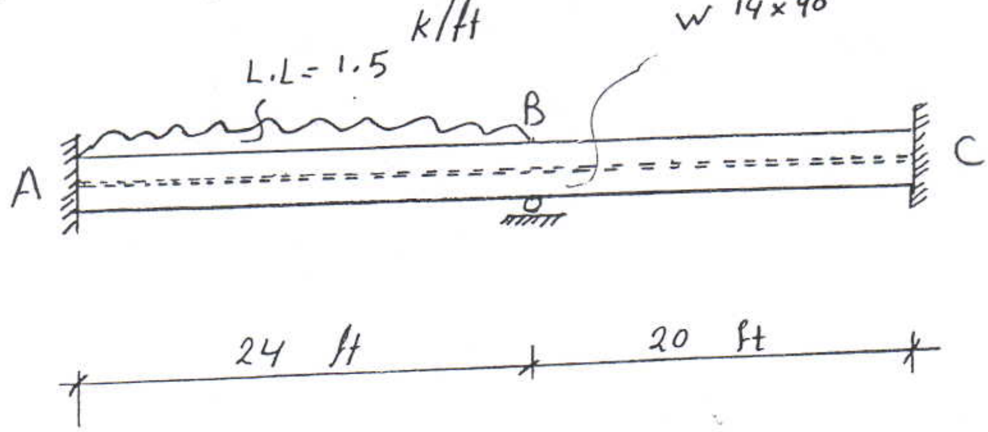
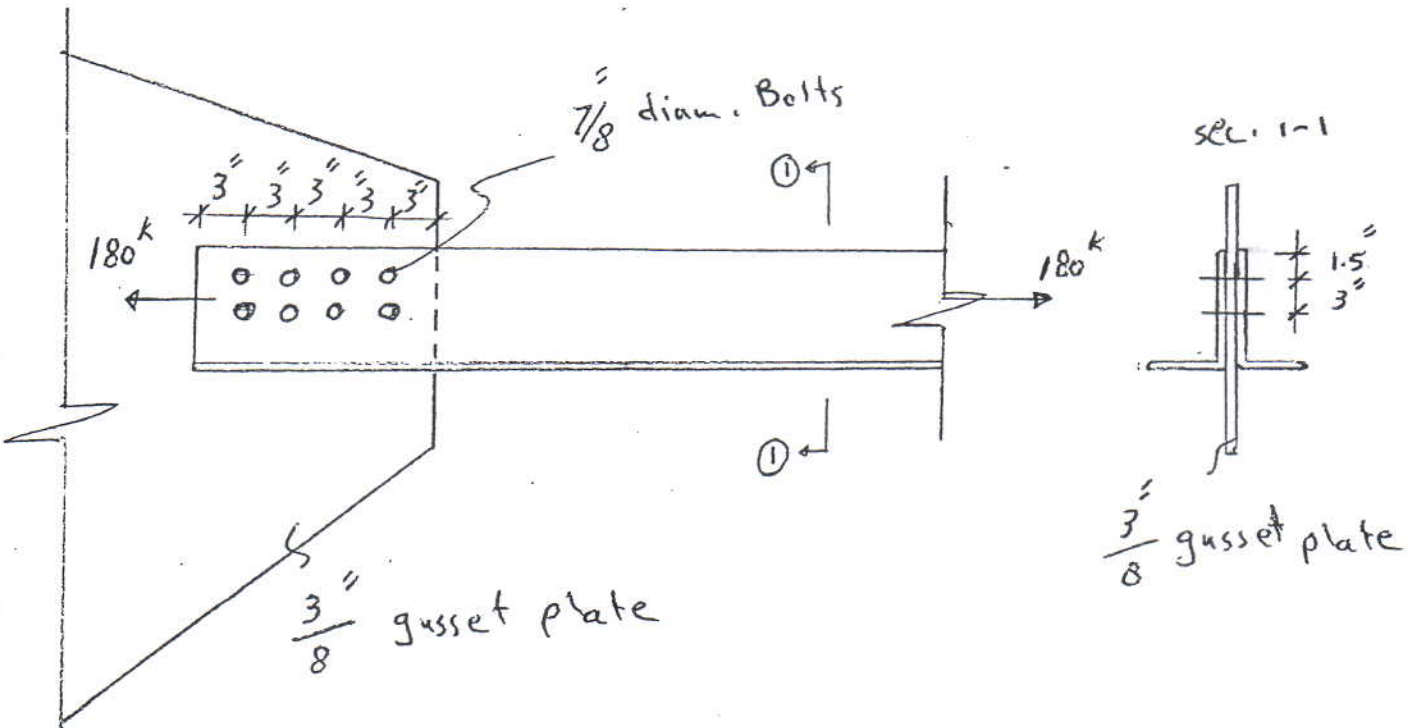
Q Check the adequacy of the beam ABC (W 14x90) shown in the fig. (moment & shear) due to the load acting using the minor axis of the beam & A36 steel is used .

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

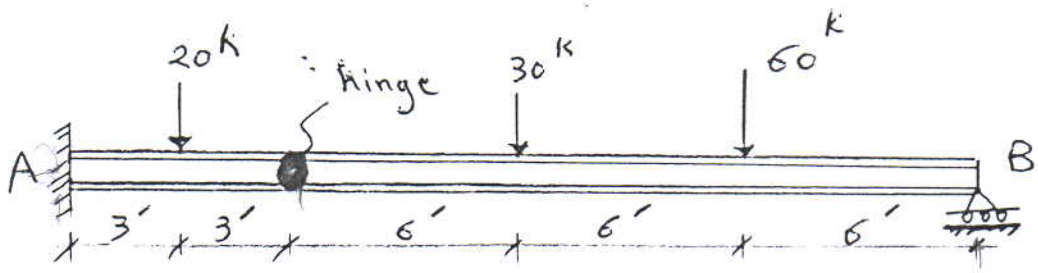
Class: Forth Class
Time : 3 Hours
Date : 14/ 6 / 2015

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

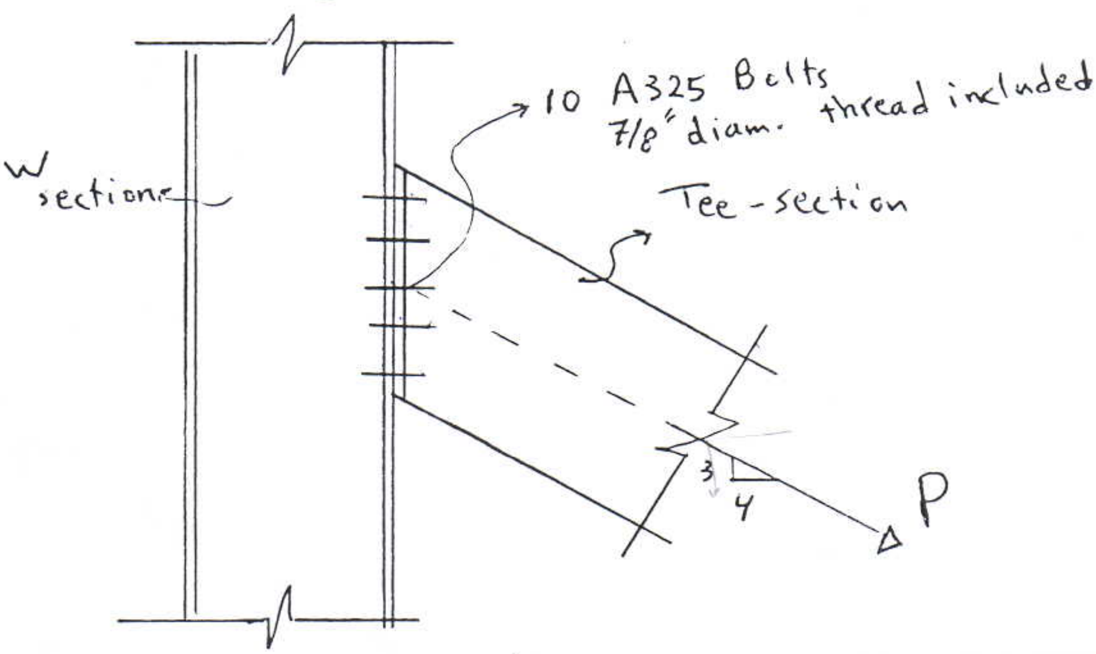
Q1/ Check the adequacy of the beam AB (W 12x152) shown in the fig. (moment, shear & deflection) due to the load acting (neglect the weight of the beam) using the major axis of the beam, A36 steel is used and the max. allowable deflection = 1 inch at the position of load (30^k) .

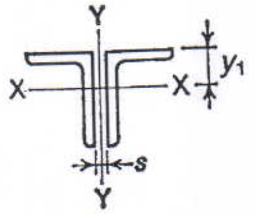
W12x152.

$A_g = 44.7$	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			Q_s*			
							Radii of Gyration Back to Back of Angles, In.			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	⅜	¾				
L 8×6 ×1	88.4	26.0	161.0	30.2	2.49	2.65	2.39	2.52	2.66	—	—	—	—
¾	67.6	19.9	126.0	23.3	2.53	2.56	2.35	2.48	2.62	—	—	—	—
½	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	—	—	—	—
¾	57.4	16.9	109.0	21.8	2.55	2.95	1.42	1.55	1.69	—	—	—	—
½	39.2	11.5	77.0	15.0	2.59	2.86	1.38	1.51	1.64	—	—	.911	.834
L 7×4 × ¾	52.4	15.4	75.6	16.8	2.22	2.51	1.48	1.62	1.76	—	—	—	—
½	35.8	10.5	53.3	11.6	2.25	2.42	1.44	1.57	1.71	—	—	.965	.897
⅜	27.2	7.97	41.1	8.88	2.27	2.37	1.43	1.55	1.68	—	—	.839	.750
L 6×4 × ¾	47.2	13.9	49.0	12.5	1.88	2.08	1.55	1.69	1.83	—	—	—	—
⅝	40.0	11.7	42.1	10.6	1.90	2.03	1.53	1.67	1.81	—	—	—	—
½	32.4	9.50	34.8	8.67	1.91	1.99	1.51	1.64	1.78	—	—	—	.961
⅜	24.6	7.22	26.9	6.64	1.93	1.94	1.50	1.62	1.76	—	—	.911	.834
L 6×3½× ⅜	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× ¾	39.6	11.6	27.8	8.55	1.55	1.75	1.40	1.53	1.68	—	—	—	—
½	27.2	8.00	20.0	5.97	1.58	1.66	1.35	1.49	1.63	—	—	—	—
⅜	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 × ½	25.6	7.50	18.9	5.82	1.59	1.75	1.12	1.25	1.40	—	—	—	—
⅜	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
¼	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: $C_c' = 126.1/\sqrt{Q_s}$

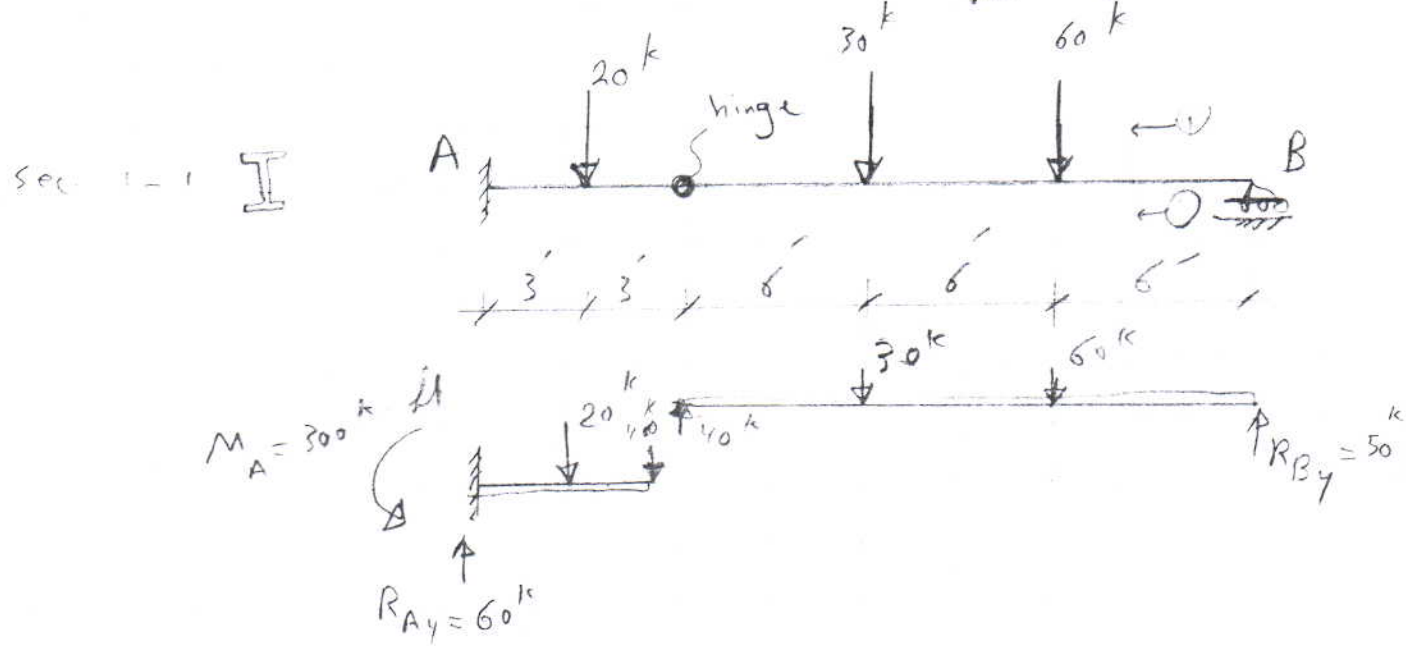
For F_y = 50 ksi: $C_c' = 107.0/\sqrt{Q_s}$

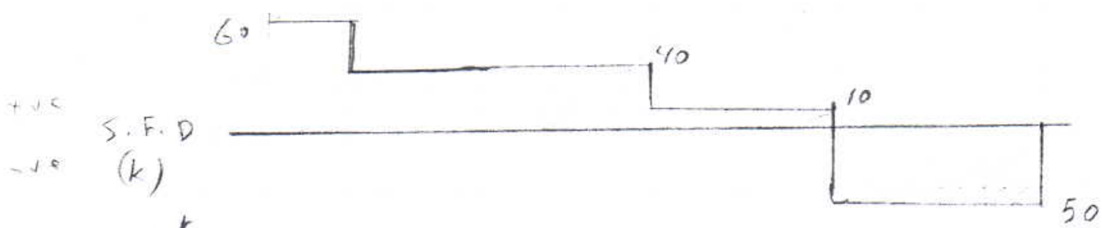
— W 12 x 152

b_f 12.48 inch

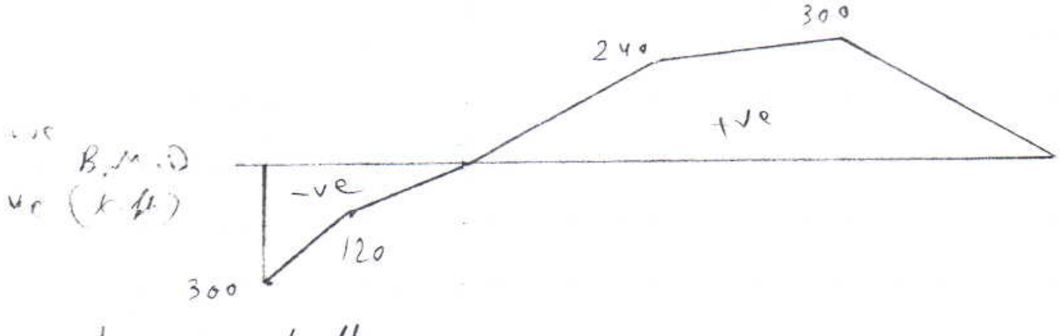
Q1

Q1/ check the adequacy of the beam AB (W12 x 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

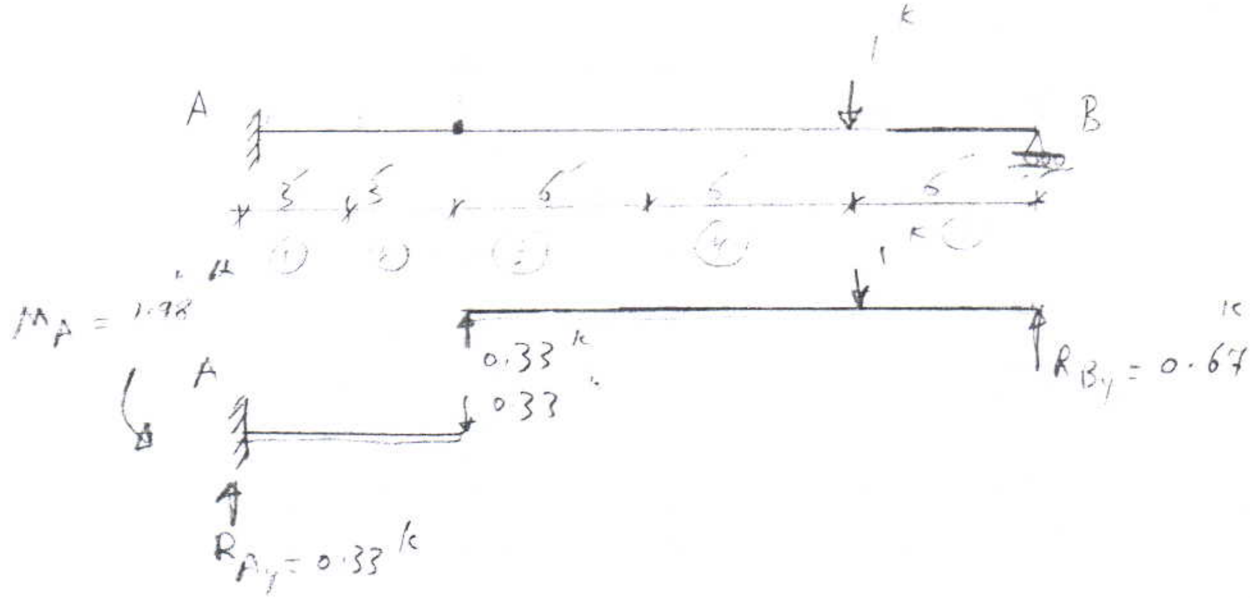




max. shear = 60 k



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 diffleection

$$L_c = \frac{76\,(b_f)}{\sqrt{F_y}} = \frac{76\,(12.18)}{\sqrt{36}} = \frac{158.\ldots}{12} = 13.17'$$

or

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

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

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

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

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

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

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

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

o.K

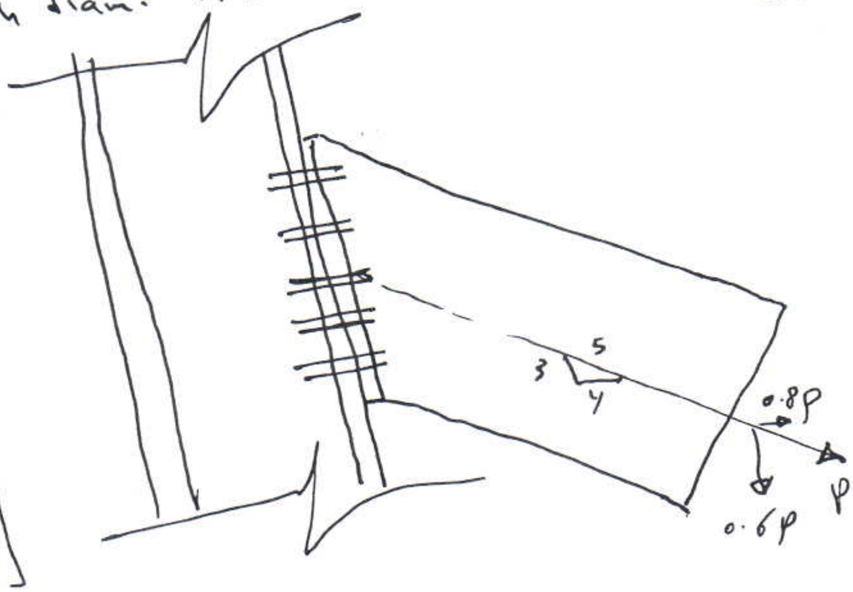
$$F_v = 0.4\,F_y = 14.4 \text{ ksi}$$

$$f_v = \frac{V_{max}}{d \cdot t_w} = \frac{60}{13.[illegible] * 0.8[illegible]} = 5 \text{ ksi} < F_v = 14.4 \text{ ksi}$$

(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

$P_s = F_s . A_b . N_b . N_s$

$0.6P = P_s$

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

for Combine shear & Tension.

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

$F_s = 17$ ksi for A 325

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

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

$T_b = 39^k$

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

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

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

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

$.6P = 102 - 0.2087 P$

$0.8087P = 102$

$\therefore P = 126.13^k$ $\qquad \therefore P \simeq 126^k$ max.