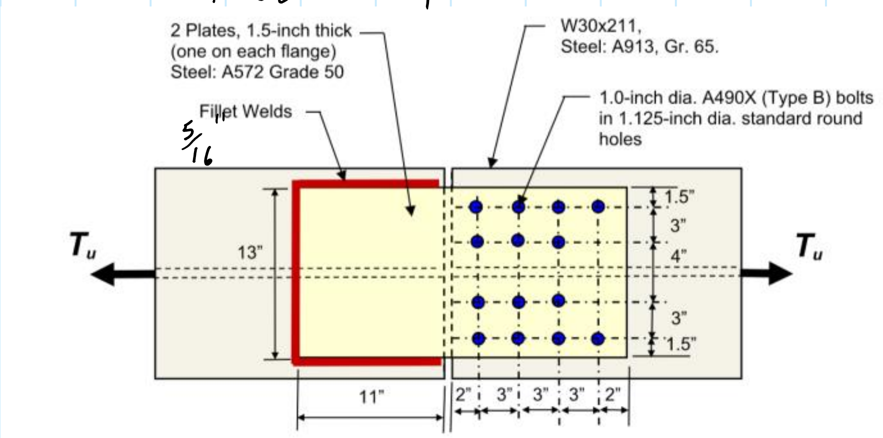
CE122N Midterm 2 Solutions

Problem 1



Failure Modes

① Weld
- BM Tension Yield
- BM Tension Rupture
- Weld Rupture

② Bolts
- Bolt shear rupture
- Bearing
- Spacing / Edge distance

Note: It may be easier to work with one side of connection, then multiply capacity by 2 at the end for weld

① Check capacity of welded connection

First, check min. size of fillet weld

Thinner part joined = W30x211 flange = t_f = 1.32"

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld,[a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[a] Leg dimension of fillet welds. Single pass welds must be used.
Note: See Section J2.2b for maximum size of fillet welds.

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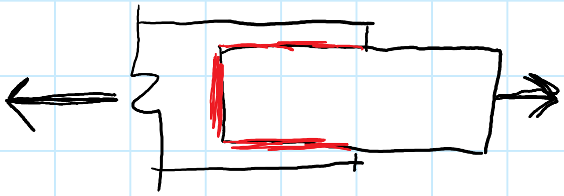
$5/8" \geq 5/16"$ OK

Compare to Max allowable = $t_f - 1/16$ = 1.2575"

$5/8" < 1.26"$ OK



Is base metal in tension or shear ?

$\Rightarrow$ TENSION

$\Rightarrow$ use tension eqn.

Base Metal Tension Yielding

We will check only the plate because the F_y is smaller & the A_g is smaller

$$T_U = \phi F_y A_{BM} = 0.9(50\text{ ksi})(1.5'' \times 13'') = 878\text{ K}$$

Base Metal Tension Rupture

Again, the plate will control, Ae of WF will be close, W 30x211 $A_g = 62.3\text{ in}^2$, if $A_{e_{WF}} = 0.6 A_g = 37\text{ in}^2 \approx \underset{\text{plate}}{2(1.5 \times 13)} = 39$ then $F_{u\,WF} = 80 > F_{u\,plate} = 65$ will decide & makes plate control

$$T_U = \phi F_u A_{BM} = 0.75(65\text{ ksi})(1.5'' \times 13'')$$
$$= 951\text{ kip}$$

Weld Rupture (use Σ weld lengths for A_{we})

$$L_w = 13 + 11 + 11 = 35''$$

$$A_{we} = L_w \times (\text{Leg size})/\sqrt{2} = 35'' \times \frac{5/8}{\sqrt{2}} = 15.5\text{ in}^2$$

$$T_U = \phi_w A_{we} F_{nw} = 0.75(15.5\text{ in}^2)(0.60 \times 70\text{ Ksi})$$
$$= 488\text{ kip}$$

($L = 11 < 100w = 31$ OK, no β reduction)

① Weld Connection Summary

Min capacity => Weld Rupture

=> $T_v = 488\,K \times 2 = \underline{976\ kip}$

② Bolt Connection Capacity

— Bolt Shear Rupture (single shear)

1" ϕ A490X Type B bolts

Hole = 1.125" dia. round holes

=> $F_{nv} = 84\ ksi$

(threads excluded)

$$A_b = \frac{\pi d_b^2}{4} = \frac{\pi (1'')^2}{4} = 0.785\ in^2$$

n = number of bolts (count them all on both sides)

$= (14 \times 2) = 28$

$$T_v = R_u \le \phi R_n = \phi F_{nv} A_{b\_total}$$

$$= 0.75\,(84\ ksi)(28 \times 0.785\ in^2)$$

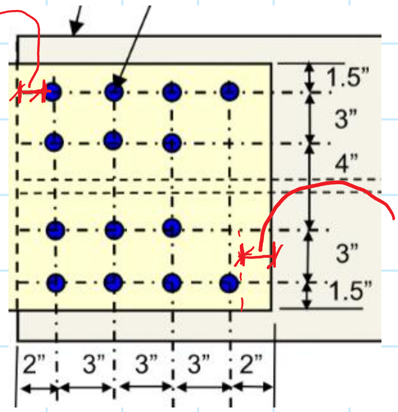
$$= \underline{1385\ Kip}$$

Bolt Bearing Failure of Connected Parts

$R_n = 1.2\, L_c\, t\, F_u \le 2.4\, d\, t\, F_u$

(assume elongation of holes is considered)

Lc worst case WF



Lc (worst case plate)

$L_c = 2" - \frac{d_H}{2} = 2 - \frac{1.125"}{2}$

$= 1.4375" = 1.44"$

use thickness of WF or plate?

$t_f F_u = (1.32")(80\text{ ksi}) = 106$

$t_{plate} F_u = (1.5")(65\text{ ksi}) = 98$ — smaller, controls

⇒ use plate as controlling case

$1.2\, L_c\, t\, F_u = 1.2(1.44")(1.5")(65\text{ ksi})$

$= 168\text{ kip}$

$2.4\, d\, t\, F_u = 2.4(1")(1.5")(65\text{ ksi})$

$= 234\text{ kip}$

$R_u < \phi R_n = 0.75 \min(\underline{168}, 234) = 126\text{ kip/bolt}$

Total capacity for connection $= \frac{T_u}{2 \times 14} \le 126\text{ k/bolt}$

$T_u \le 3528\text{ kip}$

Bolt Spacing Check

Min Bolt Spacing $\geq 2.67(1'') = 2.67''$

Min $= 3'' > 2.67''$ OK

Max Bolt Spacing $\leq (24t, 12'')$

controls

Max $= 3'' < 12''$ OK

Min Edge Distance $= 1\tfrac{1}{4}'' < 2''$ OK

TABLE J3.4 (of the AISC Spec.
Minimum Edge Distance[a] from
Center of Standard Hole[b] to Edge of
Connected Part, in.

Bolt Diameter, in.	Minimum Edge Distance
1/2	3/4
5/8	7/8
3/4	1
7/8	1 1/8
1	1 1/4
1 1/8	1 1/2
1 1/4	1 5/8
Over 1 1/4	1 1/4 × d

[a] If necessary, lesser edge distances are permitted provided the appropriate provisions from Sections J3.10 and J4 are satisfied, but edge distances less than one bolt diameter are not permitted without approval from the engineer of record.
[b] For oversized or slotted holes, see Table J3.5.

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Max Edge Distance $\leq (12t, 6)$

controls

Max $= 2'' < 6''$ OK

$\Rightarrow$ Bolt Spacing is OK

$\Rightarrow$ Bolt Summary

Bolt Shear Rupture = 1385 kip

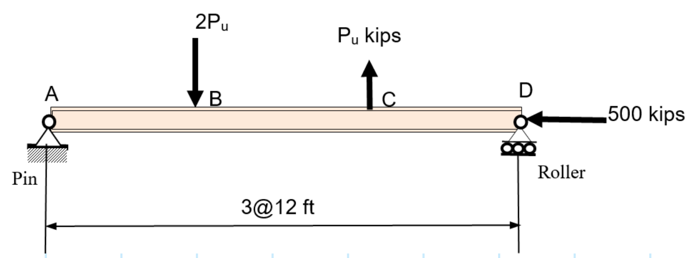
$\Rightarrow$ Weld side controls, T_u = 976 kip

(weld rupture)

Midterm Solution Problem 2

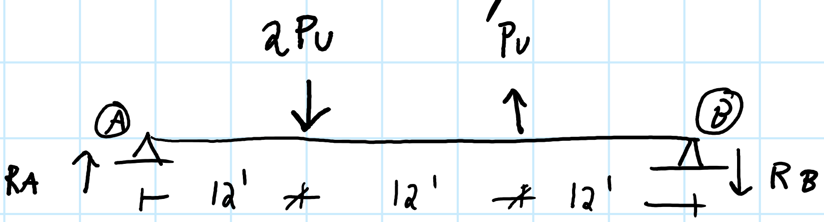
Check
- $\frac{P}{M}$ Interaction
- Shear

Problem 2 (50 points)
Calculate factored load P_u that can be applied to the beam-column shown below. Check all applicable failure modes. The effective length factors K_x and K_y are equal to 1.0. The member is a W14x109, A572 Gr. 50 steel. Lateral bracings are provided at A, B, C, and D.



Goal: solve Interaction EQN for P_u

Statics - Analyze to find M_u



$$+\circlearrowleft \Sigma M_A = 0$$

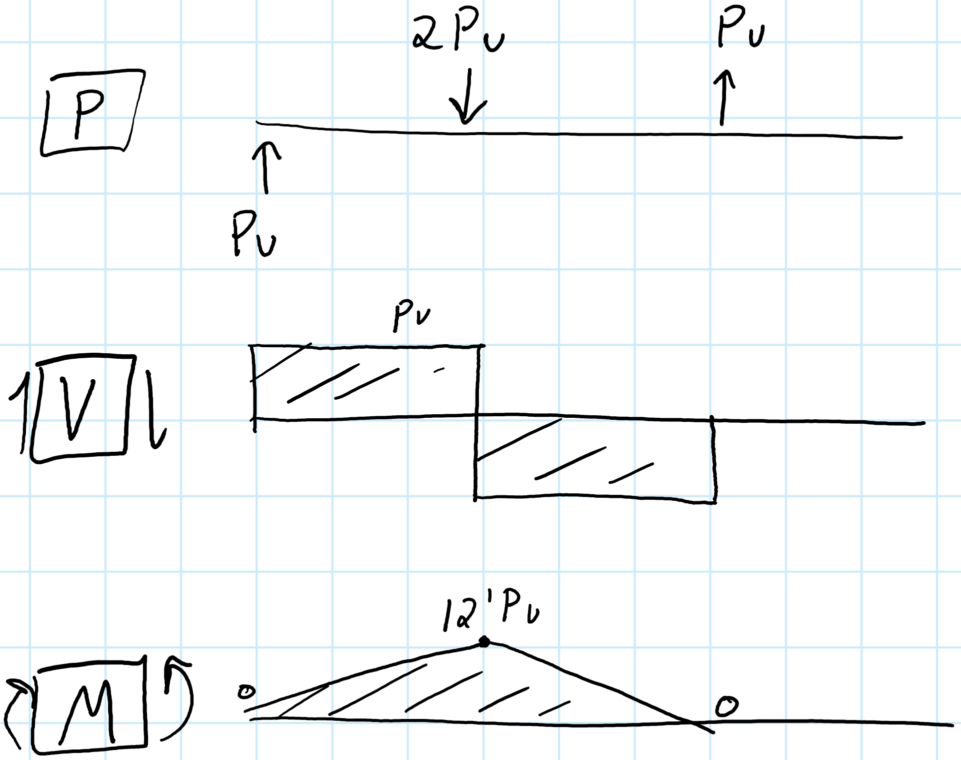
$$-2P_u(12') + P_u(24') - 36'(R_B) = 0$$

$$R_B = 0 \text{ Kip}$$

$$\Rightarrow \quad \Sigma F_y = 0$$

$$R_A - 2P_u + P_u = 0$$

$$R_A = P_u$$



(check shear at the end, it won't likely control)

$M_u = 12' P_u$ (we still need to factor up with $P-\delta$)

So far: We have P_r, M_u

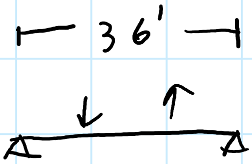
Need: P_c, $M_r = B_1 M_u$, M_c

(Proceed in any order)

$\Rightarrow$ get B_1 for M_r

Beam M_r

$$B_1 = \frac{C_m}{1 - \alpha \frac{P_r}{P_{e1}}} \geq 1.0 \qquad \alpha = 1.0$$



Transverse load between supports,
so $C_m = 1.0$

$$P_{e1} = \frac{\pi^2 E I_x}{(K_x L)^2} = \frac{\pi^2 (29000 \text{ ksi})(1240 \text{ in}^4)}{(1.0\,(432''))^2}$$

$$= 1902 \text{ K}$$

$P_r = 500$ K (given demand)

$$B_1 = \frac{1.0}{1 - \left(\frac{500}{1902}\right)} = 1.36$$

$$M_r = B_1 M_u = 1.36\,(12' P_u)$$

$$M_r = 16.3 P_u \text{ (K·ft)} = \underline{196 P_u \text{ (K·in)}}$$

Now get M_c

Beam Analysis, M_c

Get M_c for column

Check local buckling

$h/t_w = 21.7 < \lambda_p = 3.76\sqrt{E/F_y} = 91$

$\Rightarrow$ web compact

$b/t = 8.49 < \lambda_p = 0.38\sqrt{E/F_y} = 9.15$

$\Rightarrow$ flange compact

$\Rightarrow$ section compact

$\Rightarrow$ No local buckling before plastic hinge

Check LTB

$L_b = 12' \times 12 = 144''$

$L_p = 1.76\, r_y \sqrt{E/F_y} = 1.76(3.73)\sqrt{\frac{29{,}000}{50}} = 158''$

$L_b < L_p$ $\therefore$ No LTB expected prior to plastic yielding

$M_c = \phi M_n = \phi F_y Z_x$

$= 0.9\,(50\text{ ksi})(192\text{ in}^3)$

$M_c = 8640\text{ K}\cdot\text{in}$

Column P_c W14x109 $F_y = 50$

① Check Local Buckling

$$h/t_w = 21.7 < \lambda_r = 1.49\sqrt{E/F_y} = 36$$

=> web nonslender

$$b_f/t_f = 8.49 < \lambda_r = 0.56\sqrt{E/F_y} = 13.5$$

=> flange nonslender

=> Section is non-slender

=> Local buckling not expected prior to overall buckling

② $P_c = \phi_c P_n = \phi_c F_{cr} A_g$

$$\frac{L_c}{r} = \frac{432''}{6.22''} = 69.5 < 4.71\sqrt{E/F_y} = 113$$

x-x axis worst case

=> inelastic buckling

$$\Rightarrow F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$$

$$F_e = \frac{\pi^2 E}{\left(L_c/r\right)^2} = 59.3 \text{ ksi}$$

$$F_{cr} = \left(0.658^{50/59.3}\right) 50 \text{ ksi} = 35.1 \text{ ksi}$$

$$P_c = \phi F_{cr} A_g = 0.9(35.1 \text{ ksi})(32 \text{ in}^2) = \underline{1011 \text{ kip}}$$

Interaction EQN & Solve P_u

$M_c = 8640$ K·in $\quad M_r = \underline{196\,P_u}$

$P_c = 1011$ K $\quad P_r = 500$ K

$$P_r / P_c = 0.49 > 0.2$$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_r}{M_c}\right) \le 1.0$$

$$0.49 + \frac{8}{9}\left(\frac{196\,P_u}{8640}\right) \le 1.0$$

$$\boxed{P_u \le 25 \text{ Kip}}$$

Check Shear

Demand = P_u (factored)

so $V_u = P_u$

① Are stiffeners required?

$h/t_w = 21.7 < 260$

$\Rightarrow$ No stiffeners required

② Establish ϕ_v

$h/t_w = 21.7 < 2.24\sqrt{E/F_y} = 54$

& rolled shape, so $\phi_v = 1.0$

③ Establish C_{v1}

$\Rightarrow$ rolled shape with $h/t_w < 2.24\sqrt{E/F_y}$

$\Rightarrow C_{v1} = 1.0$

④ $V_u \leq \phi_v V_n = \phi_v (0.6 F_y A_w C_{v1})$

$= 1.0(0.6 \times 50ksi \times (14.3" \times .525") 1.0)$

$V_u = P_u < 225\ K$

$\Rightarrow$ bending controls, $\boxed{P_u \leq 25\ Kip}$