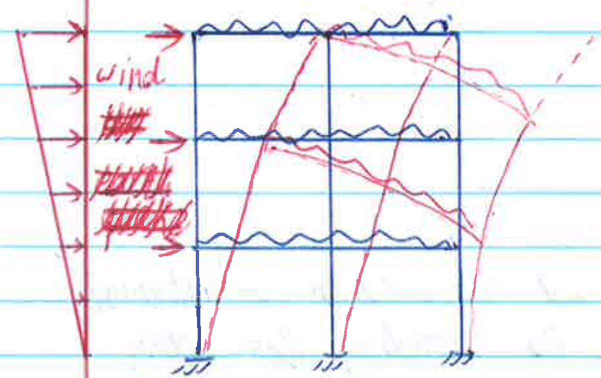
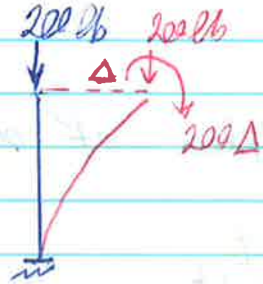
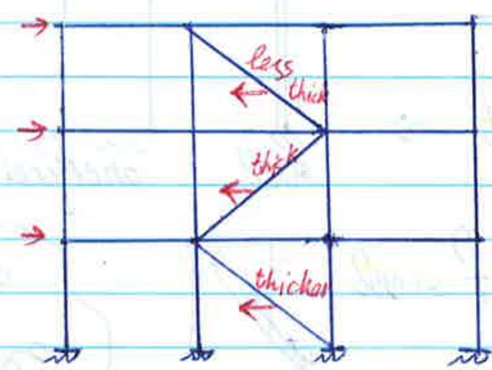
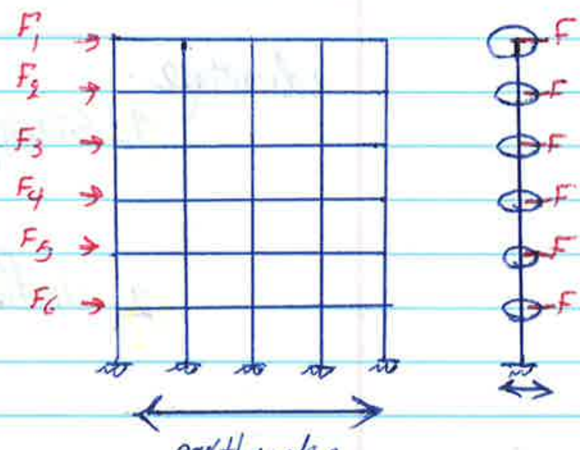
Advance Structural Steel



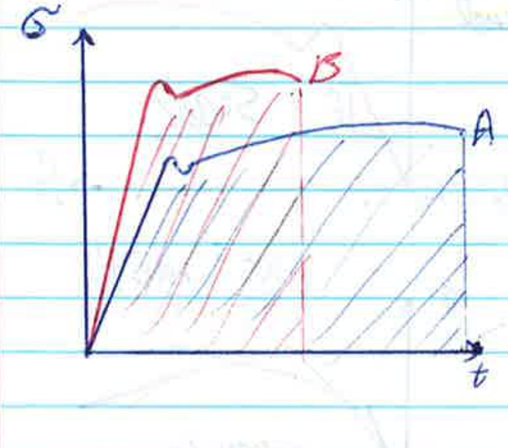


* Controlling the excessive deflection key to structural stability





$$F = \frac{w}{g}\ddot{a} = F_1 + F_2 + F_3 + F_4 + F_5 + F_6$$



$$\text{Ductility} = \frac{L - L_0}{L_0}$$

Toughness = area under chart (σ–t)
= Energy Absorbed

Redandant → to provide added safety against unkown situations.

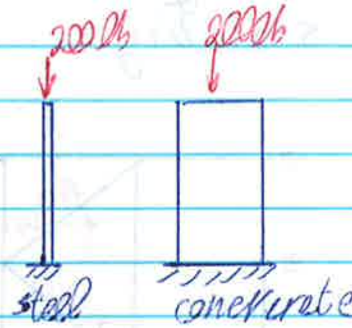
* Principles of design

1. Cost
2. Weight → $F = \frac{w}{g} \ddot{a}$ and Transportation of material
3. onstruction time → for spending less money
4. labor

* Steel Vs. Concrete

advantage

1. strength/weight ⇒

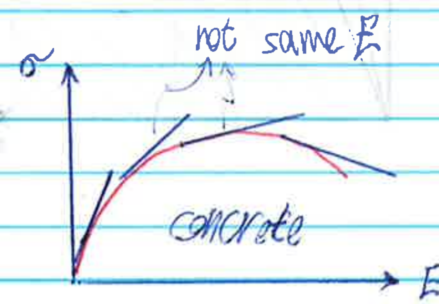


2. uniformity of steel

3. Elasticity of steel

$\delta = \frac{PL^4}{EI}$

E ↪ always 30,000 ksi



4. Permanence of steel because concrete has corrosion problem.

5. Ductile → warning (concrete has no Ductile)

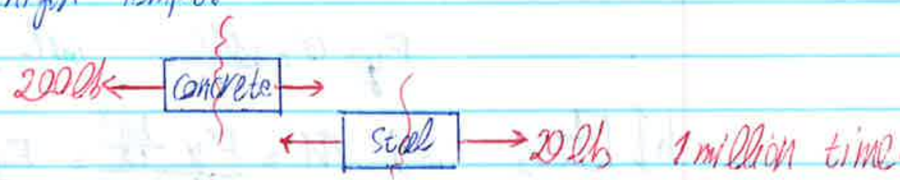
6. Tougness → steel is better in tougness means can handle loads in a period of time. not just one force.

*Problems of steel

1. Maintenance → Paint → corrosion

2. Performance in high Tempt.

3. Fatigue



4. buckling of steel

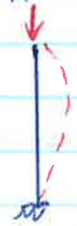
* Skeleton of steel structure consist of Tension member, Columns and beams.

* Tension members: (more area is better) (shape doesn't matter)

Cable

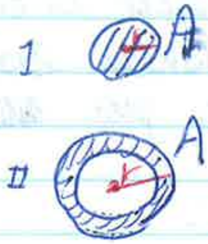
How we choose: $F = F_y A_y \Rightarrow A_y = \frac{F}{F_y}$

* compression member (column):



$$P_{cr} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 E A r^2}{L^2} \Rightarrow \text{only } r \text{ is not constant}$$

r = radius of gyration
= How much it spread out

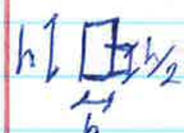


same Area but II is better in compression

Compression → more spread out better

* bending member (beam):

$$F_y = \sigma = \frac{Mc}{I} \qquad M = \frac{F_y I}{c}$$

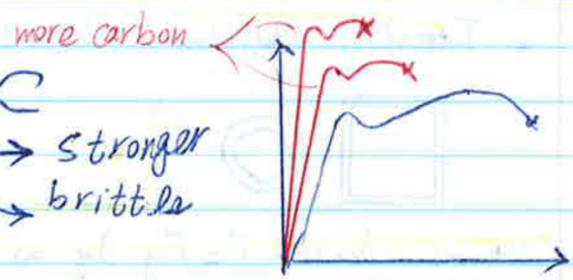


$$\Rightarrow M = \frac{F_y \frac{bh^3}{12}}{h/2} = F_y \frac{bh^2}{6}$$

$$\Rightarrow F_y \frac{bh^2}{6} = F_y \frac{b \frac{b^2}{16}}{6} = \boxed{F_y \frac{b^3}{96}}$$

$$\Rightarrow F_y \frac{bh^2}{6} = F_y \frac{\frac{b}{4} h^2}{6} = F_y \frac{b b^2}{4 \times 6} = \boxed{F_y \frac{b^3}{24}}$$

Steel → Fe + C
more carbon → stronger
↳ brittle



* with increasing 0.01 % carbon → 5 ksi increase in Steel

Cast Iron = C > 2%

Steel = 0.5% — 1.7% of C

Wrought Iron = C < 0.15%

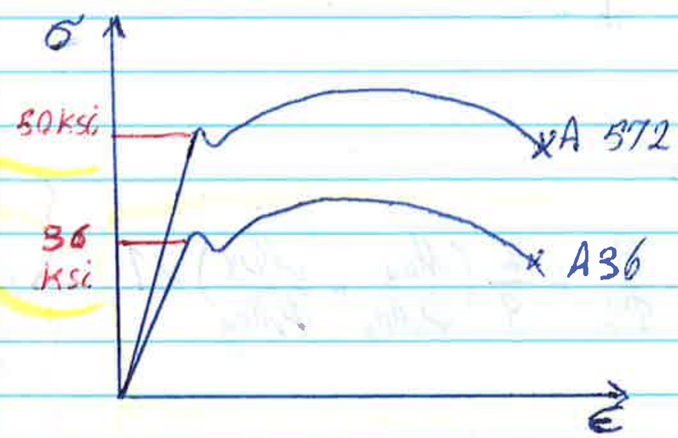
types of steel → Structural Steel:

1. Regular Steel (A36, A53, A500, A501, A529)
2. High strength / low Alloy (A572, A618, A913, A992)

→ Corrosion resistant steel (A242, A588, A847)

→ Cu vapor

* Adding low Alloys (Silicon, Manganese, Cooper) makes steel High-Strength

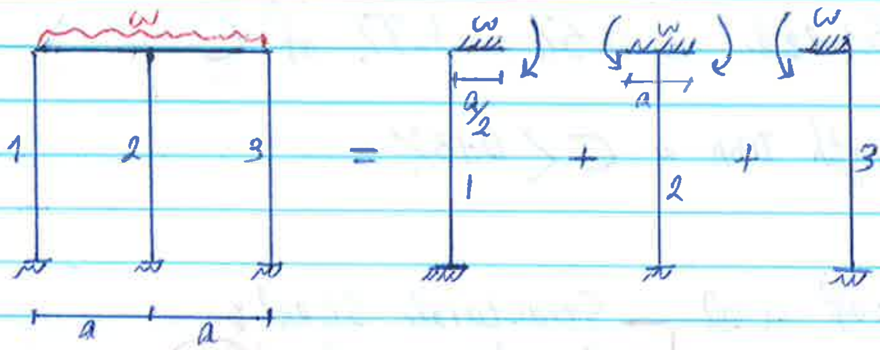


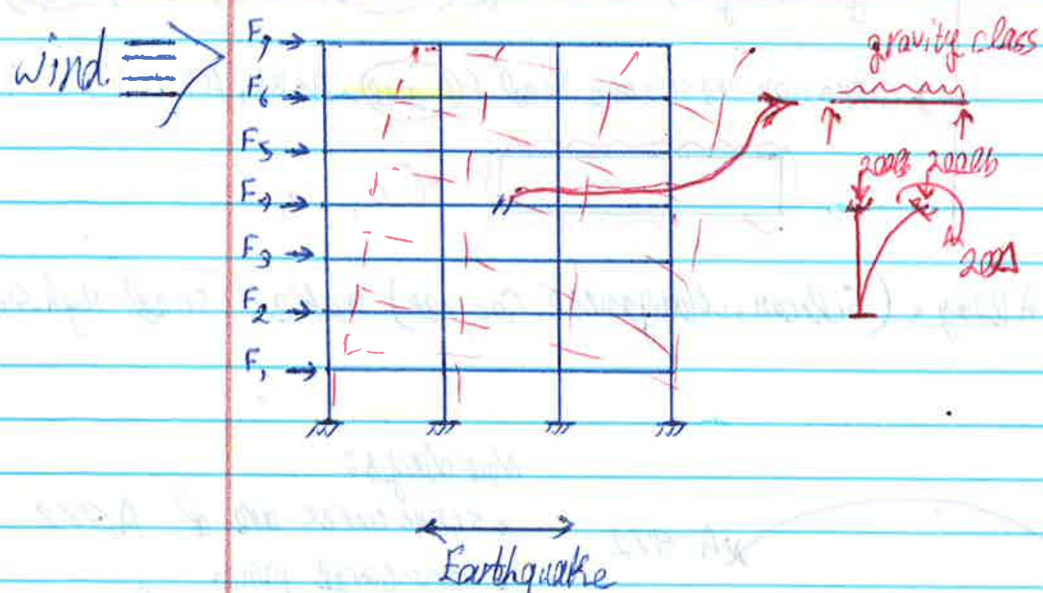
Now days:

- structures are of A572
- structural joints and connections are of A36

Chapter 11

Beam_Column





$$\frac{P_u}{\Phi P_n} \geq 0.2 \rightarrow \frac{P_u}{\Phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\Phi_b M_{nx}} + \frac{M_{uy}}{\Phi_b M_{ny}}\right) \leq 1 \rightarrow$$ like a column or tension member

$$\frac{P_u}{\Phi P_n} < 0.2 \rightarrow \frac{P_u}{2\Phi P_n} + \left(\frac{M_{ux}}{\Phi_b M_{nx}} + \frac{M_{uy}}{\Phi_b M_{ny}}\right) \leq 1 \rightarrow$$ like a beam

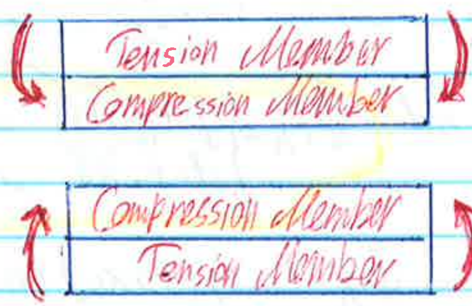
P_u: Calculated factor load

ΦP_n: Capasity of column

← under Tension → $T = \phi F_y A_g$ (indepent of length)

→ under compression ← $C = \phi \dfrac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$ (dependent to length)

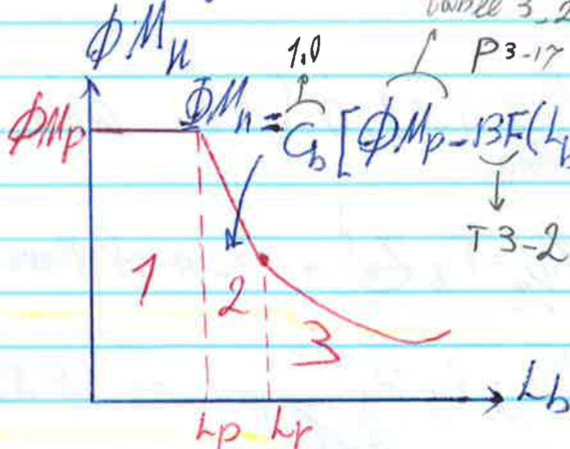
in beam:

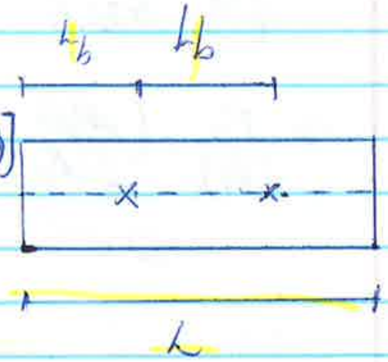


* for bracing:

table 3_2, P 3-17

$$\Phi M_n = \underbrace{C_b}_{1.0}\left[\Phi M_p - \underbrace{BF}_{T\,3\text{-}2}(L_b - L_p)\right]$$





Zone ① Plastic hinge develops by entire section reach yielding.

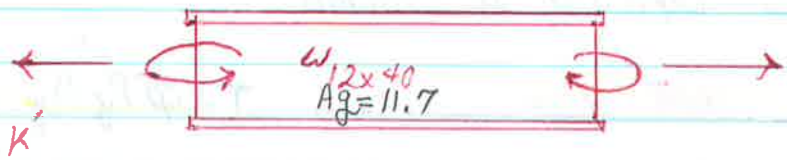
Zone ② some portion of x-section reach yielding

Zone ③ Torsional Buckling

homework: problem 11-1 next Thursday

Beam Tension Member

Exp 11.1:



$P_D = 25^k$

$P_L = 30^k$

$M_{Dy} = 10^{k'}$

$M_{Ly} = 25^{k'}$

$P_u = 1.2 \times 25 + 1.6 \times 30 = 78^k$

$M_{Uy} = 1.2 \times 10 + 1.6 \times 25 = 52^{'k}$

$\frac{P_u}{\Phi P_n} = \frac{78}{\bigcirc}$ and $\Phi P_n = \begin{cases} \Phi F_y A_g & \text{when we dont have holes} ✓ \\ \Phi F_u A_e & \text{when we have holes} ✗ \end{cases}$ → T5-1 P 5-12

$\Rightarrow \Phi P_n = (0.9)(50\,Ksi)(11.7) = 526.5$

$\Rightarrow \frac{P_u}{\Phi P_n} = \frac{78}{526.5} < 0.2 \Rightarrow$ formula H1-1b applies

$\Rightarrow \frac{P_u}{2\Phi P_n} + \left(\frac{M_{ux}}{\Phi M_{nx}} + \frac{M_{uy}}{\Phi M_{ny}} \right) \leq 1.0$ (acts like a beam)

0

$M_{Px} = F_y Z_x$ → P 3-19 of Manual

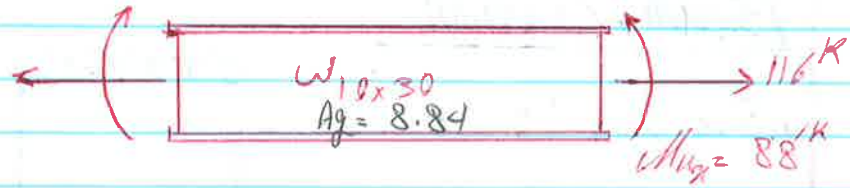
$M_{Py} = F_y Z_y$ → P 3-30 of Manual $= 16.8$

$\frac{78}{2 \times 526.5} + \left(\frac{52}{63} \right) = 0.899 \leq 1$ ✓ Design is good

$0.9 \leq \dots \leq 1$ is always a good design

Exp:

$L_b = 12 ft$



$\Phi P_n = \Phi F_y A_g = (0.9)(50 ksi)(8.84) = 397.8$

$\frac{P_u}{\Phi P_n} = \frac{116}{397.8} = 0.292 > 0.2 \Rightarrow$ (act like tension member)

$\Rightarrow$ we use H1-1a formula

$$\frac{P_u}{\Phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\Phi M_{nx}} + \frac{\overset{0}{M_{uy}}}{\Phi M_{ny}}\right) \leq 1.0$$

$$\Rightarrow \frac{116}{397.8} + \frac{8}{9}\left(\frac{88}{\bigcirc}\right.$$

Hint: in the "y" direction there is no zoning and the formula is:

$\Phi M_{ny} = \Phi F_y Z_y$ (for exp last page)

P 3-19: we have $W_{10\times30} \xrightarrow{P3-26} \begin{cases} L_p = 4.84 ft \\ L_r = 16.1 ft \end{cases} \Rightarrow L_p \leq 12 \leq L_r$ Zon 2

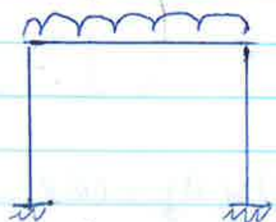
Zone 2 $\Rightarrow \Phi M_n = \overset{1.0}{C_b}[\Phi M_{px} - BF(L_b - L_p)]$

$= 1[137 - 4.61(12 - 4.84)] = 104'^K$

$\Rightarrow \frac{116}{397.8} + \frac{8}{9}\left(\frac{88}{104} + 0\right) = 1.044 > 1$ X N.G.

Beam-column

* Scenario I:

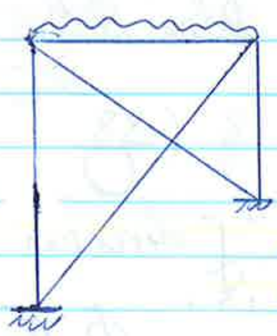


load and frame are symmetric

we call this situation: Non-translationale — nt
Non-sway
Non-lateral displacement
No-horizontal load

P_{nt}
M_{nt}

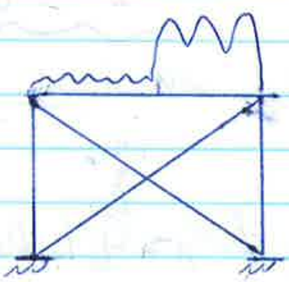
* Scenario II:



Unsymmetrical Structure but symmetrical loading — nt

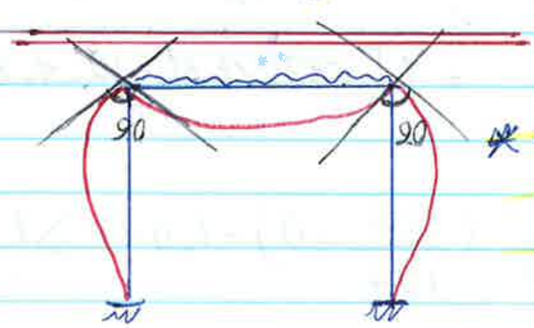
because we fixed the structure from swaying

* Scenario III:



nt

symmetric structure but non symmetric loading



* degree btw. tangent of column and beam is always 90°

fail or equilibrium

$\Sigma M_o = 0$

$\Rightarrow M'_{nt} - M_{nt} - P_u \delta = 0$

$M'_{nt} = M_{nt} + \underbrace{P_u \delta}_{\text{secondary moment}}$

solutions:
1. non linear FE analysis
2. AISC emperical method ✓ (use this)

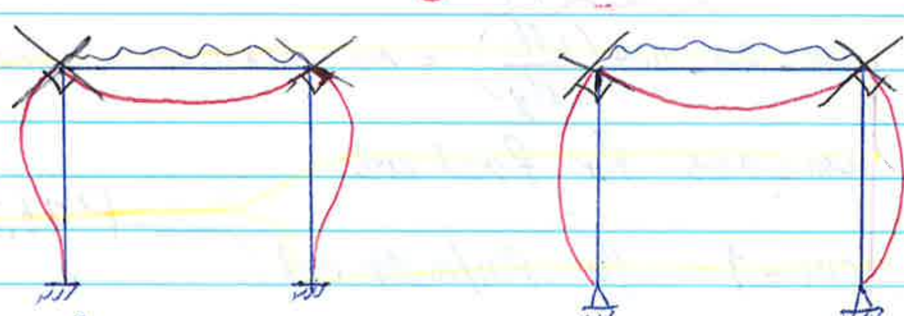
$\Rightarrow M'_{nt} = B_1 M_{nt}$

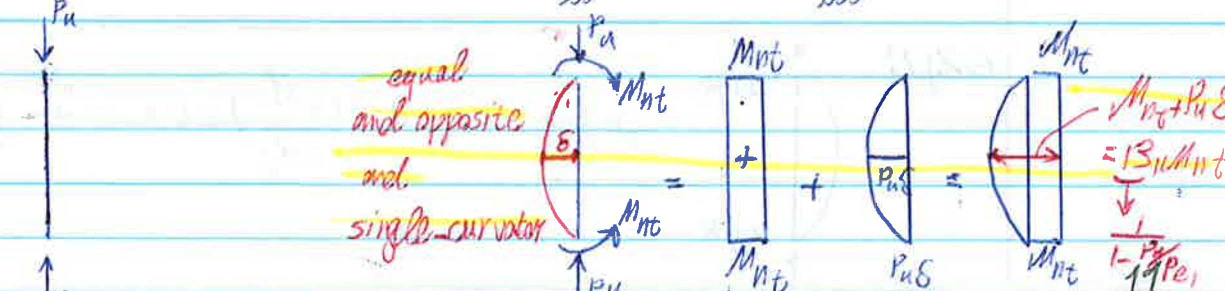
$\underbrace{B_1 = \dfrac{1}{1 - \dfrac{P_u}{P_{e1}}}}_{\text{for Single-Curvator}}$

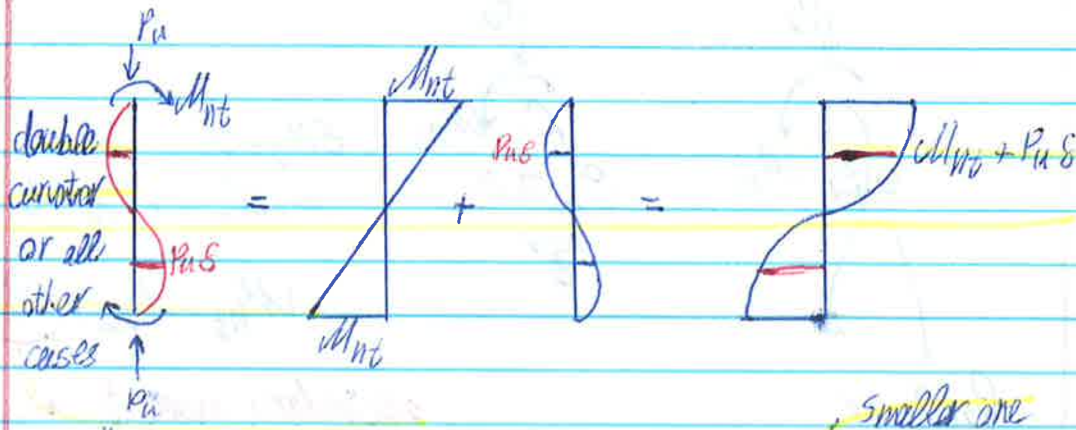
P_{e1} = buckling load = $\dfrac{\pi^2 E I}{(KL)^2}$ → T 1-1 p 1-27

braced column / Braced frame

$K \begin{cases} p\ 16.1\text{-}511 \Rightarrow \text{single column} \\ p\ 16.1\text{-}512 \Rightarrow \text{braced frame} \\ NT \Rightarrow K=1 \text{ (conservative)} \end{cases}$







$$\underbrace{\beta_1 = \frac{C_m}{1 - \frac{P_u}{P_{e_1}}}}_{\text{for other cases than single curvator}} \quad C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right)$$

M_1: smaller one

M_2: bigger one

$$\frac{M_1}{M_2} : \begin{cases} + \Rightarrow \text{for all other cases} \\ - \quad \text{for single curvator} \end{cases}$$

summary:
$$\beta_1 = \frac{C_m}{1 - \frac{P_u}{P_{e_1}}}$$

$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right)$ = No transverse load

$C_m = 0.85$ for fixed end

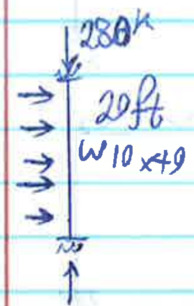
$C_m = 1$ for Pin/roller end

P 16.1-525

exp:

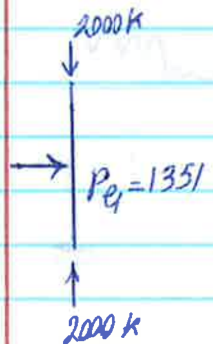
$$\Rightarrow C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6 - 0.4\left(-\frac{40}{50}\right) = 0.92$$

$$C_m = 0.6 - 0.4\left(+\frac{60}{80}\right) = 0.3$$

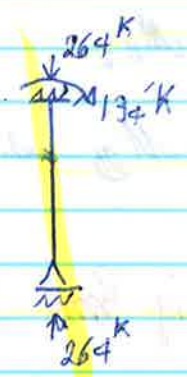


$$C_m = 1 - 0.4\frac{P_u}{P_{e1}} = 1 - 0.4\frac{280}{\dfrac{\pi^2 \times 29000 \times 272}{(1 \times 20 \times 12)^2}} = 0.92$$

$$\frac{\pi^2 EI}{(KL)^2}$$



$$C_m = 1 - 0.2\frac{P_u}{P_{e1}} = 1 - 0.2\frac{200}{1351} = 0.97$$



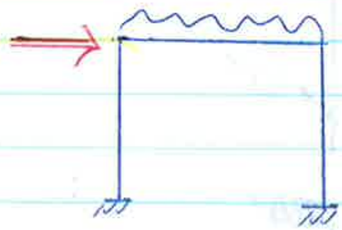
$$C_m = 0.6 - 0.4\left(\frac{\cancel{M_1}^{\,0}}{M_2}\right) = 0.6$$

in this case we dont have horizantal load so that we cant use p 16.1-525

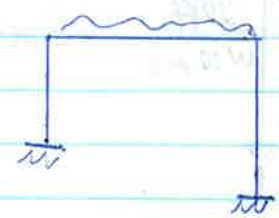
2nd Beam-Columns

LT = lateraly translational

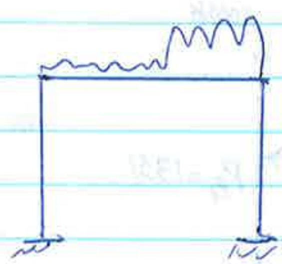
* Scenario I:

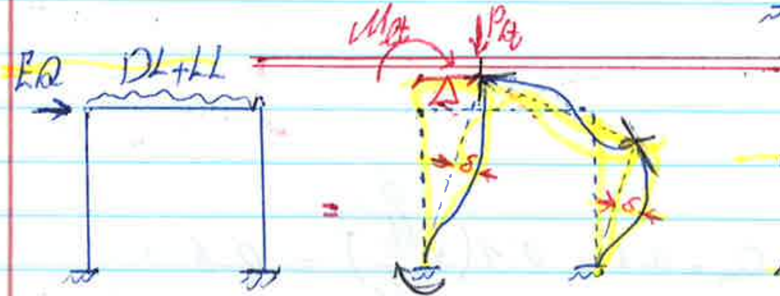


* Scenario II:



* Scenario III:



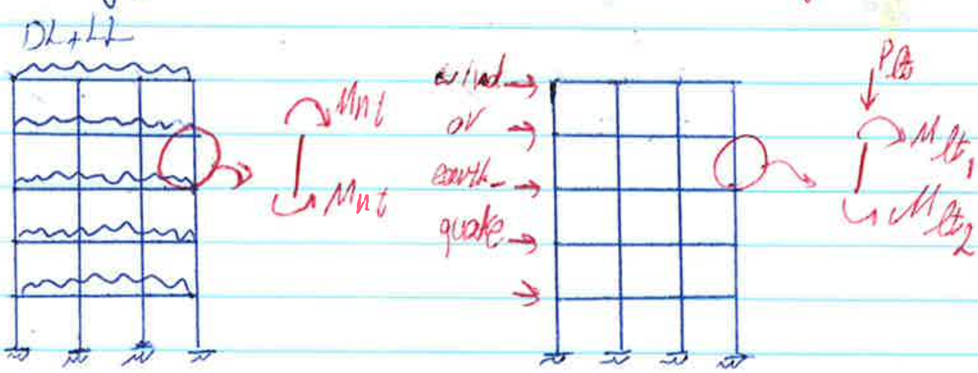


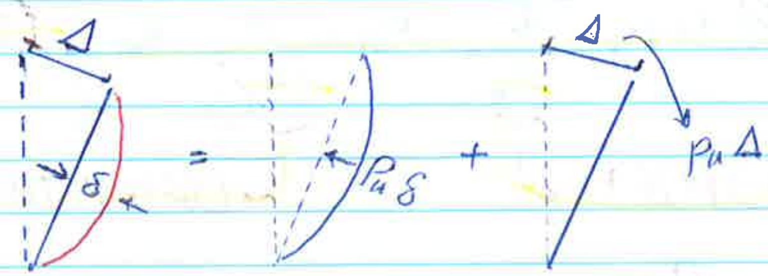
$\delta \rightarrow M_{nt}$

$\Delta \rightarrow M_{lt}$ and P_{lt}

$\Rightarrow M_u = B_1 M_{nt} + B_2 M_{lt}$

comming from DL+LL   comming from EQ





$$M_u = (M_{nt} + P_u \delta) + (M_{lt} + P_u \Delta)$$ $$P_u = P_{nt} + B_2 P_{lt}$$

$$= B_1 \underbrace{M_{nt}}_{DL + LL} + B_2 \underbrace{M_{lt}}_{\text{wind or earthquake}}$$

$$B_2 = \frac{1}{1 - \frac{P_{story}}{P_{e,story}}}$$

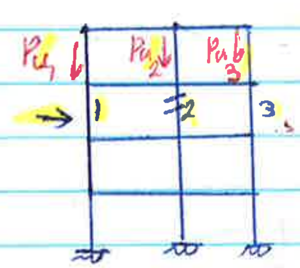
$P_{story} = \Sigma P_u$ for Entire floor

$P_{e_2,story} = \Sigma P_{e_2}$ for Entire floor

$$P_{e_1} = \frac{\pi^2 EI}{(KL)^2} \rightarrow K \approx 1.0 \text{ or braced frame}$$

$$P_{e_2} = \frac{\pi^2 EI}{(KL)^2} \rightarrow K \text{ for unbraced frame}$$

P 16.1_515 > 1.0



$$P_{story} = P_{u_1} + P_{u_2} + P_{u_3}$$

$$P_{e,story} = P_{e_{2,1}} + P_{e_{2,2}} + P_{e_{2,3}}$$

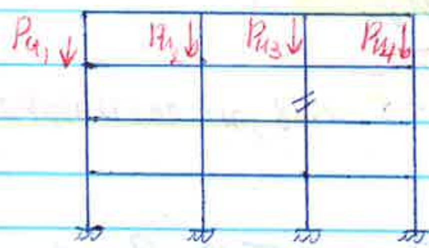
$$B_2 = \frac{1}{1 - \frac{P_{story}}{P_{e,story}}}$$

height of the floor → L ; horizontal load on the frame → H

$$\Sigma P_{e_2} = P_{e,\,Story} = \frac{HL}{\Delta_H} \times R_M = \frac{H}{\left(\frac{\Delta_H}{h}\right)} \times R_M \quad \text{Imp}$$

DRIFT → Δ_H ; DRIFT INDEX → $\frac{\Delta_H}{h}$

$$R_M = 1 - 0.15\,\frac{P_{mf}}{P_{story}}$$



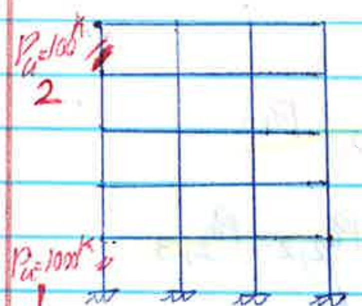
$P_{story} = P_{u_1} + P_{u_2} + P_{u_3} + P_{u_4}$

$P_{mf} = P_u$ @ columns design to take transverse load

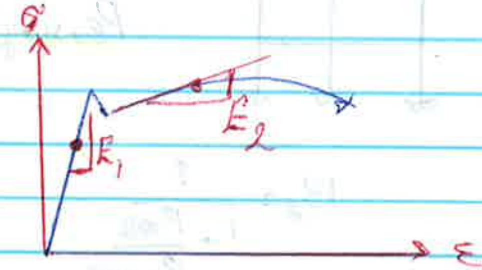
Code: $P_{mf} = \frac{1}{3}\Sigma P_u$

$$\frac{\Delta H}{L} = \begin{cases} 0.0015 - 0.003 & \text{(without load factor)} \\ 0.004 & \text{(factored load)} \end{cases}$$

Direct Analysis Method



$$P_{cr} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$



$\frac{P_u}{P_y} \leq 0.5 \rightarrow \tau_b = 1.0$

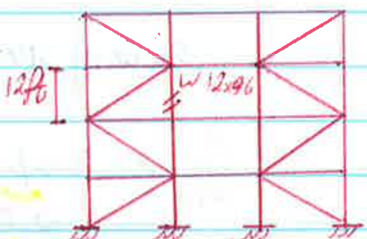
$\frac{P_u}{P_y} > 0.5 \rightarrow \tau_b = 4\left(\frac{P_u}{P_y}\right)\left(1 - \frac{P_u}{P_y}\right)$

$\Rightarrow EI \rightarrow 0.8\, \tau_b EI$

Exp 11-3:

$P_D = 175^k \quad P_L = 300^k$

$M_{Dx} = 60^{'k} \quad M_{Lx} = 60^{'k}$



single curvator

$\hookrightarrow C_m = 1.0$

nt case $\Rightarrow$

$P_u = 1.2\times175 + 1.6\times300 = 690^K = P_{nt}$

$M_{ux} = 1.2\times60 + 1.6\times60 = 168 = M_{ntx}$

$\hookrightarrow$ from now on we dont use it anymore because $M_{ux} = B_1 M_{ntx}$ So we only use "M_{ntx}"

$B_{1x} = \frac{C_m}{1 - \frac{P_u}{P_{e1}}}$ ① $\quad C_m = 0.6 - 0.4\left(-\frac{M_1}{M_2}\right) = 1.0$ ② $\quad (M_1 = 60,\ M_2 = 60)$

$P_{e1} = \frac{\pi^2 E I_x}{(KL)^2} = \frac{\pi^2 \times 29000 \times 833}{(1\times12\times12)^2} = 11498$ ③

① and ② and ③ $\Rightarrow B_{1x} = \frac{1}{1 - \frac{690}{11498}} = 1.064$

$\Rightarrow M_{ux} = B_{1x} M_{ntx} = 1.064 \times 168 = 178.8^{kft}$

$\frac{P_u}{\phi P_n} = \frac{690}{1080} = 0.639 > 0.2 \Rightarrow$ act more like a column

$\hookrightarrow$ p4-19 $\Rightarrow KL = 12' \Rightarrow \phi P_n = 1080$

$\Rightarrow \frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right) \leq 1.0$ and $L_p = 10.9 \quad L_r = 46.6$

table 3-2 $\quad L_b' = 12 \Rightarrow$ Zone 2

$\Rightarrow \phi M_{nx} = 1[551 - 5.7(12 - 10.9)] = 544.6$

$\Rightarrow 0.639 + \frac{8}{9}\left(\frac{178.8}{544.6}\right) = 0.9\ldots < 1$ ✓ design is ok

$p P_u + b_x M_{ux} + b_y M_{uy} \le 1.0 \qquad \frac{P_u}{\phi P_n} > 0.2$

$\frac{1}{2} p P_u + \frac{9}{8}\left(b_x M_{ux} + b_y M_{uy}\right) \le 1.0 \qquad \frac{P_u}{\phi P_n} < 0.2$

Solving problem using Table 6-1 and above formula:

$\frac{P_u}{\phi P_n} > 0.2 \Rightarrow p P_u + b_x M_{ux} + b_y M_{uy} \le 1.0$

$\Rightarrow 0.924\times10^{-3} \times 690 + 1.63\times10^{-3} \times 178.8 = 0.929 < 1.0$ ✓ design is ok

Exp 11-5: EI method : don't change "E"

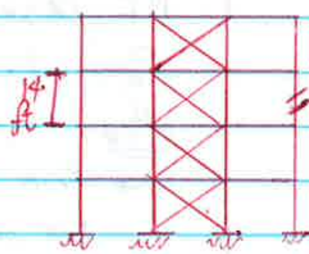
$P_u = 1.2 P_D + 1.6 P_L = 244^K$

$M_{Dx} = 60'^K$

$M_{Lx} = 80'^K$

$M_{Dy} = 40'^K$

$M_{Ly} = 60'^K$



$\Rightarrow M_{ntx} = 1.2 \times 60 + 1.6 \times 80 = 200'^K$

$M_{nty} = 1.2 \times 40 + 1.6 \times 60 = 144'^K$

single curvature

$C_m = 1.0$

$KL = 14' \Rightarrow \phi P_n = 1370$ (table 4-1)

$\frac{P_u}{\phi P_n} = \frac{244}{1370} = 0.178 < 0.2 \Rightarrow$ act like a beam ★

$M_{ux} = B_{1x} \times M_{ntx}$ and $M_{uy} = B_{1y} \times M_{nty}$

$B_{1x} = \frac{1}{1 - \frac{P_u}{P_{e1x}}} \qquad P_{e1x} = \frac{\pi^2 E I_x}{(\underbrace{K}_{\approx 1.0} L)^2} = \frac{\pi^2 \times 2900 \times 1380}{(1 \times 14 \times 12)^2} = 13995^K$

$\Rightarrow B_{1x} = \frac{1}{1 - \frac{244}{13995}} = 1.018$

hw:
11_12 / 11_11

due: next thursday

$B_{1y} = \frac{1}{1-\frac{P_u}{P_{ey}}}$ $P_{ey} = \frac{\pi^2 \times 29000 \times 495}{(1 \times 14 \times 12)^2} = 5020$

$\Rightarrow B_{1y} = \frac{1}{1-\frac{244}{5020}} = 1.051$

$\Rightarrow M_{ux} = 1.018 \times 200 = 203.6^{\,kft}$ $M_{uy} = 1.051 \times 144 = 151.3^{\,kft}$

table 6_1: $KL = 14$ $p = 0.73 \times 10^{-3}$ $\begin{cases} b_x = 1.13 \times 10^{-3} \\ b_y = 2.32 \times 10^{-3} \end{cases}$

$\Rightarrow$ Second formula: $\frac{1}{2} p P_u + \frac{9}{8}(b_x M_{ux} + b_y M_{uy})$

$= \frac{1}{2} \times 0.73 \times 10^{-3} \times 244 + \frac{9}{8}(1.13 \times 10^{-3} \times 203.6 + 2.32 \times 10^{-3} \times 151.3)$

$= 0.743 < 1$ ✓ ok but not good

Solve with direct method: from ★ will change:

$B_{1x} = \frac{C_{mx}}{1-\frac{P_u}{P_{e1x}}} = \frac{1}{1-\frac{244}{11,196}} = 1.022$

$P_{e1x} = \frac{\pi^2 \cdot 0.8\, \tau_b\, EI}{(K_1 L_x)^2} = \frac{\pi^2 \times 0.8 \times 1 \times 29000 \times 1380}{(1 \times 12 \times 14)^2} = 11,196$

$\frac{P_u}{P_y} = \frac{244}{(35.3)(50)} = 0.138 < 0.5 \Rightarrow \tau_b = 1.0$

$P_y \to A \cdot F_y$

$M_{ux} = B_{1x} M_{ntx} = 1.022 \times 200 = 204.4^{\,kft}$

$C_{my} = 0.6 - 0.4\left(-\frac{M_1}{M_2}\right) = 1.0 \Rightarrow B_{1y} = \frac{C_{my}}{1-\frac{P_u}{P_{ey}}}$

$$P_{e1y} = \frac{\pi^2\, 0.8\, \tau_b E I_y}{(K_1 L_y)^2} = \frac{\pi^2 \times 0.8 \times 1 \times 29000 \times 495}{(1 \times 14 \times 12)^2} = 4016$$

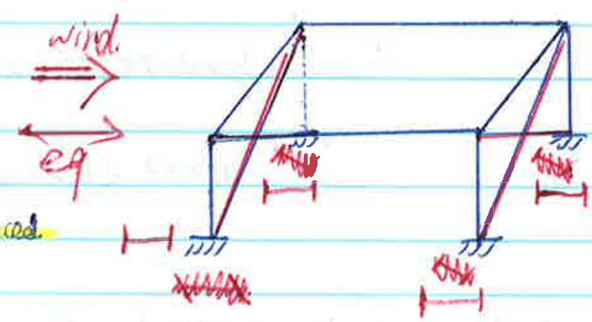
$$\Rightarrow B_{1y} = \frac{C_{my}}{1 - \frac{P_u}{P_{e1y}}} = \frac{1}{1 - \frac{244}{4016}} = 1.065 \Rightarrow M_{uy} = B_{1y} M_{nty} = 1.065 \times 144 = 153.3 \text{ k-ft}$$

AISC T 6-1 $\Rightarrow KL = 14$: $p = \quad \times 10^{-3}$, $b_x = 1.13 \times 10^{-3}$, $b_y = 2.33 \times 10^{-3}$

$$\frac{1}{2} p P_u + \frac{9}{8}(b_x M_{ux} + b_y M_{uy}) \leq 1.0 \Rightarrow 0.749 < 1 \checkmark \text{ ok but not good}$$

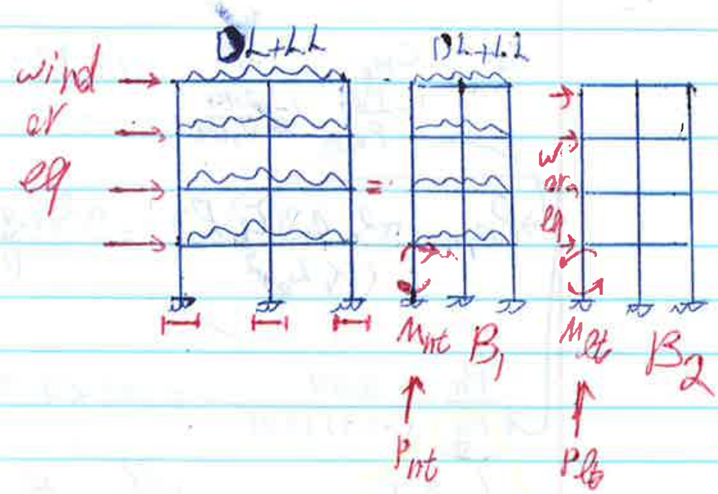
Unbraced Frame

Senario I

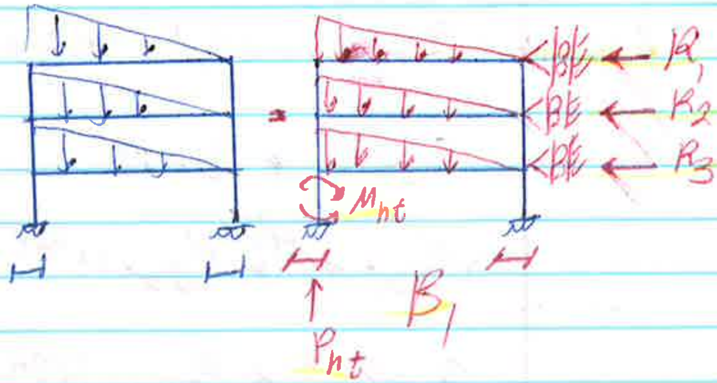


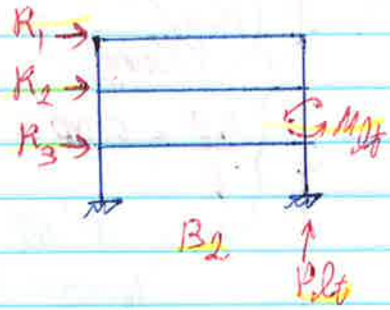
* weak axis is always braced

Senario II

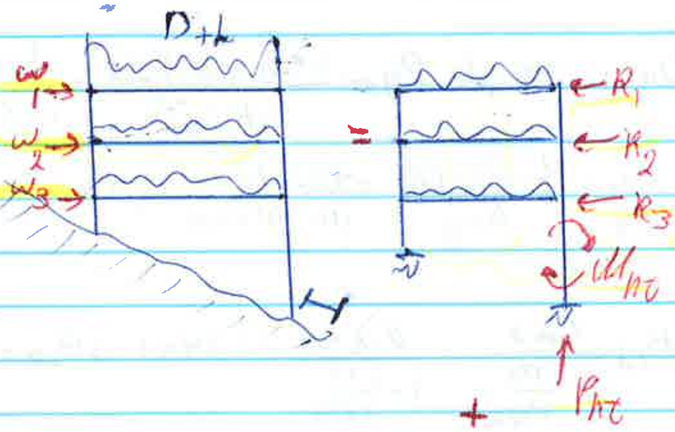


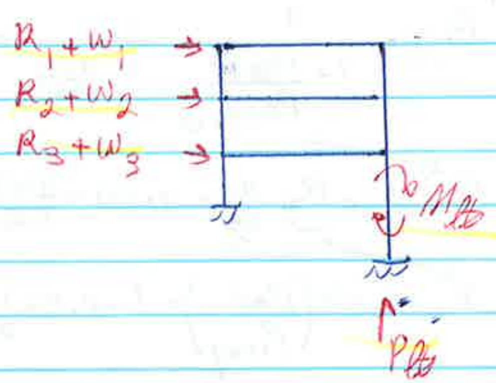
Senario III



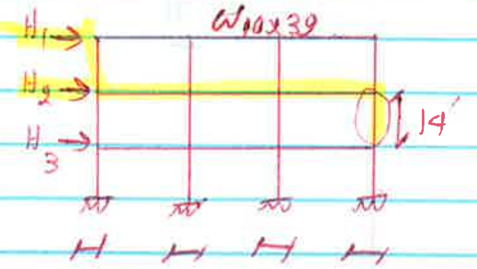


Senario IV





Exp 11.9: Direct Method. Analysis

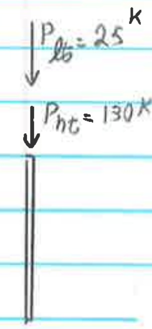


$P_{nt}=130^K$ * $P_{lt}=25^K$

$M_{ntx}=45^{kft}$ * $M_{ltx}=15^{kft}$

$C_m=0.85$ * $P_{story}=1604^K$

$P_{mf}/P_{story}=0.333$ * Story Shear $=H_1+H_2=33.4^K$



$\frac{\Delta f}{h}=0.0025$

⇒ 1/3 of all columns in this floor are designed for horizontal loads

Direct Method: $P_{e1x}=\frac{\pi^2\, 0.8\,\tau_b E I_x}{(K_1 L_x)^2}=\frac{\pi^2 \times 0.8\times1\times29000\times209}{(1\times12\times14)^2}=1696^K$

$\frac{P_u}{P_y}=\frac{P_{nt}+\overset{1}{B_2}P_{lt}}{AF_y}=\frac{130+25}{(11.5\,in^2)(50)}=0.27<0.5\Rightarrow \tau_b=1.0$

$B_{1x}=\frac{C_{mx}}{1-\frac{P_u}{P_{e1x}}}=\frac{0.85}{1-\frac{155}{1696}}=0.94<1\Rightarrow B_{1x}=1.0$

$B_{2x}=\frac{1}{1-\frac{P_{story}}{P_{e\,story,x}}}=\frac{1}{1-\frac{1604}{12692}}=1.15$

$P_{e\,story,x}=R_M\frac{H}{\Delta H/L}=0.95\times\frac{33.4}{0.0025}=12692$

$R_M=1-0.15\left(\frac{P_{mf}}{P_{story}}\right)=1-0.15\times\frac{1}{3}=0.95$

$P_u=P_{nt}+B_2P_{lt}=130+1.15(25)=158.8^K$

Homework 11-14 / 11-15
due thursday

$M_{ux} = B_{1x} M_{nt} + B_{2x} M_{lt} = 1(45) + 1.15(15) = 62.3^{kft}$

$KL \Rightarrow K_y L_y$ EI Method equibeirium. $K_y L_y = K_x L_x / (r_x / r_y)$

direct method $KL = 1.0 \times 14$

$\frac{P_u}{\phi P_n} = \frac{158.8}{306} = 0.52 > 0.2$ act like a column

$\hookrightarrow KL = 14$ T4-1: $\phi P_n = 306$

Table 6-1: $p = 3.27 \times 10^{-3}$ $b_x = 5.96 \times 10^{-3}$

$\Rightarrow 3.27 \times 10^{-3} \times (158.8) + 5.96 \times 10^{-3} (62.3) = 0.891 < 1$ ✓ OK

EI Method: $P_{e1x} = 2120$ $B_{1x} = 0.92$ $B_{2x} = 1.11$

$P_{estory, x} = 15865$

$P_{ux} = 157.8^K$ $M_{ux} = 61.7^{kft}$

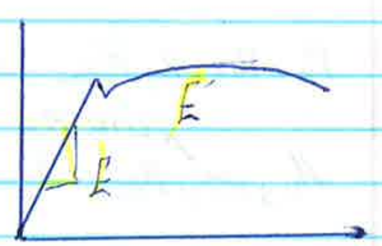
$\Rightarrow K_y L_y = 1 \times 14 = 14$ ✓ contraling one

Equivalent $K_y L_y = \frac{K_x L_x}{r_x / r_y} = \frac{1.2 \times 14}{2.16} = 18$

$\Rightarrow 0.884 < 1$ OK ✓

Direct: $\frac{\Delta H}{L} = 0.0025$ E'

EI: $\frac{\Delta H}{L} = 0.002$ E



Design of beam - column

shapes

Formula: $H_1 - 1(a)$ ✓

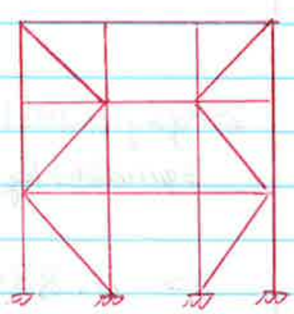
$$P_u, \quad M_{ux}, \quad M_{uy}$$

$$P_{nt}, \quad M_{ntx}, \quad M_{nty}$$
$$P_{lt}, \quad M_{ltx}, \quad M_{lty}$$
$$B_2 \quad B_{1x}, B_{2x} \quad B_{1y}, B_{2y}$$

$$P_{u,equi.} = P_u + m M_{ux} + m \overset{2.0}{u} M_{uy}$$

Exp II-13

$(KL)_x = (KL)_y = 12^{ft}$



$P_{nt} = 250^k \quad M_{ntx} = 180^{kft}$

$M_{nty} = 70^{'k}$

$P_{u,equi.} = P_u + m M_{ux} + mu M_{uy}$

$P_u = P_{nt} + B_2 \cancel{P_{lt}} = 250^k$

trial $B_1 = 1$

$M_{ux} = B_1 M_{ntx} + B_2 \cancel{M_{ltx}} = 180 \text{ kft}$

$M_{uy} = B_1 M_{nty} + B_2 \cancel{M_{lty}} = 70 \text{ kft}$

$W_{12\times72}$ and using the chart : $m=1.6$

$P_{u,equi.} = 250 + 1.6(180) + 1.6\times2(70) = 762\,K$

$KL = 12\,ft \xrightarrow{\text{Table 4-1}} W_{12\times72} \quad (\phi P_n = 806^K)$

$\frac{P_u}{\phi P_n} = \frac{762}{806} = 0.31 > 0.2$ Table 6-1 $\Rightarrow KL=12'$: $L_b=12'$, $K=1$ for Braced

$p = 1.26\times10^{-3}$, $b_x = 2.23\times10^{-3}$, $b_y = 4.82\times10^{-3}$

$(1.26\times10^{-3})(250) + 2.23\times10^{-3}(180) + 4.82\times10^{-3}(70) = 1.049 > 1$ N.G

$\Rightarrow$ Take 12×79 : $KL=12$: $p = 1.13\times10^{-3}$ $b_x = 2.02\times10^{-3}$ $b_y = 4.37\times10^{-3}$

$p\underbrace{P_u}_{250} + b_x\underbrace{M_{ux}}_{100} + b_x\underbrace{M_{uy}}_{70} = 0.952 < 1$ OK ✓

ANALYSIS by EI method:

$P_{ex} = \frac{\pi^2 EI_x}{(KL)_x^2} = \pi^2\times29000\times662/(1\times12\times12)^2 = 9138\,K$

$B_{1x} = \frac{C_{mx}}{1-\frac{P_u}{P_{ex}}} = \frac{0.85}{1-\frac{250}{9138}} = 0.87 < 1 \Rightarrow B_{1x} = 1$

$P_{ey} = \frac{\pi^2 EI_y}{(KL)_y^2} = \pi^2\times29000\times216/(1\times12\times12)^2 = 2981\,K$

$B_{1y} = \frac{C_{my}}{1-\frac{P_u}{P_{ey}}} = \frac{0.85}{1-\frac{250}{2981}} = 0.93 > 1 \Rightarrow B_{1y} = 1$

Table 6-1: $W_{12\times79}$: $KL=12$ ✓

H.W: 11-19

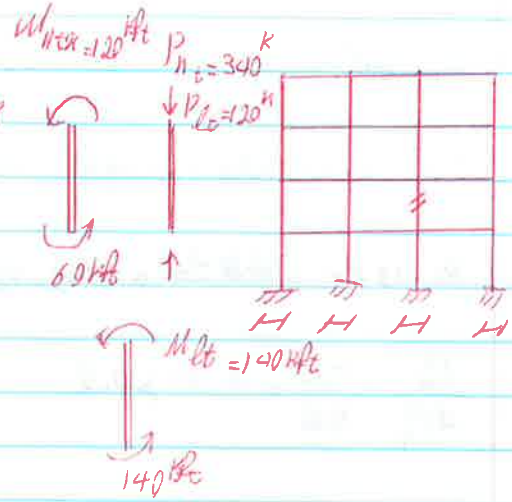
11-22

Exp11-H

$L_x = L_y = 12\,ft$ $\quad P_{story} = 2400\,k$

$K_x = 1.72$

$K_y = 1.0$ ✓✓ $\quad P_{e\,story} = 50{,}000\,k$



Assume $\beta_{1x} = 1$

$$B_{2x} = \frac{1}{1 - \frac{P_{story}}{P_{e,story}}} = \frac{1}{1 - \frac{2400}{50{,}000}} = 1.05$$

$$P_u = P_{nt} + B_2 P_{lt} = 340 + 1.05(120) = 466\,K$$

$$M_{ux} = B_1 M_{nt\,x} + B_2 M_{ltx} = 1(120) + (1.05)(140) = 267\,Kft$$

$$P_{u,equi.} = P_u + m M_{ux} = 466 + 1.6 \times 267 = 894$$

first try $\Rightarrow$ Table 4-1: $(KL)_y = 12\,ft \Rightarrow W_{12\times87}$ controlling one

$$equi.(KL)_y = (KL)_x / r_x/r_y = \frac{1.72\times12}{1.75} = 11.79 \rightarrow W_{12\times87}$$

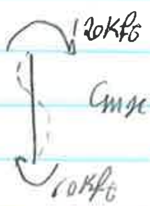
Table 6-1 and $KL = 12$, $L_b = 12$: $p = 1.02\times10^{-3}$, $b_x = 1.82\times10^{-3}$.

$$\frac{P_u}{\phi P_n} = \frac{466}{981} = 0.475 > 0.2$$

$$\Rightarrow 1.02\times10^{-3}(466) + 1.82\times10^{-3}(267) = 0.961 < 1 \text{ OK}✓$$

check $\beta_{1x} = 1.0$: $\quad P_{e1x} = \frac{\pi^2 E I_x}{(KL)^2} = \frac{\pi^2 \times 29000 \times 740}{(1\times12\times12)^2} = 10{,}214\,K$

Braced condition



$$C_{mx} = 0.6 - 0.4(M_1/M_2) = 0.6 - 0.4(+60/120) = 0.4$$

$$\Rightarrow \beta_{1x} = \frac{C_{mx}}{1 - P_u/P_{e1}} = \frac{0.4}{1 - 466/10214} = 0.42 < 1 \Rightarrow \beta_{1x} = 1✓$$

Bolt Connections

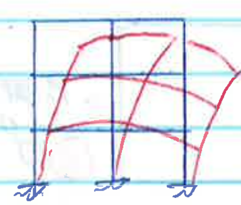
A 307: → Ordinary bolt or Common bolt
→ made out of A36 steel
→ Availible from $\frac{5}{8}$ inch to 1.5 inch
→ Only static load
+ Secondary members (like purlins)

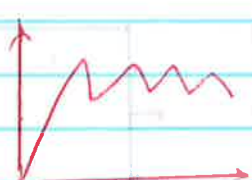
High Strength:
A 325 (Heat treated medium carbon) for Highrise building, large bridges
A 490 (Alloy steel) [Expensive] use for nuclear Reactor
Big dams
Air crafts

Bolt instalation:

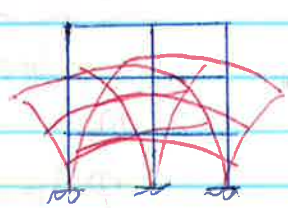
1. Snug tight not common — wrench applied a t to tighten
2. fully tensioned more common
3. slip critical

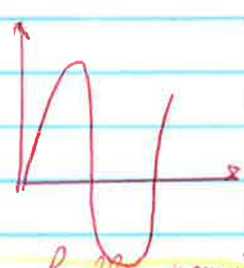
a Senario I





slip critical

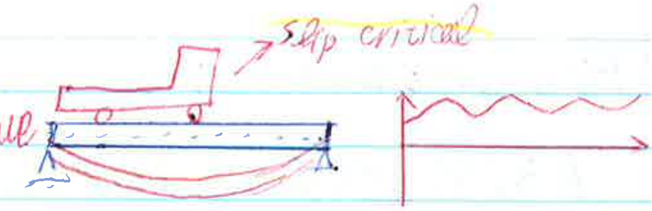




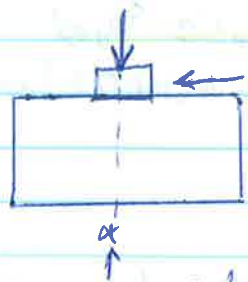
fully tension

3b Senario II :

Fatigue



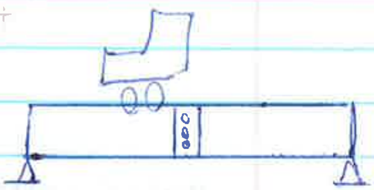
3. slip critical
→ NO SLIP OCCURES



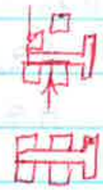
$F > \mu N$

$\phi R_n = \mu T_b$

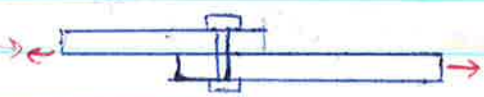
tension in the bolt

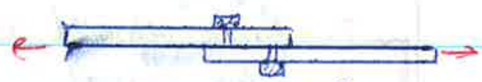


Code: $\phi R_n = \mu D_u h_f T_b n_s$ ★★★



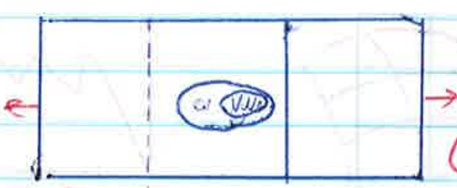
Bearing → strength failure
Slip critical → slip occures





Shearing off bolt

Ⓐ $\phi R_n = \phi F_v \times A_b$



Bearing failure of Plate

$Ⓑ_{I}$

$\phi R_n = \phi 1.2 \, l_c t F_u \le 2.4 d t F_u$

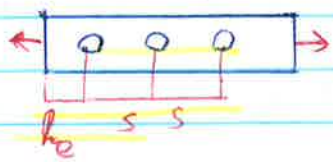
plate thickness → t ; bolt diameter → d

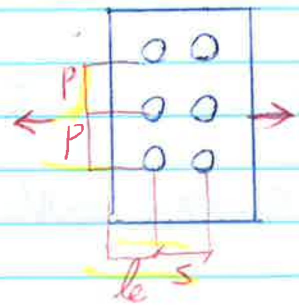
$Ⓑ_{II}$

$\phi R_n = \phi 1.0 \, l_c t F_u \le 2 d t F_u$

Bearing for seismic areas

$L_c = ?$





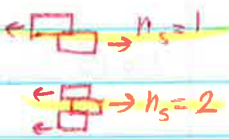
$$L_c = \min \begin{cases} L_e - \frac{hole}{2} \\ s - hole \end{cases}$$

punch → hole = d + 1/16" + 1/16" diameter

⇒ slot → hole = d + 1/16"

~~AAA~~ last page:

$$\phi R_n = \mu \cdot \overset{1.13}{D_u} \cdot \overset{1.0}{h_f} \cdot T_b \cdot n_s$$

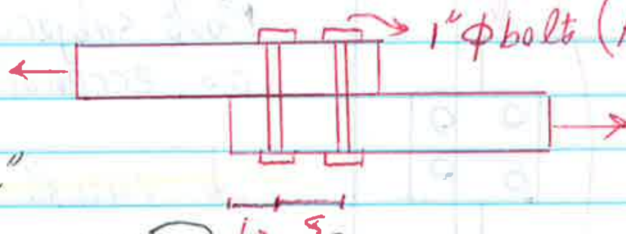


μ → 0.3 surface A

– B

– C

Exp 12-4:



1" ϕ bolt (A325_SC) (A325X)

$P_D = 27.5k$

$P_L = 40k$

$h = 1 + \frac{1}{8} = \frac{9}{8}$" ($\frac{1}{16} + \frac{1}{16} \times 1$")

$$\Rightarrow l_c = \min \begin{cases} 1.75 - \frac{1}{2}\left(\frac{9}{8}\right) = 1.187 \\ 3 - \frac{9}{8} = 1.875 \end{cases}$$

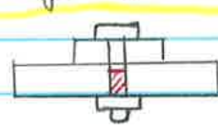
$L_e = 1.75$" ($\simeq 2$")

s (most of the time 3")

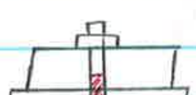
$Pu = 1.2(27.5) + 1.6(40) = 97K$

Hint: there are always minimum 2 bolt in one line

A325-X →



A325-N →



A325-SC (slip critical) → No slip

→ service load (w/o load factor)

← ultimate load (load factor)

A325_SC : $\phi R_n/_{bolt} = 1 \mu D_u h_f T_b n_s$

1" ϕ bolt and A325 ⇒ $T_b = 51$ ⇒ $= 1(0.3)(1.13)(1.0) \times 51 \times 1 = 17.29 K/_{bolt}$

$P_u = 97K \Rightarrow N = 97/17.29 = 5.61 \simeq 6$ bolt

A325_X :

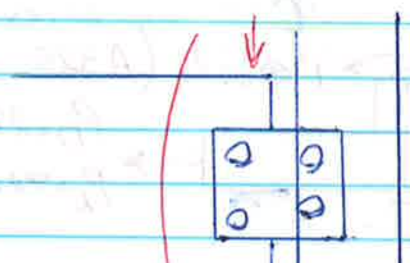
Bearing type ⇒ $\phi R_n/_{bolt} = \phi 1.5 L_c t F_u \leq \phi 2.4 d t F_u$

$= 0.75 \times 1.5 \times 1.187 \times 5/8 \times 65 \leq 0.75 \times 2.4 \times 1 \times 5/8 \times 65$

$\Rightarrow \underline{54.25} \leq 73.1 K$

Bolt Shearing ⇒ $\phi R_n/_{bolt} = 0.75(68 ksi)(\pi d^2) = \underline{40.03}$

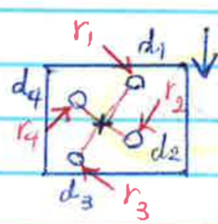
$\Rightarrow \phi R_n/_{bolt} = 40.03^K \quad N = \frac{97}{40.03} = 2.43 \simeq 3$ bolt



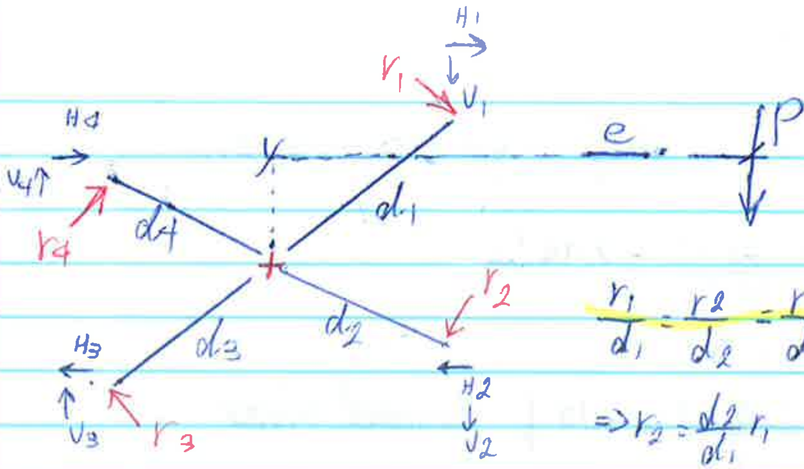
Bolt subject to eccentric Shear

→ ↓ vertical shear

→ ↻ Torsional shear



1. bolts (x/y direction)
2. draw center between c/g to bolt



$$\frac{r_1}{d_1}=\frac{r_2}{d_2}=\frac{r_3}{d_3}=\frac{r_4}{d_4}$$

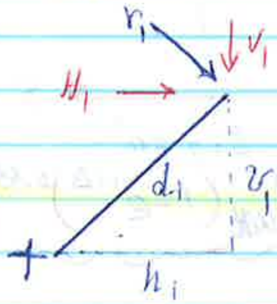
$$\Rightarrow r_2=\frac{d_2}{d_1}r_1 \qquad r_3=\frac{d_3}{d_1}r_1 \qquad r_4=\frac{d_4}{d_1}r_1$$

$$P.e=r_1d_1+r_2d_2+r_3d_3+r_4d_4$$

$$M=r_1\frac{d_1^2}{d_1}+r_2\frac{d_2^2}{d_1}+r_3\frac{d_3^2}{d_1}+r_4\frac{d_4^2}{d_1}$$

$$=\frac{r_1}{d_1}(d_1^2+d_2^2+d_3^2+d_4^2)=\frac{r_1}{d_1}\Sigma d^2$$

$$r_1=\frac{Md_1}{\Sigma d^2} \qquad r_2=\frac{Md_2}{\Sigma d^2} \qquad r_3=\frac{Md_3}{\Sigma d^2}$$



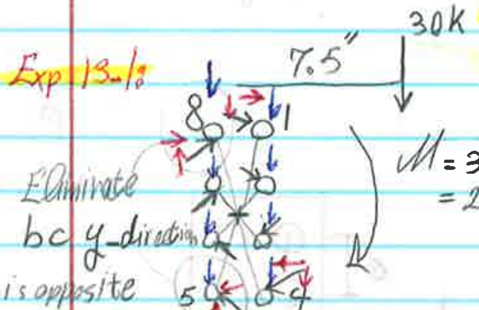
$$\frac{r_1}{d_1}=\frac{H_1}{v}=\frac{V_1}{d_1}$$

$$H_1=\frac{r_1}{d_1}v_1=\frac{M\cancel{d_1}}{\Sigma d^2}\frac{v_1}{\cancel{d_1}}=\frac{Mv_1}{\Sigma d^2}$$

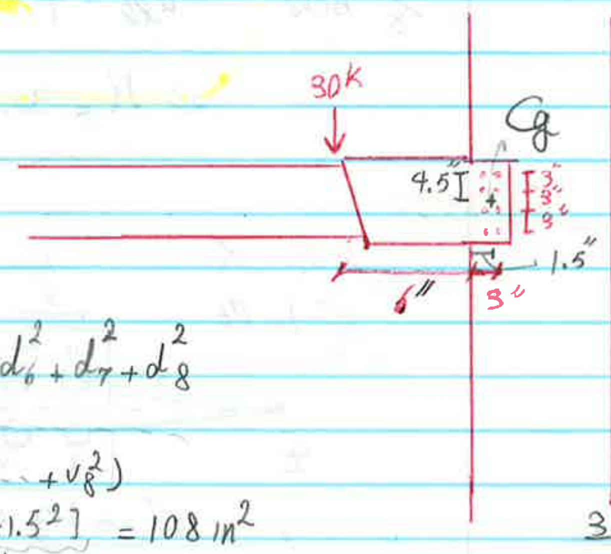
$$V_1=\frac{H_1\Sigma d^2}{M}$$

$$\begin{cases}H_1=\dfrac{MV_1}{\Sigma d^2}\\[2ex] V_1=\dfrac{M\,h_1}{\Sigma d^2}\end{cases}$$

Exp 13-1:



$$M=30\times7.5$$
$$=225K''$$



$$\Sigma d^2=d_1^2+d_2^2+d_3^2+d_4^2+d_5^2+d_6^2+d_7^2+d_8^2$$

$$=h_1^2+v_1^2+h_2^2+v_2^2+\cdots$$

$$=(h_1^2+h_2^2+\cdots+h_8^2)+(v_1^2+v_2^2+\cdots+v_8^2)$$

$$=\underbrace{[4(4.5^2)+4(1.5^2)]}_{v}+\underbrace{[8\times1.5^2]}_{h}=108\ in^2$$

HW 12-20
12 - 33
13 - 4

=> only 1 and 4 are critical and also symmetric

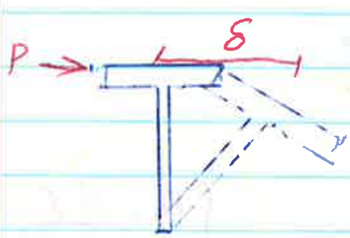
$$H_1 = \frac{M v_1}{\Sigma d^2} = \frac{225 \times 4.5}{108} = 9.38 \rightarrow$$

$$V_1 = \frac{M h_1}{\Sigma d^2} = \frac{225 \times 1.5}{108} = 3.13 \downarrow$$ Torsional shear

Vertical shear $= P/8 = \frac{30}{8} = 3.75\,k \downarrow$ =>

$$R_u = \sqrt{(3.13 + 3.75)^2 + 9.38^2} = 11.63\,K$$

*Plastic Methods:



$$R = R_{ult}\left(1 - e^{-10\Delta}\right)^{0.55}$$

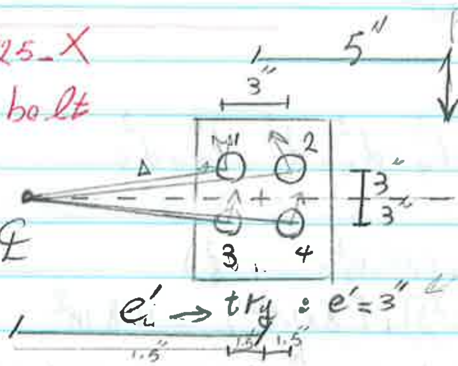
A325_x
7/8" bolt

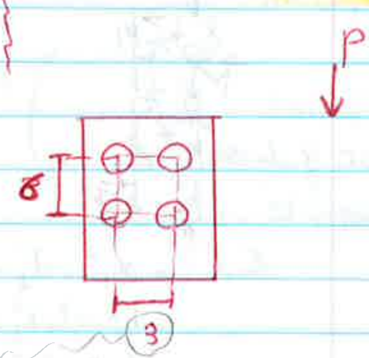
$$R_{ult} = \phi F_v A_b = 0.75 \times 68 \left[\frac{\pi}{4}\left(\frac{7}{8}\right)^2\right] = 30.6$$

$$\Rightarrow R = 30.6\left(1 - e^{-10\Delta}\right)^{0.55}$$

Exp 13_2:

A325_X
7/8" bolt





first try

$e'_u \rightarrow$ try : $e' = 3"$

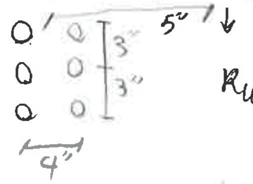
HW: $R_{ult} = 50.5$

try: $e' = 3$

ans: $e' = 2.06''$ ✓

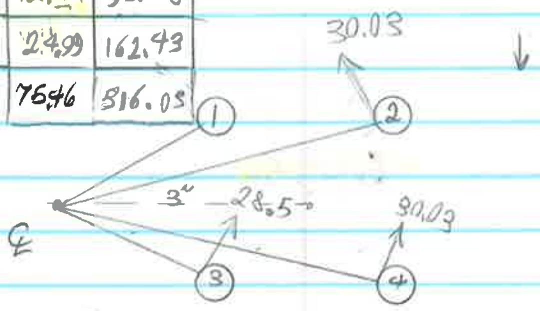
13-10 Elastic Method

13-12 AISC Table (Plastic)



	Δ	h	v	$d=\sqrt{h^2+v^2}$	R	R_v	$R \times d$ (M)
1		1.5	3	3.3541	28.5	12.74	95.48
2	0.34	4.5	3	5.4083	30.03	24.99	162.43
3		1.5	3	3.3541	28.5	12.74	95.58
4	0.34	4.5	3	5.4083	30.03	24.99	162.43
					Σ	75.46	516.03

$$\Delta = \frac{\overset{\Delta_4}{0.34}}{\underset{d_4}{5.4083}} \times \overset{d_3}{3.354} = \boxed{0.211}$$



$M = Pd \Rightarrow$ $P = \frac{516.03}{3+5} = 64.5 \neq 75.46 \Rightarrow e' \times \rightarrow$ because its bigger

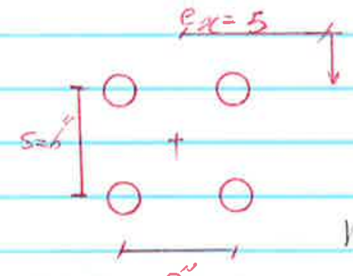
$3 = e'$, 5

5''

we have to use smaller $e' \Rightarrow e' = 2.5$

$e' = 2.4''$

Exp



base on the chart in the book

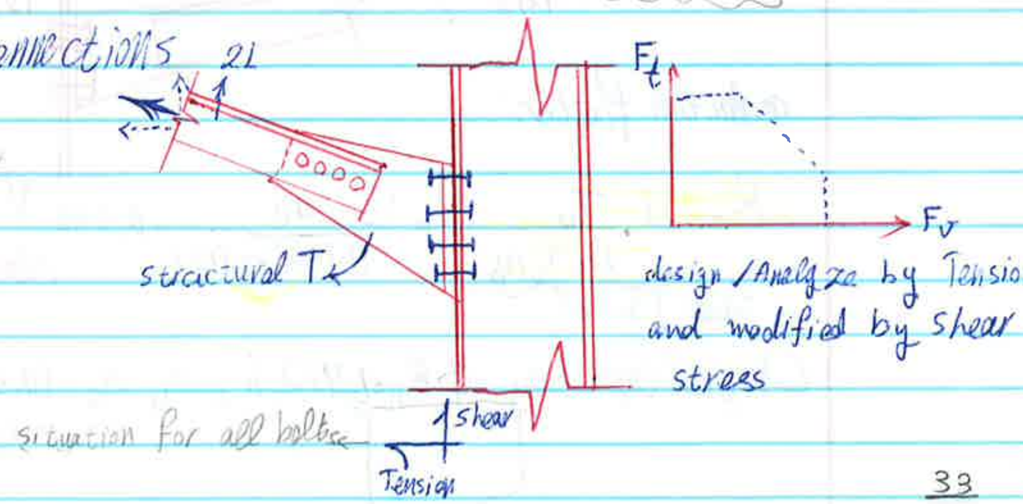
$n = 2$ $C = 2.19$ table

$r_n = F_v \times A_b = 68\left(\frac{\pi}{4}\left(\frac{7}{8}\right)^2\right) = 40.8$

$R_n = C r_n = 2.1 \times 40.8 = 91.4$

$\phi R_n = 0.75 \times 91.4 = 68.6^k = P$

Braeing connections



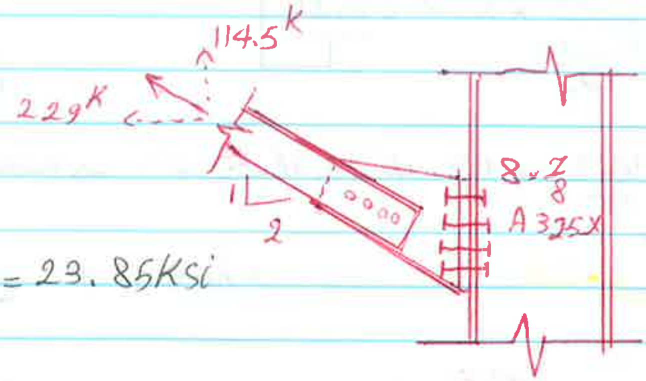
design / Analyze by Tension and modified by shear stress

hw: 13-17

so instead of $F_{nt} \rightarrow \phi$ shear
we use:

$$F'_{nt} = 1.3\, F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v < F_{nt}$$

Exp13-5:



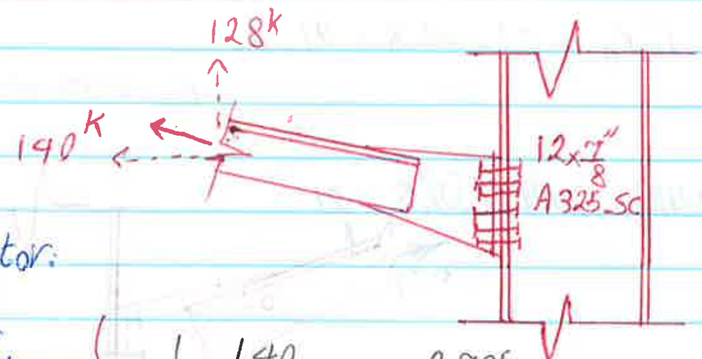
$$f_v = \frac{114.5^K}{8(0.6\,in^2)} = 23.85\,Ksi$$

$$f_t = \frac{229^K}{8(0.6)} = 47.7\,Ksi$$

chart: $F_{nt} \rightarrow 90$; $F_{nv} = 68\,ksi$

$$F'_{nt} = 1.3(90)^{nt} - \frac{90}{0.75 \times 68} \times 23.85 = 74.9 < 90 \checkmark OK$$

Exp



reduction factor:

$$R_{sc} = 1 - \frac{T_u}{D_u T_b n_b} = 1 - \frac{140}{(1.13)(39)(12)} = 0.735$$

Table J3-1

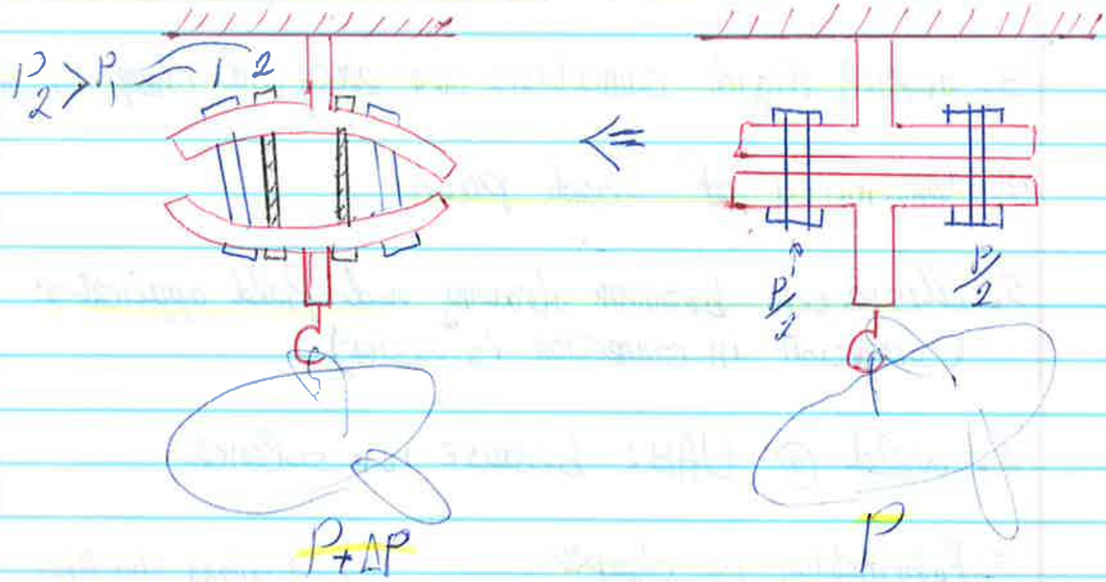
class 13

Original capacity: $\phi R_n = 1 \mu D_u h_f T_b n_s = (0.5)(1.13)(1.0)(39)(1)$
$= 22.03$

$\phi R_n' = 0.795 \times 22.03 = 16.2^K / \text{bolt} \quad \times 12 = 194.4^K$

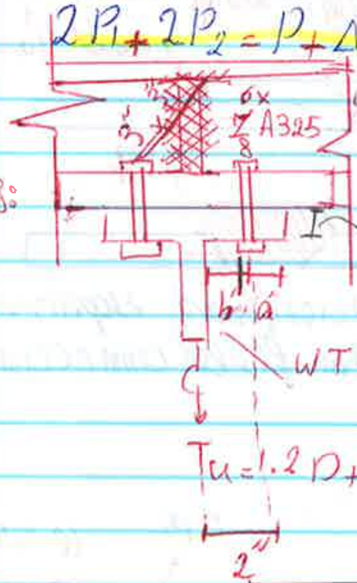
$\Rightarrow 194.4 > 128 \ \checkmark \ OK$

PRYING ACTION



$2P_1 + 2P_2 = P + \Delta P$

Exp13.8:



$t_{min} = \sqrt{\dfrac{4.44\,T b'}{p F_u}} \leq t_f$

p → Spacing of the bolt in other direction

I → $t_f = 0.565$ (table)

$b' = 2 - \dfrac{0.345}{2} - \dfrac{7/8}{2} = 1.39''$

$T_u = 1.2D + 1.6L = 100^K$

$T = \dfrac{T_u}{6} = \dfrac{100}{6} = 16.67^K / \text{bolt}$

A36 → $F_u = 58$

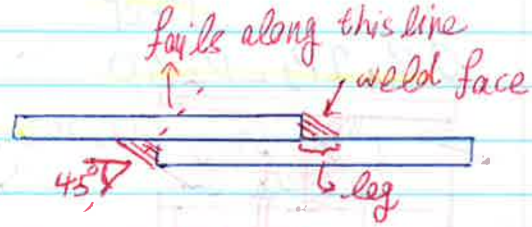
2"

$\Rightarrow t_{min} = \sqrt{\dfrac{4.44 \times 16.67}{3 \times 58}} = 0.769 > t_f = 0.565$ ✗ N.G

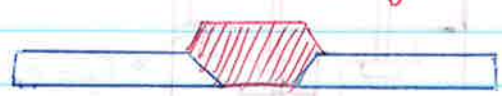
Weld Connection

1. Over 15% saving $ (nuts, bolts, washers...)
2. Its pretty easy for complex shapes
3. making Rigid connections are easy and cheap or moment connection
4. Continuity of load path.
5. Mismatch between drawing and field application (correction in connection is easier)
6. Weld @ UAB: because its silence
7. Fabrication is cheaper.

weld → Fillet



weld → Groove



50 – 100% more expensive than fillet connection

$$\text{weld capacity/inch} = (0.707 \times \text{size}) \times \overset{0.75}{\phi}\, \underbrace{\overset{(0.6\times 70)}{F_w}}_{\text{Shear capacity}}$$

most of the times use rod E 70ksi

length of the well (maximum length side)

$\beta = 1.2 - 0.002\left(\frac{\ell}{w}\right) \leq 1.0$ } $\ell/w \leq 100 \Rightarrow$ don't need "β"

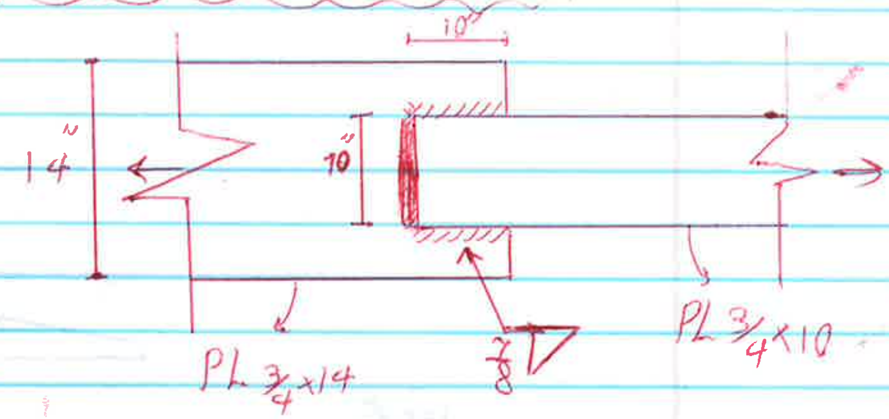
the minimum size and maximum size of fillet weld → P16.1_111

maximum size { $< \frac{1}{4}''$ max weld $= t_{min}$

$\geq \frac{1}{4}''$ max weld $= t_{min} - \frac{1}{16}''$

Exp 14_2:

$\phi R_n = ?$



weld capacity/inch $= (0.707 \times \frac{7}{8}) \times (0.75)(0.6 \times 70)$

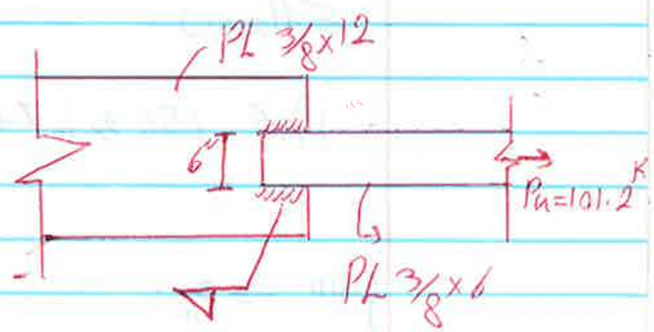
$\phi R_n =$ weld capacity/in $\times 30 = 194.9^K$

$\beta = 1.2 - 0.002(\ell/w) = 1.2 - 0.002(10/\frac{7}{8}) = 1.2 - 0.002 \times 22.88 = 1.18 > 1$

$\Rightarrow \beta = 1.0$

$\Rightarrow \phi R_n = 194.9^K$ ✓

Exp 14_3:



min thickness → $\frac{3}{16}$ (table)

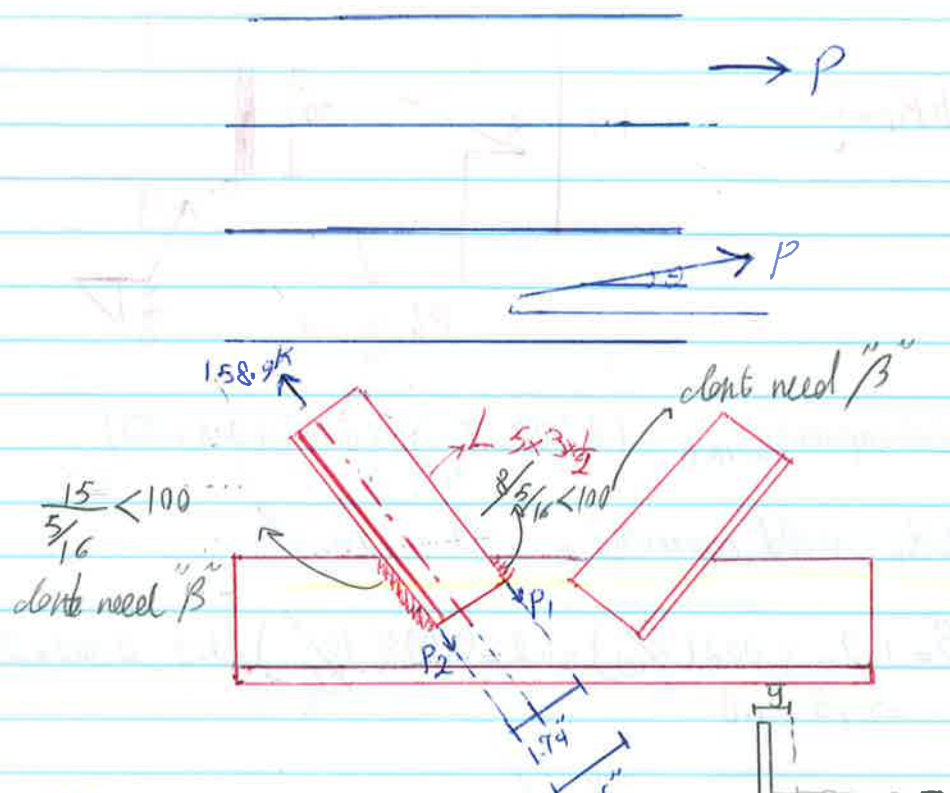
max thickness → $t_{min} - \frac{1}{16} = \frac{3}{8} - \frac{1}{16} = \frac{5}{16}$

weld capacity/inch $= (0.707 \times \frac{5}{16}) \times 0.75 \times (0.6 \times 70) = 9.28\ ^{K}/_{in}$

weld length : $L = \frac{101.2}{9.28} = 14.59'' \simeq 15''$

Symmetric : 7.5" on top and 7.5" in the bottom

$l/w = 7.5/\frac{5}{16} = 24 < 100$ ∴ dont need to calculate "β"



$\Sigma M_0 = 0$

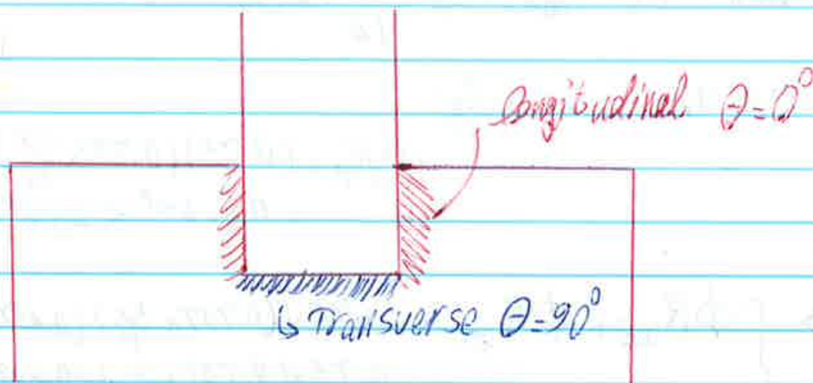
$P_1 \times 5 - 158.9 \times 1.74 = 0 \Rightarrow P_1 = 55.3^{K}$

$P_2 = 158.9 - 55.3 = 103.6^{K}$

$\begin{cases} \min \rightarrow \frac{3}{16} \\ \max \rightarrow \frac{1}{2} - \frac{1}{16} = \frac{7}{16} \end{cases}$ ⇒ we wonna be in the middle $\frac{5}{16}$

$$\text{weld capacity}/\text{in} = (0.707 \times 5/16)(0.75)(0.6 \times 70) = 6.96\ K/\text{in}$$

$$P_1 \rightarrow L_1 = \frac{55.3}{6.96} = 7.95'' \simeq 8''$$



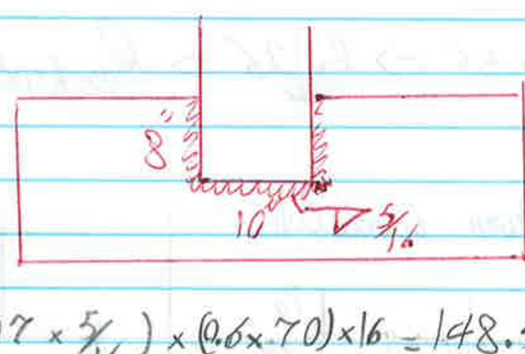
$$(1 + 0.5 \sin^{1.5}\theta)\, A_w$$

Case I (AISC) : $R_n = \underbrace{R_{nwl}}_{\text{without } \theta}^{\text{longitudinal}} + \overbrace{R_{nwt}}^{\text{transverse}}$

Case II : $R_n = 0.85 R_{nwl} + 1.5 R_{nwt}$

} choose larger one

Exp 14-5:

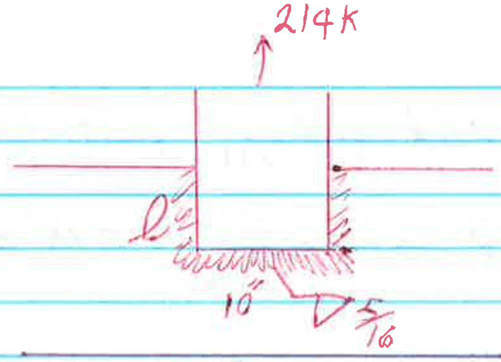


$$R_{nwl} = (0.707 \times \tfrac{5}{16}) \times (0.6 \times 70) \times 16 = 148.5$$

$$R_{nwt} = (0.707 \times \tfrac{5}{16}) \times (0.6 \times 70) \times 10 = 92.8$$

$$\Rightarrow \begin{cases} R_n = R_{nwl} + R_{nwt} = 148.5 + 92.8 = 241.3 \\ R_n = 0.85 \times 148.5 + 1.5 \times 92.8 = 265.4 \end{cases} \Rightarrow R_n = 265.4 \checkmark$$

Exp: design problem



min. Vs max. → $\frac{5}{16}$ is choosen ✓

with Transverse only:

because its only Transverse

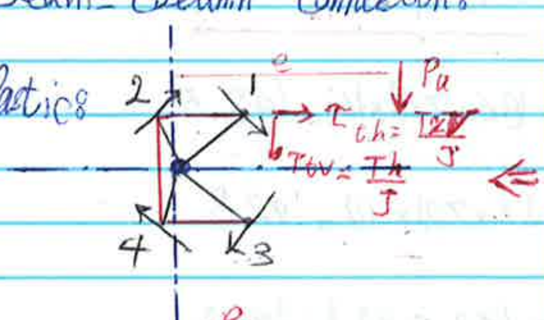
$$\phi R_n = (0.75)(0.707 \times \tfrac{5}{16})(0.6\times70)(10)(1.5)$$
$$= 62.64^k < 214^k \times \text{ Not enough}$$

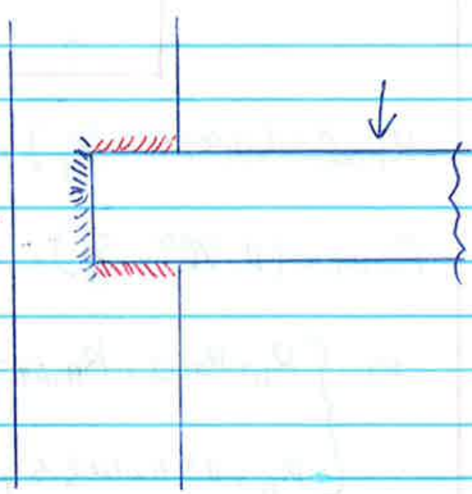
$$\Rightarrow \begin{cases} \phi R_{wl} + \phi R_{wt} = 0.75\times(0.707\times\tfrac{5}{16})(0.6\times70)(l) + 0.75\times(0.707\times\tfrac{5}{16})(0.6\times70)(10) \\ \quad = 6.96\,l + 41.76 = 214^k \Rightarrow l = 24.75'' \\ 0.85\,\phi R_{wl} + 1.5\,\phi R_{wt} = 0.85(0.75)(0.707\times\tfrac{5}{16})(0.6\times70)(l) + 1.5(0.75)(0.707\times\tfrac{5}{16})(0.6\times70)(10) \\ \quad = 5.916\,l + 62.636 = 214^k \Rightarrow l = 25.56'' \end{cases}$$

$$\Rightarrow l_{total} = 25.56 \Rightarrow l_{total} = 26'' \Rightarrow l_{eachside} = 13''$$

*** Beam_Column Connection:**

elastic:





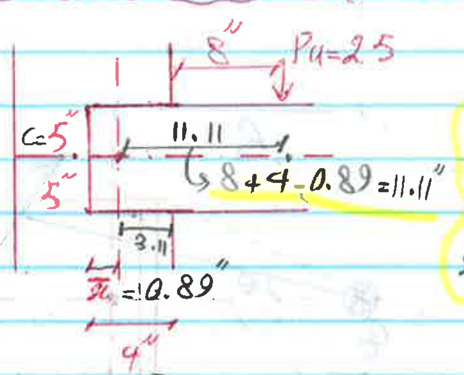
Transverse: P_u → $P_u . e$

1. Direct shear $\tau_D = \dfrac{P_u}{A_w}$

2. Torsional shear $\tau_t = \dfrac{Tc}{J}$; $J = I_x + I_y$

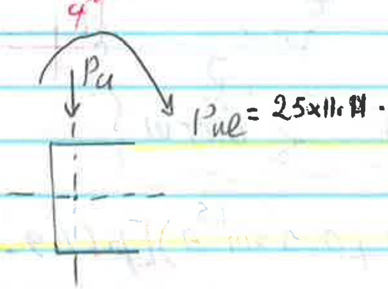
$$\tau = \sqrt{\tau_D^2 + \tau_t^2} \;\left\{ = \sqrt{\left(\tau_D + \frac{Th}{J}\right)^2 + \left(\frac{Tv}{J}\right)^2} \right\}$$

Exp 14.8:



$A = (4'' \times 1)\,2 + 10'' \times 1 = 18\ in^2$

$\bar{x} = \dfrac{[(4\times1)\times2]\times2}{18} = 0.89''$



$I_{x-x} = 283.3''$ and $I_{y-y} = 28.4$

$J = 283.3 + 28.4 = 311.7$

$\tau_D = \dfrac{P_u}{A_w} = \dfrac{25}{18} = 1.39\ K/in$

$T_{th,1} = \dfrac{Tv}{J} = \dfrac{(25\times11.11)\times5}{311.7} = 4.46\ K/in$

$T_{tv,1} = \dfrac{Th}{J} = \dfrac{(25\times11.11)\times3.11}{311.7} = 2.77\ K/in$

$\Rightarrow \tau = \sqrt{(2.77+1.39)^2 + (4.46)^2} = 6.10\ K/in$

$\Rightarrow 6.1 = (0.75)(0.707\,w)(0.6\times70) \Rightarrow w = 0.274''$

$\simeq 5/16''$

✱ Solve by Table 8-8:

$al = 11.11 \Rightarrow a = \dfrac{11.11}{l=10} \Rightarrow a = 1.11$

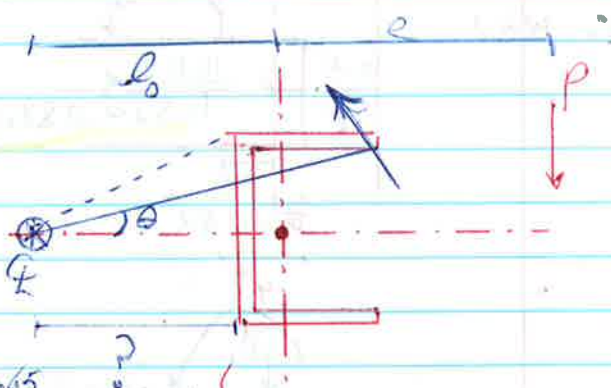
HW: 14-14 / 14-31

$KL = 4'' \Rightarrow K = 4/\ell = 0.4$

Table: Coefficient $= 1.31$

$$D = \frac{25}{0.75 \times 1.31 \times 1 \times 10} = 2.54/_{16} \simeq 3/_{16}$$ ✓

Table 8.3: $C_1 = 1$ (E70)



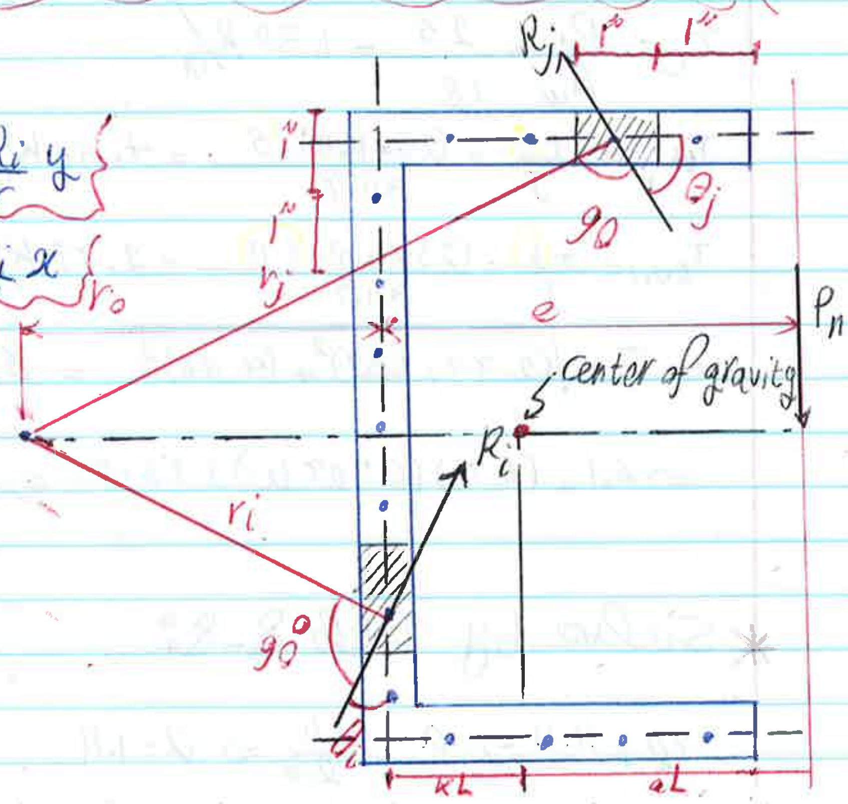
∗ Plastic method:

$$\Delta_{max} = 1.087\, w (\theta + 6)^{-0.65} < 0.17 w$$

$$R = 0.6\, F_{E70xx} A_w (1 + 0.5 \sin^{1.5}\theta)[p(1.9 - 0.9\phi)]^{0.3}$$

$R_x = \frac{R_i}{r} y$

$R_y = \frac{R_i}{r} x$



step1) Split weld into 1" section

step2) Calculate the θ for each element

$$R_i = 0.6\, F_{Exx} t_e (1 + 0.5 \sin^{1.5}\theta)\left[\frac{\Delta_i}{\Delta_m}\left(1.9 - 0.9\frac{\Delta_i}{\Delta_m}\right)\right]^{0.3}$$

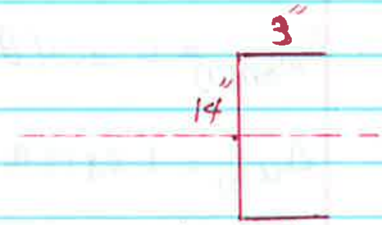
$$= 0.6 \times 70 \times (0.707 \times \tfrac{1}{4})(1 + 0.5 \sin^{1.5}\theta)\left[\frac{\Delta_i}{\Delta_m}\left(1.9 - 0.9\frac{\Delta_i}{\Delta_m}\right)\right]^{0.3}$$

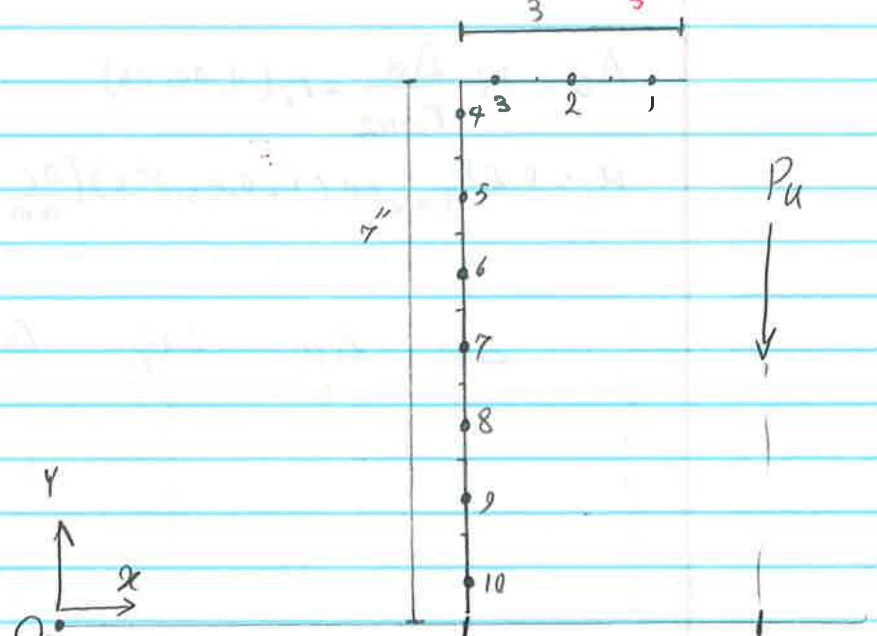
Δ_i = deformation of element $= r_i \dfrac{\Delta_u}{r_{crit}}$

$\Delta_m = 0.209($ θ

exp:

Because of symmetry we solve for $\frac{1}{2}$





try: $e = 6"$

$r_y = \sqrt{8.5^2 + y^2}$

$\sqrt{x^2+y^2}$ $\tan^{-1}\frac{x_i}{y_i}$

Horizontal	length	x	y	r_i	(radians) θ_j	(degree) θ_j
1	1	8.5	7	11.011	0.882	50.5
2	1	7.5	7	10.259	0.820	47
3	1	6.5	7	9.552	0.748	42.9
vertical						
4	1	6	6.5	8.846	0.825	47.3
5	1	6	5.5	8.139	0.742	42.5
6	1	6	4.5	7.5	0.644	36.9
7	1	6	3.5	6.946	0.528	30.3
8	1	6	2.5	6.5	0.395	22.6
9	1	6	1.5	6.185	0.245	14
Point ① 10	1	6	0.5	6.021	0.083	4.8

$$\Delta_{m_{max(1)}} = 0.209(\theta+2)^{-0.32} a = 0.209(50.5+2)^{0.32}(0.25) = 0.0143$$

$$\Delta_{u_{(1)}} = 1.087(\theta+6)^{-0.65} a = 1.087(50.5+6)^{0.65}(0.25) = 0.01973$$

$$\Delta_i = r_i \frac{\Delta u}{r_{crit}} = r_i(0.00179)$$

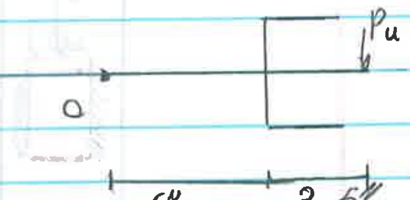
$$R_i = 0.6 F_{Exx} te(1+0.5\sin^{1.5}\theta)\left[\frac{\Delta_i}{\Delta m}\left(1.9-0.9\frac{\Delta_i}{\Delta m}\right)\right]^{0.3}$$

	Δm	Δu	$\Delta u/r_i$	Δi	$\Delta i/\Delta m$	R_i
1						
2						
3						
4						
5						
6						

HW: repeat this exp / 14_17

	R_i	r_i	x	y	$(R_i)_x$	$(R_i)_y$	$R_i r_i$
1							
2							
3							
					sum ~	~	~

$\Sigma M_{O} = 0 \Rightarrow P_u(6+3.5) = \Sigma R_i r_i$
$= 721.54 \times 2$



$P_n = 2(721.94)/(6+3.5) = 152K$

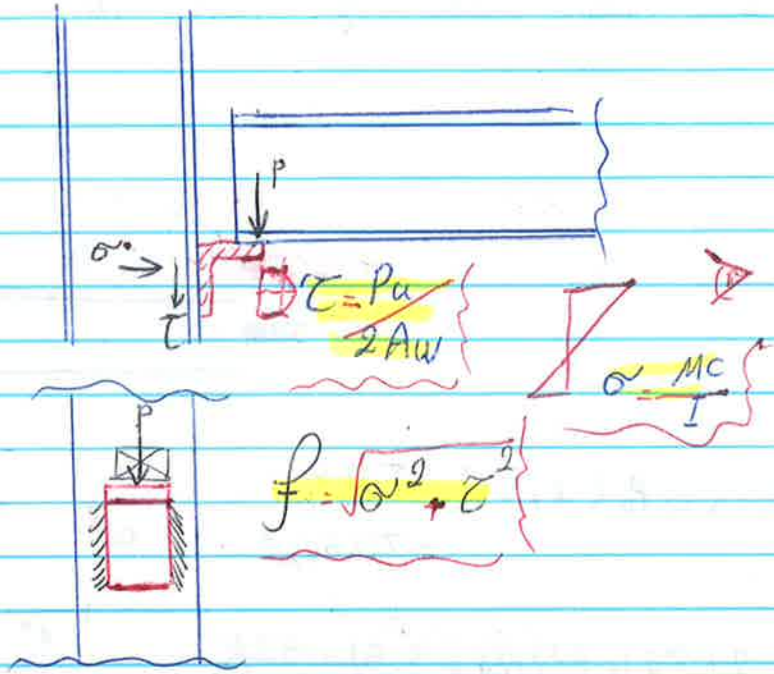
$\Rightarrow$ Try Not good $\Rightarrow$ Use bigger $e \Rightarrow e = 6.5''$ and solve again

Solve by chart : (Table 8-8)

$al = 3.5'' \Rightarrow a = \frac{3.5}{14} = 0.25 \quad KL = 3'' \Rightarrow K = \frac{3}{14} = 0.214$

$\Rightarrow C = 2.573$ (Table)

$P_u = D_{min} \times \phi \times C \times C_1 \times l = \left(\frac{0.25}{1/16}\right) \times 0.75 \times 2.573 \times 1 \times 14 = 111^K$

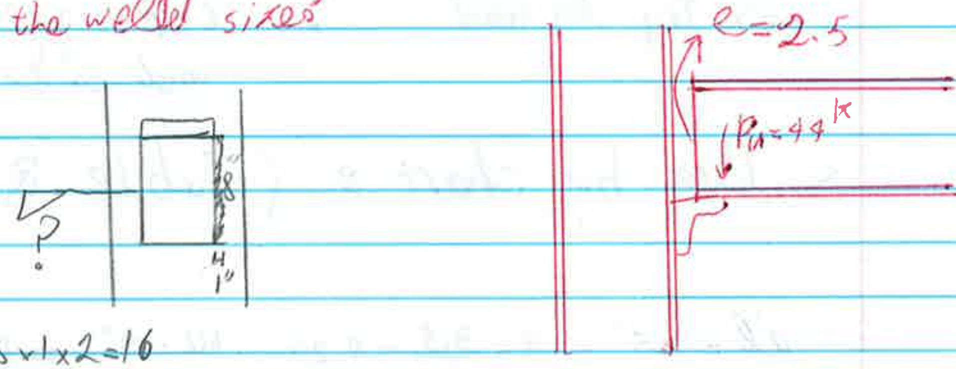


$\tau = \frac{P_u}{2A_w}$

$\sigma = \frac{Mc}{I}$

$f = \sqrt{\sigma^2 + \tau^2}$

Exp: whats the weld size?
14-11



$2A_w = 8 \times 1 \times 2 = 16$

$I = 2 \times \frac{bh^3}{12} = 2 \times \frac{1 \times 8^3}{12} = 85.3$

$\Rightarrow \tau = \frac{P_u}{2A_w} = \frac{44}{16} = 2.75\ K/in$

$\sigma = \frac{Mc}{I} = \frac{(44 \times 2.5)(4)}{85.3} = 5.16\ K/in$

$\Rightarrow f = \sqrt{(5.16)^2 + (2.75)^2} = 5.85\ K/in$

weld capacity/in $= (0.707 \times \beta)(0.7 \times 70)(0.75) = 5.85$

$\Rightarrow \beta = 0.263 = 5/16$ "

solve by table: $al = 2.5 \Rightarrow a = 0.31$

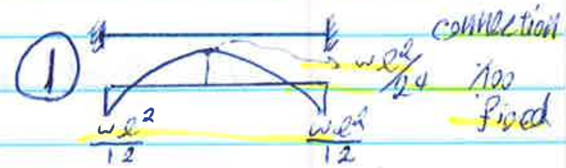
$Kl = 0 \Rightarrow K = \checkmark$

$P_{min} = \frac{44}{0.75 \times 1 \times 3.09 \times 8} = 2.3 \sim 3$ → ics out of plane condition

↳ $\frac{3}{16}$

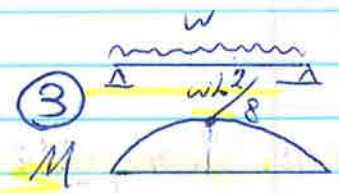
* Building Connections (chapter 15)

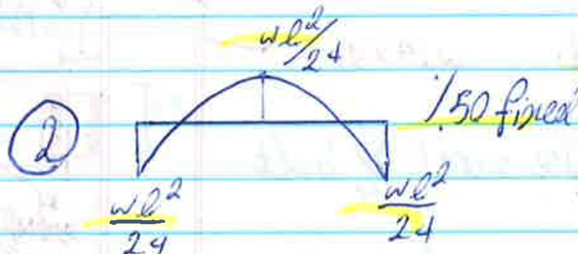
1. FR _ Fully Restrained _ Rigid connection _ Take moment _ Fixed

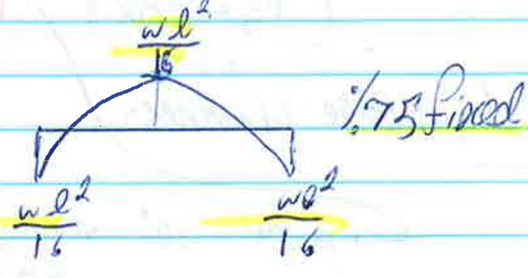


2. PR _ Partially Restrained _

3. simple connection _ shear connection



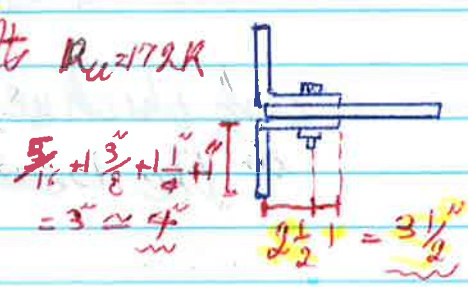




Hint: in ① we always use $\frac{3}{4}$" φ A325_N @ 3" unless posted

Exp 15.)

① bolt $R_u = 172K$



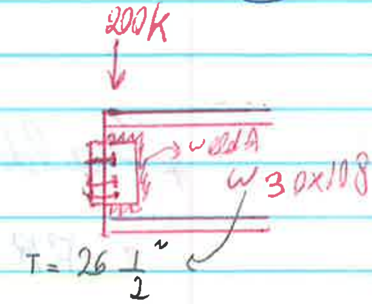
whole plate   table 10-1 (No. of bolts = 6)

$\Rightarrow \{4 \times 3\frac{1}{2} \times \frac{5}{16}\}$ ⇒ < web of beam)

(check the length of angle)

in welding { welding of angle to beam is always called weld A
{ ~ ~ ~ ~ column ~ ~ weld B

exp8 weld A and bolt to column

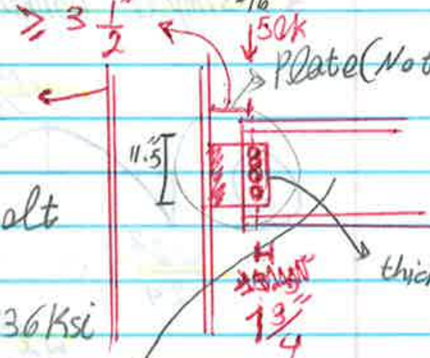


use T 10-3 & only left part cause we only have weld A

use lower size of weld => $L = 20$ in and $\phi R_n = 223K$

=> $L = 20"$, $\frac{3}{16}"$

exp. Shear plate/shear tab
15-14



we go T 10-9a & make sure { 3/4" bolt
{ $F_y = 36$ Ksi

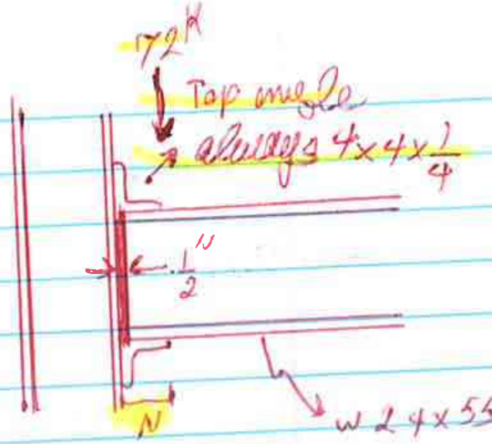
=> $L = 11\frac{1}{2}"$ and $\frac{1}{4}"$ plate thickness

$3\frac{1}{2} + 1\frac{3}{4} = 5\frac{1}{4}$

Beam web strength T 10.1 uncoped
$351 \, K/in$ => $351 \times 0.38 = 133.4$
$133.4K > 50K$ ✓
beam can handle the shear force

web thickness of the beam

Exp 15.5 Seated beam Connection

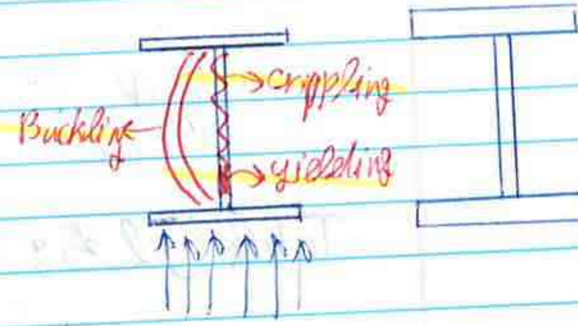


for yielding:

$\phi R_n = \phi R_{n_1} + N(\phi R_{N_2})$

web crippling

$\phi R_n = \phi R_3 + N(\phi R_4)$



use T9-4: pick $\phi R_1 / \phi R_2 / \phi R_3 / \phi R_4$

$\Rightarrow$ yielding $\Rightarrow 72 = 49.6 + N(19.8) \Rightarrow N = 1.13''$

crippling $\Rightarrow 72 = 63.7 + N(5.61) \Rightarrow N = 1.48''$

$\} > 1\frac{1}{2}''$ ✓

$\downarrow$ AISC: must be $3\frac{1}{2}''$ or $4''$

$\Rightarrow$ So we use $N = 3\frac{1}{2}''$ ✓

use T 10-6 (unstiffened):

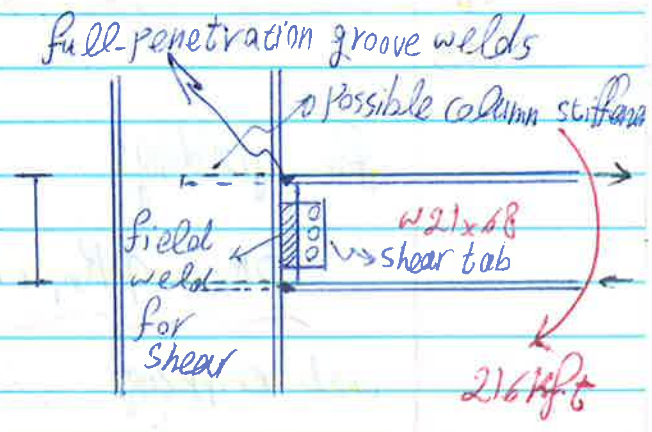
base on $1\frac{1}{2}'' \Rightarrow$ angle thickness $= 3/4$

$\Rightarrow$ Angle $= 8 \times 3\frac{1}{2} \times \frac{3}{4}$

Moment_Resistant connection

Exp 15-6:

effective: 21.1 _ 0.685



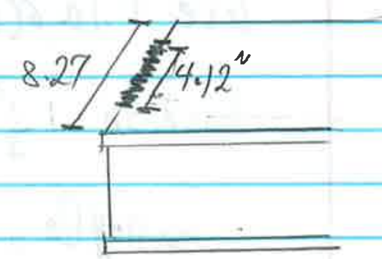
$$T = C = \frac{216 \times 12}{21.1 - 0.685} = 127^K$$

Table J2.5: Strength is controled by the base Metal

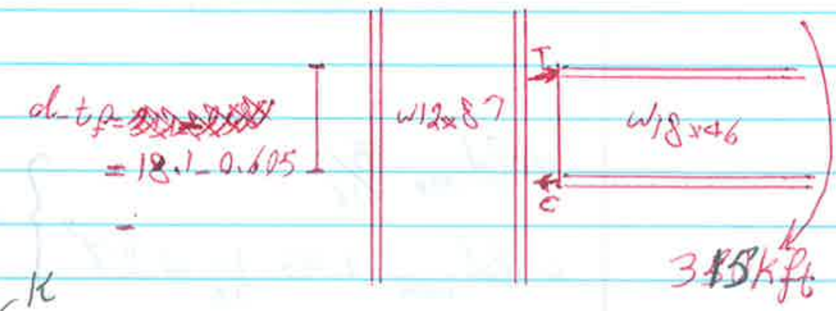
$$\left(\underbrace{F_{y_{metal}} = 50ksi}_{controlling} \text{ and } F_{y_{weld}} = 70ksi \right)$$

$$A_w = \frac{127^K}{\phi F_y} = \frac{127}{0.9 \times 50} = 2.82 \, in^2$$

$$\left.\begin{array}{l} \Rightarrow b_w \times 0.685 = 2.82 \Rightarrow b_w = 4.12'' \\ W_{21\times68}: b_f = 8.27'' \end{array}\right\} \Rightarrow b_w < b_f \quad \checkmark$$



Exp use som of 15.7 (modified.)



$$T = C = \frac{315 \times 12}{18.1 - 0.605} = 216^{k}$$

$$A_w = \frac{216}{\phi F_y} = \frac{216}{0.9 \times 59} = 4.8'' \quad \text{and} \quad b_w = \frac{4.8}{0.605} = 8'' \left. \right\} b_w > b_f \; X$$

$W18x46$: $b_f = 6.06''$

$$b_f = \frac{4.2}{t_f} \Rightarrow t_f = \frac{\overset{A_w}{4.8}}{\underbrace{6.06}_{b_f\, W18x46}} \approx 1.338'' = 1\frac{1}{3}''$$

$$b_w = \frac{4.8}{1.338} = 3.6 < 5$$

$\Rightarrow$ Plates $1\frac{1}{3}'' \times 5'' \times 13''$

*check the plates to see if they can handle $T = C = 216^{K}$:

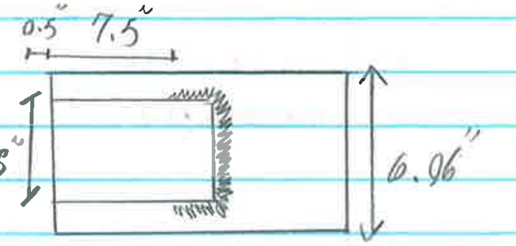
Tension: $\phi F_y A_g = (5'' \times 1.338)(0.9(50)(\quad) = 216^{K}$ ✓

compression: $C = \phi F_{cr} \times A = 32(5 \times 1.338) = \ldots > 216^{K}$ ✓

$$\left. \begin{array}{l} KL/r = \dfrac{0.65 \times 8}{0.384''} = 13.5 \\ r = \dfrac{t}{\sqrt{12}} = \dfrac{1.33}{\sqrt{12}} = 0.384'' \end{array} \right\} \text{Table 4-22} \Rightarrow \phi F_{critical} = 32 \text{ ksi}$$

{15-16
{15-17

check the filled weld:



$weld_{min} = \frac{5}{16}"$

$weld_{max} = 1.33 - \frac{1}{16} = 1.27"$

} we have to use the maximum size

weld capacity $= (0.707 \times 1.27)(0.6 \times 70)\, 0.85\, L > 216^K \Rightarrow L = 5.7"$

① Flange local bending of the column:

Exp 5-7

$R_n = 6.25 \times t_f^2 \times F_{yf} = 6.25 \times 0.81^2 \times 30 = 205$

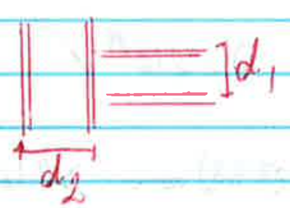
$\phi = 0.9$ (LRFD) $\phi R_n = 0.9 \times 205 = 184.5^K < 216^K$ ✗

column will bend

② web of the column yielding:

$d_1 > d_2$: $R_n = (5K + N)\, F_{yw}\, t_w = (5 \times 1.41 + 6.0a)(50)(0.33) = 337.6^K$

$\phi = 1$ (LRFD) $\Rightarrow \phi R_n = 1 \times 337.6 > 216^K$ ✓



③ web crippling of the column

$d_{beam} - t_{f\,beam} > \frac{d_{beam}}{2}$: $R_n = 0.8\, t_w^2 \left(1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{E\, F_{yw}\, t_f}{t_w}}$

$R_n = 0.8 \times 0.515^2 \left[1 + 3\left(\frac{8.06}{12.5}\right)\left(\frac{0.505}{0.81}\right)^{1.5}\right]\sqrt{\frac{\cdots \times 0.81}{0.515}} = 417.5$

$\phi = 0.75$ (LRFD) $\} \quad > 216^K$ ✓

W12x87 Column fail at bending : 2 solution

a. W12x96

b. use stiffener ✓

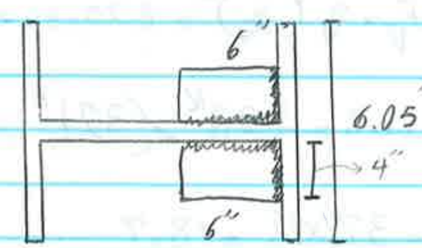
$$\left.\begin{array}{l} \Rightarrow R_w = 216^K \\ \phi R_n = 184.5 \end{array}\right\} \text{min Area} = \frac{216 - 184.5}{50} = 0.63\ in^2$$

code: Min width: $\left.\underbrace{\frac{\overset{\text{beam}}{b_f}}{3} - \frac{\overset{\text{column}}{t_w}}{2}}\right\} = \frac{6.06}{13} - \frac{0.515}{2} = 1.76''$

$b \times t = \text{Area} = 0.63 \Rightarrow t = \frac{0.63}{1.76} = 0.358 \approx 3/8$

$b = \frac{0.63}{3/8} \simeq 1.68'' < 4'' \Rightarrow b = 4''$ (min must be 4")

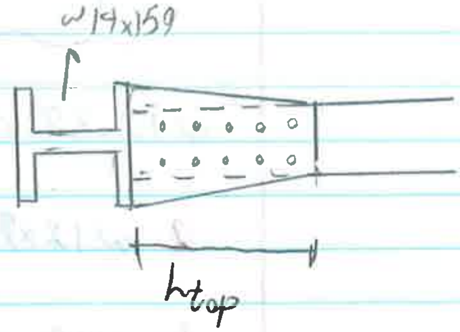
length: $d/2 - t_f = \frac{12.5}{2} - 0.81 = 5.46 \simeq 6''$



*Connection that make plastic hinge in high loads:

review 16.1-64 important

Exp: 3/4 bolts



$F_y = 50\,ksi$

$F_u = 65\,ksi$

$\frac{F_y}{F_u} = \frac{65}{50} = 0.83 > 0.8$

hole diameter $= \frac{3}{4} + \frac{1}{8} = \frac{7}{8}''$

beam flang

$F_u A_{fn} = 65(6.45 - 2 \times \frac{7}{8} \times 0.645) = 346\,K$ ①

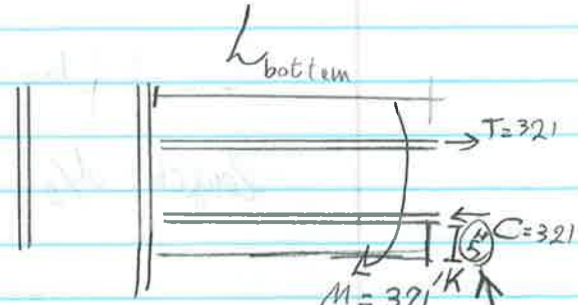
$Y_t F_y A_{fg} = 1.1 \times 50 \times 6.45 = 355\,K$ ②

$\Rightarrow$ ① < ②

$\Rightarrow M_n = \frac{F_u A_{fn}}{A_{fg}} S_x = (\quad)(12) = 412$

(50, 8.32, 92.1, 645)

$\Rightarrow \phi M_n = 0.9 \times 412 = 371\,Kft$



$T = C = \frac{371}{13.89} \times 12 = 321^K$

$\frac{3}{4}$ bolts: $\phi F_n = 0.75 F_v \times \frac{\pi}{4}(\frac{3}{4})^2 = 0.75 \times 90 \times 0.442 = 29.8^K/bolt$

$\Rightarrow$ max bolt = 8 : $8 \times 29.8 = 238^K < 321^K$ X Not good

Required Moment $= \frac{371 \times 12}{238} = 18.7$

(T and C must < 238)

-13.89

Extra arms $= 4.8'' \simeq 5$

$\Rightarrow T_{new} = C_{new} = \frac{371\times12}{13.89+5} = 236^K$

$\Rightarrow$ each bolt: $\frac{236^K}{8\,b} = 29.5^K < 29.8^K$ ✓

* extra beam lengths (WT5×24.5)

$\phi V_n = \overbrace{\phi(0.6F_y)(t_w\times L) \Rightarrow 236 = 1}^{code}(0.6\times50)\times0.34\times L \Rightarrow L_{bottom} = 23.1 \simeq 24''$

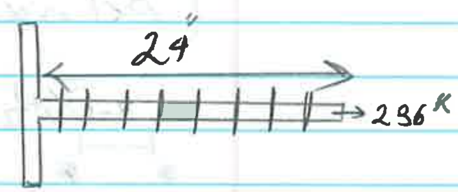
* bolts on top of the beam connected to T:

$\phi R_n = 0.75\times F_v\times 0.4418 = 0.75\times48\times\overbrace{0.4418}^{A_{bolt}} = 15.9^K$

$\Rightarrow$ # of bolts $= 236/15.9 = 14.8 \approx 16 \Rightarrow$ 8 @ each row and 3" spacing

$\Rightarrow L_{top} = (7\times3)+(2\times1)+\overset{extra}{(1)} = 24''$

* T_{stub} design (top T):



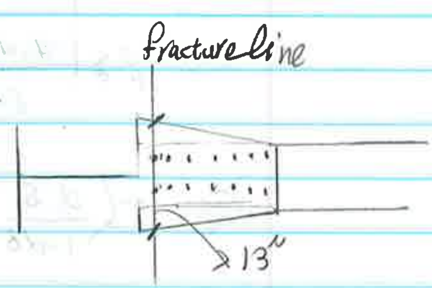
yielding $\Rightarrow T_u = \phi F_y A_g$

$A_g = \frac{236}{0.9\times50} = 5.2\,in^2 \Rightarrow 13\,t_w = 5.2 \Rightarrow t_w = 0.4''$

Fracture $\Rightarrow T_u = \phi F_u A_n$

$A_n = \frac{236}{0.75\times65} = 4.8\,in^2$

$\Rightarrow (13 - 2\times\frac{7}{8})\,t_w = 4.8 \Rightarrow t_w = 0.43''$



HW: repeat whole thing with column: $W_{16\times100}$

→ design top T without any prying Action:

$$t_f \geq \sqrt{\frac{4.44\,Tb'}{\omega F_u(1+\alpha\delta)}} \geq 0.85'' \quad \left(T = \frac{236}{2} = 118^{K} \,/\, \alpha = 0 \text{ (w/o prying)} \quad b' = b - \frac{d_b}{2} = \frac{\text{gage}}{2} - \frac{t_w}{2} - \frac{d_b}{2} = \frac{4}{2} - \frac{t_w}{2} - \frac{3/4}{2} \approx 1.25' \text{ (Assume)} \right.$$

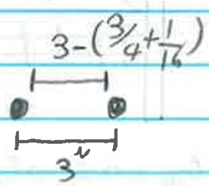
$$\Rightarrow t_f \geq \frac{4.44(118)(1.25)}{14\times65\times1} \geq 0.85'' \quad \checkmark \text{ ok}$$

→ design top T with prying Action:

$$t_f \geq \sqrt{\frac{4.44\,Tb'}{\omega F_u(1+\alpha\delta)}} \quad \begin{cases} \alpha = ? \\ \delta = ? \end{cases}$$

$$\beta = \left(\frac{\overset{\text{bolt}}{B}}{\underset{\text{bolt capacity}}{T}} - 1\right)\frac{a'}{\underset{\approx 1.25''}{b'}} = \left(\frac{29.8}{29.5} - 1\right)(1.25) = 0.013$$

$$\begin{cases} \beta > 1 \Rightarrow \alpha = 1 \\ \beta < 1 \Rightarrow \alpha = \min\left(1, \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right)\right) = 0.018 \end{cases}$$



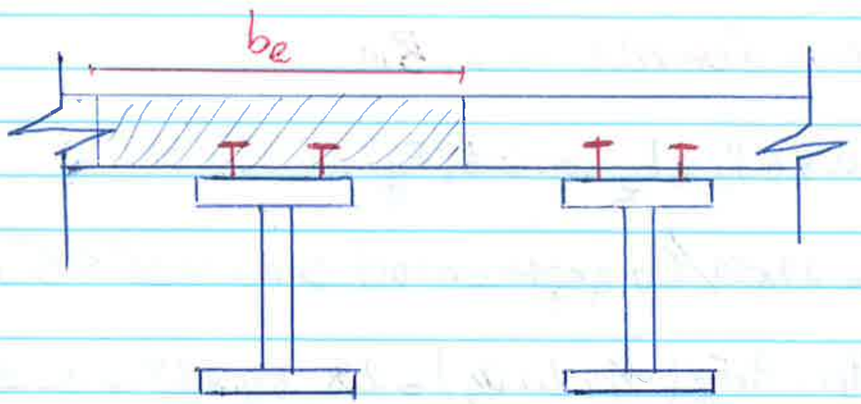
$$\delta = \frac{3 - \left(\frac{3}{4} + \frac{1}{16}\right)}{3} = 0.73$$

$$\Rightarrow t_f \geq \frac{4.44\times118\times1.25}{65(1+0.018\times0.73)} = 0.84'' \geq 0.84'' \text{ OK} \checkmark$$

$$Q = T\left(\frac{\alpha\delta}{1+\alpha\delta}\right)\left(\frac{b'}{a'}\right) = 118\left(\frac{0.018\times0.73}{1+0.018\times0.73}\right)\left(\frac{1.375}{1.875}\right) = 0.01\,T$$

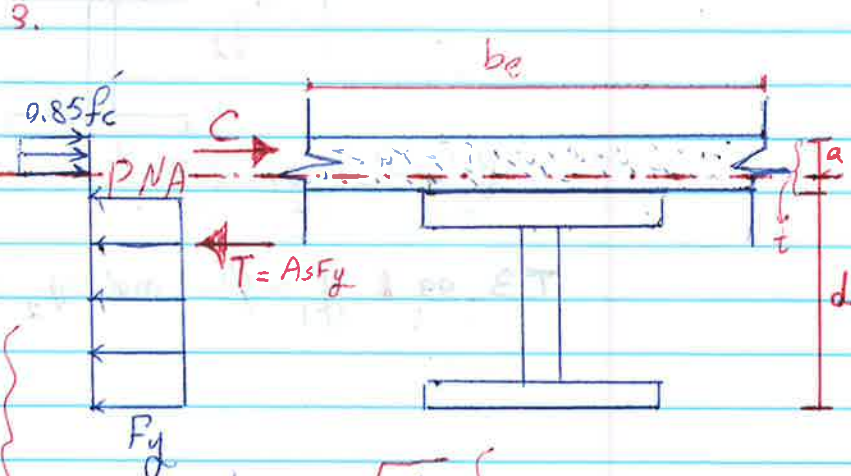
$$\begin{cases} a' = a + d_b/2 = \frac{b_f - g}{2} + \frac{d_b}{2} = \frac{7-4}{2} + \frac{3/4}{2} = 1.875'' \\ b' = b - d_b/2 = g/2 - \frac{t_w}{2} - \frac{d_b}{2} = \frac{4}{2} - \frac{0.5}{2} - \frac{3/4}{2} = 1.375 \end{cases}$$

* Composite structures:



AISC: $\frac{b_e}{2} \geq$
1. $\frac{1}{8}$ beam span
2. $\frac{1}{2}$ x beam ℄ to ℄
3. Distance from beam ℄ to slab edge

AASHTO: $b_e \geq$
1. $\frac{1}{4}$ beam span
2. 12 x slab
3.



$$\phi M_n = \phi M_p$$

$$= A_sF_y\left(\frac{d}{2} + t - \frac{a}{2}\right)$$

$$\frac{h}{t_w} \leq 3.76\sqrt{\frac{E}{F_y}}$$

Ex 16.1 $b_e = 100\,in$ beam $W_{30\times99}$ ($d = 30\,in$)

Thickness of concrete deck = 5 in

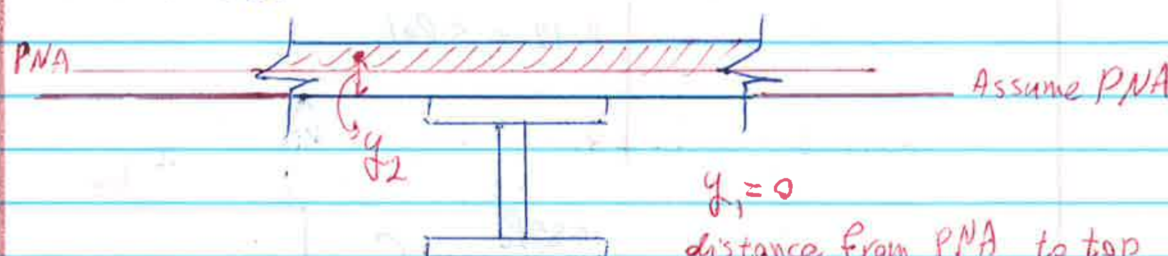
$C=T \Rightarrow 0.85 f'_c \times b_e \times a = A_s F_y$

$\Rightarrow a = 29\times50/(0.85\times4Ksi\times100) \Rightarrow a = 4.26'' < 5''$ ✓ OK (CASE "A")

$\phi M_p = A_s F_y\left(\frac{d}{2} + t - \frac{a}{2}\right) = 29\times50\left(\frac{29.7}{2} + 5 - \frac{4.26}{2}\right) = 25694\ K\text{ft}$

Case A Case B Case C

* in case A, AISC assume that PNA is at the bottom of concrete slab



$y_1 = 0$
distance from PNA to top of flang

$y_2 = 5 - \frac{4.26}{2} = 2.86''$

T3-99 : $y_1 = \phi$ and $y_2 = 2.86 \Rightarrow \phi M_p = 1926$ ✓

$$C = 0.85 f'_c b_e t + F_y b_f t_f$$

$$T = F_y (A_s - b_f t_f)$$

$C > T \rightarrow$ PNA within fleng

$C < T \rightarrow$ PNA within web

Exp $\quad$ beam $= W_{30\times116}$ $\quad b_e = 80 \quad t = 4$

$$\Rightarrow C = 0.85 \times 4 \times 80 \times 4 + 50 \times 10.5 \times 0.85 = 1534\,K$$

$$T = 50(34.2 - 10.5 \times 0.85) = 1264\,K$$

$C > T \Rightarrow$ Case B

$\Rightarrow$ Case B: $0.85 f'_c b_e t + F_y b_f \bar{y} = F_y (A_s - b_f \bar{y})$

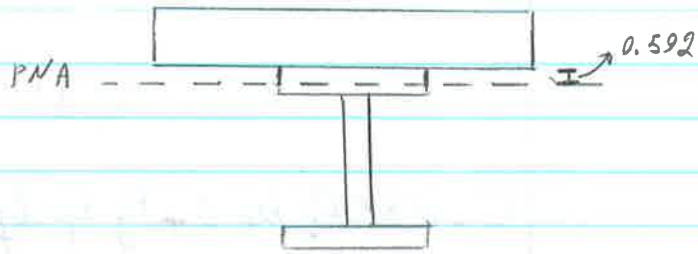
$$\Rightarrow \bar{y} = \frac{F_y A_s - 0.85 f'_c b_e t}{2 F_y b_f} = \frac{50 \times 34.2 - 0.85 \times 4 \times 80 \times 4}{2 \times 50 \times 10.5} = 0.592$$

$$\phi M_n = \phi M_p = 0.85 f'_c b_e t \left(\frac{t}{2} + \bar{y}\right) + 2 F_y b_f \bar{y} \left(\frac{\bar{y}}{2}\right) + F_y A_s \left(\frac{d}{2} - \bar{y}\right)$$

$$= 0.85 \times 4 \times 80 \times 4 \left(\frac{4}{2} + 0.592\right) + 2 \times 50 \times 80 \times 0.592 \left(\frac{0.592}{2}\right) + 50 \times 33.2 \left(\frac{80}{2} - 0.592\right)$$

$$= 2074\,Kft$$

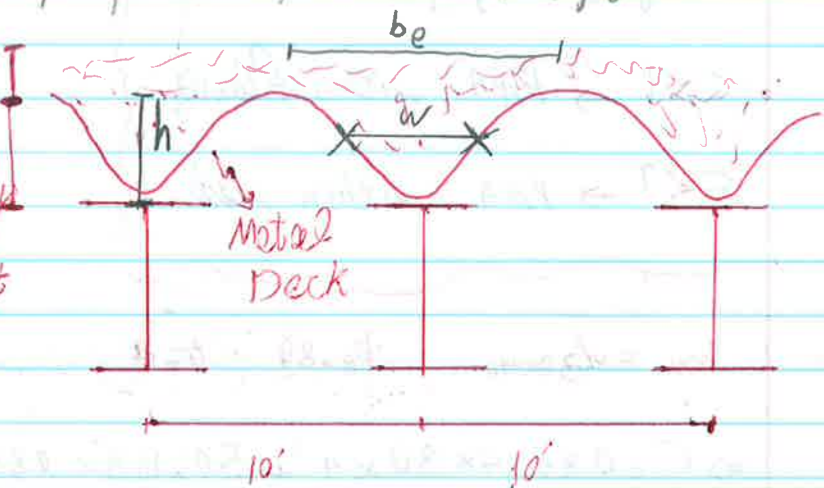
hw: 16_1



$\begin{cases} y_1 = 0.592 \\ y_2 = 2'' \text{ (half of the slab)} \end{cases}$

table 3_19 $\Rightarrow \phi M_p =$ (interpolate) $= 2070$ Kft

Exp 16.3



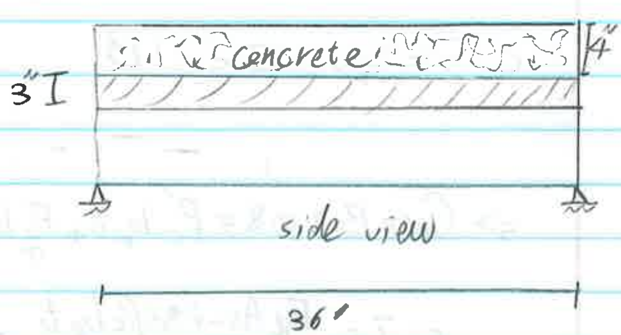
include beam weight
$DL = 0.78\ K/ft$

$LL = 2\ K/ft$

$Y_{con} = 3'' + 4''$.

$W_u = 1.2D + 1.6L = 1.2 \times 0.78 + 1.6 \times 2 = 4.14\ K/ft$

$M_u = wl^2/8 = 670.7$



$b_e/2 \rightarrow \ngtr \frac{1}{8}\text{ span} = \frac{1}{8}(36 \times 12) = 54''$

$\rightarrow \ngtr 0.5(10 \times 12) = 60''$

$\Rightarrow b_e = 54 \times 2 = 108''$

Y_{con} = distance of top of slab to top of beam $= 7''$

$a \simeq t_{slab}/2 \rightarrow 1.5''$ try
$= 4/2 = 2''$

$Y_1 = 0$; $Y_2 = Y_{con} - a/2 = 7 - 2/2 = 6''$

T 3-19 ($Y_1 = \phi$, $Y_2 = 6''$) ∴ W_{18-46}

Shored → use some bamboes or... to handle the load before concrete get dry

unshored →

①

$$w = \frac{4}{12} \times 110 \times 10 + \underbrace{46}_{W18\times46} = 413 \text{ lb/ft}$$

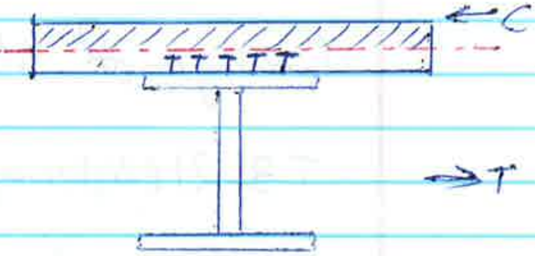
$$M = \frac{0.413 \times 36^2}{8} = 66.9^{kft} < \phi M_{p(W18\times46)} \checkmark$$

② check deflection

$$\Delta = \frac{M l^2}{C_1 I_x} = \frac{66.9 \times 36^2}{161 \times 712} = 0.76''$$

P2-9 $\Delta_{allow} = \frac{L}{180} = \frac{36\times12}{180} = 2.5''$

$0.76'' < 2.5''$ ✓ OK



first try: $\sum Q_n = A_s F_y \Rightarrow W_{18\times46}$

↳ Total shear resistant by stars

$$\sum Q_n = 13.5 \times 50 = 675 K$$

$$\Rightarrow C = 675K$$

$$a = \frac{C}{0.85 f'_c b_e} = \frac{675}{0.85 \times 4 \times 108} = 1.84 < 4''$$ within slab ⇐ NA

$Y_2 = Y_{con} - a/2 = 7 - \frac{1.84}{2} = 6.08''$

T3-19 ($Y_1 = 0$, $Y_2 = 6.08''$):

TFL
$Y_1 = 0 \rightarrow \Sigma Q_n = 675^K$

③ $Y_1 = 0.303 \rightarrow \Sigma Q_n = 492^K$ ✡ $\hookrightarrow \phi M_n = 677\ Kft > 671\ Kft$ required

$a = \frac{\Sigma Q_n}{0.85 f'_c\, b_e} = \frac{494}{0.85 \times 4 \times 108} = 1.35''$

Actual $Y_2 = 7 - \frac{1.35}{2} = 6.33$

$Y_1 = 0.303$

$\phi M_n = 678 + \frac{0.33}{0.5}(697 - 678)$

$= 690.5 > 670.7$ OK ✓

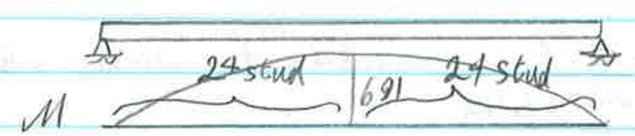
USE $W_{18 \times 46}$

Design of studs (stars)

T3-21 (lightweight concrete, $f'_c = 4Ksi$): $\frac{W}{h} > 1.5$

capacity / stud $= 21.2^K$

$\Sigma Q_n = 494^K \Rightarrow N = \frac{494}{21.2} = 23.3 \simeq 24$ $\frac{3}{4}''$ stud



$\Rightarrow 48$ stud

hw: 16-8

Check live load Deflection

$$\Delta \not> \frac{span}{360} = \frac{36 \times 12}{360} = 1.2$$

$$M_L = \frac{L \times L''}{8} = \frac{2 \times 36''}{8} = 324K'$$

$$C_1 = 161$$

$$\Delta_L = \frac{ML^2}{C_1 I} = \frac{324 \times 36^2}{161 \times}$$

T3-20:

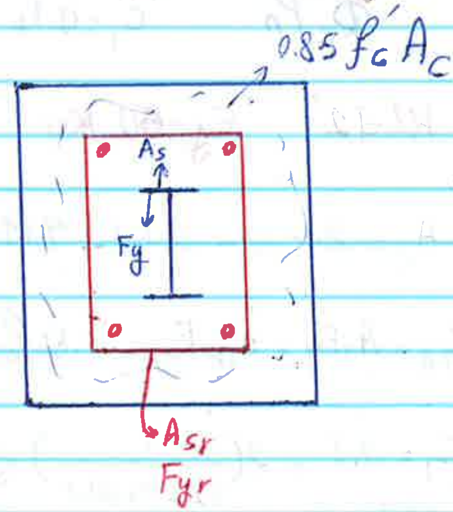
$$I = 200 + \frac{0.33}{0.5}(2090 - 2000) = 2059$$

$$\Delta = \frac{ML^2}{C_1 I} = \frac{324 \times 36^2}{161 \times 2059} = 1.27'' > 1.2$$

CK: $V_u = \frac{4.14 \times 36}{2} = 74.5$

ck → ϕV_n => Table 3-2 (195^k) for steel section
$W_{18 \times 46}$

Composite columns

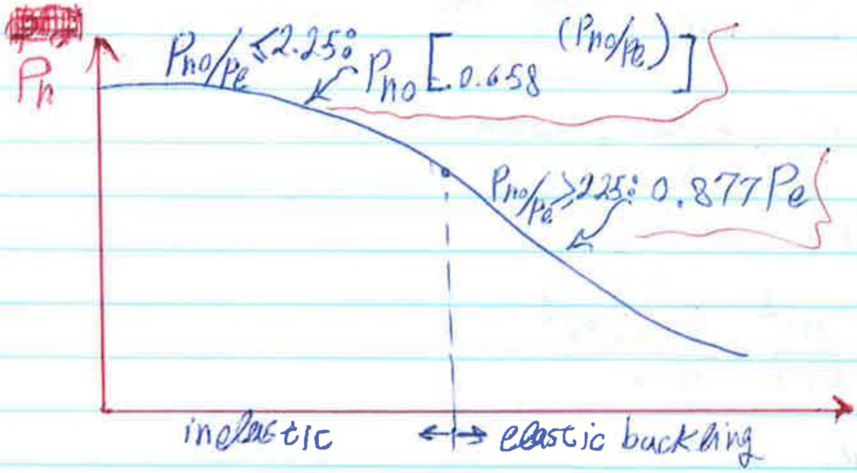


1. Small size column
 55-60% cost saving

2. Corrosion + Fire protection

3. Very efficient construction

$$P_{no} = A_s F_y + A_{sr} F_{yr} + 0.85 f'_c A_c$$



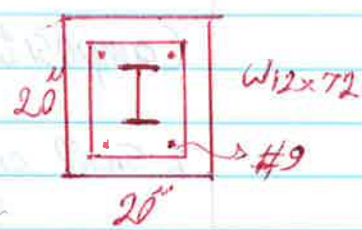
$$P_e = \frac{\pi^2 EI_{effective}}{(KL)^2}$$

$EI_{effective}$ = effective stiffness + composite

$$= E_s I_s + 0.5 E_s I_r + C_1 E_c I_c$$

$\phi_c P_n$ (ϕ_c = 0.75) $\quad C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.3$

Exp II 1: $KL = 12'$ $\quad F_y = 50$ ksi $\quad f'_c = 3.5$ ksi



$$A_c = 20 \times 20 - 21.1 - 4(1\,in^2) = 374.9\,in^2$$

$$P_{no} = A_s F_y + A_{sr} F_{yr} + 0.85 f'_c A_c = 21.1 \times 50 + 4(1) \times 60 + 0.85 \times 3.5 \times 374.9 = 2410^K$$

$$C_1 = 0.1 + 2\left(\frac{21.1}{374.9 + 21.1}\right) \leq 0.3 \Rightarrow 0.2066 \leq 0.3 \checkmark OK$$

(weak axis) y axis $\quad I_c = \frac{20 \times 20^3}{12} - 195 = 13138$

$$EI_{eff} = E_s I_{sy} + 0.5 E_s I_r + C_1 E_c I_c = 29000 \times 195 + 0.5(29000)(4 \times 1 \times 7.5^2)$$
$$+ 0.2066 \times 3267 \times 13138 = 17.785 \times 10^6\ K\,in^2$$

$P_e = \frac{\pi^2 E I_{eff}}{(KL)^2} = \frac{\pi^2 \times 29000 \times 17.785 \times 10^6}{12 \times 12} = 8465$

$\frac{P_{no}}{P_e} = \frac{2410}{8465} = 0.28 < 2.25$

$\Rightarrow P_{no}\left[0.658^{(P_{no}/P_e)}\right] = 2410\left[0.658^{\frac{2010}{8465}}\right] = 2139$

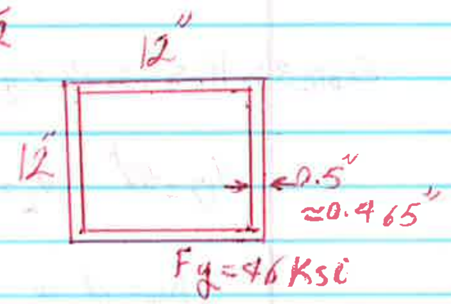
$\Rightarrow \phi_c P_n = 0.75 \times 2139 = 1604\ K$

En case column: steel is inside

Concrete fill column: steel is outside

Exp 172: HSS (High Strength Structure) $12 \times 12 \times \frac{1}{2}$

$(KL)_x = (KL)_y = 16'$ $\qquad \phi P_n = ?$



check the steel thick: $b/t \le 2.26\sqrt{E/F_y}$

$\Rightarrow \frac{12}{0.462} = 25.81 < 2.26\sqrt{\frac{29000}{50}}$ ✓ ok

$P_{no} = A_s F_y + C_2 f'_c \left(A_c + A_{sr}\left(\frac{E_s}{E_c}\right)\right) = (20.9)(46) + 0.85 \times 4(123.1 + 0) = 1380K$

$C_2 = 0.85$ rectangular

$C_2 = 0.95$ circular

$A_c = 12 \times 12 - 20.9 = 123.1\ in^2$

$EI_{eff} = E_s I_s + E_{sr} I_{sr} + C_3 E_c I_c = 29000 \times 457 + 0 + 0.85 \times 3492 \times 1271 = 17.026 \times 10^6$

$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.9 \Rightarrow 0.6 + 2\left(\frac{20.9}{123.1 + 20.9}\right) = 0.85 < 0.9$ ✓

$E_c = w_c^{1.5}\sqrt{f'_c} = 145^{1.5}\sqrt{4} = 3492$

$I_c = \frac{12\times12^3}{12} - \underbrace{453}_{I_s} = 1271$

$P_e = \frac{\pi^2 EI_{eff}}{(KL)^2} = \frac{\pi^2\times 29000\times 17.026\times10^6}{(12)^2} = 4558\ K$

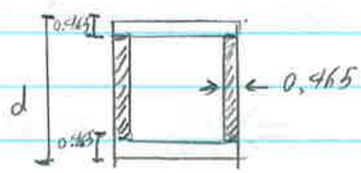
$P_{no}/P_e = \frac{1380}{4558} = 0.3 < 2.65 \Rightarrow P_n = P_{no}\left[0.65^{P_{no}/P_e}\right] = 1380\left[0.658^{1380/4558}\right]$
$= 1216\ K$

$\phi P_n = 0.75\times1216 = 912K$

Exp17.3: HSS $12\times12\times\frac{1}{2}$ $(KL)_x = (KL)_y = 16'$

$V_D = 50^K$ $V_L = 100^K \Rightarrow V_u = 1.2\times50 + 1.6\times100 = 220^K$

$h_w = d - 3t = 12 - 3(0.465) = 10.605$
$A_w = 2h_w t_w = 2\times10.605\times0.465 = 9.86$



$V_n = 0.6 F_y A_w = 0.6\times 46\times 9.86 = 272^K$

$\phi V_n = 0.9\times272 = 244.8 > 220^K$ OK ✓

Exp17.4: $(KL)_x = (KL)_y = 15'$ HSS $10\times10\times3/8$ Filled wit $f'_c = 4ksi$
$F_y = 46\ ksi$

$\phi P_n = ?$

T4-15 (KL = 15, HSS) $\Rightarrow$ $575\ K = \phi P_n$

HW: 17-1
17-4

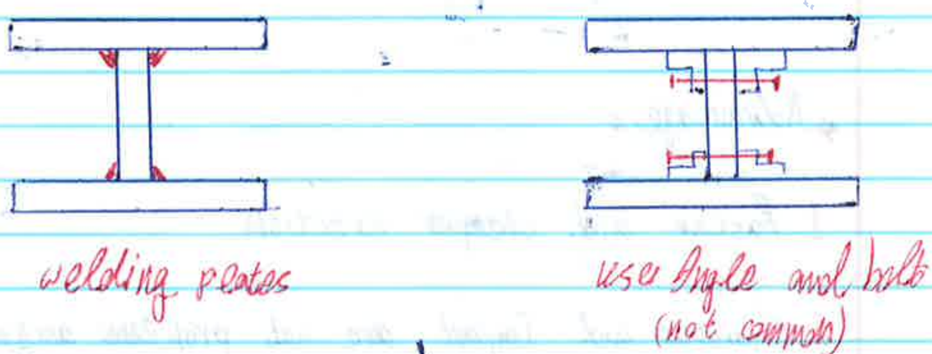
Exp17-5: HSS 20x20 x $\frac{5}{8}$ $(KL)_x = 24$ $(KL)_y = 12$

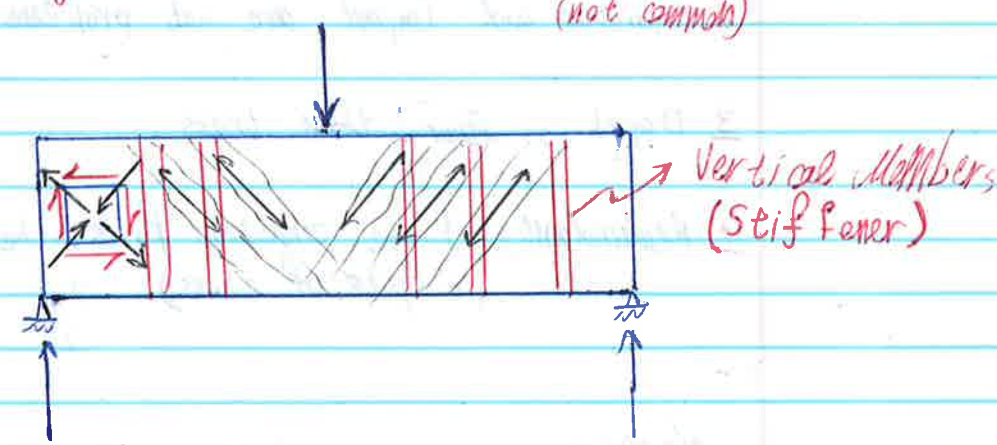
$(KL)_y = 12 \Rightarrow$ equivalent $(KL)_y = \frac{(KL)_x}{r_x/r_y} = \frac{24}{1.54} = 15.58 \Rightarrow KL = 15.58$

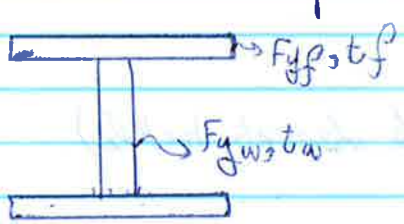
T 4-15: $\phi P_n = 1653K$

PLATE GIRDER

1. Steel bridges
2. beams with very large span







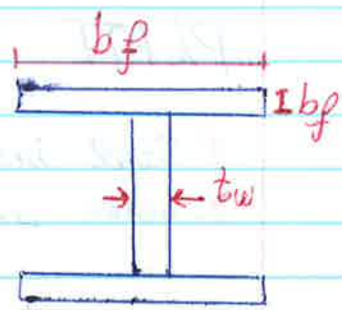
Bridges (Highway): 80′-150′ ⇒ > 200′

Railing Bridges: 50′-180′ ⇒ > 400′

1 depth of the girder = $\frac{1}{6}$ to $\frac{1}{15}$ of span

very high load concentrated load — very low load concentrated load

$\left(\frac{1}{10} \text{ to } \frac{1}{12}\right)$



2. t_f : 1" to 2"

3. $b_f \approx \dfrac{\text{depth of the girder}}{4}$

4. $t_w \approx \dfrac{3}{4}$ flange thickness

* Advantage:

1 Faster and cheaper erection

2 Vibration and Impact are not problems anymore

3 Depth is lower than truss

4 Redundant: if one connection fails, the beam is not going to fail. (in spite of truss)

* Names:

1 Built up I-section (web doesnt buckle)

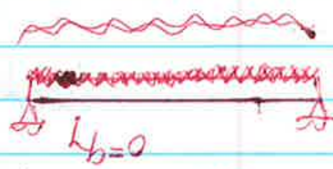
2 Built up plate section (make sure web must buckle)

Expl 82: depth of built up I-section = 60"

span = 70'   DL (without beam) = 1.1 K/ft

LL = 3K/ft

A36 steel → full lateral bracing in compression flange



$d_{web} = \cancel{\frac{1}{6}}$ to $\frac{1}{15} = \frac{70 \times 12}{15} \approx 60''$ ✓ OK

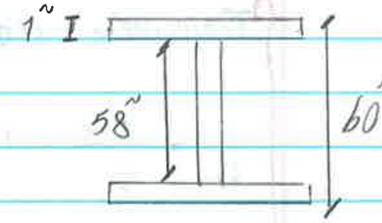
$L_b = 0$

$\phi M_n = \phi M_p$

$t_f = 1''$ to $2'' = 1''$

$b_f = d/4 = \frac{60}{4} = 15''$

$t_w = 3/4\, t_f = 0.75''$



DL of beam: flangs: $2(15 \times 1) = 30\ in^2$

$0.75 \times 58 = 43.5\ in^2$

$73.5\ in^2$

steel weight = 490 lb/ft³

$\Rightarrow$ I_beam: $\frac{490}{\frac{73.5}{144}} = 250.1$ lb/ft

$= 0.25$ K/ft

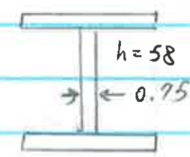
$W_u = 1.2(1.1 + 0.25) + 1.6(3) = 6.42$ K/ft

shear $V_u = \frac{W_u L}{2} = 6.42 \times \frac{70}{2} = 224.7$ K

moment $M_u = \frac{W_u l^2}{8} = 6.42 \times \frac{70^2}{8} = 3932$ K ft

I-section → web must not buckle

Table B4.1: $\frac{h}{t_w} \le 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29000}{36}} = 106.7$



$t_w = \frac{106.7}{h} = \frac{106.7}{58} = 0.544'' = \frac{9}{16}'' \approx \frac{1}{16}''$

$\Rightarrow$ web: $\frac{9}{16}'' \times 58''$

check for shear:

$\phi V_n = \phi\, 0.6 F_y A_w C_v$ section G 16.1-68, 16.1-69

Transverse stiffeners are not required where $h/t_w \le 2.46\sqrt{\frac{E}{F_y}}$

$\frac{h}{t_w} = \frac{58}{9/16} = 103.1 \not< 2.4\sqrt{\frac{29000}{36}} = 69.82 \Rightarrow$ stiffener is needed

$V_u < \phi V_n$ Equ G2.1 with $K_v = 5$

$\frac{h}{t_w} = 103.1 < 260 \Rightarrow K_v = 5$ without stiffener

Try the one in the middle: $h/t_w = 103.1 > 1.37\sqrt{\frac{K_v E}{F_y}} = 1.37\sqrt{\frac{5\times 29000}{36}} = 86.94$

$\Rightarrow C_v = \frac{1.51\, E K_v}{(h/t_w)^2 F_y} = \frac{1.51\times 29000\times 5}{(103.1)^2\ 36} = 0.572$

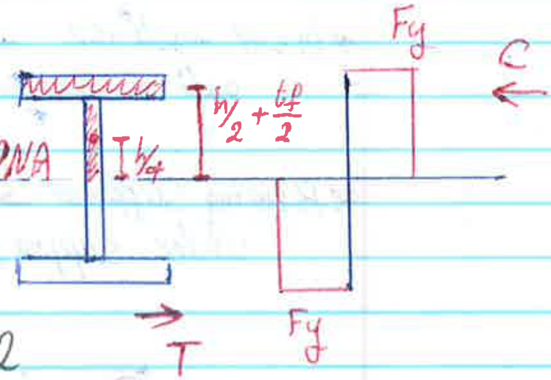
$\phi V_n = (0.9)\,0.6 F_y A_w C_v = (0.9)(0.6\times 36)(58\times\frac{9}{16})(0.572) = 403.1^K$

$403.1^K > 224.7^K = V_u$ ✓ OK

* web handle the shear and flange handle the moment

Check Moment capacity:

$$M_p = F_y Z$$



Z = Plastic Section Modulus
= Moment of area about PNA

$$= \left(A_f \times \left(\frac{h}{2}+\frac{t_f}{2}\right) + \left(\frac{h}{2}\times t_w\right)\frac{h}{4}\right)2$$

$$= A_f(h+t_f) + 2\left(\frac{h}{2}\right)t_w\left(\frac{h}{4}\right) \quad ①$$

$$M_u = \phi M_p = \phi F_y Z \Rightarrow Z_{required} = \frac{M_u}{\phi F_y} \quad ②$$

① and ② $\Rightarrow M_u/\phi F_y \Rightarrow A_f = \_\_\_$ ✓

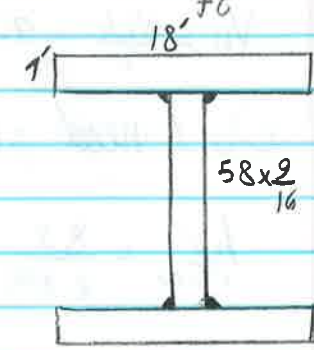
$$\Rightarrow A_f = \frac{M_u}{\phi F_y(h+t_f)} - \frac{t_w h^2}{4(h+t_f)} = \frac{3932\times 12}{0.9\times 36(58+1)} - \frac{\frac{9}{16}\times 58^2}{4(58+1)} = 16.66\ in^2 \simeq 18\ in^2$$

→ PL 18×1 for flange

$$\frac{b_f}{2t_f} < 0.38\sqrt{\frac{E}{F_y}} \Rightarrow \frac{18}{2\times 1} = 9 < 10.79 \text{ ✓OK}$$

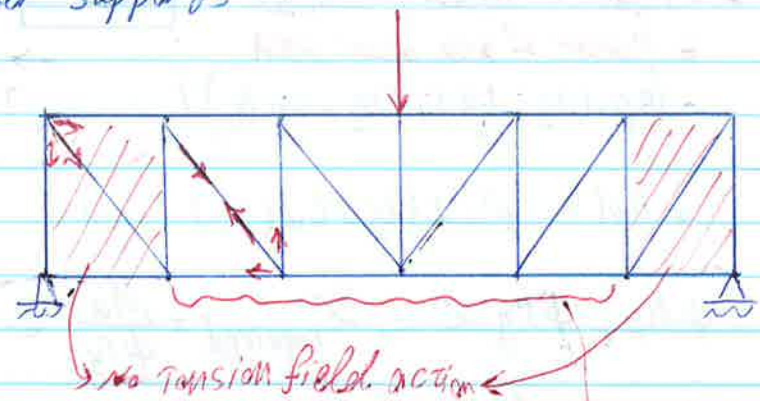
Check the girder weight:

$$\frac{\frac{9}{16}\times 58 + 2(18\times 1)}{144} \times 490\ lb/ft^3 = 233.5\ lb/ft < 250\ lb/ft \text{ ✓OK}$$



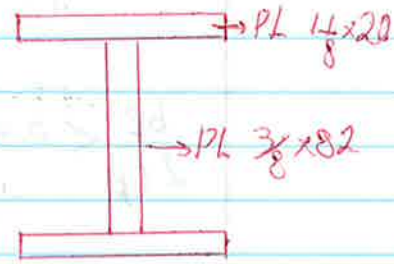
* Intermediate stiffener → stiffeners at middle that have a gap at bottom

* Bearing Stiffener → stiffeners at first and end of the beam or under supports



Exp 18.5

$L = 65'$ $\quad D = 1.1\ K/ft$ without beam
$L = 2K/ft$



$A = 2(1\frac{1}{8} \times 20 + \frac{3}{8} \times 82) = 75.75$

$w/ft = \frac{75.75}{144} \times 490 \Rightarrow 258\ K/ft$

$w_u = 1.2(1.1 + 0.258) + 1.6(2) = 4.83\ K/ft$

$V_u = \frac{W_u L}{2} = 4.83 \times \frac{65}{2} = 156.89\ K$

Do I need stiffener:

$h/t_w = \frac{82}{0.375} = 219 < 260 \Rightarrow K_v = 5$

$1.37\sqrt{\frac{K_v E}{F_y}} = 1.37\sqrt{\frac{5\times 29000}{36}} = 86.95 < h/t_w$

G2.5 $\rightarrow C_v = \frac{1.51 E k_v}{(h/t_w)^2 F_y} = \frac{1.51\times 29000\times 5}{219\times 36} = 0.1268$

$\Rightarrow \phi V_n = 0.9(0.6 F_y) A_w C_v = 0.9\times(0.6\times 36)\times(82\times 0.375)(0.1268) = 77.87^K$

$\Rightarrow 77.87^K < V_u = 156.98K$

* T 3-16a (for End Parts) right side is $\frac{V_u}{A_w}$

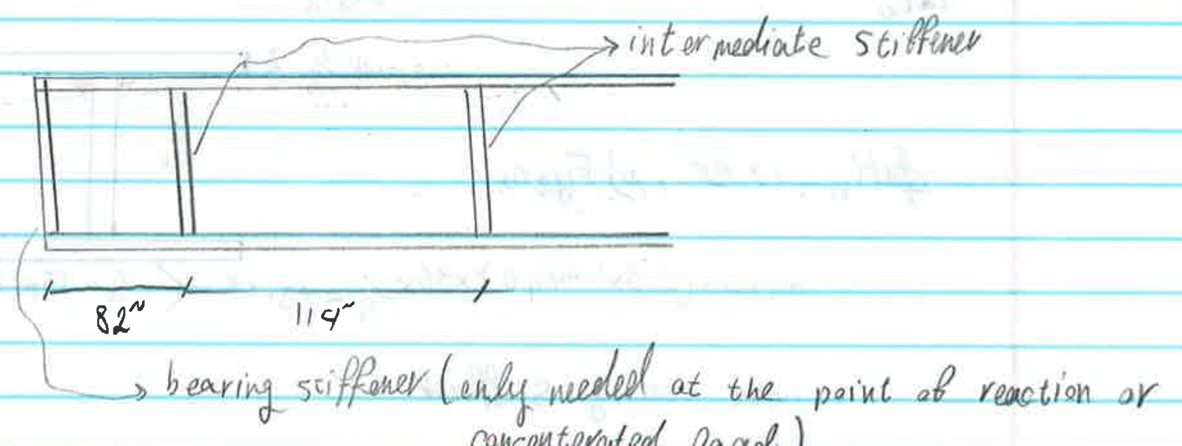
* T 3-16b (for middle parts)

V_u @ 82 in of edge = 123.97

T 3-16b : $\frac{V_u}{A_w} = \frac{123.97}{31.59} = 3.92$ ksi

$\phi V_n = 77.87 < 123.97 \Rightarrow$ need stiffener

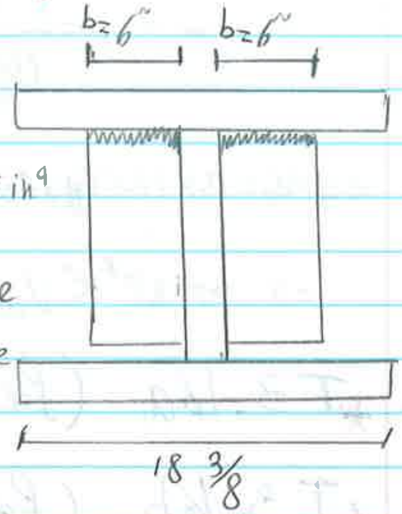
next, use table $\Rightarrow a/h = 1.4 \Rightarrow a : 1.4\times 82 = 114.8''$



$$J = \frac{2.5}{(a/h)^2} - 2 = \frac{2.5}{(82/82)^2} - 2 = 0.5$$

Transverse: $I_{st} = b\, t_w^3\, j = 82 \times (3/8)^3 \times 0.5 = 2.16\ in^4$

b: min($\omega = 82$, $h = 82$)



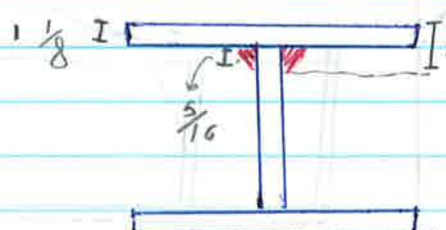
$$I_{st} = \frac{bh^3}{12} = \frac{1/4 \times 6^3}{12} \times 2 = 9\ in^4 >> 2.16\ in^4$$

so we can use just one plate at one side, doesnt have to be at both sides

Sec G2.2 and J10.8:

bearing stiffener must connected to the top

for bearing:



$K = 1\frac{1}{8} + \frac{5}{16} = 1.44$

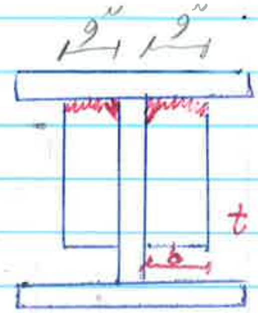
$$\phi R_n = (2.5K + N) F_{ye} t_w$$

$$= (1.0)(2.5 \times 1.44 + 0) \times 36 \times 3/8 = 48.6^K < V_u = 156.98^K$$

$\Rightarrow$ need bearing stiffener

hw 18-5
18-7

TB4-1:



$\frac{b}{t} \leq 0.56\sqrt{\frac{E}{F_y}}$

$\Rightarrow \frac{9}{t} \leq 0.56\sqrt{\frac{29000}{36}}$

$\Rightarrow t \approx \frac{5}{8}''$

Plates:

$I_{Plates} = \frac{5}{8} \times (9\times2+\frac{3}{8})^3/12 = 323\ in^4$

$A_{Plates} = 2(9\times\frac{5}{8}) + (12\ t_w)\times\frac{3}{8}'' = 12.94$ ($t_w = \frac{3}{8}''$)

$r = \sqrt{I/A} = \sqrt{\frac{323}{12.94}} = 5''$

$KL = 0.75\times h = 0.75\times 82 = 61.5$

$\Rightarrow \frac{KL}{r} = \frac{61.5}{5} = 12.3$ and T4-22 : $\phi F_y = 32.17$

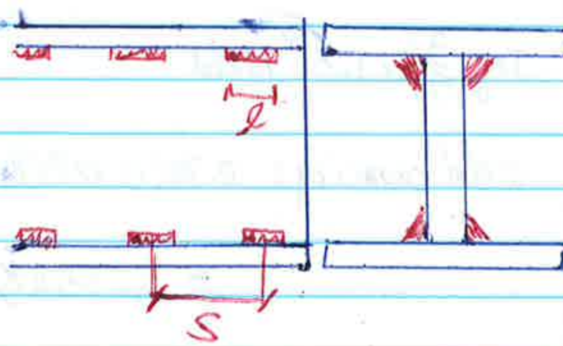
$\phi P_n = 32.17\times12.94 = 416 > V_u = 156.98$ ✓ ok

only Area of plate

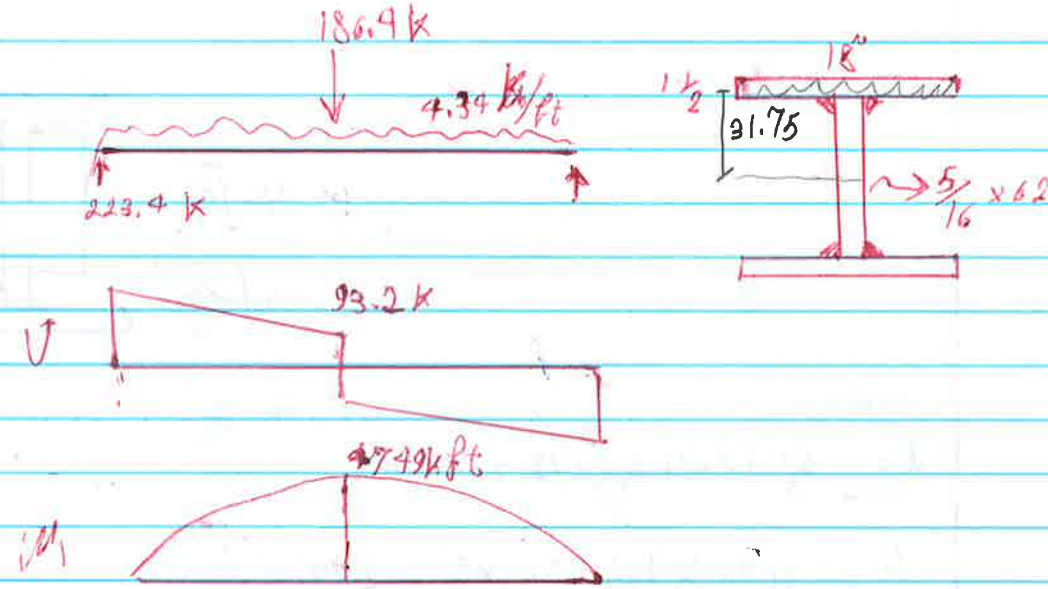
AISC $\rightarrow 1.8F_y A_{pd} > V_u \Rightarrow 1.8\times36\times0.75(9-0.5)(\frac{5}{8})(2)$
$= 516k > 156.96$ ✓ ok

$A_{pd} = (9-0.5)(\frac{5}{8})\times2$

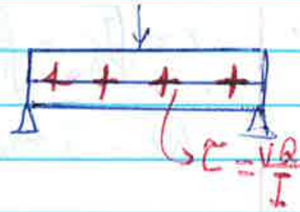
Connection Design



Exp:



$$\left\{\begin{array}{l} \tau = \dfrac{\overset{223.4}{V} Q}{\underset{= 60642\ in^4}{I}} = 3.158\ k/in \\ \\ Q = (18 \times 1.5) \times 31.75 = 857.2 \end{array}\right.$$

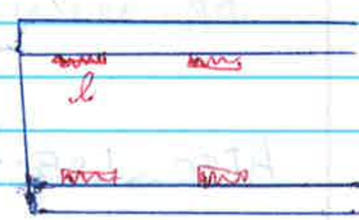


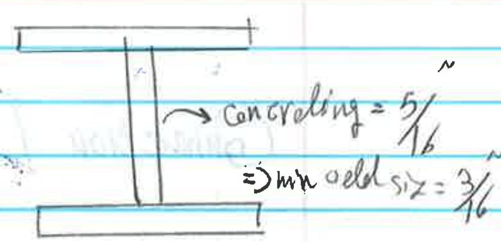
weld size:

$l_{min} = 4w \geq 1.5''$

$= 4(3/16) \geq 1.5'' \Rightarrow 0.75'' \geq 1.5$

$\Rightarrow l = 1.5''$





Try $3/16 \times 1.5''$ weld

weld capacity: $0.75(0.6 \times 70)(0.707 \times 3/16) \times \underbrace{2}_{\text{both side}}$

$= 8.352\ k/in$

our capacity

weld size

Web capacity : (1) yielding : $0.6\,F_y \times \frac{5}{16} = 0.6 \times 36 \times \frac{5}{16} = 6.75\ K/in$

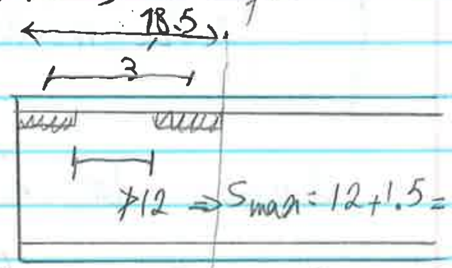
(2) fracture: $0.75\,(0.6 \times \overset{F_u}{58})\ \frac{5}{16} = 8.156\ K/in$

$\Rightarrow$ each weld can handle: $6.75 \times 1.5 = 10.13\,K$

$$\tau = 3.158 = \frac{10.13}{S} \Rightarrow S = 3.21'' \simeq 3''$$

Code : $S_{max} \leq 0.75 \sqrt{\frac{E}{F_y}}\ t_f \ngtr 12''$

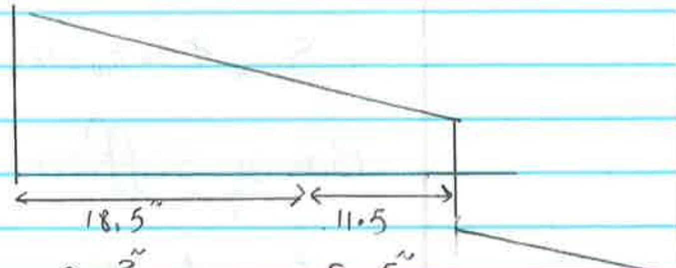
$$\Rightarrow S_{max} \leq 0.75 \sqrt{\frac{29000}{36}}\ 1.5 \ngtr 12'' \Rightarrow 31.9'' \ngtr 12''$$



$\ngtr 12 \Rightarrow S_{max} = 12 + 1.5 = 13.5''$

$$\frac{V_u Q}{I} = \frac{10.13}{5''} \Rightarrow V_u = \frac{10.13}{5''} \times \frac{60640}{857.2} = 143.3^K$$

$$\Rightarrow V_u = 223.4 - 4.34x = 143.3 \Rightarrow x = 18.5''$$

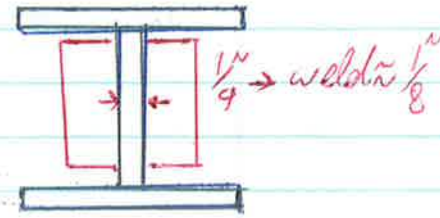


s = 3" 　　 S = 5"

74 space @ 3" 　　 28 space @ 5"

= 18'6" 　　 = 11'6"

Transverse Stiffener



weld length $4w \not< 1.5''$

$\Rightarrow 4 \times \frac{1}{8} \not< 1.5'' \Rightarrow .5 \not< 1.5''$

$\Rightarrow$ weld length $= 1.5''$

$$\begin{cases} \text{weld: } \phi F_w/in \Rightarrow 0.75(0.6 \times 70)(0.707)\frac{1}{8} \times \underset{\text{both side}}{2} \times \underset{\text{2 stiffener}}{2} = 5.568 \, k/in \\ \text{web: } \begin{cases} 1.\ \text{yielding} \Rightarrow 1 \times (0.6 \times 36)(\frac{1}{4}) = 5.4 \, k/in \\ 2.\ \text{fracture} \Rightarrow 0.75(0.6 \times 58)(\frac{1}{4}) = 6.525 \, k/in \end{cases} \end{cases}$$

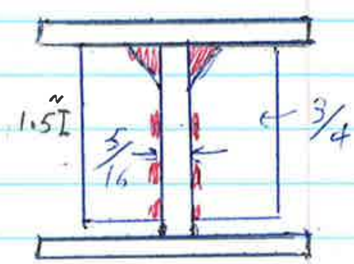
2 place $\phi F_w = 5.4 \times 2 = 10.8$

AISC: $f\ 0.045 \sqrt{\frac{F_y^3}{E}}\ h = 0.045 \sqrt{\frac{36^3}{29000}} \times 62 = 3.536$

$\Rightarrow \tau = 3.536 = \frac{10.8\ k/in \times 1.5}{S} \Rightarrow S = 4.5''$

$S_{max} \not< 16\, t_w = 16 \times \frac{5}{16} = 5'' \Rightarrow S = 4.5''$ ✓

Bearing Stiffener



$t_{min} = t_w = 5/16$

weld$_{size}$ $= 3/16 \rightarrow$ min

min length $\Rightarrow 4w \not< 1.5 \Rightarrow 4 \times \frac{3}{16} \not< 1.5 \Rightarrow 0.75 \not< 1.5 \Rightarrow l = 1.5''$

weld → $0.75(0.6\times70)(0.707\times\frac{3}{16})\times2 = 8.352\ K/in$

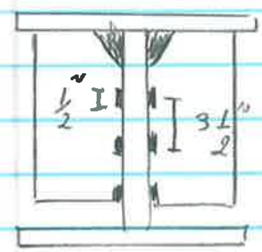
web → 1. yielding : $1(0.6\times36\times\frac{5}{16}) = 6.75\ K/in$

2. fracture: $0.75(0.6\times58\times\frac{5}{16}) = 8.156\ K/in$

2 plate → $2\times6.75 = 13.5\,K/in$

shear flow $= \dfrac{R_{max}}{\text{length availible } h} = \dfrac{223.4}{62-2\times\frac{5}{16}} = 3.662$

$\tau = \dfrac{13.5\ K/in\times1.5}{S} = 3.662 \Rightarrow S = 5.53 \approx 5\frac{1}{2}''$



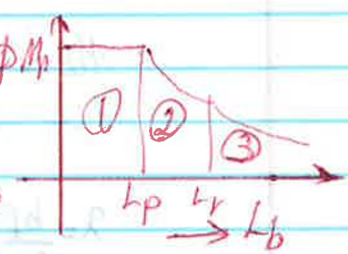
chapter F eq 1_4 to 1_15 8

ϕM_n for plate girder

second check

1. Lateral Torsional Buckling (LTB) 2

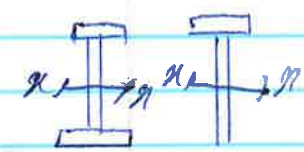
ϕM_p if $L_b = 0$



third check

2. Flang Local Buckling (FLB) 3

gonna happen if the flang is non compact



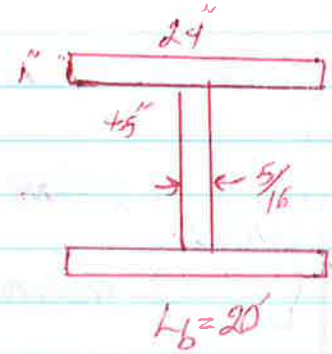
3. (TFY) unsymmetric section around x-x

first check

4. Compression Flange Yielding (CFY) 1

(Y)

Exp 18.4



① is the web compact? $\Rightarrow h/t_w > 3.76\sqrt{\frac{E}{F_y}}$

$\Rightarrow 144 > 106.72 \Rightarrow$ non compact web (plate girder)

② is the flang compact? $\Rightarrow b/t_f > 0.38\sqrt{E/F_y}$

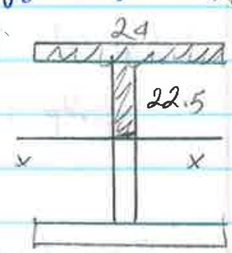
$\frac{24/2}{1} > 0.38\sqrt{\frac{29000}{36}} \Rightarrow 12 > 10.79$

$\Rightarrow$ the flang is non compact

checking steps : 1. Y
2. LTB
3. FLB

1. Y $M_n = R_{pc}\, F_y\, S_{xc}$ $\quad R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] < \frac{M_p}{M_{yc}}$

$$\frac{M_p}{M_{yc}} = \frac{\cancel{F_y} Z}{\cancel{F_y} S} = \frac{2\left(24\times1\times(22.5+0.5)\right) + 22.5\times0.3125\times\frac{22.5}{2}}{1132} = 1.068$$



$\lambda = \frac{b_f}{2t_f} = 12 \qquad \lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 10.79$

$\lambda_{rf} = 0.95\sqrt{\frac{E}{F_y}} = 15.58$

$\Rightarrow R_{pc} = 1.022$: $M_n = 1.022 \times 36 \times \frac{1132}{12} = 3624$ (Y)

2. LTB   Zone 2 or 3?

$L_b = 20'$ $L_p = 1.1\sqrt{E/F_y} = 17.42$ $L_r = 63.67'$

$\Rightarrow$ Zone 2 : $M_n = C_b\left[R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L S_{xc})\left[\frac{L_b - L_p}{L_r - L_p}\right]\right] \leq R_{pc} M_{yc}$

$C_b \to$ table 3-1

now we use $C_b = 1.0$

$\Rightarrow M_n = 1.0[\ \ \ ] = 3560$ (LTB)

3. FLB $M_n = \left[R_{pc} M_{yc} - (R_{pc} M_{yc} - F_L S_{xc})\left[\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right]\right]$

eq F.4.6a $\Rightarrow F_L$ ✓ $\frac{S_{xt}}{S_{xc}} = 1 \to$ (bcause of symmetry)

S_{xt}: tension, S_{xc}: compression

$\Rightarrow F_L = 0.7 F_y = 25.2$

$\Rightarrow M_n = 3334$

$\phi M_n = 0.9 \times 3334 = 3001$ Kft

■ **FIGURE 6.3**

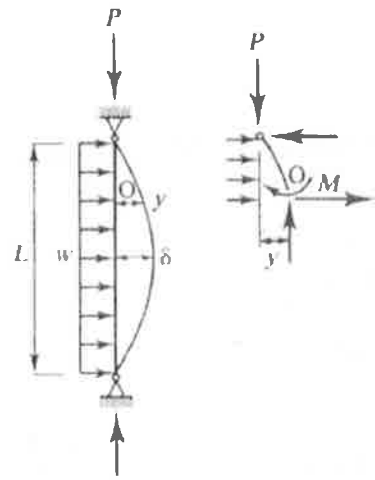


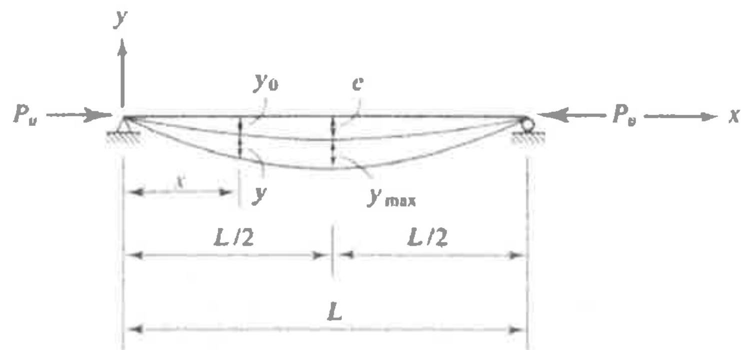
Figure
shows a beam-column with an axial load and a transverse uniform load. At an arbitrary point O, there is a bending moment caused by the uniform load and an additional moment Py, caused by the axial load acting at an eccentricity from the longitudinal axis of the member. This secondary moment is largest where the deflection is largest — in this case, at the centerline, where the total moment is $wL^2/8 + P\delta$. Of course, the additional moment causes an additional deflection over and above that resulting from the transverse load. Because the total deflection cannot be found directly, this problem is nonlinear; and without knowing the deflection, we cannot compute the moment.

Ordinary structural analysis methods that do not take the displaced geometry into account are referred to as *first-order* methods. Iterative numerical techniques, called *second-order* methods, can be used to find the deflections and secondary moments, but these methods are impractical for manual calculations and are usually implemented with a computer program. Most current design codes and specifications, including the AISC Specification, permit the use of either a second-order analysis or the *moment amplification method*. This method entails computing the maximum bending moment resulting from flexural loading (transverse loads or member end moments) by a first-order analysis, then multiplying by a *moment amplification factor* to account for the secondary moment. An expression for this factor will now be developed.

Figure 6.4 shows a simply supported member with an axial load and an initial out-of-straightness. This initial crookedness can be approximated by

$$y_0 = e \sin \frac{\pi x}{L}$$

■ **FIGURE 6.4**



where e is the maximum initial displacement, occurring at midspan. For the coordinate system shown, the moment – curvature relationship can be written as

$$\frac{d^2y}{dx^2} = -\frac{M}{EI}$$

The bending moment M is caused by the eccentricity of the axial load P_u with respect to the axis of the member. This eccentricity consists of the initial crookedness y_0 plus additional deflection y resulting from bending. At any location, the moment is

$$M = P_u(y_0 + y)$$

Substituting this equation into the differential equation, we obtain

$$\frac{d^2y}{dx^2} = -\frac{P_u}{EI}\left(e \sin \frac{\pi x}{L} + y\right)$$

Rearranging gives

$$\frac{d^2y}{dx^2} + \frac{P_u}{EI} y = -\frac{P_u e}{EI} \sin \frac{\pi x}{L}$$

which is an ordinary, nonhomogeneous differential equation. Because it is a second-order equation, there are two boundary conditions. For the support conditions shown, the boundary conditions are

$$\text{At} \quad x = 0, \quad y = 0 \quad \text{and at} \quad x = L, \quad y = 0$$

That is, the displacement is zero at each end. A function that satisfies both the differential equation and the boundary conditions is

$$y = B \sin \frac{\pi x}{L}$$

where B is a constant. Substituting into the differential equation, we get

$$-\frac{\pi^2}{L^2} B \sin\frac{\pi x}{L} + \frac{P_u}{EI} B \sin\frac{\pi x}{L} = -\frac{P_u e}{EI} \sin\frac{\pi x}{L}$$

Solving for the constant gives

$$B = \frac{-\dfrac{P_u e}{EI}}{\dfrac{P_u}{EI} - \dfrac{\pi^2}{L^2}} = \frac{-e}{1 - \dfrac{\pi^2 EI}{P_u L^2}} = \frac{e}{\dfrac{P_e}{P_u} - 1}$$

where

$$P_e = \frac{\pi^2 EI}{L^2} = \text{the Euler buckling load}$$

$$\therefore\ y = B \sin\frac{\pi x}{L} = \left[\frac{e}{(P_e/P_u) - 1}\right] \sin\frac{\pi x}{L}$$

$$M = P_u(y_0 + y)$$

$$= P_u\left\{e \sin\frac{\pi x}{L} + \left[\frac{e}{(P_e/P_u) - 1}\right] \sin\frac{\pi x}{L}\right\}$$

The maximum moment occurs at $x = L/2$:

$$M_{\max} = P_u\left[e + \frac{e}{(P_e/P_u) - 1}\right]$$

$$= P_u e\left[\frac{(P_e/P_u) - 1 + 1}{(P_e/P_u) - 1}\right]$$

$$= M_0\left[\frac{1}{1 - (P_u/P_e)}\right]$$

where M_0 is the unamplified maximum moment. In this case, it results from initial crookedness, but in general it can be the result of transverse loads or end moments. The moment amplification factor is therefore

$$\frac{1}{1 - (P_u/P_e)} \qquad (6.4)$$

As we describe later, the exact form of the AISC moment amplification factor can be slightly different from that shown in Expression 6.4.

Table 1.
Revised Table of *m* for Steel Beam-Column

Values of *m*														
F_y	36 ksi							50 ksi						
KL (ft)	10	12	14	16	18	20	22 and over	10	12	14	16	18	20	22 and over
1st Approximation														
All Shapes	2.0	1.9	1.8	1.7	1.6	1.5	1.3	1.9	1.8	1.7	1.6	1.4	1.3	1.2
Subsequent Approximation														
S4,5,6	1.3	1.0	0.8	0.7	0.6	0.5	0.5	1.1	0.9	0.8	0.7	0.6	0.5	0.5
W,M4	3.1	2.3	1.7	1.4	1.1	1.0	0.8	2.4	1.8	1.4	1.1	1.0	0.9	0.8
W,M5	3.2	2.7	2.1	1.7	1.4	1.2	1.0	2.8	2.2	1.7	1.4	1.1	1.0	0.9
W,M6	2.8	2.5	2.1	1.8	1.5	1.3	1.1	2.5	2.2	1.8	1.5	1.3	1.2	1.1
W8	2.5	2.3	2.2	2.0	1.8	1.6	1.4	2.4	2.2	2.0	1.7	1.5	1.3	1.2
W10	2.1	2.0	1.9	1.8	1.7	1.6	1.4	2.0	1.9	1.8	1.7	1.5	1.4	1.3
W12	1.7	1.7	1.6	1.5	1.5	1.4	1.3	1.7	1.6	1.5	1.5	1.4	1.3	1.2
W14	1.5	1.5	1.4	1.4	1.3	1.3	1.2	1.5	1.4	1.4	1.3	1.3	1.2	1.2