(Summer 2020)

Instrutor Engr Amjad Islam : Duration : 4hours

Subject : Steel Structure : Marks : 50

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betch : 2016

Dep : BS (Civil Engineering)

Q no# 01 ?

Given data lightest W-shape column

A – 36 Steel

D.L = 60K

L.L = 110K

pin supported at top and Bottom

$K_x l_x = 36ft$ $K_y l_y = 18ft$.

Ais / LRFD method.

Sol: Required capacity = $(1.2 \times 60) + (1.6 \times 110)$

$= 248K$

Enter design strength table of manual with KL = 18ft and P = 248K.

Sompossible section

W12x53 P = 320 $\frac{r_x}{r_y} = 2.11$

W14 x 61 P = 364 $\frac{r_x}{r_y} = 2.44$

W10 x 49 P = 301 $\frac{r_x}{r_y} = 1.71$

W8x58 P = 300K $\frac{r_x}{r_y} = 1.74$

Now

$\frac{K_x l_x}{K_y l_y} = \frac{36}{18} = 2$

try $W12 \times 53$ · $\frac{rx}{ry} = 2.11$

$$\frac{rx}{ry} > \frac{KxLx}{KyLy}$$

$rx = 5.23$, $ry = 2.48$, $A = 15.6 in^2$

$$\frac{KxLx}{rx} = \frac{36 \times 12}{5.23} = 82.6$$

$$\frac{KyLy}{ry} = \frac{18 \times 12}{2.48} = 87.09$$

$\frac{KL}{r} = 87.09$ selected.

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{Fy}{E}}$$

$$\frac{87.09}{\pi}\sqrt{\frac{36}{29,000}}$$

$$= 0.97 < 1.5$$

$$fcr = 0.658^{\lambda^2} \times fy$$

$$= 0.658^{(0.97)^2} \times 36$$

$$fcr = 24.28$$

$P_n = A_g f_{cr}$

$P_n = 15.6 \times 24.28$

$P_n = 378.78K$

$\phi P_n = 0.85 \times 378.78$

$= 321.96 > 248K$

OK

So

use $W_{12} \times 53$

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Qno#02 7722 8 5

Givendata:

D.L = 1.5 Kips

L.L = 4.5 Kips

fy = 36 KSi

l = 52 ft.

Sol:

the moment of inertial required to Satisfy the deflection limit Can be Calculated from eqn (5-11a) form book However the value Of Δ by multiply by 0.95 for the case of three Concentral load (table 5-4) the maximum moment due to Concentrated load = of

= 1.5 + 4.5 = 6Kips

$M = 1\frac{1}{2} \times 6 \times 26 - 6 \times 13$

M = 156 ft. Kips

Scanned with CamScanner

$$Eq(1) \rightarrow I = \frac{5}{48} \times \frac{ml^2}{E\Delta} \times 0.95$$

$$I = \frac{5}{48} \frac{(156 \times 12)(52 \times 12)^2}{29{,}00\left(\frac{52}{360} \times 12\right)}$$

$$I = 1510.51 \, in^4$$

try W24x62, $I_x = 1550 \, in^4$

$b_f = 7.04 \, in$

$d/A_f = 5.72$

$$L_c = \frac{76 \cdot b_f}{\sqrt{f_y}} = \frac{76 \times (7.04)}{\sqrt{36}} = 89'' = 7.41'$$

$$L_c = \frac{20{,}000}{f_y \frac{d}{A_f}} = \frac{20{,}000}{36 \times 5.72} = 97.12'' = 8.09'$$

$L > L_c$ from table 5.2

$$C_b = 1.13$$

$$= \sqrt{\frac{102{,}000 C_b}{f_y}} = \sqrt{\frac{102{,}000 \times 1.13}{36}} = 57$$

$$\sqrt{\frac{510{,}000 c_b}{f_y}} = \sqrt{\frac{510{,}000 \times 1.13}{36}} = 127$$

$$\frac{L}{r_T} = \frac{13 \times 12}{1.71} = 91.22$$

Condition

$$\sqrt{\frac{102{,}000 c_b}{f_y}} \le \frac{L}{r_T} \le \sqrt{\frac{510{,}000 c_b}{f_y}}$$

so $F_b = \left[\frac{2}{3} - \frac{f_y (L/r_T)^2}{1530 \times 10^3 \times c_b}\right] \times f_y$

$= \left(\frac{2}{3} - \frac{36 (91.22)^2}{1530 \times 10^3 \times 1.13}\right) \times 36$

$F_b = 17.76$ ksi allowable

the beam self wt $= \frac{62 \text{ lb}}{\text{ft}} = 0.062 \text{ k/ft}$

$M = \frac{wl^2}{8} = \frac{1}{8}(0.062)(52)^2$

$M = 20.95$ k.ft

Total $M = 156 + 20.95 = 176.95$

$S_x = 131$

$f_b = \frac{M}{S_x} \Rightarrow \frac{176.95 \times 12}{131} = 16.2$ ksi

Use $W_{24} \times 62$. $f_b < F_b$ — ok

QNo# 03?

7/7/22

Given data: A-36 Doule angle tension member

$L = 18ft$

$D.L = 50K$

$L.L = 150K$

The connection is bearing type with A-325 bolt with 3/4 inch dia (Standard hole)

use two line of bolt use ASD method

Sol:→ A36 load Doule angle tension?

Sol:- Total load = D.L + L.L

= 5 + 150

= 200 kips

or

100 kips/angle

For yielding at gross area allowable Stress are

$0.6fy \rightarrow 0.6 \times 36$

$= 22 ksi$

3/4

pto

for fracture at the net area allowable stresses are

$0.5\,f_u = 0.5 \times 58$

$= 29\,ksi$

since the connection bolted so

$A_g \neq A_n$

now $A_e = 0.85 A_n$

for yielding

$A_g \times 22 = 100$

$A_g = \frac{100}{22}$

$A_g = 4.54\ in^2$

For fracture

$29 \times A_e = 100$

$A_e = 3.44\ in^2$

$A_n = \frac{A_e}{0.85} = \frac{3.44}{0.85}$

$A_n = 4.04\ in^2$

Assume 15% deduction in gross area 10
for hole

So $A_g = \frac{A_n}{0.85} \Rightarrow A_g = \frac{4.04}{0.85}$

$A_g = 4.76$

for $L\ 4 \times 4 \times \frac{5}{8}$ $A_g = 4.61 \approx 4.76$ OK

$r_x = 1.20$ $r_y = 1.20$. with $\frac{3}{8}$ in Gusset plate

$\frac{L}{r_{min}} = \frac{18 \times 12}{1.20} = 180 = 300$ OK OK

Bolt design: using A-325 bolt with thread included in shear plane as dia = $\frac{3}{4}$"

$A = \frac{\pi d^2}{4} = \frac{\pi}{4}(0.75)^2 \Rightarrow 0.441\ in^2$

~~A = 0.44~~ in^2

allowable bolt ~~or~~ shear = 21 Ksi

Since bolt are in double shear so allowable shear per bolt = $2 \times 21 \times 0.44$

$= 18.5K$

allowable bolt bearing stress = 1.2 fu

= 1.2 × 58

= 69.6 ksi

Allowble bearing Stress on two $\frac{5}{8}$" thick angle

Long legs = 69.6 × 2 × $\frac{5}{8}$ × 0.75

= 65.25 > 18.5

So shear governs.

number of bolt = $\frac{200}{18.5}$

= 10.81

use 10 bolt

Design of gusset plate

Bearing Stress = 1.2 fu

= 1.2 × 58

= 69.6 ksi

so

allowble bearing = 69.6 × 10 × 0.75 × t = 200

t = 0.38 in

use $\frac{3}{4}$" G.P.

Checking various limit states.

yielding $= 0.6 f_y A_g$

$= 0.6 \times 36 \times (8 \times 0.75)$

$= 129.6 < 200K$

Not OK

try $L7 \times 4 \times \frac{1}{2}$ $A_g = 5.25$

$r_x = 2.25$, $r_y = 1.11$ with $\frac{3}{8}''$ G.P

$\frac{L}{r_{min}} = \frac{18 \times 12}{1.11}$ $194.59 < 300K$ OK

allowble bearing on two $\frac{1}{2}''$ thick angle

long leag s $= 69.6 \times 2 \times \frac{1}{2} \times 0.75$

$= 52.2 > 18.5$ So Shear governs

Checking varuis limit state

Yielding $= 0.6 f_y A_g$

$= 0.6 \times 36 \times (14 \times 0.75)$

$= 226.8 > 200K$ → OK

fracture $= 0.5 \times f_u \times A_e$

$= 0.5 \times 58 \times 0.85 \left(14 - \left(\frac{3}{4}\right) \times 2\right) \times \frac{3}{4}$

$= 231K > 200K$

→ OK

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