CIVE 3205
Example AC30
Built-up Column Sections
Feb 26, 2020

N.M. Holtz

Revisions:

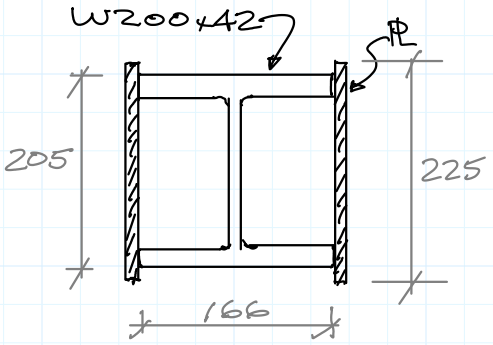
- Feb 26/20 – original posting

AC30-1

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 ∴ N.G.

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



provide ≈ 10mm for welding ea. side
∴ width ≈ 205 + 10 + 10 = 225

W200 x 42

$A = 5310$ mm^2 $\quad b = 166$ $\quad t = 11.8$
$I_x = 40.9 \times 10^6$ mm^4 $\quad d - 2t = 181$
$I_y = 9.00 \times 10^6$ mm^4 $\quad w = 7.2$

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: $\frac{b_{eff}}{t} = \frac{166}{2 \times 11.8} = 7.0 \ll 35.8$ 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

∴ local buckling is OK.

AC30-1 (continued)

Compute section properties

$I_x = 40.9 \times 10^6 + 2 \times 7 \times \frac{225^3}{12}$

$= 54.19 \times 10^6 \text{ mm}^4$

$A = 5310 + 2 \times 7 \times 225$

$= 8460 \text{ mm}^2$

$r_x = \sqrt{\frac{54.19 \times 10^6}{8460}} = 80.03 \text{ mm}$

$I_y = 9.00 \times 10^6 + 2 \times 7 \times 225 \left(\frac{166}{2} + \frac{7}{2}\right)^2$

$= 32.57 \times 10^6 \text{ mm}^4$

$r_y = \sqrt{\frac{32.57 \times 10^6}{8460}} = 62.05 \text{ mm}$

ii) overall strength

$\left(\frac{kL}{r}\right)_{max} = \frac{1.0 \times 4500}{62.05} = 72.52$

$F_e = \frac{\pi^2 \times 200000}{72.52^2} = 375.3$

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

$n = 1.34$

$C_r = \phi A F_y \left(1 + \lambda^{2n}\right)^{-1/n}$

$= 0.9 \times 8460 \times .35 \left(1 + 0.9657^{2.68}\right)^{-1/1.34}$

$= 1644 \text{ kN} \quad >> \quad 950 \text{ kN} \quad \text{O.K.}$

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

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

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

$= 28.97 \times 10^6\ mm^4$

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

ii) overall strength

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

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

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

$n = 1.34$

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

$C_r = 1510\ kN \quad >> \quad 950\ kN$

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

Use W200 x 42
PL 6x225 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 $n = 0.93$.

$n = 0.93$

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

$C_r = 1200\ kN > 900\ kN$ O.K.