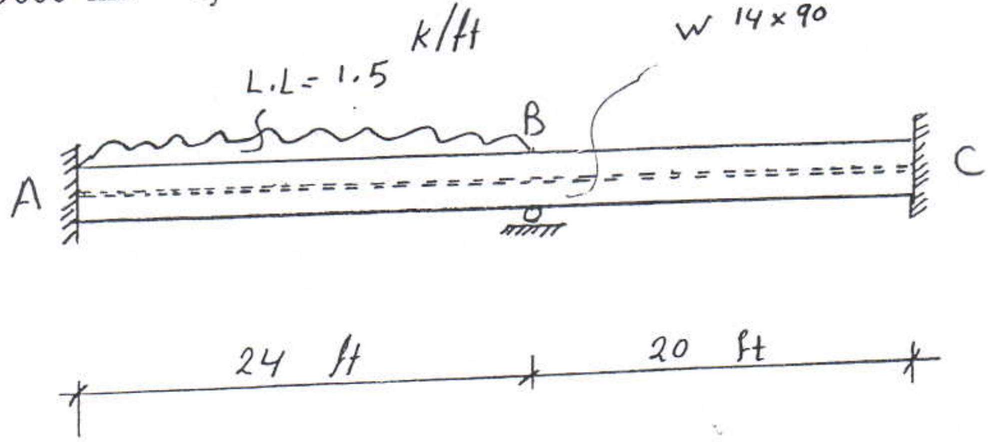
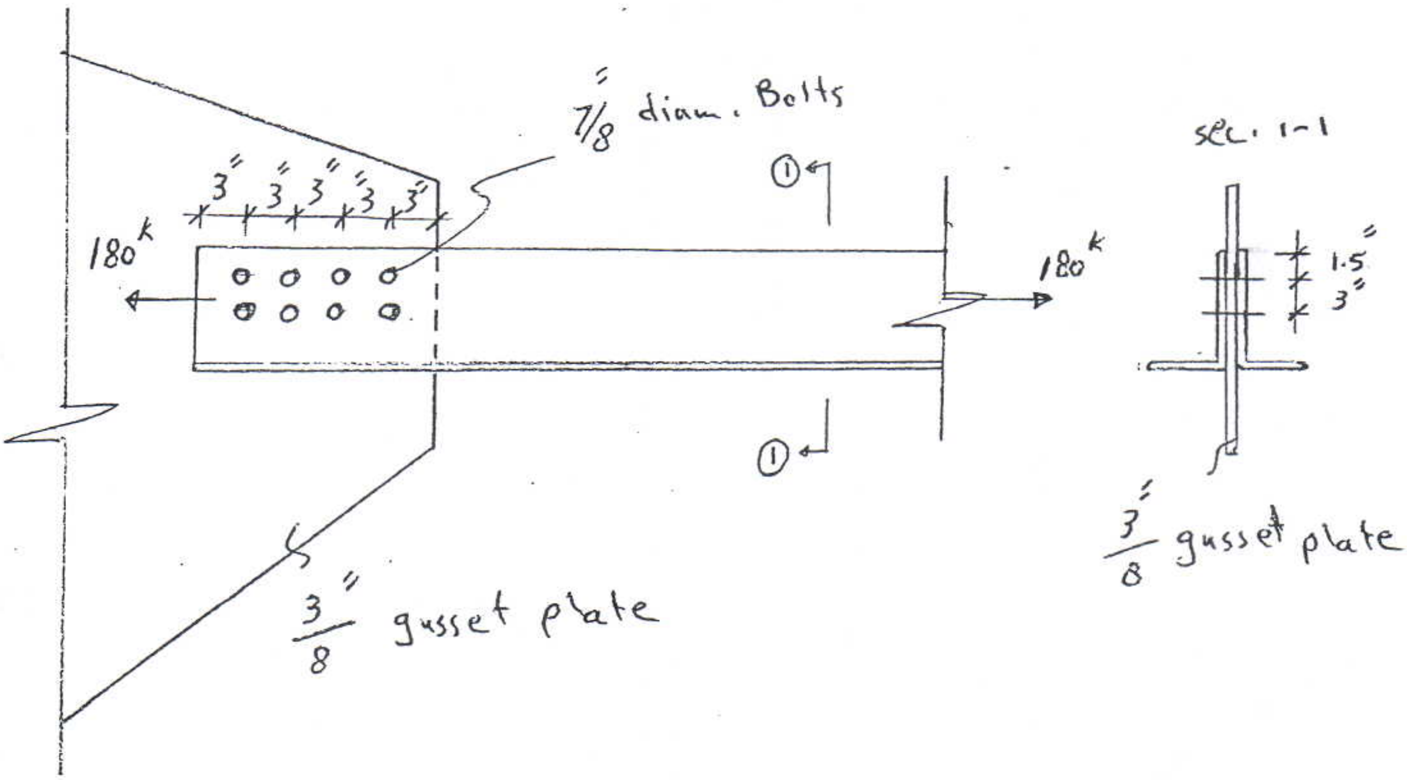
Q Check the adequacy of the beam ABC (W 14x90) shown in the fig. (moment & shear) due to the load acting using the minor axis of the beam & A36 steel is used .

W14x90

$A_g = 26.5$	in^2	$I_x = 999$	in^4
$d = 14.02$	in	$S_x = 143$	in^3
$t_w = 0.44$	in	$r_x = 6.14$	in
$b_f = 14.52$	in	$I_y = 362$	in^4
$t_f = 0.71$	in	$S_y = 49.9$	in^3
$E = 29000$ ksi		$r_y = 3.70$	in



Q4/ Select the lightest double-angle as a tension member like that shown in the fig. to support load (180 k). Use A36 steel for all elements & 7/8 inch diam. bolts in standard angle gages &edge distance . Assume the length of the member is (15 ft) & two unequal leg angles long legs back to back are used .



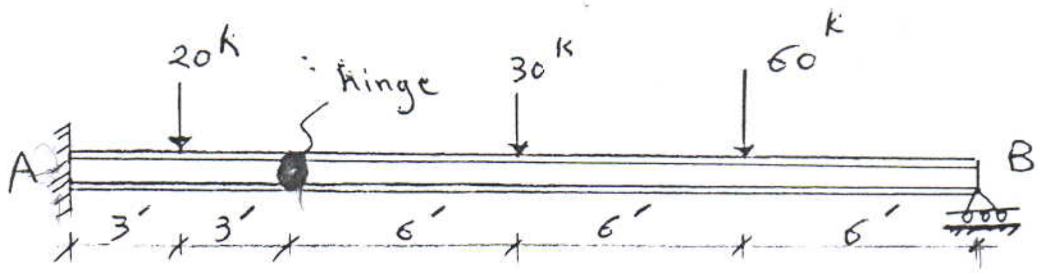
University Of Technology
Building and Construction Eng. Dept.
Final Exam 2014-2015
Subject : Structural Design- steel
Branch : Building& construction management
Examiner : Dr. Zeyad M. Ali
Class: Forth Class
Time : 3 Hours
Date : 14/ 6 / 2015

Note1 : Answer three questions only. **Note2 : Use the AISC specification.**

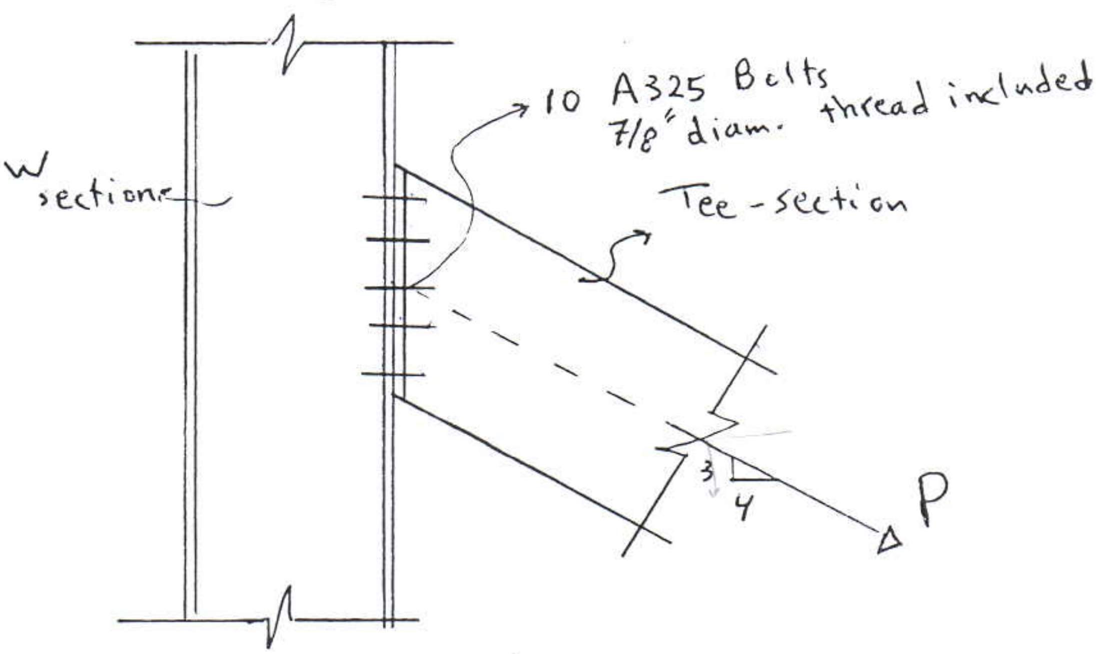
Q1/ Check the adequacy of the beam AB (W 12x152) shown in the fig. (moment, shear & deflection) due to the load acting (neglect the weight of the beam) using the major axis of the beam,A36 steel is used and the max. allowable deflection = 1 inch at the position of load (30^k) .

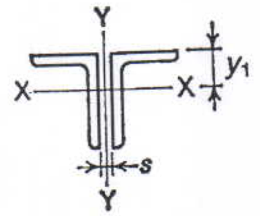
W12x152.

A_g = 44.7	in^2	I_x = 1430	in^4
d = 13.7	in	S_x = 209	in^3
t_w = 0.87	in	r_x = 5.66	in
b_f = 12.48	in	I_y = 454	in^4
t_f = 1.4	in	S_y = 72.8	in^3
r_T = 3.44	in	r_y = 3.19	in
d/A_f = 0.79	in^{-1}	E = 29000	ksi



Q2/Given the tension- shear connection as shown in the fig. what is the allowable load (P) if A36 steel are used for all sections & a slip- critical connection type with (ten A325 bolts 7/8 inch diam. thread included for shear plane)? Assume that all spacing & edge distance requirement are satisfied & the resultant force acts through the center of the connection .





DOUBLE ANGLES
Two unequal leg angles
Properties of sections
Long legs back to back

Designation	Wt. per Ft 2 Angles	Area of 2 Angles	AXIS X-X				AXIS Y-Y Radii of Gyration Back to Back of Angles, In.			Q_s* Angles in Contact		Angles Separated	
			I	S	r	y				$F_y =$ 36 ksi	$F_y =$ 50 ksi	$F_y =$ 36 ksi	$F_y =$ 50 ksi
	Lb.	In.^{2}	In.^{4}	In.^{3}	In.	In.	0	3/8	3/4				
L 8×6 ×1	88.4	26.0	161.0	30.2	2.49	2.65	2.39	2.52	2.66	—	—	—	—
3/4	67.6	19.9	126.0	23.3	2.53	2.56	2.35	2.48	2.62	—	—	—	—
1/2	46.0	13.5	88.6	16.0	2.56	2.47	2.32	2.44	2.57	—	—	.911	.834
L 8×4 ×1	74.8	22.0	139.0	28.1	2.52	3.05	1.47	1.61	1.75	—	—	—	—
3/4	57.4	16.9	109.0	21.8	2.55	2.95	1.42	1.55	1.69	—	—	—	—
1/2	39.2	11.5	77.0	15.0	2.59	2.86	1.38	1.51	1.64	—	—	.911	.834
L 7×4 × 3/4	52.4	15.4	75.6	16.8	2.22	2.51	1.48	1.62	1.76	—	—	—	—
1/2	35.8	10.5	53.3	11.6	2.25	2.42	1.44	1.57	1.71	—	—	.965	.897
3/8	27.2	7.97	41.1	8.88	2.27	2.37	1.43	1.55	1.68	—	—	.839	.750
L 6×4 × 3/4	47.2	13.9	49.0	12.5	1.88	2.08	1.55	1.69	1.83	—	—	—	—
5/8	40.0	11.7	42.1	10.6	1.90	2.03	1.53	1.67	1.81	—	—	—	—
1/2	32.4	9.50	34.8	8.67	1.91	1.99	1.51	1.64	1.78	—	—	—	.961
3/8	24.6	7.22	26.9	6.64	1.93	1.94	1.50	1.62	1.76	—	—	.911	.834
L 6×3½× 3/8	23.4	6.84	25.7	6.49	1.94	2.04	1.26	1.39	1.53	—	—	.911	.834
5/16	19.6	5.74	21.8	5.47	1.95	2.01	1.26	1.38	1.51	—	—	.825	.733
L 5×3½× 3/4	39.6	11.6	27.8	8.55	1.55	1.75	1.40	1.53	1.68	—	—	—	—
1/2	27.2	8.00	20.0	5.97	1.58	1.66	1.35	1.49	1.63	—	—	—	—
3/8	20.8	6.09	15.6	4.59	1.60	1.61	1.34	1.46	1.60	—	—	.982	.919
5/16	17.4	5.12	13.2	3.87	1.61	1.59	1.33	1.45	1.59	—	—	.911	.834
L 5×3 × 1/2	25.6	7.50	18.9	5.82	1.59	1.75	1.12	1.25	1.40	—	—	—	—
3/8	19.6	5.72	14.7	4.47	1.61	1.70	1.10	1.23	1.37	—	—	.982	.919
5/16	16.4	4.80	12.5	3.77	1.61	1.68	1.09	1.22	1.36	—	—	.911	.834
1/4	13.2	3.88	10.2	3.06	1.62	1.66	1.08	1.21	1.34	—	—	.804	.708

* Where no value of Q_s is shown, the angles comply with the noncompact section criteria of Specification Sect. B5.1 and may be considered fully effective.

For $F_y = 36$ ksi: $C_c' = 126.1/\sqrt{Q_s}$

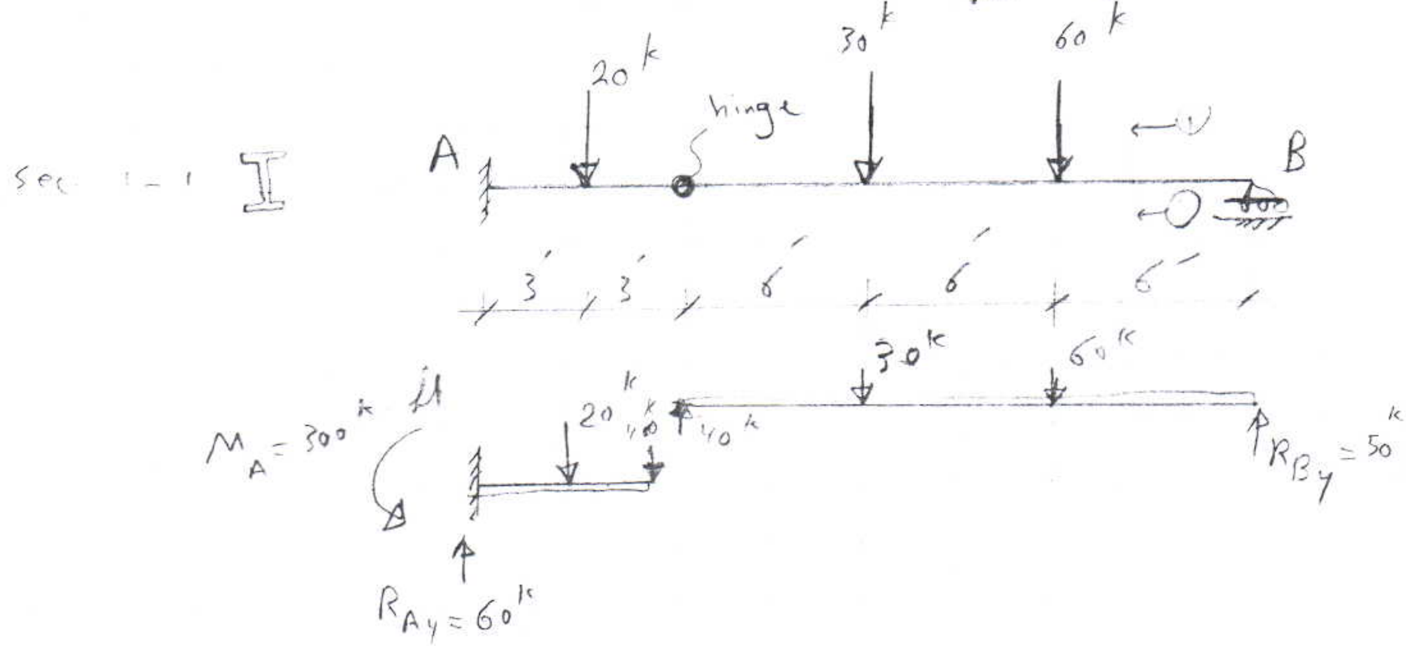
For $F_y = 50$ ksi: $C_c' = 107.0/\sqrt{Q_s}$

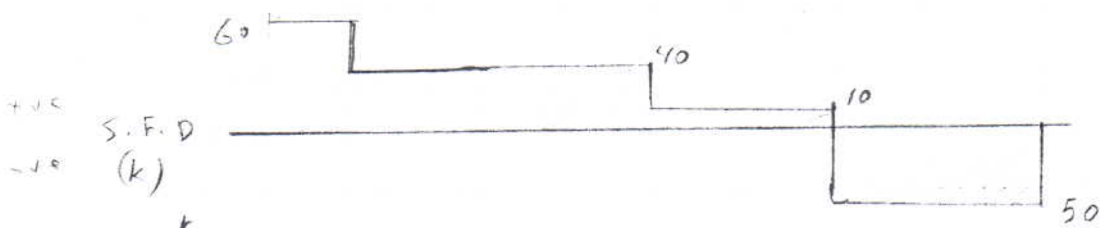
— W 12 × 152

b_f 12.48 inch

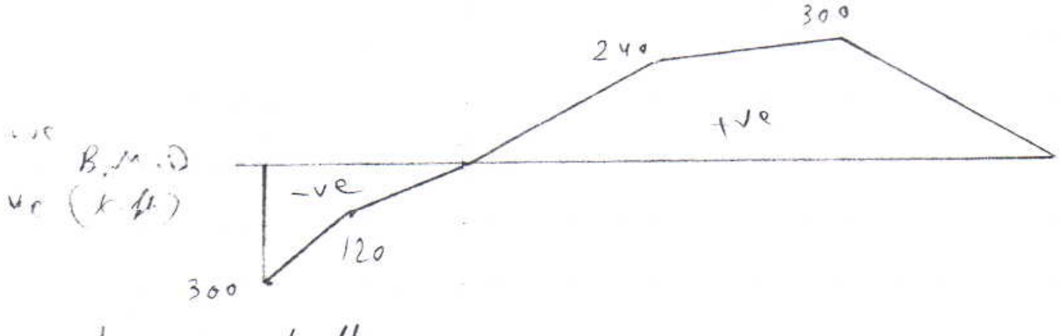
Q1

Q1/ check the adequacy of the beam AB (W12 × 152) shown in the fig (neglect the weight of the beam) (moment, shear & deflection) due to the load acting? Given EI is Constant, A-36 steel is used and the max allowable deflection = 1 inch at the position of load 60 k

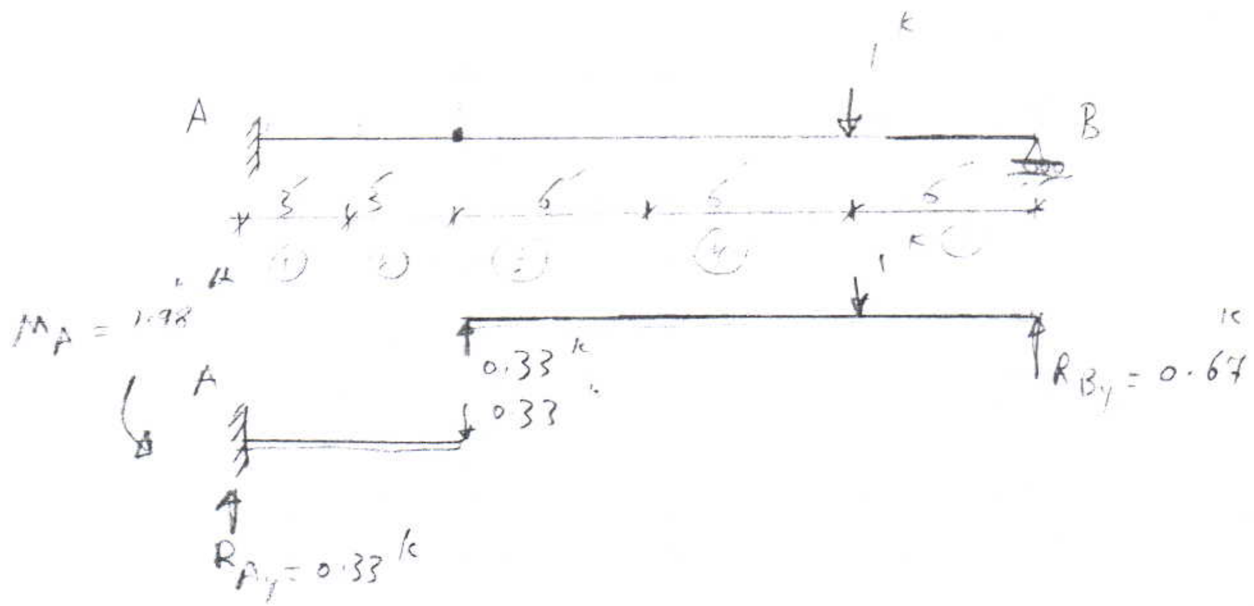




max. shear = 60 k



max. moment = 300 k.ft



Section	Origin	Limit (ft)	EI	M	m
1	1	0 - 3	Constant	$(60x - 300)$	$(0.33x - 1.98)$
2	3	0 - 3	"	$(-40x)$	$(-0.33x)$
3	3	0 - 6	"	$(40x)$	$(0.33x)$
4	3	0 - 6	"	$40(6+x) - 30x$	$0.33(6+x)$
5	5	0 - 6	"	$50x$	$0.67x$

$$(1 \cdot \Delta_v)EI = \int_0^3 \left(19.8x^2 - 217.8x + 594\right)dx + \int_0^3 \left(13.2x^2\right)dx + \int_0^6 \left(13.2x^2\right)dx + \int_0^6 \left(3.3x^2 + 99x + 475.2\right)dx + \int_0^6 33.5x^2\,dx$$

$$(1\Delta)EI = 6.6x^3\Big|_0^3 - 108.9x^2\Big|_0^3 + 594x\Big|_0^3 + 4.4x^3\Big|_0^3 + 4.4x^3\Big|_0^6 + 1.1x^3\Big|_0^6 + 49.5x^2\Big|_0^6 + 475.2x\Big|_0^6 + 11.16x^3\Big|_0^6$$

$$1 \cdot \Delta = \frac{9330.66}{EI} < 1''$$

$$\frac{9330.66 * 144}{(29 * 10^6)(1430)} < 1''$$

o.k for diffection

$$L_c = \frac{76(b_f)}{\sqrt{F_y}} = \frac{76(12.18)}{\sqrt{36}} = \frac{[illegible]}{12} = 13.17'$$

or

$$L_c = \frac{20000\, A_f}{F_y \cdot d} = \frac{20000}{36\ ([illegible])\ 12} = 58.6'$$

$$\therefore L_c = 13.17' < L = 24'$$

$$\frac{L}{r_T} = \frac{24 \times 12}{3.44} = 83.72$$

$$\sqrt{\frac{102000\, C_b}{F_y}} = 53.23 < \frac{L}{r_T}$$

$$\sqrt{\frac{510000\, C_b}{F_y}} = 119 > \frac{L}{r_T}$$

$$\boxed{F_b = 0.6\, F_y = 21.6\ ksi}$$

$$f_b = \frac{M}{S_x} \leq F_b$$

$$= \frac{300 * 12}{209} = 17.22\ ksi < 21.6\ ksi = F_b$$

o.K

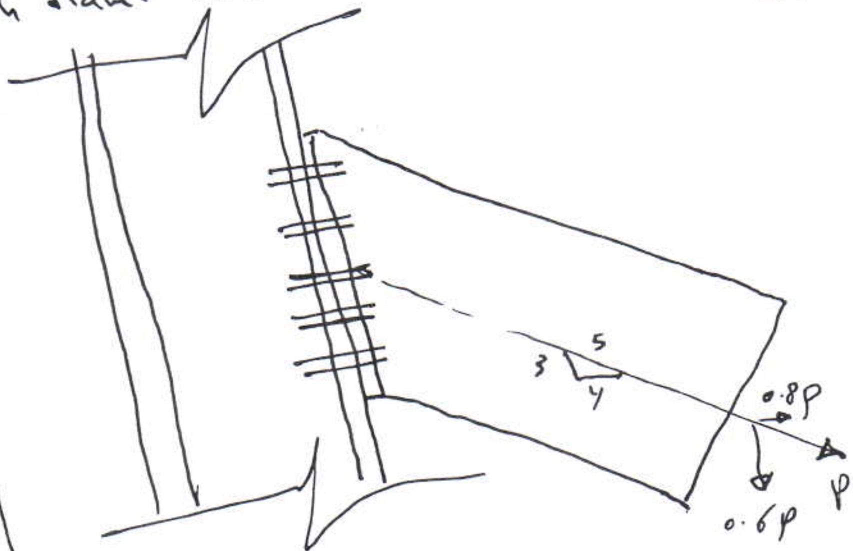
$$F_v = 0.4\, F_y = 14.4\ ksi$$

$$f_v = \frac{V_{max}}{d \cdot t_w} = \frac{60}{13.71 * 0.87} = 5\ ksi < F_v = 14.4\ ksi$$

(o.K)

The beam is adequat

Q/2 Determine the capacity of the structural Tee section Connected to the Column shown in the fig.
All structural steel is A36. A 325 SC in standard holes.
Bolts (total of 10) are 7/8 inch diam.
A 325SC [slip - critical Connection]



For shear only

$$P_s = F_s \cdot A_b \cdot N_b \cdot N_s$$

$$\boxed{0.6P = P_s}$$

but $(F_s)_{new} = F_s \left[1 - \frac{f_t * A_b}{T_b} \right]$

for Combine shear & Tension.

$P_s = (F_s)_{new} \cdot A_b \cdot N_b \cdot N_s$ & $(F_s)_{new} = F_s \left[1 - \frac{f_t \cdot A_b}{T_b} \right]$

$F_s = 17$ ksi for A 325

$$f_t = \frac{0.8P}{10 \frac{\pi (7/8)^2}{4}} = 0.133 P \text{ ksi}$$

$$A_b = \frac{\pi (7/8)^2}{4} = 0.6 \text{ in}^2$$

$$T_b = 39^k$$

$$\therefore (F_s)_{new} = 17 \left[1 - \frac{0.133P * 0.6}{39} \right]$$

$$= 17 \left[1 - 2.046 \times 10^{-3} P \right]$$

$$\therefore P_s = \underbrace{17 \left[1 - 2.046 \times 10^{-3} P \right]}_{(F_s)_{new}} \underbrace{\left[0.6 * 10 \times 1 \right]}_{A_b * N_b * N_s}$$

$$0.6P = 102 \left[1 - 2.046 \times 10^{-3} P \right]$$

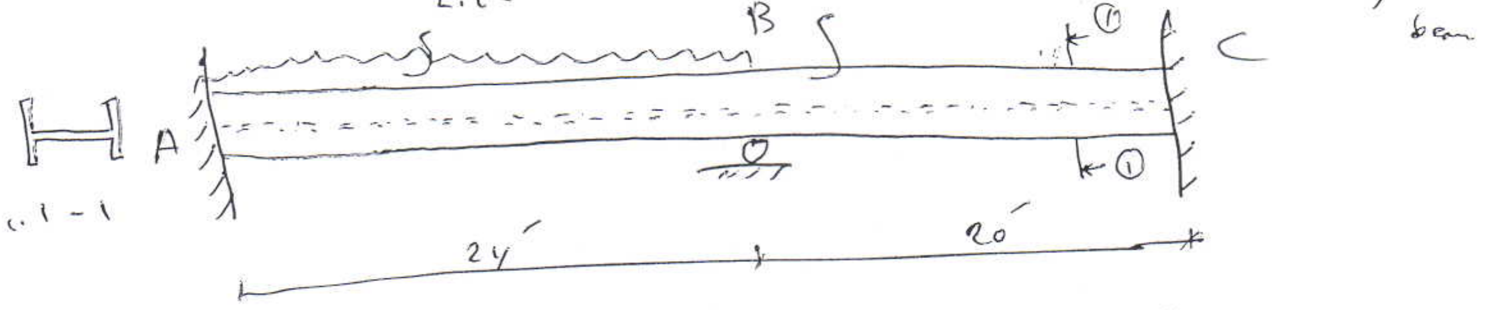
$$.6P = 102 - 0.2087 P$$

$$0.8087P = 102$$

$$\therefore P = 126.13^k \qquad \therefore P = 126^k \text{ max.}$$

3/ Check the adequacy of the beam shown in the fig. due to the Loading acting and according to the AISC specification). Given A36 for all steel, EI is Constant, W 14 x 90 and using the minor axis of the beam

L.L = 1.5 k/ft



W 14 x 90

$A_g = 26.5 \; in^2$
$d = 14.02 \; in$
$t_w = 0.44 \; in$
$b_f = 14.52 \; in$
$t_f = 0.71 \; in$

$I_x = 999 \; in^4$
$S_x = 143 \; in^3$
$r_x = 6.14 \; in$

$I_y = 362 \; in^4$
$S_y = 49.9 \; in^3$
$r_y = 3.7 \; in$

$E = 29000 \; ksi$

① F.E.M

span AB :-

$$M_{FAB} = \frac{-WL^2}{12} = \frac{(1.5 + 0.09)(24)^2}{12} = -76.32 \; k.ft$$

$$M_{FBA} = \frac{WL^2}{12} = 76.32$$

span BC :-

$$M_{FBC} = \frac{-WL^2}{12} = \frac{(0.09)(20)^2}{12} = -3$$

$$M_{FCB} = \frac{WL^2}{12} = 3$$

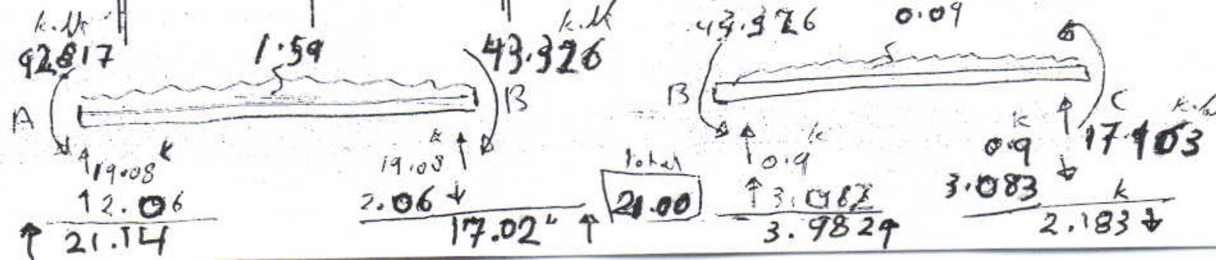
② $k = I/L$ span $AB = \frac{1}{24} \times 120 = 5$

$BC = \frac{1}{20} \times 120 = 6$

joint	A	B		C
member	AB	BA	BC	CB
$k = I/L$	5	5	6	6
$D.F = k/\Sigma k$	—	0.45	0.55	—
F.E.M	−76.32	76.32	−3	3
Bal	—	−32.994	−40.326	—
	—	—	—	−20.163
	−16.497	—	—	—
	—	—	—	—
Σ	−92.817	43.326	−43.326	−17.163

$\therefore$ max. moment = 92.817 k/ft

max. shear = 21.14 k



check the adequacy of the beam

for ① bending :-

$$f_b = \frac{M_{max.}}{S_y} = \frac{92.81k\ 12}{49.9} = 22.32 \text{ ksi}$$

For Compact shapes $F_b = 0.75 F_y$

$$\frac{b_f}{2t_f} = \frac{14.52}{2(0.71)} = 10.22$$

$$\frac{65}{\sqrt{F_y}} = 10.83 \qquad \therefore \frac{b_f}{2t_f} < \frac{65}{\sqrt{F_y}}$$

$$\text{so } F_b = 0.75(36)$$

$$= 27 \text{ ksi}$$

$$\therefore f_b = 22.32 < F_b = 27 \text{ ksi} \quad (\text{o.k})$$

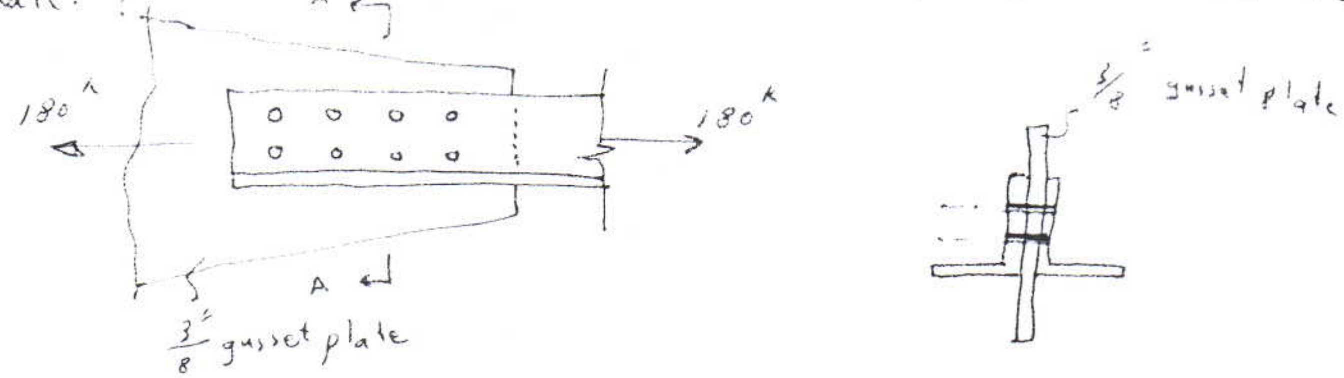
② shear :-

$$f_v = \frac{21.1}{t_w.\,d} = \frac{21.1}{(0.44)(14.02)} = 3.42 \text{ ksi}$$

$$\text{but } F_v = 0.4 F_y = 14.4 \text{ ksi}$$

$$\therefore f_v < F_v \quad (\text{o.k})$$

Q4/ ...port a Load $P = 180^k$. Use A36 steel and $\frac{7}{8}$ in. diameter bolts. The ...ngth of the member is (15 ft). Use standard angle gages, (1.5 in.) ...e distance and (3 in.) pitch. Assume a total of eight fasteners are adequate.



A36 $F_y = 36$ ksi & $F_u = 58$ ksi

$$\boxed{A_{req.} = \frac{180}{0.6 F_y} = 8.333 \text{ in}^2}$$

$$(A_e)_{req.} = \frac{180}{0.5 F_u} = 6.207 \text{ in}^2$$

$$A_{eff} = 0.85 A_{net}$$

$$(A_{net})_{req.} = \frac{6.207}{0.85} = 7.300 \text{ in}^2$$

effective hole diam. $= \frac{7}{8} + \frac{1}{8} = 1''$

two holes / angle

$$(A_g)_{req.} = (A_{net})_{req.} + A_{holes} = 7.300 + (2 * 2)(1 \times t)$$

$$\boxed{(A_g)_{req.} = 7.300 + 4t}$$

$$\left(\frac{L}{r}\right)_{max.} = 240 \quad \text{(Tension member)}$$

$$r_{min} = \frac{L}{240} = \frac{15(12)}{240} = \boxed{0.75'' = r_{min.}}$$

L $6 \times 4 \times \frac{5}{8}$

wt./ft = 40 lb

$t = \frac{5}{8}$

$$(A_g)_{req.} = 7.300 + 4\left(\frac{5}{8}\right) = 9.8 \text{ in}^2$$

$\therefore$ actual area $= 11.7$ in² o.k.

//

Try (2) L $7 \times 4 \times \frac{1}{2}$

wt/ft = 35.8 lb

$A_g = 10.5 \ in^2$ $r_y = 1.57$ in $r_x = 2.25$ in

$(A_g)_{req} = 8.275 + 4t$

$= 8.275 + 4(\frac{1}{2}) = 10.275 \ in^2 < (A_g = 10.5 \ in^2$

ok

Try (3) L $8 \times 4 \times \frac{1}{2}$

$\boxed{wt./ft = 39.2 \ lb}$ $A_g = 11.5$ $r_y = 1.51$ in $r_x = 2.59$ in

$(A_g)_{req} = 7.300 + 4t$

$= 7.300 + 4(\frac{1}{2}) = 9.300 \ in^2 < A_g = 11.5 \ in^2$

o.k

$r_y = 1.51$ in > 0.75 in o.k

$\therefore$ use L $7 \times 4 \times \frac{1}{2}$

The Lightest angle as a tension member.

wt/ft = 35.8 lb