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Problem #1

(1)

Given DATA:

A-36 Steel

Length = 36 ft

D.L = 60 K

L.L = 110 K

Pin supported at top & bottom

$K_x L_x$ = 36 ft

$K_y L_y$ = 18 ft

Required:

Select the lightest W-shape

AISC/LRFD Method?

Solution:

Design Capacity = (1.2 × 60) + (1.6 × 110) =

= 248 K

Enter the design strength tables in Part 2 of the Manual with KL = 18 ft and P = 248 K

Some possible (2) Selection are

$W_{14} \times 61$ $\quad P = 364$ $\quad r_x/r_y = 2.44$

$W_{12} \times 53$ $\quad P = 380$ $\quad r_x/r_y = 2.11$

$W_{10} \times 49$ $\quad P = 301$ $\quad r_x/r_y = 1.71$

$W_{8} \times 58$ $\quad P = 300$ $\quad r_x/r_y = 1.74$

Now

$$\frac{K_x L_x}{K_y L_y} = \frac{36}{18} = 2$$

Try $W_{12} \times 53$ $\quad r_x/r_y = 2.11$

$$r_x/r_y > \frac{K_x l_x}{K_y l_y}$$

$r_x = 5.23$ $\quad r_y = 2.48$ $\quad A = 15.6 in^2$

$$\frac{K_x l_x}{r_x} = \frac{36 \times 12}{5.23} = 82.6$$

$$\frac{K_y l_y}{r_y} = \frac{18 \times 12}{2.48} = 87.09$$

(3)

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

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{e}}$$

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

$$= 0.97 < 1.5$$

$$F_{cr} = 0.658^{\lambda_c^2} \times F_y$$

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

$$F_{cr} = 24.28$$

$$P_n = A_g F_{cr}$$

$$= 15.6 \times 24.28$$

$$P_n = 378.78\ K$$

$\phi P_n = 0.85 \times 378.78$

$= 321.96 > 248$

ok

$\checkmark$ ✓

Use $W_{12} \times 53$

Q:2 Problem:

Given DATA:

Live Load = 4.5 Kips
Dead Load = 1.5 Kips
Length = 52 ft
Live Load deflection = 1/360 of the span
F_y = 36 Ksi

Required:

Determine the lightest W-shape section

Solution:

The Moment of Inertial required to satisfy the deflection limit can be Calculated from Eq. (5-11a) from book. However, the value of Δ by this equation should be multiplied by 0.95 for the case of three concentrated loads (Table 5-4).
The maximum moment due to the concentrated loads of $1.5 + 4.5 = 6$ Kips

(6)

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

$$M = 156 \text{ ft-Kips}$$

and Eq (5-11 a) yields

So

$$I = 0.95 \times \frac{5}{48} \frac{ML^2}{E\Delta}$$

Putting values

$$I = 0.95 \times \frac{5}{48} \times \frac{156 \times 12 (52 \times 12)^2}{29{,}000 \left(52 \times \frac{12}{360} \right)}$$

$$I = 1434.98 \text{ in}^4$$

From the Manual's moment of-Inertia Selection tables we find the lightest Section to be the $W_{24} \times 62$ for which, $I_x = 1550 \text{ in}^4$
$b_f = 7.04$ in, $d/A_f = 5.72$

~~Try~~ ~~W~~...

Try $W_{24\times62}$, $I_x = 1550 \text{ in}^4$

$b_f = 7.04$ in

$d/A_f = 5.72$

$$L_e = \frac{76 b_f}{\sqrt{F_y}} \Rightarrow \frac{76\times(7.04)}{\sqrt{36}}$$

$L_e = 89'' = 7.41'$

$$L_c = \frac{20{,}000}{F_y \frac{d}{A_f}} \Rightarrow \frac{20{,}000}{36\times5.72} = 97.2'' = 8.09'$$

$L > L_c$ from table 5.2

$C_b = 1.13$

$$\sqrt{\frac{102{,}000\ C_b}{F_y}} = \sqrt{\frac{102{,}000\times1.13}{36}} = 57$$

$$\sqrt{\frac{510{,}000\,C_b}{F_y}} = \sqrt{\frac{510{,}000 \times 1.3}{36}} = 127$$

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

Condition

$$\sqrt{\frac{102{,}000\,C_b}{F_y}} \le \frac{L}{r_T} \le \sqrt{\frac{510{,}000\,C_b}{F_y}}$$

So

$$F_b = \left[\frac{2}{3} - \frac{F_y (L/r_T)^2}{1530 \times 10^3 \times 1.13}\right] F_y$$

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

$F_b = 17.76$ Ksi allowable

The beam self weight $= 62\,\frac{lb}{ft} = 0.062\,k/ft$

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

$M = 20.95 \text{ k ft}$

Total $M = 156 + 20.95$

$M = 176.95$ $S_x = 131$

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

$$f_b < F_b$$

ok

Use $W_{24} \times 62$

Q: 3 Problem :-

Given DATA :-

Length = L = 18 ft

Dead Load = D.L = 50 K

Live Load = L.L = 150 K

Bearing Type Connection

A 325 bolts

Bolt diameter = 3/4" (Standard holes)

Thread, not excluded from shear plane

Use two lines of bolts.

Required: Determine an A-36 double tension member ?

Use ASD Method ?!

Solution:

Design Load = D.L + L.L

= 50 + 150

= 200 Kips

OR

100 Kips/Angle

→ For yielding at the gross area allowable stresses are

$$0.6 F_y = 0.6 \times 36$$
$$= 22 ksi$$

→ For Fracture at the net area allowable stresses are

$$0.5 F_u = 0.5 \times 58$$
$$= 29 \ ksi$$

→ Since the connection is not bolted

So $A_g \neq A_n$

Now $A_e = 0.85 A_n$

For Yielding

$$A_g \times 22 = 100$$

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

$$\boxed{A_g = 4.54 \ in^2}$$

For fracture

$$29 \times A_e = 100$$

$$A_e = 3.44 \text{ in}^2$$

$$A_n = A_e / 0.85 \Rightarrow 3.44 / 0.85$$

$$\boxed{A_n = 4.04 \text{ in}^2}$$

⇒ Assume 15% deduction in gross area for Holes

So,

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

$$\boxed{A_g = 4.76 \text{ in}^2}$$

For $L4 \times 4 \times \frac{5}{8}$ $\quad A_g = 4.61 \cong 4.76$ ok

$r_x = 1.20$ $\quad r_y = 1.20$ With $\frac{3}{8}$ in Gusset plate

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

ok

Bolts Design:

Using A325 bolts with threads Included in shear plane

as dia = $3/4''$

$$Area = \frac{\pi}{4}(d)^2 \Rightarrow \frac{\pi}{4}(0.75)^2$$

$$\boxed{A = 0.441\ in^2}$$

Allowable bolts shear = 21 ksi

Since bolts are in double shear

So Allowable shear per bolt =

$$2 \times 21 \times 0.44 = 18.5\ K$$

Allowable bolt bearing stress = $1.2 F_u$

1.2×58

$= 69.6\ K$

Allowable on two $\frac{5}{8}''$ thick angle

Allowable shear per bolt =

$2 \times 21 \times 0.44$

$= 18.5$ Kips

Allowable bearing on two 7/8" thick angle long legs =

$69.6 \times 2 \times \frac{7}{16} \times 0.75$

$\Rightarrow 45.68$ kips > 18.5 kips

So
Shear governs

Now
Number of bolts $\Rightarrow \frac{200}{18.5}$

$\Rightarrow 10.81$

So we will use 12 bolts

Design of Gusset plate:

Bearing stress $= 1.2 F_u$

$\Rightarrow 1.2 \times 58$

$= 69.6$ ksi

long legs $= 69.6 \times 2 \times \frac{5}{8} \times 0.75 = 65.25 > 18.5$

So shear governs