \ll 35.8$ o.k.

web: $\frac{h}{w} = 181/7.2 = 25.1 < 35.8$ o.k

PL : $\frac{h}{w} = \frac{205}{7} = 29.3 < 35.8$ OK

(205 is appropriate h = dist between welds

$\therefore$ local buckling is OK.

Compute section properties

$$I_x = 40.9\times10^6 + \frac{2\times7\times225^3}{12}$$
$$= 54.19\times10^6 \text{ mm}^4$$

$$A = 5310 + 2\times7\times225$$
$$= 8460 \text{ mm}^2$$

$$r_x = \sqrt{\frac{54.19\times10^6}{8460}} = 80.03 \text{ mm}$$

$$I_y = 9.00\times10^6 + 2\times7\times225\left(\frac{166}{2}+\frac{7}{2}\right)^2$$
$$= 32.57\times10^6 \text{ mm}^4$$

$$r_y = \sqrt{\frac{32.57\times10^6}{8460}} = 62.05 \text{ mm}$$

ii) overall strength

$$\left(\frac{KL}{r}\right)_{max} = \frac{1.0\times4500}{62.05} = 72.52$$

$$F_e = \frac{\pi^2\times200000}{72.52^2} = 375.3$$

$$\lambda = \sqrt{\frac{350}{375.3}} = 0.9657$$

$$n = 1.34$$

$$C_r = \phi A F_y \left(1+\lambda^{2n}\right)^{-1/n}$$
$$= 0.9\times8460\times.35\left(1+0.9657^{2.68}\right)^{-1/1.34}$$
$$= 1644 \text{ kN} \gg 950 \text{ kN} \qquad \text{O.K.}$$

Try 6mm PL

i) local buckling

$$\frac{h}{w} = \frac{205}{6} = 34.2 < 35.8$$

However edge restraint is likely less than that of a regular web & the $\frac{670}{\sqrt{F_y}}$ limit may not be fully justified

Though Table 1 allows this for cover plates, which are similar

$\therefore$ OK

Section properties

$A = 5310 + 2 \times 6 \times 225 = 8010 \text{ mm}^2$

$I_y = 9.00 \times 10^6 + 2 \times 6 \times 225 \times \left(\frac{166}{2} + \frac{6}{2}\right)^2$

$= 28.97 \times 10^6 \text{ mm}^4$

$r_y = \sqrt{\frac{28.97 \times 10^6}{8010}} = 60.14$

ii) overall strength

$\left(\frac{KL}{r}\right)_{max} = \frac{1 \times 4500}{60.14} = 74.83$

$F_e = \frac{\pi^2 \times 200000}{74.83^2} = 352.5$

$\lambda = \sqrt{\frac{350}{352.5}} = 0.9964$

$n = 1.34$

$C_r = 0.9 \times 8010 \times .35 \times \left(1 + .9964^{2.68}\right)^{-1/1.34}$

$\underline{\underline{C_r = 1510 \text{ kN}}} \qquad >> \quad 950 \text{ kN}$

Its not really practical to make the plates thinner or narrower.

Use W200 × 42
PL 6×225 continuously welded to each side, full height

P.S.

As per commentary for § 13.3.1, because of higher residual stresses induced by welding, it might be appropriate to use $n = 0.93$.

$n = 0.93$

$C_r = 0.9 \times 8010 \times .35 \times \left(1 + .9964^{1.86}\right)^{-1/0.93}$

$C_r = 1200 \text{ kN} > 900 \text{ kN}$ O.K.