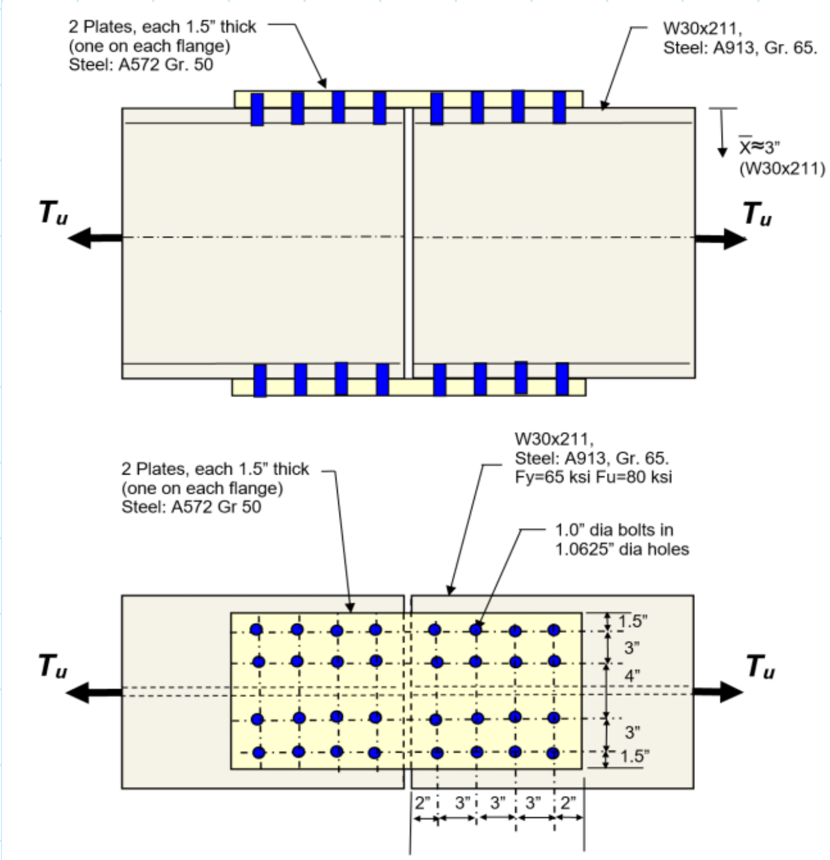


W30x211

$A_g = 62.3 \ in^2$

$d = 30.9 \ in$

$t_w = 0.775 \ in$

$b_f = 15.1 \ in$

$t_f = 1.32$

$b_f/t_f = 5.74$

$h/t_w = 34.5$

$F_y = 65 \ ksi$

$F_u = 80 \ ksi$

$\boxed{1}$ Check Gross Area Yielding

W30x211

$T_u < \phi_t P_n$

$\phi_t = 0.9 \qquad P_n = F_y A_g = 65 \ ksi \ 62.3 \ in^2 = 4050 \ kip$

$T_u < 0.9(4050 \ k) = \underline{3645 \ kip}$

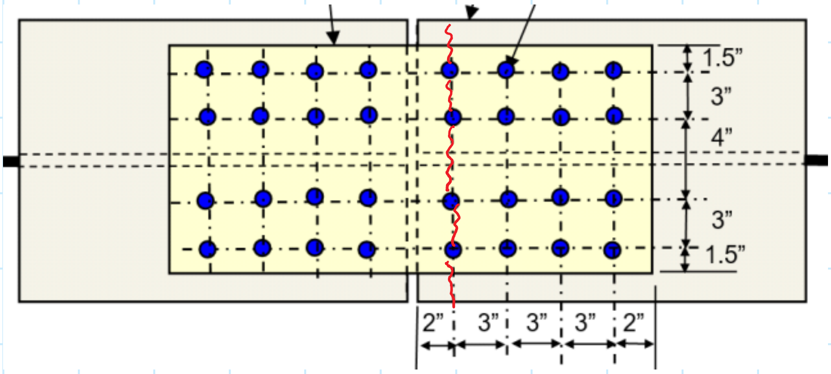
2 PL 13 x 1.5

$T_u < \phi_t P_n$

$\phi_t = 0.9 \qquad P_n = F_y A_g = 50 \ ksi (2 \times 13'' \times 1.5'') = 1950 \ kip$

$T_u < 0.9(1950 \ kip) = \underline{1755 \ k}$

$\boxed{2}$ Check Net Section Rupture



Summary of all

W 30x211 Bolted side

2 PL 13x3/4 Bolted side

Total = 2 checks

hole ϕ = (diameter + 1/16)

= 1.0625 + 1/16

= 1.125"

① W 30x211 Bolted side

$T_u \le \phi P_n = \phi F_u A_e = \phi F_u A_n U$

$A_n = A_g - n(\text{hole } \phi) t_f$

$= 62.3 - 8(1.125") \, 1.32$

$= 50.42 \text{ in}^2$

$U = 1 - \bar{x}/L$

$\bar{x} = b_f \times t_f \times \frac{t_f}{2} + \frac{T}{2} \times t_w \times \left(\frac{T}{4} + t_f\right) / \Sigma \text{Area}$ (T = 26.5)

$= \left(15.1 \times 1.32 \times \frac{1.32}{2}\right) + \left(\frac{26.5}{2} \times 0.775 \times \left(\frac{26.5}{4} + 1.32\right)\right) / \Sigma \text{Area}$

$= 13.155 + 81.6 / \Sigma \text{Area}$

$= \frac{94.74}{\Sigma \text{Area}} = \frac{94.74}{A_g/2} = \frac{94.74}{62.3/2} = 3.04"$

$= 3"$

$$U = 1 - \frac{\bar{x}}{L} = 1 - 3/9 = 0.67$$

$$T_u \le \phi F_u A_n U = 0.75\,(80\text{ ksi})(50.42\text{ in}^2)(0.67)$$

$$\underline{T_u \le \phi P_n = 2027\text{ kip}}$$

3 $\underline{2}$ PL 13x1.5 Bolted side

$U = 1.0$

$$A_n = A_g - n(\text{hole }\phi)t = \underline{2}\left(13 \times 1.5 - 4(1.125)\,1.5\right)$$

$$= \underline{2}\,(12.75)$$

$$= 25.5\text{ in}^2$$

$$T_u \le \phi F_u A_n U = 0.75(25.5\text{ in}^2)(1.0)(65\text{ ksi})$$

$$= 1243\text{ kip}$$

$$\underline{T_u \le \phi P_n = 1243\text{ kip}}$$

3 Block Shear Rupture

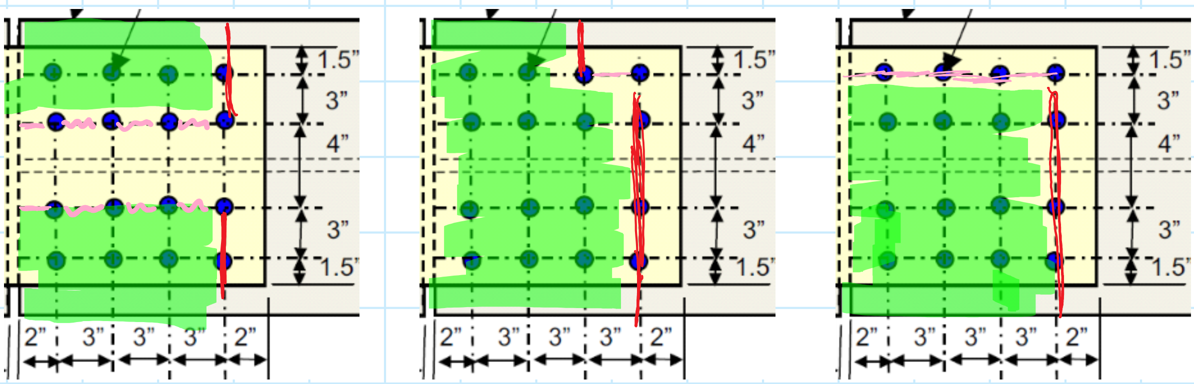
Identify what to check

W 30 x 211 => (?) Bolted side
2PL $13 \times \frac{3}{4}$ => (?) Bolted side

we have to determine which shapes to check

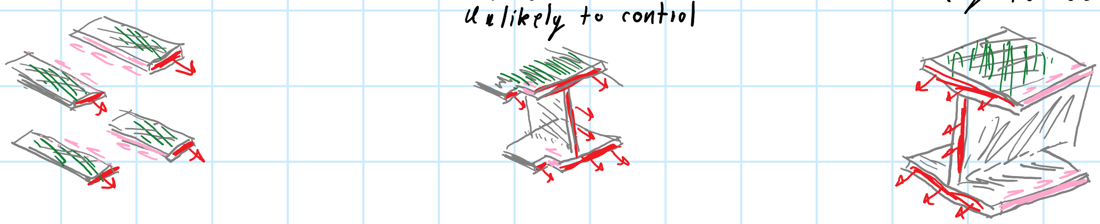
① W30x211 Bolted Side

Draw potential shapes



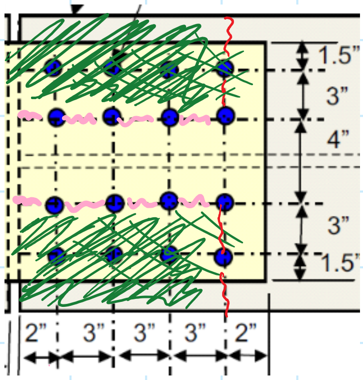
Cuts through entire web. Unlikely to control

Cuts through entire web. Unlikely to control



By observation, any shape that rips through the web adds a lot of area! The first shape rips out of the flange only & has the smallest area, ∴ lowest capacity

① continued BSR



$A_{nt} = t_f[b_f - 4" - n(1.125")] \times 2 = 1.32[15.1 - 4 - 3(1.125)]2$

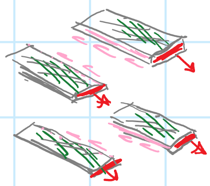
$= 20.4 \ in^2$

$A_{nv} = 4(11" - 3.5(1.125")) \, 1.32"$

$= \underline{37.3 \ in^2}$

$A_{gv} = 4(11") \, 1.32"$

$= \underline{\underline{58.08 \ in^2}}$



BSR $T_u < \phi R_n$

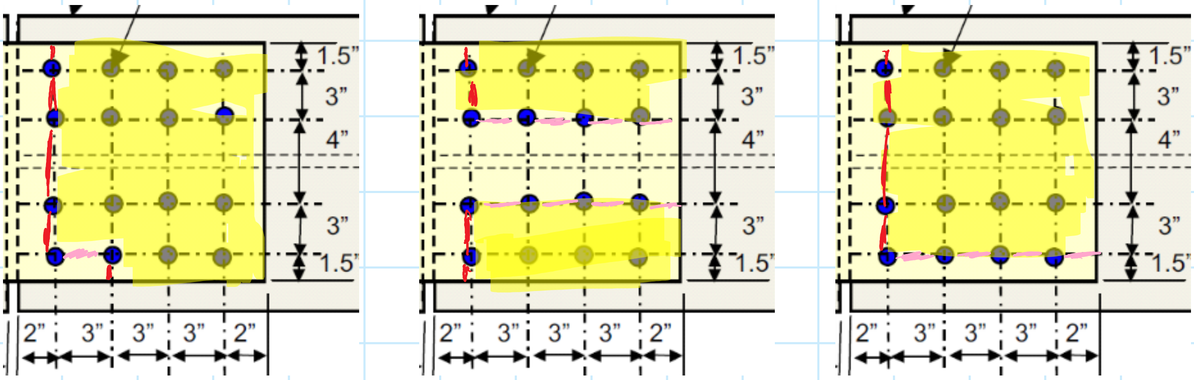
$< \phi \min(0.6 F_u A_{nv} + U_{bs} F_u A_{nt}, \ 0.6 F_y A_{gv} + U_{bs} F_u A_{nt})$

$\phi(0.6(80)(37.3) + 80(20.4), \ 0.6(58)(58) + 80(20.4))$

$\phi(\underline{\underline{3422}}, \ 3650)$

$T_u < 0.75(3422) = 2567 \text{ kip}$

(2) BSR 2PL18x1.5 Bolted Side



(A) (B) (C)

Before plugging in numbers, look at perimeter....

(A) has ≈ 16"

(B) has ≈ 31" w/o subtracting holes

(C) has ≈ 22"

clearly (A) is of minimal perimeter

(A) $A_{nt} = [13" - 4(1.125")] \times 2 \times 1.5" = 25.5 \text{ in}^2$

$A_{nv} = 2[3" - 1.125"] 1.5" = 5.63 \text{ in}^2$

$A_{gv} = 2[3"]1.5" = 9 \text{ in}^2$

$$T_u < \phi R_n = \phi \min\left(\begin{array}{l} 0.6(65\text{ ksi})(5.63\text{ in}^2) + 65\text{ ksi}(25.5\text{ in}^2), \\ 0.6(50\text{ ksi})(9\text{ in}^2) + 65\text{ ksi}(25.5\text{ in}^2) \end{array}\right)$$

$$= 0.75(1877)$$

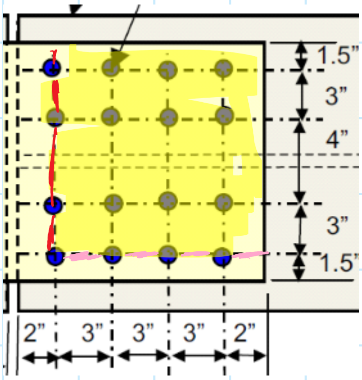
$$T_u < \phi R_n = 1407 \text{ Kip}$$

Conclusion:

Lowest T_u from 2PL13x1.5 - Net Section Rupture

$T_u \leq 1243$ Kip

Even if the answer is incorrect, final answer should state capacity, failure mode, and from What material.



C

Since many students may have checked C, it is shown here
For completeness.

$$A_{nt} = \underline{2}\left(11.5'' - 3.5\left(1.125''\right)\right) 1.5'' = 22.69 \text{ in}^2$$

$$A_{nv} = \underline{2}\left(11'' - 3.5\left(1.125''\right)\right) 1.5'' = 21.19 \text{ in}^2$$

$$A_{gv} = 2\left(11''\right) 1.5'' = 33 \text{ in}^2$$

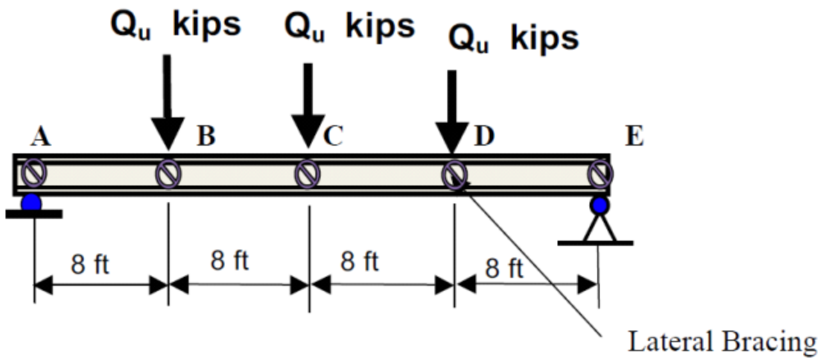
$$T_u \le \phi R_n = \phi \min\left(0.6\,(65\text{ ksi})(21.19\text{ in}^2) + 65\text{ ksi}\,(22.69\text{ in}^2),\ 0.6\,(50\text{ ksi})(33\text{ in}^2) + 65\text{ ksi}\,(22.69\text{ in}^2)\right)$$

$$= \phi\,(2301\text{ kip})$$

$$= 0.75\,(2301\text{ kip})$$

$$T_u \le \phi R_n = 1726\text{ kip}$$

(note case A has $T_u = 1407$ kip, so perimeter based estimate for uniform thickness plate was accurate for predicting Case A to control)



Elevation of the Beam

W 24x84 Find max Q_u

Beam - Flexure

① Check Plastic Hinge

$M_u < \phi_b M_p = \phi_b F_y Z_x = 0.9 \times 50\,ksi \times 224\,in^3$

$M_u \leq \phi_b M_p = \underline{10{,}080\ k \cdot in}$

② Check Local Buckling Web/Flange

$b/t = 5.86 < 0.38\sqrt{E/F_y} = 9.15$

=> Compact, no flange local buckling prior to M_p

$h/t_w = 45.9 < 3.76\sqrt{E/F_y} = 91$

=> compact, no web local buckle prior to M_p

③ Check LTB $(r_y = 1.95)$

$L_b = 8' \times 12 = 96''$

$L_p = 1.76 r_y \sqrt{E/F_y} = 82.6''$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}}$$

$r_{ts} = 2.37$

$J = 3.7$

$S_x = 196$

$h_o = 23.3$

$C = 1.0$

$L_r = 244$

$\Rightarrow \quad L_p < L_b < L_r$

$\Rightarrow$ inelastic LTB expected Prior to M_p!

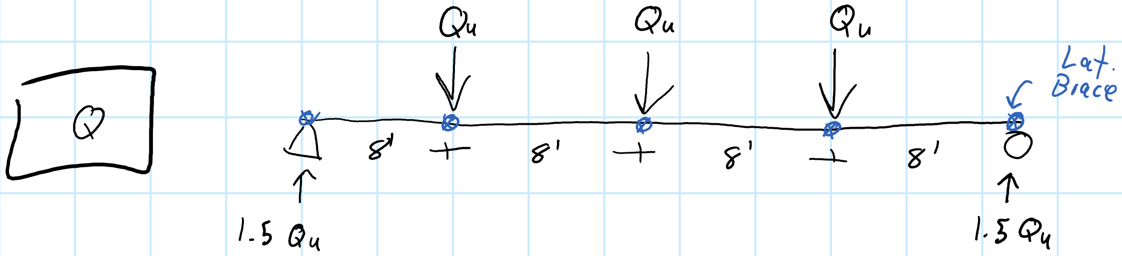
$\Rightarrow$

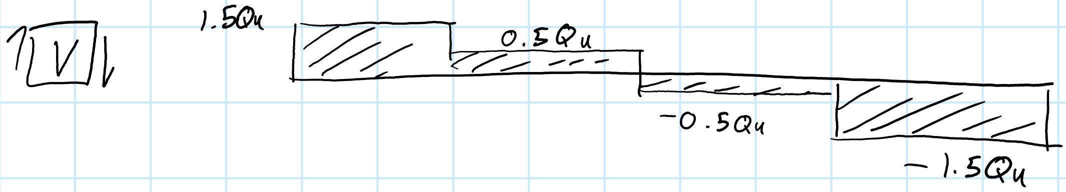
2. Inelastic LTB

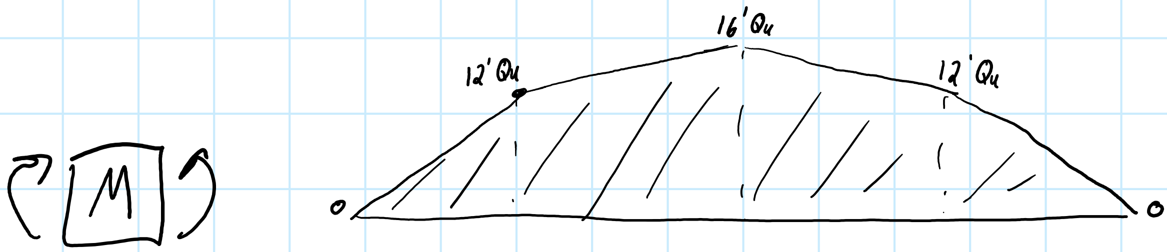
$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

Now find C_b from moment diagram

$L_b = 96 \quad L_r = 244 \quad L_p = 82.6$

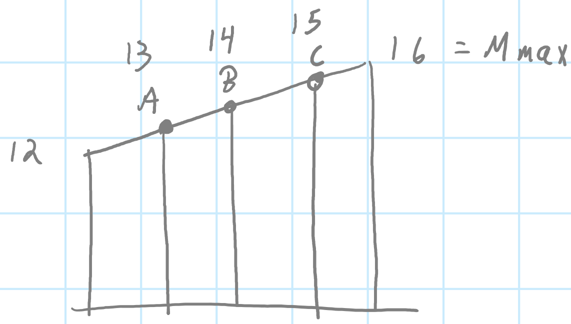






$$C_b = \frac{12.5\,M_{max}}{2.5\,M_{max} + 3M_A + 4M_B + 3M_C}$$

$\Rightarrow$ More uniform & larger diagrams can have lower C_b, so we can tell that the middle sections will control (since each length is the same)



$$C_b = \frac{12.5(16)}{2.5(16) + 3(13) + 4(14) + 3(15)}$$

$$C_b = \underline{1.11}$$

we are tasked with finding $\underline{\text{max}}$ Q_u, it would not be appropriate to use a $Q_u = 1.67$ (triangle) which is $\underline{\text{wrong}}$ (under-conservative) or a $Q_u = 1.0$ ("conservative assumption")

$$C_b = 1.11$$

$C_b = 1.11 \quad L_b = 96 \quad L_r = 244 \quad L_p = 82.6$

$$M_n = 1.11\left(11{,}200\,k\cdot in - (11{,}200 - 0.7\times 50\times 196)\left(\frac{96-82.6}{244-82.6}\right)\right)$$

$$1.11 \times [11{,}200 - (4340 \times 0.083)]$$

$$= 12032\ k\cdot in > M_p$$

since LTB capacity > M_p

Take $M_n = M_p$!

Overall, $M_u \le \phi_b M_p = \boxed{10{,}080\ k\cdot in \text{ OR } 840\ k\cdot ft}$

$$16' Q_u \le 840\ k\cdot ft$$

$$Q_u \le 52.5\ kip$$

Now check Shear Capacity

$$1.5\,Q_u \le \phi V_n$$

SHEAR

① Are stiffeners required?

$$h/t_w = 45.9 < 260$$

$\Rightarrow$ No stiffeners req. (OK as is)

② Establish ϕ_v

$$h/t_w = 45.9 < 2.24\sqrt{E/F_y} = 53.9$$

$\Rightarrow \phi_v = 1.0$ (rolled shapes only)

③ Establish C_{v1}

since $h/t_w < 2.24\sqrt{E/F_y}$

$C_{v1} = 1.0$

full depth x web thickness

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

$$= 1.0(0.6 \times 50\,ksi \times 24.1'' \times 0.47'' \times 1.0)$$

$$V_u \le \phi_v V_n = 339.8 \text{ Kips}$$

$$1.5 Q_u \le 339.8 \text{ K}$$

$$Q_u \le 227 \text{ Kip} \qquad < 52.5 \text{ Kip}$$

Flexural Bending Controls

$$\boxed{Q_u = 52.5 \text{ Kip}}$$

Plastic Hinge Formation Controls