University Of Technology
Building and Construction Eng. Dept.
Final Exam – 2015/2016
Subject :Structural Design (steel) Class: Fourth
Branch :Building & management of structural projects Time : 3 hrs
Examiner: Dr. Iqbal N. Gorgis
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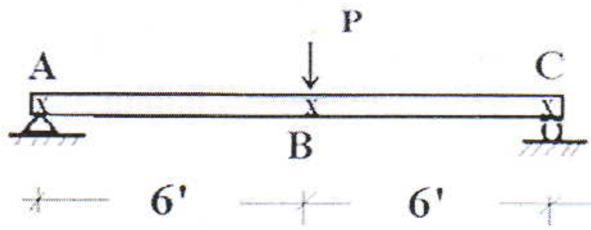
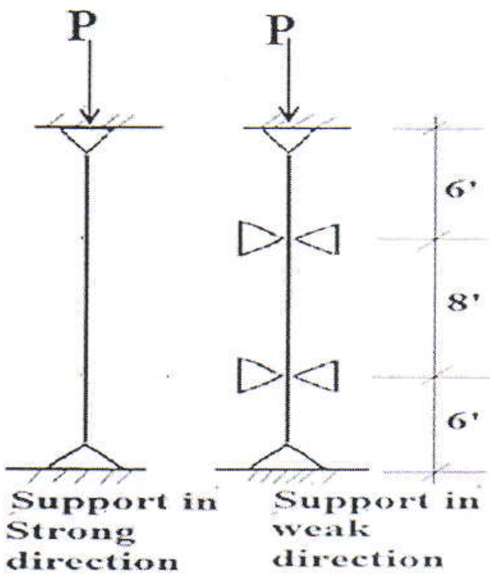
Date : 29 / 5 /2016

Note: **Solve only <u>three</u> questions, Use the AISC specifications.**

Q.1: Determine the allowable live load (P) which satisfies the flexural and shear strength requirements for the beam W18x65 shown below. Using A-36 steel and the beam was braced at points ABC (the distributed load is only **beam weight**).



Q. 2: Select a lightest W-shape for the pin ended column which support an axial compression force of P = 600 kip shown in figure below, Use A572 grade 50 steel, then check local stability.



Q. 1 :

W18x65 specification P-22, 23

Designation	Area A	Depth d		Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Distance k_1
	In.^{2}	In.		In.		In.	In.		In.		In.	In.	In.
W 18× 71	20.8	18.47	18½	0.495	½	¼	7.635	7⅝	0.810	13/16	15½	1½	⅞
× 65	19.1	18.35	18⅜	0.450	7/16	¼	7.590	7⅝	0.750	¾	15½	1 7/16	⅞

Nominal Wt. per Ft	Compact Section Criteria $\frac{b_f}{2t_f}$	F_y'	$\frac{d}{t_w}$	F_y'''	r_T	$\frac{d}{A_f}$	Elastic Properties Axis X-X I	S	r	Axis Y-Y I	S	r	Plastic Modulus Z_x	Z_y
Lb.		Ksi		Ksi	In.		In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{3}	In.^{3}
65	5.1	—	40.8	39.7	1.97	3.22	1070	117	7.49	54.8	14.4	1.69	133	22.5

$\frac{76*b_f}{\sqrt{fy}} = \frac{76*7.59}{\sqrt{36}*12} = 8'(control)$ and $\frac{20000}{\frac{d}{A_f}\sqrt{fy}} = \frac{20000}{3.22*36*12} = 14.38'$

$L_c = 8'$ $and\ Lb(between\ laterally\ supports) = 6'$

Then full laterally supported beam. Check for compact section or not:

$\frac{b_f}{2t_f} \leq \frac{65}{\sqrt{fy}}$ $5.1 \leq \frac{65}{\sqrt{36}} = 10.8$ $o.k.$

$\frac{d}{t_w} \leq \frac{640}{\sqrt{fy}}$ $40.8 \leq \frac{640}{6} = 106.67$ $o.k.\ the\ beam\ section\ is\ compact.$

Then $Fb = 0.66fy = 0.66 * 36 = 24\ ksi$

For flexural $M = Sx * Fb = 117 * \frac{24}{12} = 234\ kip - ft$

$$M = \frac{Wd * L^2}{8} + \frac{P * L}{4}$$

$$234 = \frac{0.065 * 12^2}{8} + \frac{P * 12}{4} \quad \rightarrow\rightarrow P = 77.61 \frac{kip}{ft}$$

For shear

$$f_v(\,allowable) = 0.4 * fy = 0.4 * 36 = 14.4\ ksi$$

$$F_v = \frac{V}{d * tw} = \frac{V}{18.35 * 0.45} = 14.4 \quad \rightarrow\rightarrow\rightarrow V = 118.908\ kip$$

$$V = 118.908 = \frac{Wd * L}{2} + \frac{P}{2} = \frac{0.065 * 12}{2} + \frac{P}{2}$$

$$P = \frac{236.84kip}{ft}$$

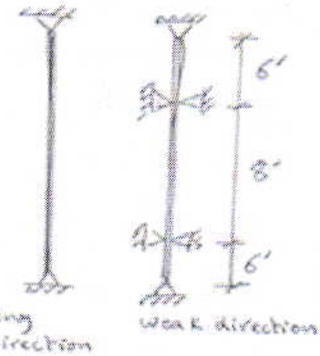
The maximum P= 77.61 k/ft.

Q. 2 :

select W-shape , $F_y = 50$ ksi , $P = 600^k$

$L_x = 20'$, $L_y = 8'$ (larger span)

$K_x = K_y = 1$



assume $\frac{KL}{r} = 100 \xrightarrow[C-50]{\text{table}} F_a = 14.71$ ksi

$\therefore A = \frac{600}{14.71} = 40.79 \text{ in}^2$

$r_x = \frac{1 * 20 * 12}{100} = 2.4''$

$r_y = \frac{1 * 8 * 12}{100} = 0.96''$

from Manual we can try

W 12 X 152

W 14 X 145 → lightest one

$\therefore$ Try W14 X 145 : $A = 42.7 \text{ in}^2$, $r_x = 6.33''$, $r_y = 3.98''$

$f_a = \frac{600}{42.7} = 14.05$ ksi

$\left(\frac{KL}{r}\right)_x = \frac{1 * 20 * 12}{6.33} = 37.9$

$\left(\frac{KL}{r}\right)_y = \frac{1 * 8 * 12}{3.98} = 24.12$

$\left(\frac{KL}{r}\right)_x$ controls $\xrightarrow[C-50]{\text{table}} F_a = 26.12$ ksi

check $\frac{f_a}{F_a} = \frac{14.05}{26.12} = 0.53 << 1$ choose smaller section

Try W 14 X 82 : $A = 24.1 \text{ in}^2$, $r_x = 6.05''$, $r_y = 2.48''$

$f_a = \frac{600}{24.1} = 24.9$ ksi

$\left(\frac{KL}{r}\right)_x = \frac{1 * 20 * 12}{6.05} = 39.7$

$\left(\frac{KL}{r}\right)_y = \frac{1 * 8 * 12}{2.48} = 38.7$

$\left(\frac{KL}{r}\right)_x$ controls $\xrightarrow[C-50]{\text{table}} F_a = 25.87$ ksi

$\frac{f_a}{F_a} = \frac{24.9}{25.87} = 0.96$ o.k $\therefore$ **Use W 14 X 82**

Check local buckling:

$$\frac{b_f}{2t_f} \leq \frac{95}{\sqrt{fy}} \qquad \frac{10.1}{2 * 0.855} = 5.906 \leq \frac{95}{\sqrt{50}} = 13.435$$

$$\frac{d - 2t_f}{t_w} \leq \frac{253}{\sqrt{fy}} \qquad \frac{14.3 - 2 * 0.855}{0.51} = 28.04 \leq 35.779$$

Q. 3 :

Tensile yielding on gross section $P_n = F_y A_g = (50 \text{ ksi})(3.75 \text{ in}^2) = 187.5 \text{ k}$

Tensile rupture on net section $A_e = UA_g = (0.87)(3.75 \text{ in}^2) = 3.26 \text{ in}^2$

$$P_n = F_u A_e = (65 \text{ ksi})(3.26 \text{ in}^2) = 211.9 \text{ k}$$

For tensile yielding ($\phi_t = 0.90$) $\phi_t P_n = (0.9)(187.5) = 168.7 \text{ k}$

For tensile rupture ($\phi_t = 0.75$) $\phi_t P_n = (0.75)(211.9) = 158.9 \text{ k}$ ← Control

$$\text{Maximum weld size} = \frac{1}{2} - \frac{1}{16} = \frac{7}{16} \text{ in}$$

Use $\frac{5}{16}$-in weld (largest that can be made in single pass)

$$\text{Effective throat } t \text{ of weld} = (0.707)\left(\frac{5}{16}\text{ in}\right) = 0.221 \text{ in}$$

Design strength/in of
$\frac{5}{16}$-in welds ($\phi = 0.75$) $= (0.75)(0.60 \times 70)(0.221)(1) = 6.96 \text{ k/in}$

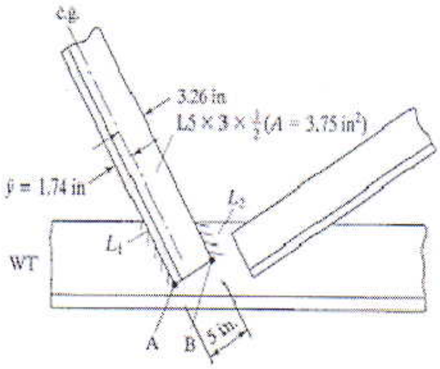
$$\text{Weld length reqd} = \frac{158.9}{6.96} = 22.83 \text{ in}$$

Taking moments about point A $(158.9)(1.74) - 5.00P_2 = 0$

$$P_2 = 55.3 \text{ k}$$

$$L_2 = \frac{55.3 \text{ k}}{6.96 \text{ k/in}} = 7.95 \text{ in (say, 8 in)}$$

$$L_1 = 22.83 - 7.95 = 14.88 \text{ in (say, 15 in)}$$



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Solution of question:

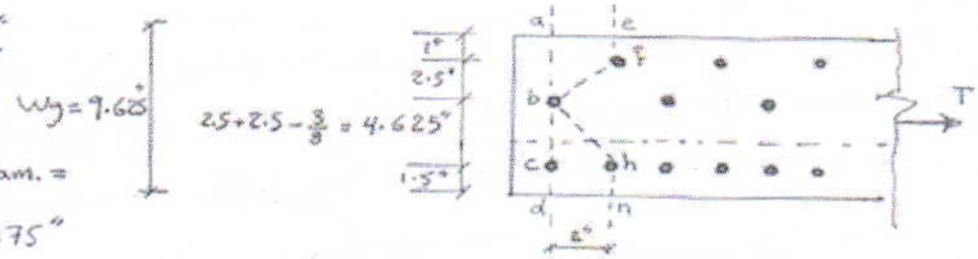
Q. 4 :

$w_g = 6'' + 4'' - \frac{3}{8}''$

$= 9.625''$

effective hole diam. =

$\frac{3}{4}'' + \frac{1}{8}'' = 0.875''$



line (abcd): $W_n = 9.625'' - 2(0.875'') = 7.875''$

line (efbcd): $W_n = 9.625'' - 3(0.875'') + \frac{2^2}{4(2.5)} = 7.4''$

line (efbhn): $W_n = 9.625'' - 3(0.875'') + \frac{2^2}{4(2.5)} + \frac{2^2}{4(4.625)} = 7.616''$

ملاحظة: لا يتم ضرب $W_n * \frac{12}{11}$ لأن البرغي المتروك في ع غير داخل في تحمل قوة الشد

$\therefore W_n = 7.4''$

$A_n = \overset{\text{Double angle}}{2}(W_n * t) = 2(7.4 * \frac{3}{8}) = 5.55 \text{ in}^2$

$A_e = A_n * C_t = 5.55 * 0.85 = 4.7175 \text{ in}^2$

$\therefore$ for gross section $T_{max.} = 0.6 * F_y * A_g = 0.6(36)(7.22) = 156^k$

for net section $T_{max.} = 0.5 * F_u * A_e = 0.5(58)(4.7175) = 136.8^k$

For bolts capacity :- * الجزء الثاني من المثال يخص موضوع الـ connection

Shear capacity for one bolt $= F_v * (\text{area})_{\text{bolts}} * 2$

$= 21.0 * \frac{\pi (3/4)^2}{4} * 2 = 18.55^k$

bearing capacity for one bolt $= F_p *$ bolt diam. $* t_{min.}$

$= 1.2 F_u * \frac{3}{4}'' * \frac{3}{8}'' = 19.575^k$

$\therefore$ Shear capacity control

$\therefore T_{max.} = 18.55 * 6 = 111.3^k$

$\therefore$ The tensile force (T) permitted by AISC $= 111.3^k$