Steel structure prestressed

↑
Coverage

01716966401
Alon something
book reseller

Chapter 8

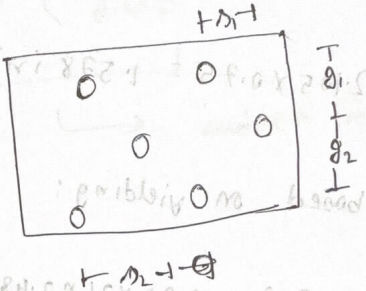
9.8 Prestressed

Recommendation
ltr
✓

Additional
Salmon & Johnson
Steel structures
Design and behavior

Staggered fastener

Fig. 208

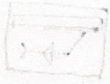


Failure propagation

hence is why we min (An)

Strength capacity always lowest

Ex – 2



fract < yield

↳ ~~sudden~~ impact force

Limit states

Servicibility limit state

excess buckling, - - -

- - -

Design consideration

ASD

not used anymore, from indian

LS - -

Loads and load cfts

DL, LL, Roof Live Ld, wind ld, E

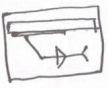
Rain water ld

- - -

Steel code on BNBC, from AISC

Gross area (A_g)

Net area (A_n)

welded → 

Shear lag →

Eff. area $A_e = A_n U$

$U = 1$ when plate

$A_n = A_g - A_h = 5.77 - \left[\frac{1}{2}\left(\frac{5}{8} + \frac{1}{8}\right)\right] 2$

thickness ↵ ↳ always add ↳ two bolt crossing

tolerance

Failure will always happen through shortage path i.e., bolt

AISC Table 1-7 [be given]

A36 steel → $f_y = 36$ ksi

$f_u = 58$ ksi

Steel struct.

Limit states

A predefined unacceptable condition

Advantages of struct steel

Disadv.

Fatigue

Brittle fracture

susceptibility to temperature

Progressive collapse → 9/11

Ex. 03

$$A_n = A_g - A_h = 2.48 - \left[\left(\frac{7}{8} + \frac{1}{8}\right)\frac{3}{8}\right]2 = 2.105 \text{ in}^2$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1}{4} = 0.75$$

$$A_e = A_n U = 2.105 \times 0.75 = 1.578 \text{ in}^2$$

Design strength based on yielding:

$$\phi_t P_n = 0.90 F_y A_g = 0.90 \times 36 \times 2.48 = 80.35 \text{ kip}$$

Design strength based on fracture:

$$\phi_t P_n = 0.75 F_u A_e = 0.75 \times 58 \times 1.578 = 68.7 \text{ kip}$$

← certainty higher

hole dia → add $1/16''$

bolt dia → add $1/8''$ } for uncertainty

$\Sigma \frac{S^2}{4g} +$

$A_g = 12 \times 1/4 = 3\ in^2$

Path AD: $A_n = 3 - 2\left[\frac{1}{4}\left(\frac{15}{16} + \frac{1}{16}\right)\right] = 2.5\ in^2$

Path ABD: $3 - 3\left[\frac{1}{4}\left(\frac{15}{16} + \frac{1}{16}\right)\right] + \left[\frac{2.125^2}{4 \times 4} + \frac{2.125^2}{4 \times 2.5}\right] \times \frac{1}{4} = 2.43 in^2$

Path ABC: $3 - 3\left[\frac{1}{4}\left(\frac{15}{16} + \frac{1}{16}\right)\right] + \left[\frac{2.125^2}{4 \times 2.5} + \frac{1.875^2}{4 \times 4}\right] \frac{1}{4} = 2.42 in^2$

LO6

28/01

Compnession memben Design as column

For steel struct, buckling failure most common

Buckling resist — bracing

Example 9

Changed

$KL = 28'$

$\frac{KL}{r} = 137$

$C_c = 113.4$ } buckling

$F_e = 15.25$

$F_{cr} = 13.4$

$\phi_c P_n = 216$

Biaxial bending

Use crushing failure config: Aesthetic, where casualty won't occur

Never use on hospital, schools, ...

Design

2 methods

Example 11 →

Example 6.10.1 →

By default use LRFD method
ASD when it is certain
whole process is centrally
monitored and ensured

Example 14/

$$I = \sum \frac{bh^3}{12} + Ad^2$$

$$= \frac{1/2 \times 12^3}{12} + 0 + \left\{ \frac{8 \times 1^3}{12} + (1 \times 8) \times (6 + 1/2)^2 \right\} \times 2 = 749.33 \text{ in}^4$$

$$S = I/c = \frac{749.3}{1 + 12/2} =$$

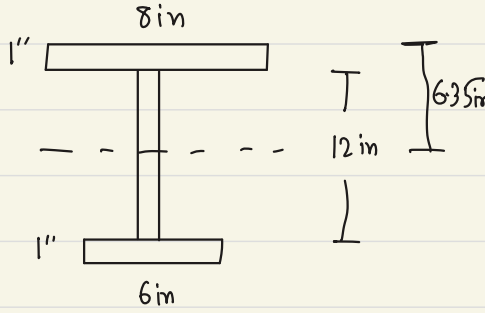
Example 15/

Example 16/

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{6 \times 0.5 + 6 \times 7 + 8 \times 13.5}{8 + 6 + 6}$$

$$= 7.65'' \text{ (from btm)} \; ; \; 6.35 \text{ in from top}$$

$$y_1 = 8 \times 1 \times (6.35 - 0.5) +$$



L08 25/02

Bolt connection

Example 25, 26, 27

Page 488

Prestressed concrete (imp for theory as well; e.g., adv.)
Prestressing loss (next)

Rough / CT-3

31/03

Compression member

Structure that prone to buckling than crushing high ratio of eff. len to it rad of gyration

Buckling : Sudden lateral instability of a slender structure caused by axial loading before yield stress is reached.

$L \rightarrow KL$ (eff. length)

Crushing : when direct stress from an axial ld exceeds material's compressive strength

Radius of gyration (r) : Average distance of an object's mass from the axis of rotation

How spread out a section is from its center

↓ → Mass is bunched up near the middle (thin rod)

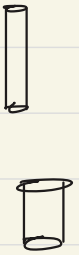
↑ → " " pushed toward edges (hollow pipe)

measures stability of a section against buckling

$$\sqrt{\frac{I}{A}}$$

Slenderness ratio : $\frac{KL}{r}$ ↑ → easy to buckle

↓ → resists buckling →



Critical coeff. : $C_c = 4.71\sqrt{\frac{E}{F_y}}$

Slenderness ratio: $SR = \frac{KL}{r}$

Short col → Crushing

Long col → Buckling

If $\frac{KL}{r} < C$, $F_{cr} = 0.658^{F_y/F_e} F_y$

$> C$, $F_{cr} = 0.877 F_e$

$F_e = \frac{\pi^2 E}{(KL/r)^2}$

Steel products

Shape → W-, M-, S-, HP- shapes → I, ...

Channels → C

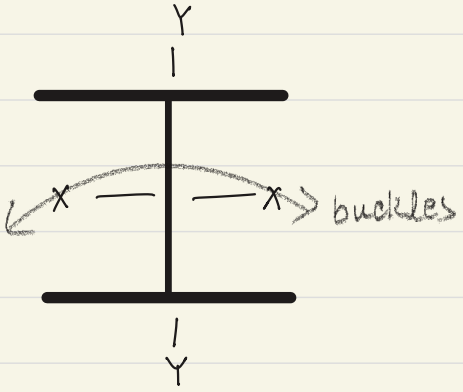
Angles → L

...

W24-55

→ 24" deep

55 lb/ft



Usually, $r_y \downarrow$ $r_x \uparrow$

Buckles in x-z plane

Y-Y axis (weak axis)

Strength → A992 → Yield stren, $F_y = 50$ ksi

Tensile ultimate stren, $F_u = 65$ ksi

A36 → $F_y = 36$ ksi

$F_u = 58-80$ ksi

Example 9

$L = 25'$

W14X61 $\rightarrow$ $r_y = 2.45$, $A_g = 17.9\ in^2$

Fixed hinge $\rightarrow$ $K = 0.8$

A992 $\rightarrow$ $F_y = 50\ ksi$, $F_u = 65\ ksi$

Capacity = ?

$$SR = \frac{KL}{r} = \frac{(0.8 \times 25) \times 12}{2.45} = 97.96$$

$$C_c = 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.43$$

$SR < C_c$ $\rightarrow$ Short col $\rightarrow$ failure by crushing

$$F_{cr} = 0.658^{F_y/F_e} F_y$$

$$= 0.658^{\frac{50}{29.83}} \times 50 = 24.79\ ksi$$

$$\therefore \phi P_u = \phi A_g F_{cr} = 0.9 \times 17.19 \times 24.79\ kip$$

$$= 383.53\ kip$$

$$F_e = \frac{\pi^2 E}{(SR)^2} = \frac{\pi^2 \times 29000}{(97.96)^2}$$

$$= 29.83\ ksi$$

Example 9 (L = 35')

$L = 35'$

W14X61 $\rightarrow$ $r_y = 2.45$ in, $A_g = 17.9$ in^2

Fixed hinge $\rightarrow$ $K = 0.8$

A992 $\rightarrow$ $F_y = 50$ ksi, $F_u = 65$ ksi

Capacity = ?

$$SR = \frac{KL}{r} = \frac{(0.8 \times 35) \times 12}{2.45} = 137.14$$

$$C_c = 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.43$$

$SR > C_c \rightarrow$ Long column, failure by buckling

$$F_{cr} = 0.877\, F_e$$

$$= 0.877 \times 15.22 = 13.35 \text{ ksi}$$

$$\therefore \phi P_n = \phi A_g F_{cr}$$

$$= 0.9 \times 17.9 \times 13.35 \text{ kip}$$

$$= 215.07 \text{ kip}$$

$$F_e = \frac{\pi^2 E}{(SR)^2}$$

$$= \frac{\pi^2 \times 29000}{(137.14)^2} \text{ ksi}$$

$$= 15.22 \text{ ksi}$$

Biaxial bending : Bends at both x-x & y-y axis

x-x axis : $SR = \frac{KL_x}{r_x}$

y-y axis : $SR = \frac{KL_y}{r_x}$

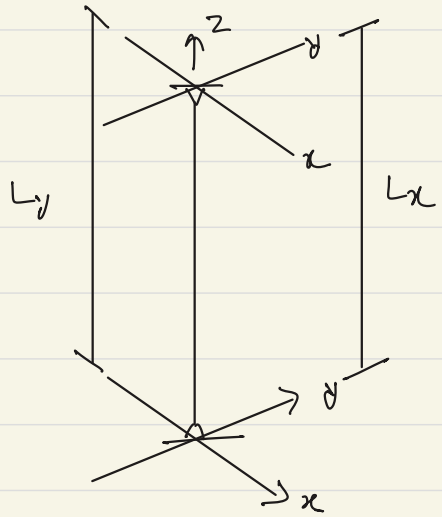
$r \uparrow$ → mats spread out

hollow pipe

resists buckling

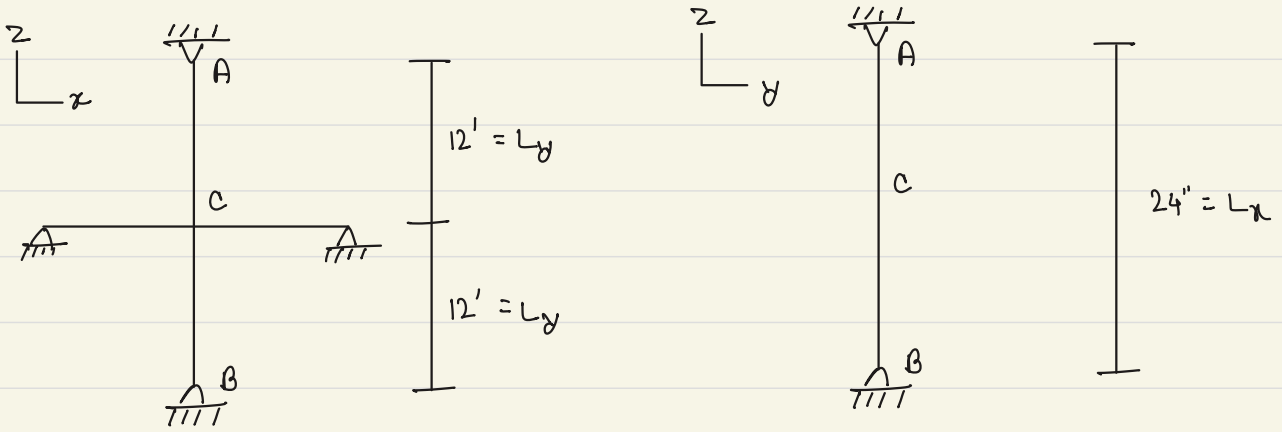
$SR \uparrow$ → Easy to buckle

Design w/ highest SR



Example 10

A36 → $F_y = 36$ ksi, $F_u = 58$ ksi ; $A_g = 11.5$ in^2, $r_x = 4.27$ in, $r_y = 1.98$ in



$$SR = \max\left[\frac{KL_{y1}}{r_y}, \frac{KL_{y2}}{r_y}, \frac{KL_x}{r_x}\right]$$

$$= \max\left[\frac{2\times12\times12}{1.98}, \frac{1\times24\times12}{4.27}\right] = \max[72.73, 67.45] = 72.73$$

$$C_c = 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{36}} = 133.68 > SR \rightarrow \text{short col} \rightarrow \text{crushing}$$

$$F_{cr} = 0.658^{F_y/F_e} F_y \qquad F_e = \frac{\pi^2 E}{(SR)^2} = \frac{\pi^2 \times 29000}{(72.73)^2} = 54.11 \text{ ksi}$$

$$= 0.658^{\frac{36}{54.11}} \times 36 = 27.25 \text{ ksi}$$

$$\therefore \phi P_n = \phi A_g F_{cr} = 0.9 \times 11.5 \times 27.25 = 282.04 \text{ kip}$$

Example 11 Design by trial & error method

$L = 25'$

Fixed hinge $\rightarrow K = 0.80$

$DL = 135^K$, $LL = 145^K$

A992 $\rightarrow F_y = 50$ ksi

W14 shape

$KL = 0.8 \times 25' = 20'$

$$C_c = 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.43$$

$$P_u = 1.2DL + 1.6LL = 1.2\times135 + 1.6\times145 = 394^K$$

Trial 1:

$$F_{cr} = \tfrac{2}{3} \times 50 = 33.3 \text{ ksi}$$

$$P_u = \phi P_n = \phi F_{cr} A_g \Rightarrow A_g = \frac{P_u}{\phi F_{cr}} = \frac{394}{0.9\times33.3}$$

$$= 13.15 \text{ in}^2 \qquad [W14\times48]$$

$$SR = \frac{KL}{r} = \frac{20\times12}{1.91} = 125.65 > C_c \qquad [\text{Slender col.}]$$

$$F_{cr} = 0.877 F_e \qquad F_e = \frac{\pi^2 E}{(SR)^2} = 18.13 \text{ ksi}$$

$$= 0.877 \times 18.13 = 15.9 \text{ ksi}$$

$$\phi P_n = 0.9\times14.1\times15.9 = 201.77 \text{ kip} < 394^K \qquad [\text{not OK}]$$

Trial 2:

$$F_{cr} = \frac{33.3+15.9}{2} = 24.6 \text{ ksi}$$

$$A_g = \frac{394}{0.9 \times 24.6} = 17.8 \text{ in}^2 \qquad [\text{W14} \times 61 \rightarrow A_g = 17.9 \text{ in}^2;\ r = 2.45 \text{ in}]$$

$$SR = \frac{20 \times 12}{2.45} = 97.96 \quad < C_c \rightarrow \quad \text{Stocky col}$$

$$F_{cr} = 0.658^{\frac{F_y}{F_e}} F_y \qquad F_e = \frac{\pi^2 E}{(SR)^2} = \frac{\pi^2 \times 29000}{97.96^2} \text{ ksi}$$

$$= 0.658^{\frac{50}{29.83}} \times 50 = 24.79 \text{ ksi} \qquad = 29.83 \text{ ksi}$$

$$\phi P_n = 0.9 \times 17.9 \times 24.79 = 399.37 \text{ kip} \qquad [> P_u = 394 \text{ kip}] \quad [\text{ok}]$$

Example 6.10.1

A992 → $F_y = 50$ ksi

Pinned-end → $K = 1.0$

$L = 16'$

$DL = 115^K$; $LL = 125^K$

W section

$KL = 16'$

$C_c = 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.43$

$P_u = 1.2DL + 1.6LL = 338^K$

Trial 1.

$F_{cr} = \frac{2}{3}50 = 33.3$ ksi

$A_g = \frac{338}{0.9 \times 33.3} = 11.28$ in^2 [W10X39 → $A_g = 11.5$ in^2, $r = 1.98$]

$SR = \frac{KL}{r} = 96.97 < C_c$

$F_{cr} = 0.658^{F_y/F_e} \times F_y = 25.14$ ksi $\qquad F_e = \frac{\pi^2 E}{(SR)^2} = 30.44$ ksi

$\phi P_n = 0.9 \times 11.5 \times 25.14 = 260$ ksi [N.G]

Trial 2:

$$F_{cr} = \frac{33.3 + 25.14}{2} = 29.22 \text{ ksi}$$

$A_g = 12.85 \text{ in}^2$ [W21X44^C → $A_g = 13 \text{ in}^2$, $r = 1.26$]

$SR = 152.38 > C_c$

$F_{cr} = 0.877 F_e$ $F_e = 12.34 \text{ ksi}$

$= 10.82 \text{ ksi}$

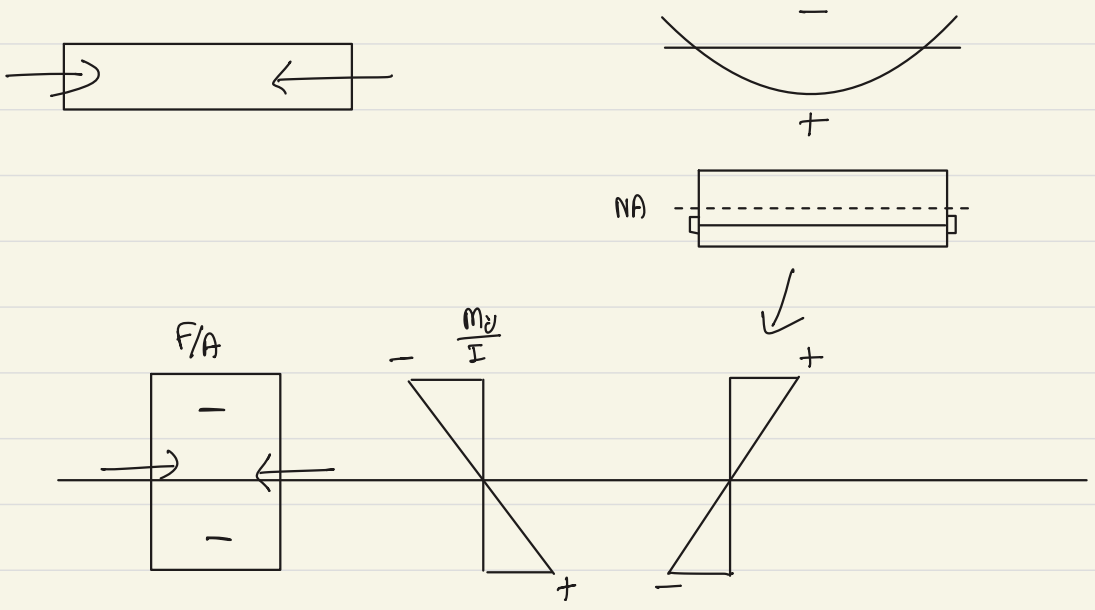
$\therefore \phi P_n = 126 \text{ ksi}$ (NG)

Trial 3:

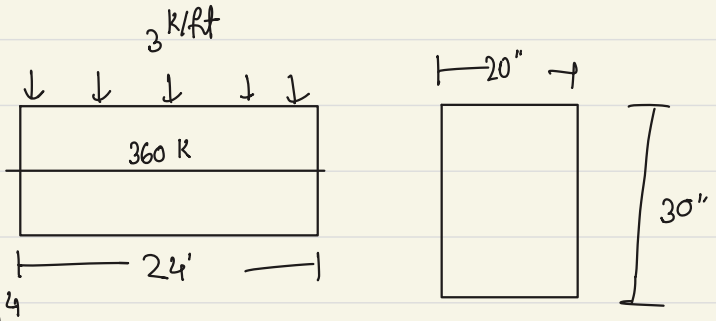
$F_{cr} = 20 \text{ ksi}$

$A_g = 18.78$ [

P495 [no formula in xm]



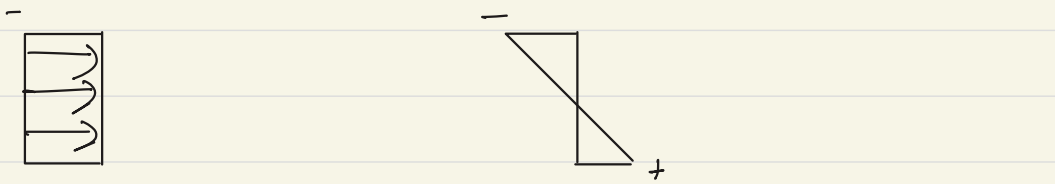
Example 27:



$F = 360$ kip

$A = bh = 20 \times 30 = 600$ in^2

$I = \frac{bh^3}{12} = \frac{20 \times 30^3}{12} = 45{,}000$ in^4



$F/A = -\frac{360}{600} = -0.6$ ksi

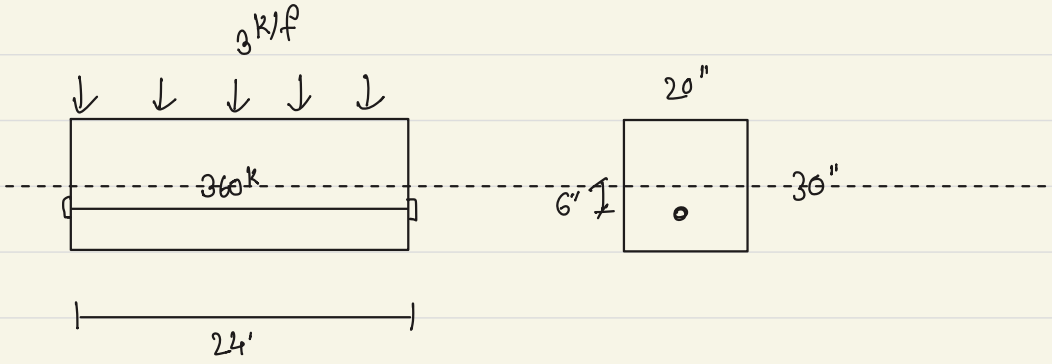
$\frac{M_y}{I} = \frac{\frac{3}{12} \times (24 \times 12)^2}{\frac{8}{45000}} \times 15 = 0.864$ k-in

$f = \frac{F}{A} \pm \frac{M_y}{I}$

$f_{top} = -0.6 - 0.864 = -1.464$ ksi

$f_{btm} = -0.6 + 0864 = +0.264$ ksi

Example 26:



$A = bh = 20 \times 30 = 600 \text{ in}^2$

$I = \frac{bh^3}{12} = \frac{20 \times 30^3}{12} = 45000 \text{ in}^4$

$F/A = \frac{-360}{600} = -0.6 \text{ ksi}$

$\frac{M_x}{I} = 0$

$\frac{F e_y}{I} = \frac{360 \times 6 \times 15}{45000} = 0.72 \text{ ksi}$

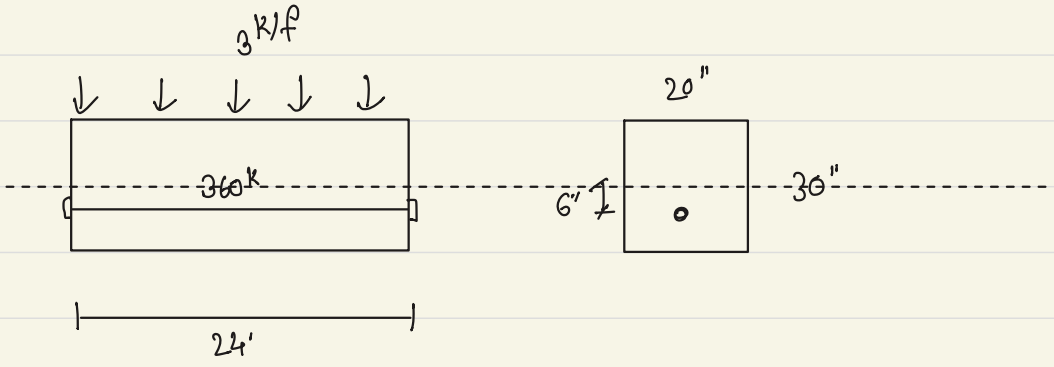
$f_{top} = -0.6 + 0.72 = 0.12 \text{ ksi}$

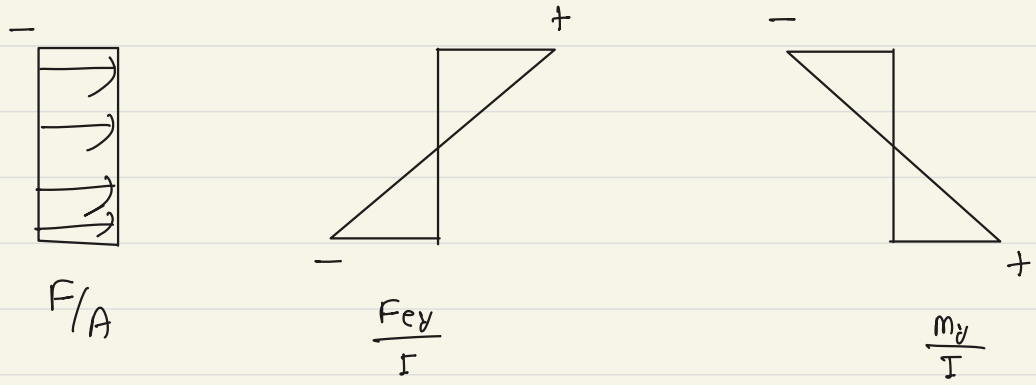
$f_{btm} = -0.6 - 0.72 = -1.32 \text{ ksi}$

Example 25:

$$A = bh = 20 \times 30 = 600 \text{ in}^2$$

$$I = \frac{bh^3}{12} = \frac{20 \times 30^3}{12} = 45{,}000 \text{ in}^4$$





$$F/A = \frac{-360}{600} = -0.6 \text{ ksi}$$

$$\frac{Fey}{I} = \frac{360 \times 6 \times 15}{45000} = 0.72 \text{ ksi}$$

$$\frac{M_y}{I} = \frac{\left(\frac{3}{12} \times (24 \times 12)^2\right) \times \frac{1}{8} \times 15}{45000} = 0.864 \text{ k-in}$$

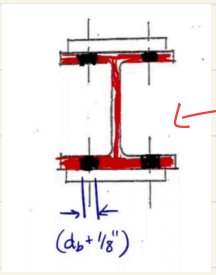
$$\therefore f_{top} = -0.6 + 0.72 - 0.864 = -0.744 \text{ ksi}$$

$$f_{btm} = -0.6 - 0.72 + 0.864 = -0.456 \text{ ksi}$$

Rough / Final

21/04

Design of steel structures



A_g = red + black

A_n = A_g - black

Tension member:

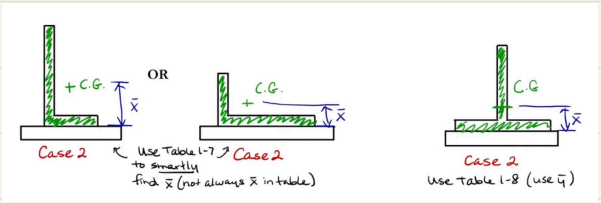
Gross-area (A_g): Total available cross-sectional area

Net-area (A_n): A_g - Area(hole)

Shear lag: (specially notable on angles (L-shape)) All parts of a section may not be uniformly connected. As a result, stress distribution vary.

To account for this, use shear lag factor (U)

Eff. area, $A_e = U A_n$; $U = \begin{cases} 1 - \frac{\bar{x}}{l} & \leftarrow \text{dist btwn first to last bolt} \\ 1.0 & \text{for plate} \end{cases}$



LRFD: Load resistance and factor design

$$\phi P_u \leq \phi P_n$$

$\phi P_n = 0.90 F_y A_g$ → Yielding criteria

$= 0.75 F_u A_e$ → Fracture criteria

P375, Ex 1 L6x6x ½, ⅝" bolt, c.g. 1.67", eff. area

$A_g = 5.77 \text{ in}^2$

$A_n = 5.77 - \frac{1}{2}\left[\frac{5}{8} + \frac{1}{8}\right] \times 2 = 5.02 \text{ in}^2$

$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{1.67}{6} = 0.72$

$\therefore A_e = U \times A_n = 3.61 \text{ in}^2$

P376, Ex. 2 Plate ½x5, ⅝" bolt A36 steel; det^n tensile stren

$A_g = \frac{1}{2} \times 5 = 2.5 \text{ in}^2$

$A_n = 2.5 - \frac{1}{2}\left[\frac{5}{8} + \frac{1}{8}\right] \times 2 = 1.75 \text{ in}^2$

$U = 1; \quad A_e = 1.75 \text{ in}^2$

A36 steel

$F_y = 36$ ksi

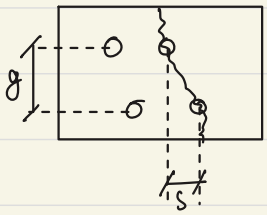
$F_u = 58$ ksi

Design

- Yield stren = $\phi F_y A_g = 0.9 \times 36 \times 2.5 = 81$ ksi
- Fracture stren = $\phi F_u A_e = 0.75 \times 58 \times 1.75 = 76.13$ ksi

Staggered fasteners:

Add additional $\left(\frac{s^2}{4g} \times t\right)$ for each staggering for A_n



Ex. 4 $A_g = 3.81\ in^2$

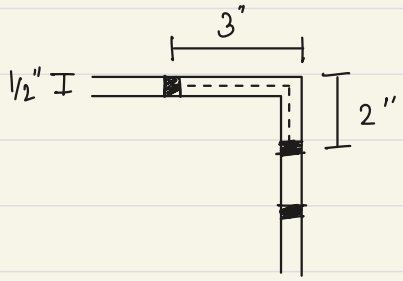
For line abe: $A_n = 3.81 - \frac{7}{16}\left[\frac{5}{8} + \frac{1}{8}\right] \times 1 = 3.48\ in^2$

For line abcd: $A_n = 3.81 - \frac{7}{16}\left[\frac{5}{8} + \frac{1}{8}\right] \times 2 + \left[\frac{2^2}{4 \times 3} \times \frac{7}{16}\right] = 3.30\ in^2$

---

g to be considered along the center line of holes

$g = 3 + 2 - 1/2"$



Ex 5

$A_g = 2.89\ in^2$

For line abc: $A_n = 2.89 - 2\left[\frac{5}{16}\left(\frac{3}{4}+\frac{1}{8}\right)\right] = 2.34\ in^2$

For line debc: $A_n = 2.89 - 3\left[\frac{5}{16}\left(\frac{3}{4}+\frac{1}{8}\right)\right] + \frac{1.5^2}{4\times(2.25+2-5/16)}\,5/16$

$= 2.11\ in^2$

$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.756}{1.5+1.5}\ 0.748$

$\therefore A_e = U A_n = 1.58\ in^2$

$\therefore$ Design capacity based on yielding $= \phi F_y A_g = 0.9 \times 36 \times 2.89 = 93.64^K$

fracture $= \phi F_u A_e = 0.75 \times 58 \times 1.58 = 68.73^K$

Servic ld $= 1.2DL + 1.6LL$

$= 1.2 \times 25 + 1.6 \times 25 = 70^K$

$\therefore$ Not adaquate

Ex 8 Select angle tension member

$L = 15'$

$DL = 35^K$; $LL = 70^K$; $WL = 60^K$;

A36 steel → $F_y = 36$ ksi; $F_u = 58$ ksi

3/4" bolt

$t_{min} = 1/4$ in

Demand ld,

$$P_u = 1.2DL + 1.6LL = 1.2\times35 + 1.6\times70 = 154^K$$

$$1.2DL + 0.5LL + 1WL = 1.2\times35 + 0.5\times70 + 1\times60 = 137^K$$

Required gross area,

Based on yielding: $P_u = \phi F_y A_g \Rightarrow A_g = \dfrac{P_u}{\phi F_y}$

$$\Rightarrow A_{g1} = \frac{154}{0.9\times36} = 4.75 \text{ in}^2$$

Based on fracture: $P_u = \phi F_u A_e$

$$\Rightarrow P_u = \phi F_u U (A_g - 1.75t)$$

$$\Rightarrow A_g = \frac{P_u}{\phi F_u U} + 1.75t$$

$$\Rightarrow A_{g2} = \frac{154}{0.75\times58\times0.8} + 1.75t$$

$$= 4.425 + 1.75t$$

$A_e = UA_n$

$A_n = A_g - t\left[\frac{3}{4} + \frac{1}{8}\right]\times2$

$\therefore A_g - 1.75t$

$U = 0.8$ when 4 or more bolts in one row

Minimum radius of gyration:

$$r_{min} = \frac{L}{300} = \frac{15\times12}{300} = 0.6 \text{ in}$$

Trial #	t	A_{g1}	A_{g2}	n_2	Section
1.	1/4	4.75	4.86		None
2.	5/16	4.75	4.97		None
3.	3/8	4.75	5.08		None
4.	7/16	4.75	5.19	0.867	L8x4x7/16

Procedure: iterate through L-shape's third dimension matching for varying thickness

If the first found shape of such thickness has area less than required, move to next thickness

Thickness to try: 1/4, 5/16, 3/8, 7/16

or infer from table

Compression member:

Eff. len = KL

Slenderness ratio, $S.R = \frac{KL}{r}$; $C_c = 4.71\sqrt{\frac{E}{F_y}}$

Fail by crushing $\longleftrightarrow$ $S.R < C_c \rightarrow$ stocky col

$F_{cr} = 0.658^{F_y/F_e} F_y$

Fail by buckling $\longleftrightarrow$ $S.R > C_c \rightarrow$ slender col

$F_{cr} = 0.877\, F_e$

$F_e = \frac{\pi^2 E}{(SR)^2}$

Ex. 9 $L = 25'$

W14 X 61 $\rightarrow$ $r = 2.45$ in, $A_g = 17.9$ in^2

Fixed - hinge $\rightarrow$ $K = 0.80$

A992 $\rightarrow$ $F_y = 50$ ksi; $F_u = 65$ ksi

$$SR = \frac{KL}{r} = \frac{0.8 \times 25 \times 12}{2.45} = 98$$

$$C_c = 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.4$$

$\therefore$ stocky col; failure by crushing

$$F_e = \frac{\pi^2 E}{(SR)^2} = \frac{\pi^2 \times 29000}{(98)^2} = 29.8 \text{ ksi}$$

$$F_{cr} = 0.658^{F_y/F_e} F_y = 24.77 \text{ ksi}$$

$$\phi P_n = \phi F_{cr} A_g = 0.9 \times 24.77 \times 17.9 = 400 \text{ kip}$$

$L_{new} = 35'$

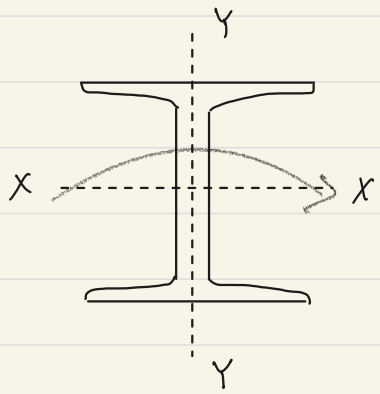
$$SR = \frac{35 \times 12 \times 0.8}{2.45} = 137.14 > C_c$$

$\therefore$ slender col

$$F_{cr} = 0.877\, F_e = 0.877 \times \frac{\pi^2 \times 29000}{(137.14)^2} = 13.35 \text{ ksi}$$

$$\therefore \phi P_n = 0.9 \times 13.35 \times 17.9 = 215 \text{ kip}$$

Biaxial loading:

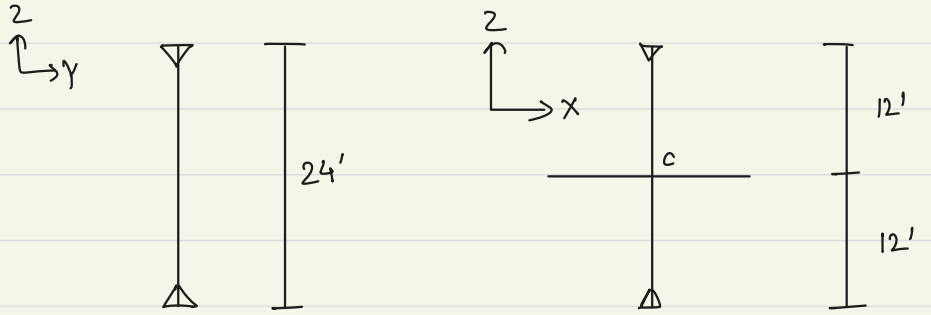


Y–Y → weak axis $SR = \frac{KL_y}{r_y}$

buckles in this axis

X–X → strong axis $SR = \frac{KL_x}{r_x}$

Ex. 10



$r_x = 4.27$ in; $r_y = 1.98$ in; $A_g = 11.50$ in^2

Steel A36 → $F_y = 36$ ksi; $F_u = 58$ ksi

$$SR = SR_{max} = \max\left[\frac{1\times 24}{4.27}, \frac{1\times 12}{1.98}\right]\times 12 = \max(67.45, 72.73)$$

$$C_c = 4.27\sqrt{\frac{E}{F_y}} = 4.27\times\sqrt{\frac{29000}{36}} = 121.19$$

∴ Stocky col

$$F_{cr} = 0.658^{F_y/F_e} F_y$$

$$= 0.658^{36/54.4} \times 36 = 27.25 \text{ ksi}$$

$$F_e = \frac{\pi^2 E}{(SR)^2} = \frac{\pi^2 \times 29000}{(72.73)^2}$$

$$= 54.11 \text{ ksi}$$

$$\therefore \phi P_n = 0.9 \times 27.25 \times 11.50 = 282 \text{ kip}$$

Ex. 11 $L = 25'$;

Fixed-hinge $\rightarrow$ $K = 0.80$

$DL = 135^K$, $LL = 145^K$

A992 $\rightarrow$ $F_y = 50$ ksi, $F_u = 65$ ksi

} Select W14 shape

$KL = 0.8 \times 25 = 20'$

$C_c = 4.77\sqrt{\frac{E}{F_y}} = 4.77\sqrt{\frac{29000}{50}} = 114.9$

$P_u = 1.2DL + 1.6LL = 1.2 \times 135 + 1.6 \times 145 = 394^K$

Trial 1:

$F_{cr,assumed} = \frac{2}{3} \times 50 = 33.3$ ksi

$P_u = \phi P_n = \phi A_g F_{cr} \Rightarrow A_g = \frac{P_u}{\phi F_{cr}}$

$= \frac{394}{0.9 \times 33.3} = 13.15 \text{ in}^2$

Select W14X48 $\rightarrow$ $A_g = 14.1 \text{ in}^2$, $r_y = 1.91$

$\therefore SR = \frac{KL}{r} = \frac{20 \times 12}{1.91} = 125.7 > C_c$

$\therefore F_{cr} = 0.877 F_e = 0.877 \times \frac{\pi^2 \times 29000}{(125.7)^2} = 15.89$ ksi

$\phi P_n = 0.9 \times 14.1 \times 15.89 = 202$ kip $< P_u$

Trial 2:

$$F_{cr} = \frac{33.3 + 15.89}{2} = 24.6 \text{ ksi}$$

$$A_g = \frac{P_u}{\phi F_{cr}} = \frac{394}{0.9 \times 24.6} = 17.8 \text{ in}^2$$

Sel W14X61 $\rightarrow A_g = 17.9 \text{ in}^2$, $r_y = 2.45$ in

$\therefore SR = 97.96$

$$F_{cr} = 0.658^{F_y/F_e} F_y$$

$$F_e = \frac{\pi^2 \times 29000}{(97.96)^2}$$

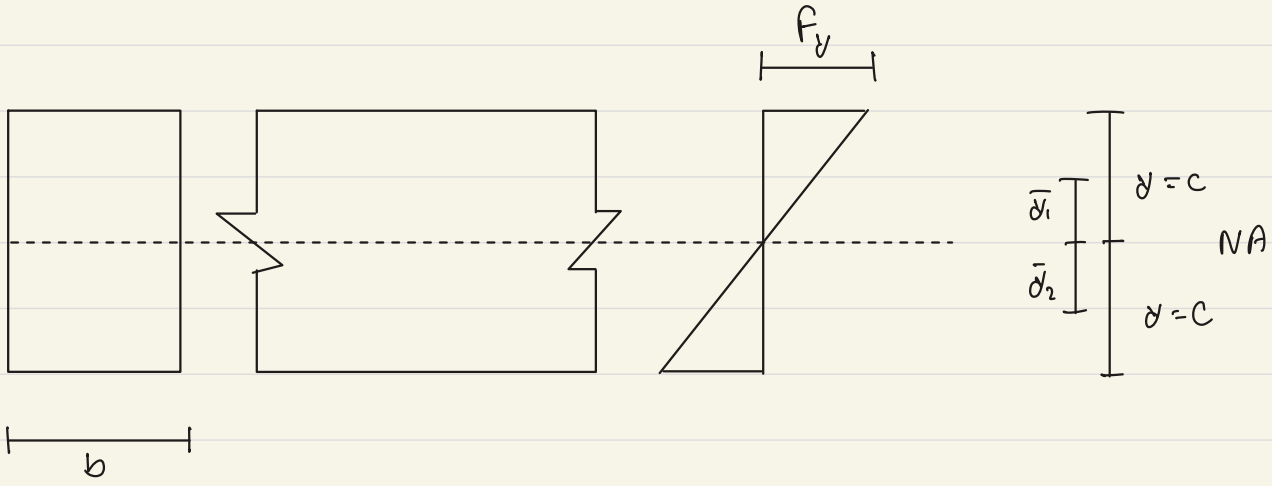
$$= 0.658^{\frac{50}{29.83}} \times 50 = 24.79 \text{ ksi}$$

$\therefore 29.83$ ksi

$$\therefore \phi P_n = 0.9 \times 24.79 \times 17.9 = 399 \text{ ksi} > P_u$$

Plastic moment

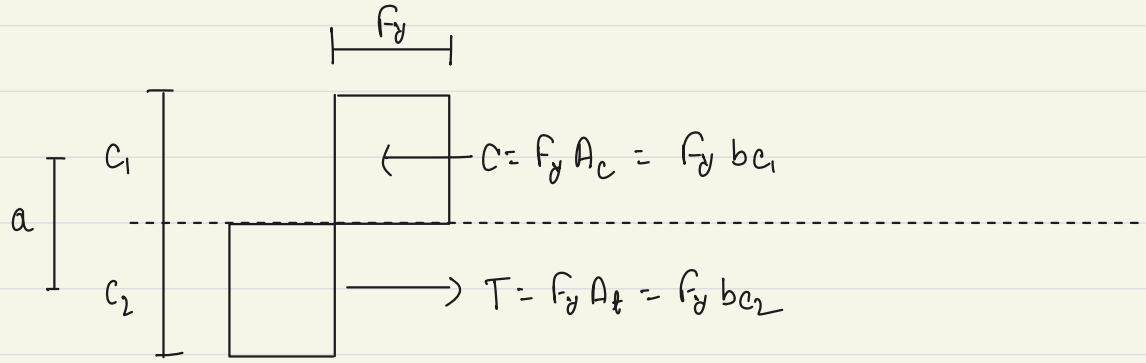
Yield moment: moment at which extreme edge fiber reaches its yield point



$$f_y = \frac{Mc}{I} \Rightarrow f_y = \frac{M}{S} ; \quad S = \frac{I}{c}$$

$$\Rightarrow M = f_y S$$

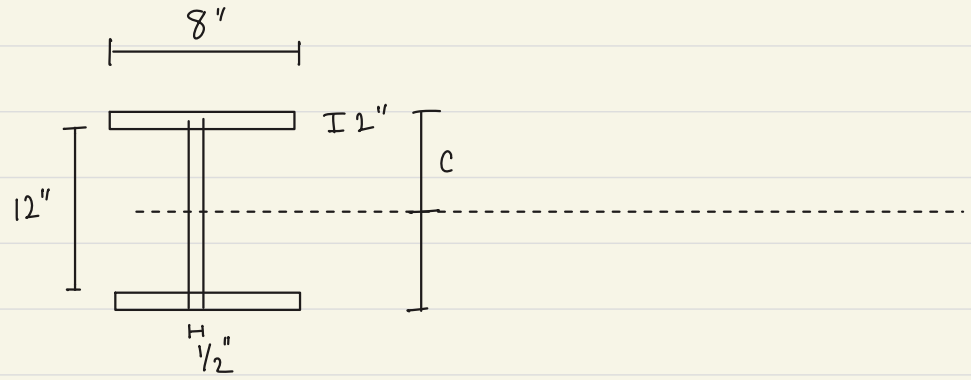
Plastic moment: moment at which whole section becomes plastic.



$$M_p = Ca = Ta$$

$$\Rightarrow f_y A_c a = f_y A_t a$$

Ex. 14



$S = \frac{I}{C}$

$= \frac{749.33}{7} = 107 \text{ in}^3$

$C = 1" + 6" = 7"$

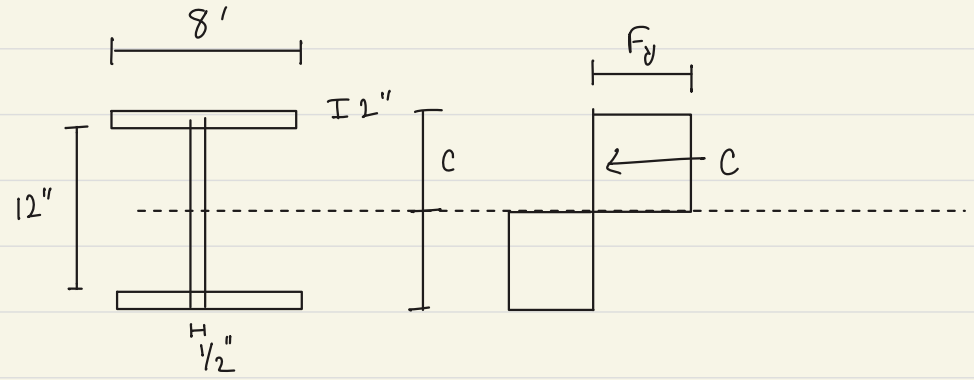
$I = \frac{0.5 \times 12^3}{12} + 2\left[\frac{8 \times 1^3}{12} + (8 \times 1) \times (6 + 0.5)^2\right]$

$= 749.33 \text{ in}^4$

$\therefore M_y = F_y S = 50 \times 107 = 5350 \text{ K-in}$

$= 446 \text{ K-ft}$

Ex. 15



$\therefore M_p = Ca = C \times 2\bar{y}_1$

$= 550 \times 5.54 \times 2 = 6094 \text{ Kip-in}$

$C = F_y \times (6 \times 0.5 + 8)$

$= 550 \text{ Kip}$

$\bar{y}_1 = \frac{3 \times 6 \times 0.5 + 6.5 \times 8}{6 \times 0.5 + 8}$

$= 5.54 \text{ in}$

Ex. 15 (alt)

$S = A_c\, a = 121.88 \text{ in}^3$

$A_c = 0.5 \times 6 + 8 = 11 \text{ in}^2$

$a = 2\bar{y} = 11.08$

$\therefore M_p = F_y S = 50 \times 121.88 = 6094 \text{ k-in}$

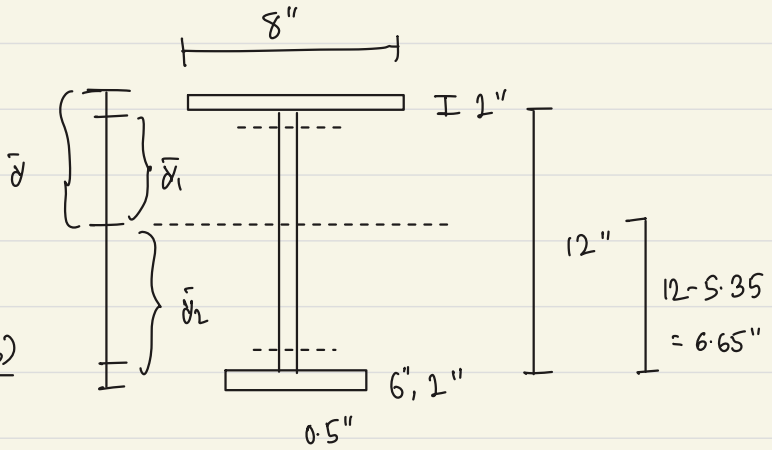
Ex. 16

$S = A_c a$

Moment arm:

$$\bar{y} = \frac{(0.5 \times 8 \times 1) + (7 \times 12 \times 0.5) + (13.5 \times 6)}{8 + 6 + 6}$$

$= 6.35$ in (from top)



$$\bar{y}_1 = \frac{\left(\frac{5.35}{2} \times 5.35 \times 0.5\right) + \{(5.35 + 0.5) \times 8\}}{5.35 \times 0.5 + 8} = 5.05 \text{ in}$$

$$\bar{y}_2 = \frac{(6.65 \times 0.5 \times 6.65 \times 0.5) + \{(6.65 + 0.5) \times 6\}}{6.65 \times 0.5 + 6} = 5.79 \text{ in}$$

$\therefore a = \bar{y}_1 + \bar{y}_2 = 10.84 \text{ in}$

Section modulus:

$S = A_c a = (5.35 \times 0.5 + 8 \times 1) \times 10.84$

$= 115.72 \text{ in}^3$

$\therefore M_p = F_y S$

$= 5786 \text{ k-in}$

Local buckling strength

Classification of steel section based on width to thickness ration (λ):

$\lambda \uparrow \rightarrow \uparrow$ slender $\rightarrow$ greater chance of local buckling

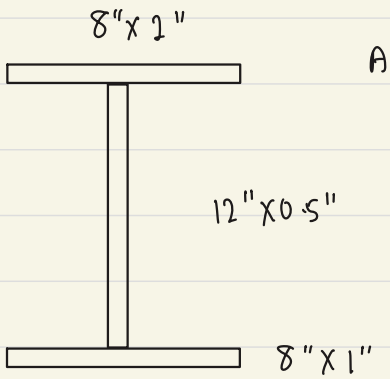
if $\lambda < \lambda_p \rightarrow$ compact $\rightarrow$ full mat stren

$\lambda_p < \lambda < \lambda_r \rightarrow$ non-compact $\rightarrow$ may reach elastic/inelastic buckling prior to plastic moment

$\lambda > \lambda_r \rightarrow$ slender $\rightarrow$ def. reaches buckling

	λ_p	λ_r	
Flange →	0.38	1	$\times \sqrt{\frac{E}{F_y}}$
web →	3.76	5.7	

Ex 17



Flange:

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 9.15$$

$$\lambda_r = 1\sqrt{\frac{E}{F_y}} = 24.08$$

$$\lambda = \frac{8}{2x1} = 4 < \lambda_p \rightarrow \text{compact}$$

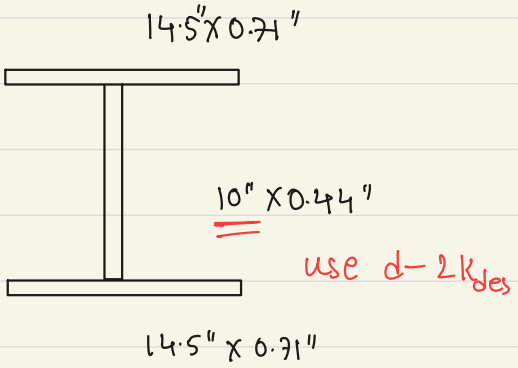
Web:

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 90.55$$

$$\lambda_r = 5.7\sqrt{\frac{E}{F_y}} = 137.27$$

$$\lambda = \frac{12}{0.5} = 24 < \lambda_p \rightarrow \text{compact}$$

Ex. 18 W14 X 90



Flange:

$$\lambda = \frac{14.5}{0.71 \times 2} = 10.21$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 9.15$$

$$\lambda_r = \sqrt{\frac{E}{F_y}} = 24.08$$

$\therefore$ non-compact

$$\sqrt{\frac{E}{F_y}} = 24.08$$

web:

$$\lambda = \frac{10}{0.44} = 22.73$$

$\lambda_p = 3.76 \times \cdots = 90.5$

$\lambda_r = 5.7 \times \ldots = 137.26$

$\}$ $\therefore$ compact

Ex. 19 W16 x 31 of A992

Demand:

$DL = 450 + SW = 450 + 31 = 481$ lb/ft

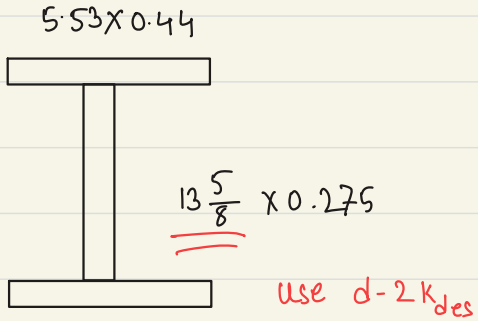
$LL = 550$ lb/ft

$\therefore w_u = 1.20L + 1.6LL = 1460$ lb/ft ; $M_u = \frac{wL^2}{8} = 164.25$ k-ft

$= 1.46$ K/ft

Strength:

Flange check:



$\lambda = \frac{6.53}{0.44 \times 2} = 6.28$

$\lambda_p = 0.38 \times \cdots = 9.15$

$\therefore$ compact

$\sqrt{\frac{E}{F_y}} = 24.08$

Web check:

$\lambda = \frac{13\ 5/8}{0.275} = 49.5$

$\lambda_p = 3.76 \times \cdots = 90.54$

$\therefore$ compact

$\therefore M_p = F_y Z_x$

$= 50 \times 54 = 2700$ k-in $= 225$ k-ft

$\therefore \phi M_p = 0.9 M_p = 202.5$ k-ft

Ex. 20 W14x90, A992

DL = 600 + 90 = 690 lb/ft

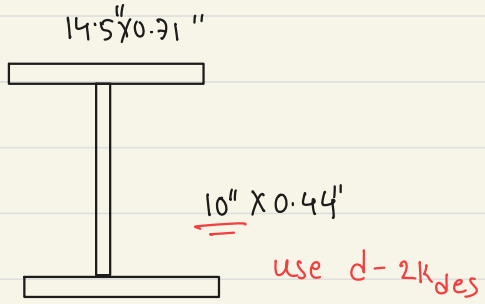
LL = 1200 lb/ft

Demand: $w_u = 1.2DL + 1.6LL = 2748$ lb/ft

$= 2.748$ K/ft

$\therefore M_u = \frac{w_u L^2}{8} = 696$ K-ft

Capacity:



Flange check:

$\lambda = \frac{14.5}{2x0.71} = 10.21$

$\lambda_p = 0.38x\sqrt{\quad} = 9.15$

$\lambda_r = 2\sqrt{\quad} = 24.08$

$\sqrt{\frac{E}{F_y}} = 24.08$

$\therefore$ non-compact

web check:

$\lambda = \frac{10}{0.44} = 22.73$

$\lambda_p = 3.76\sqrt{\quad} = 90.54$

$\therefore$ compact

$$\therefore M_n = M_p - \left[(M_p - 0.7 F_y S_x) \frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right]$$

$$= F_y Z_x - \left[(F_y Z_x - 0.7 F_y S_x) \frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right]$$

$$= 50 \times 157 - \left[(50 \times 157 - 0.7 \times 50 \times 143) \frac{10.21 - 9.15}{24.08 - 9.15} \right] \text{ k-in}$$

$$= 7850 - 202 \text{ k-in}$$

$$= 7648 \text{ k-in} = 637.33 \text{ k-ft}$$

$$\therefore \phi M_n = 0.9 M_n = 573 \text{ k-ft} < (M_u = 696)$$

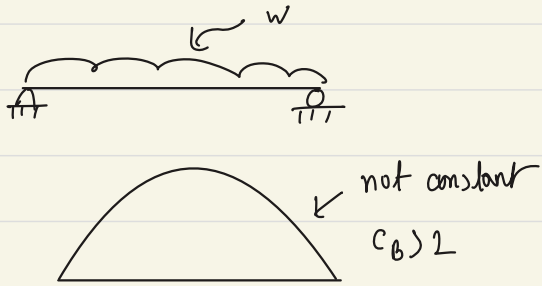
$\therefore$ not adequate

$$M_n = M_p - (M_p - 0.7 M_y) \frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}$$

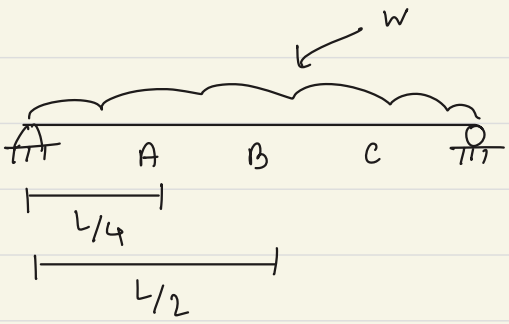
Moment gradient

Moment is not same across whole span of beam portion subjected to less bending, helps hold up high stressed portion $\longrightarrow$ $C_B > 1$

If constant bending $\rightarrow$ $C_B = 1$



Ex. 22

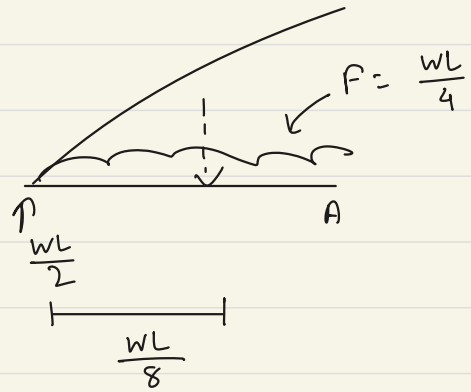


$$M_A = \frac{WL}{2} \times \frac{L}{4} - \frac{WL}{4} \times \frac{L}{8}$$

$$= \frac{WL^2}{8} - \frac{WL^2}{32} = \frac{3}{32} WL^2$$

$$M_B = \frac{WL^2}{8} = M_{max}$$

$$M_C = \frac{3}{32} WL^2$$

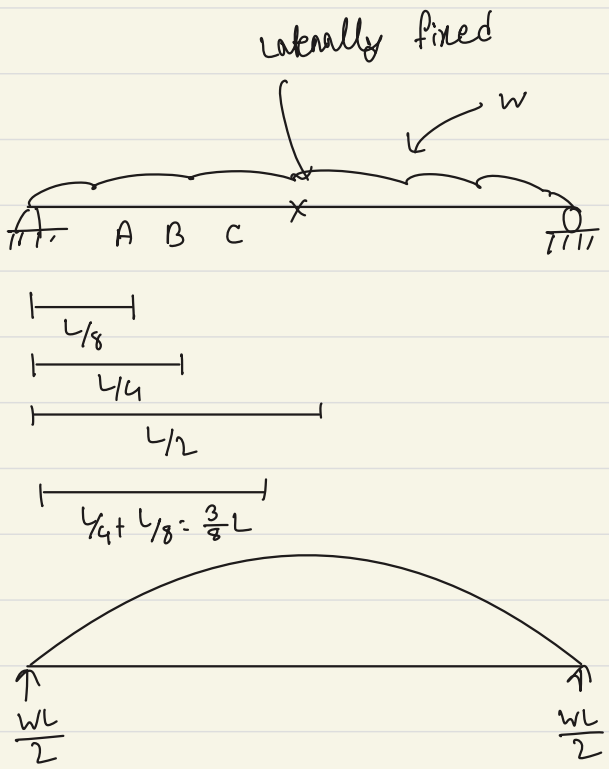


$$\therefore C_B = \frac{12.5\, M_{max}}{2.5\, M_{max} + 3 M_A + 3 M_C + 4 M_B}$$

$$= \frac{12.5 \times 1/8 \; WL^2}{\left(2.5 \times 1/8 + 3\, 3/32 + 3\, 3/32 + 4\, 1/8\right) WL^2}$$

$$= 1.14$$

Ex. 22



$$M_A = \frac{WL}{2} \times \frac{L}{8} - \frac{WL}{8} \times \frac{WL}{16} = \frac{7}{128} WL^2$$

$$M_B = \frac{WL}{2} \times \frac{L}{4} - \frac{WL}{4} \times \frac{L}{8} = \frac{3}{32} WL^2$$

$$M_C = \frac{WL}{2} \times \frac{3L}{8} - \frac{W3L}{8} \times \frac{3L}{16} = \frac{15}{128} WL^2$$

$$M_{max} = \frac{WL^2}{8}$$

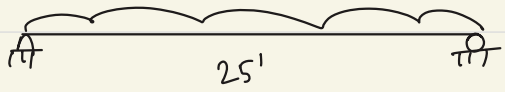
$$\therefore C_B = \frac{12.5\, M_{max}}{2.5 M_{max} + 3M_A + 3M_C + 4M_B}$$

$$= \frac{12.5 \times 1/8}{2.5 \times 1/8 + 3 \times 7/128 + 3\frac{15}{128} + 4\frac{3}{32}}$$

$$= 1.3$$

Beam design

Ex. 24 Lightest section : A992



DL = 1.2 K/ft ; LL = 2 K/ft

Demand :

$$w_u = 1.2DL + 1.6LL = 1.2(1.2) + 1.6 \times 2 = 4.64 \text{ K/ft}$$

$$m_u = \frac{wL^2}{8} = 362.5 \text{ K/ft}$$

Trial 2: W10X77 ($\phi_b m_p = 366$ K-ft)

$$w_u = 1.2(1.2 + 0.0.077) + 1.6 \times 2 = 4.73 \text{ K/ft}$$

$$m_u = \frac{wL^2}{8} = 369.5 > \phi_b m_p$$

Trial 2: W18X50 ($\phi_b m_p = 379$ K-ft)

$$w_u = 1.2(1.2 + 0.05) + 1.6 \times 2 = 4.7 \text{ K/ft}$$

$\therefore m_u = 367.2$ K-ft [OK]

Deflection due to LL:

$$D_{max} = \frac{L}{360} = \frac{25\times12}{360} = 0.83 \text{ in}$$

$$D = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384}\times\frac{(2/12)(25\times12)^4}{29000\times800}$$

$$= 0.76'' < 0.83 \text{ in [OK]}$$

$I = 800 \text{ in}^4$

Deflection due to both LL & DL

$$D_{max} = \frac{L}{240} = 1.25 \text{ in}$$

$$\Delta = \frac{5}{384}\times\frac{wL^4}{EI} = \frac{5}{384}\times\frac{(1.2+0.05+2)\times\frac{1}{12}(25\times12)^4}{29000\times800}$$

$$= 1.23 \text{ in} < 1.25 \text{ in} \quad [OK]$$

Shear check:

$$V_u = \frac{w_u L}{2} = \frac{4.7\times25}{2} = 58.75 \text{ kip}$$

$$\phi V_n = 192 \text{ kip} \quad (OK)$$

$\therefore$ W18X50

Bearing strength:

Criteria: $R_n = 1.2 F_u l_c t \leq 2.4 d t F_u$

$l_c \rightarrow$ clean spacing of bolt (on to the end)

$l_e \rightarrow$ center to end of member

$R_n \rightarrow$ capacity

Spacing criteria: $l_c \geq 2.67 d$ ..... only in here c/c spacing

$l_e \geq (d + 0.25'')$

Check → tension member bearing stren

connecting " " "

bolt shear stren

Example 25

Tension member: $5" \times 1/2"$

Plate: $3/8"$

Demand:

$$P_u = 1.2DL + 1.6LL = 1.2 \times 15 + 1.6 \times 45 = 90 \text{ kip}$$

Capacity:

Spacing check! $2.67d = 2.67 \times \frac{3}{4} = 2\text{in} < 2.5"$ [ok]

$d + 0.25 = 1" < 1.25"$ [OK]

Hole dia: $3/4 + 1/16 = 0.813\text{in}$

Tensim member:

Edge bearing:

$$R_n = 1.2 F_u l_c t \leq 2.4 F_u d t \qquad l_c = 1.25 - \frac{0.813}{2} = 0.844\text{in}$$

$$= 1.2 \times 58 \times 0.844 \times 1/2 \leq 2.4 \times 58 \times 3/4 \times 1/2$$

$$= 29.37 < 52.2$$

$$\therefore 29.37 \text{ kip / bolt}$$

Innen bearing: $R_n = 1.2 F_u l_{ct} \leq 2.4 F_u d t$ | $l_c = 2.5 - 0.813 = 1.687\text{in}$

$$= 1.2 \times 58 \times 1.687 \times 1/2 \leq 2.4 \times 58 \times \tfrac{3}{4} \times \tfrac{1}{2}$$

$$= 58.7 \leq 52.2 : 52.2 \text{ kip/bolt}$$

∴ Bearing on tension member = $2 \times 29.37 + 2 \times 52.2 = 163$ kip > 90 kip

Bearing on plate:

Edge: $R_n = 1.2 F_u l_c t \leq 2.4 F_u d t$

$= 1.2 \times 58 \times 0.844 \times \frac{3}{8} \leq 2.4 \times 58 \times \frac{3}{4} \times \frac{3}{8}$

$\therefore 22.02 \leq 39.15$

$= 22.02$ kip/bolt

$l_c = 1.25 - \frac{0.813}{2} = 0.844$ in

Inner: $R_n = 1.2 F_u l_c t \leq 2.4 F_u d t$

$= 1.2 \times 58 \times 1.687 \times \frac{3}{8} \leq 2.4 \times 58 \times \frac{3}{4} \times \frac{3}{8}$

$= 44 \leq 39.15$

$= 39.15$ kip / bolt

∴ $R_n = 2 \times 22.02 + 39.15 \times 2 = 122.34$ kip ≥ 90 kip

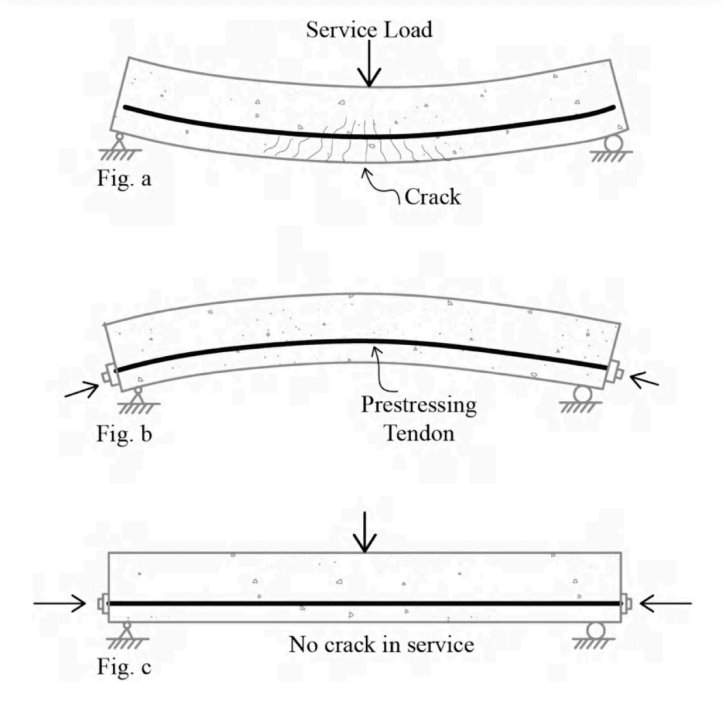
Shear stren of bolt:

$\phi R_n = \phi_n F_n A_b = 0.75 \times 4 \times 48 \times \frac{\pi \times 0.75^2}{4} = 63.6$ kip < 90 kip [not OK]

∴ not adequate

Prestressed concrete:

- Figure 9.69(a): Regular reinforced concrete members are weak in resisting tension but strong in resisting compression. Crack develops on the tension side due to service loads even though reinforcements are present there.
- Figure 9.70(b): To prevent excessive tension at the service period, concrete is compressed at the construction stage by steel wires known as tendon. The term *pre* refers to this stage, which comes earlier of service period of the beam, and the term stress is referred to the applied compression by the tendon.
- Figure 9.71(c): When this prestressed beam enters into service period, in order to develop cracks, the preapplied compression must be overcome by the tension due to service load.
- As a result, prestressed members deflect less, carry a greater load, or span a greater distance than a conventionally reinforced member of the same size, proportion, and weight.
- There are two types of prestressing: pretensioning and posttensioning.



9.8.1 PRETENSIONING

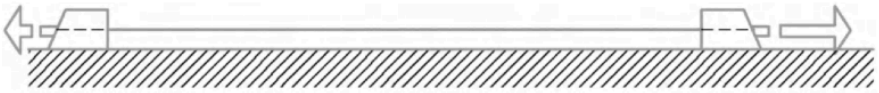


Fig. a



Fig. b

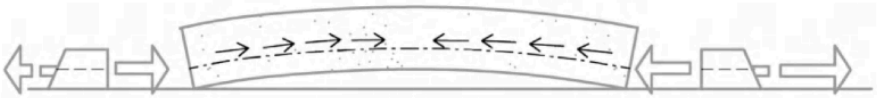


Fig. c

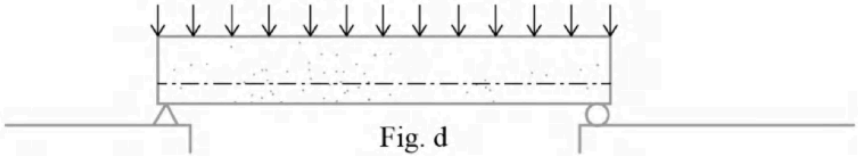


Fig. d

- *Pretensioning* prestresses a concrete member by stretching the reinforcing tendons before the concrete is cast.
- Figure 9.72(a): Steel tendons are first stretched across the casting bed between two abutments until a predetermined tensile force is developed.
- Figure 9.72(b): Concrete is then cast in formwork around the stretched tendons and fully cured. The tendons are placed eccentrically in order to reduce the maximum compressive stress to that produced by bending alone.
- Figure 9.72(c): When the tendons are cut or released, the tensile stresses in the tendons are transferred to the concrete through bond stresses. The eccentric action of the prestressing produces a slight upward curvature or camber in the member.
- Figure 9.72(d): The deflection of the member under loading tends to equalize its upward curvature.

9.8.1 PRETENSIONING

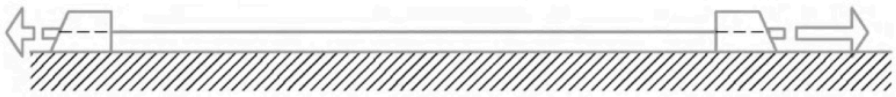


Fig. a



Fig. b

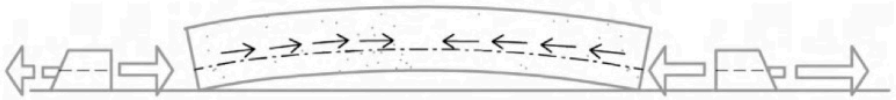


Fig. c

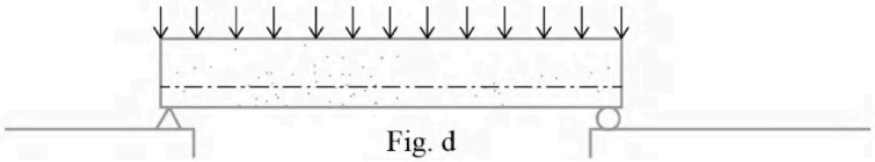


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- Figure 9.72(d): The deflection of the member under loading tends to equalize its upward curvature.

9.8.3 Advantages and Disadvantages of Prestressed Concrete

Advantages

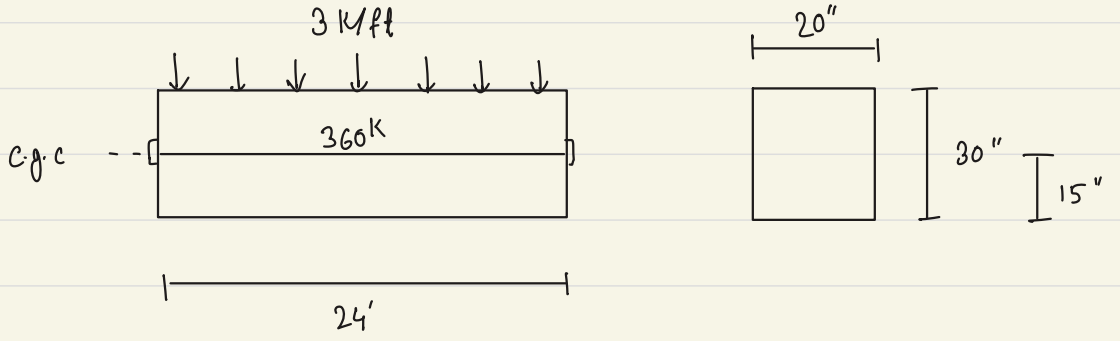
- It is possible with prestressing to utilize the entire cross sections of members to resist loads. Thus, smaller members can be used to support the same loads, or the same size members can be used for longer spans.

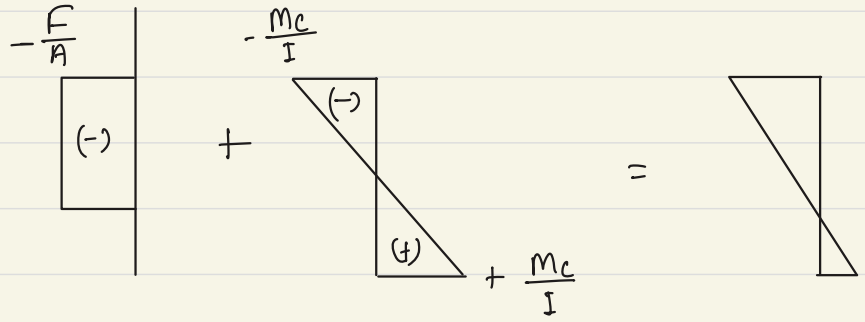
- Prestressed members are crack-free under working loads and, as a result, look better and are more watertight, providing better corrosion protection for the steel. Furthermore, crack-free prestressed members require less maintenance and last longer than cracked reinforced concrete members.
- The negative moments caused by prestressing produce camber in the members, with the result that total deflections are reduced.
- Other advantages of prestressed concrete include the following: reduction in diagonal tension stresses, sections with greater stiffnesses under working loads, and increased fatigue and impact resistance compared to ordinary reinforced concrete.

Disadvantages

- Prestressed concrete requires the use of high-strength concretes and steels and the use of more complicated formwork, with resulting higher labor costs.
- Closer quality control is required in manufacture.
- Losses in the initial prestressing forces. When the compressive forces from prestressing are applied to the concrete, it will shorten somewhat, partially relaxing the cables. The result is some reduction in cable tension, with a resulting loss in prestressing forces. Shrinkage and creep of the concrete add to this effect.
- Additional stress conditions must be checked in design, such as the stresses occurring when prestress forces are first applied and then after prestress losses have taken place, as well as the stresses occurring for different loading conditions.
- Cost of end anchorage devices and end beam plates that may be required.

Example 27





$$\therefore \frac{Mc}{I} = \frac{2592 \times 15}{45000} = 0.864 \text{ ksi}$$

$$\frac{F}{A} = \frac{360}{600} = 0.6 \text{ ksi}$$

$$\therefore f_{top} = -0.6 - 0.864 = -1.464 \text{ ksi}$$

$$f_{btm} = -0.6 + 0.864 = 0.264$$

$$I = \frac{bh^3}{12} = \frac{20 \times 30^3}{12}$$

$$= 45000 \text{ in}^4$$

$$M = \frac{WL^2}{8} = \frac{3 \times 24^2}{8}$$

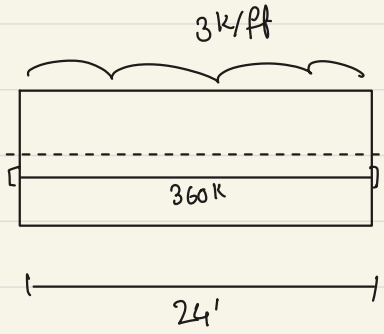
$$= 216 \text{ K-ft}$$

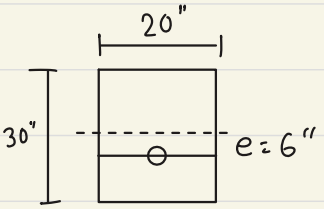
$$= 2592 \text{ K-in}$$

$$A = bh = 20 \times 30 \text{ in}^2$$

$$= 600 \text{ in}^2$$

Ex. 26





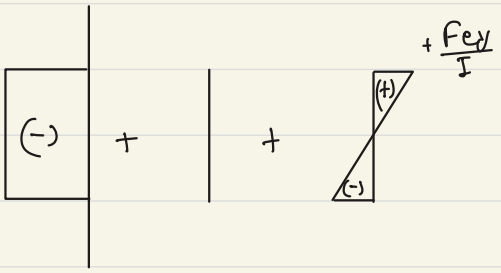
$\frac{F}{A} = \frac{360}{600} = 0.6 \text{ ksi}$

$I = 45000 \text{ in}^4$

$A = 600 \text{ in}^2$

$\frac{Mc}{I} = 0$

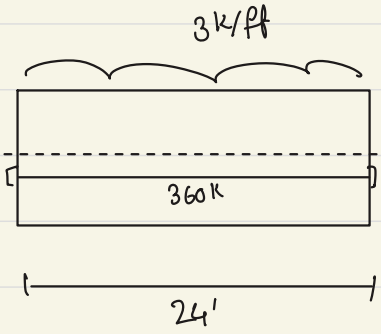
$\frac{Fey}{I} = \frac{360 \times 6 \times 15}{45000} = 0.720 \text{ ksi}$

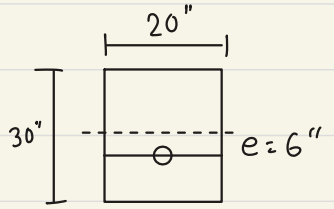


$\therefore f_{top} = -0.6 + 0.72 = 0.12 \text{ ksi}$

$f_{bfm} = -0.6 - 0.72 = 1.32 \text{ ksi}$

Ex. 25

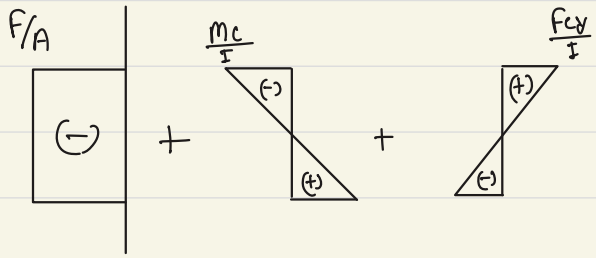




$\frac{Mc}{I} = 0.864$ ksi

$\frac{Fev}{I} = 0.720$ ksi

$\frac{F}{A} = 0.6$ ksi

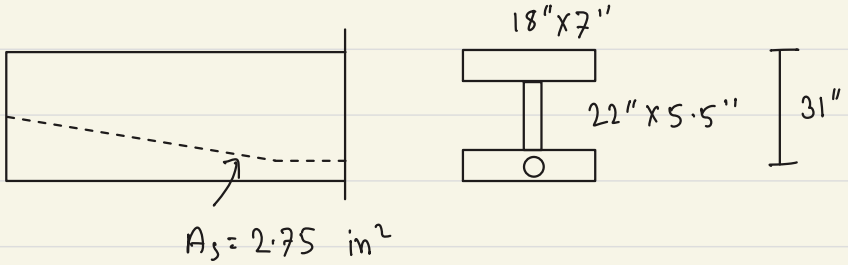


$\therefore f_{top} = -0.6 - 0.864 + 0.720 = -0.744$ ksi

$f_{btm} = -0.6 + 0.864 - 0.720 = -0.456$ ksi

Example 28:

$f_{se} = 160$ ksi; $f_{pu} = 270$ ksi; $f_c' = 7$ ksi



$$\rho = \frac{A_s}{bd} = \frac{2.75}{18 \times 31.5} = 0.00485$$

$$\therefore f_s = f_{pu}\left(1 - 0.5\rho \frac{f_{pu}}{f_c'}\right)$$

$$= 270\left(1 - 0.5 \times 0.00485 \frac{270}{7}\right) = 244.745 \text{ ksi}$$

$$C = T \Rightarrow 0.85 f_c' A_c = f_s A_s$$

$$\Rightarrow 0.85 \times 7 \times A_c = 244 \times 2.75 \Rightarrow A_c = 113.12 \text{ in}^2$$

$A_{flange} = 18 \times 7 = 126 \text{ in}^2 > A_c$

$\therefore$ Rectangular beam in effect

$$\therefore M_n = A_s f_s \left(d - \frac{a}{2}\right)$$

$$= 2.75 \times 244 \left(31 - \frac{6.27}{2}\right)$$

$$= 18697 \text{ k-in}$$

$$= 1558 \text{ k-ft}$$

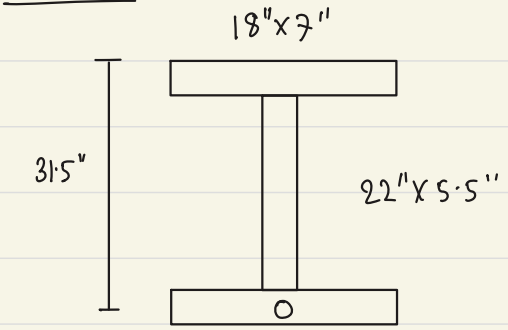
$$0.85 f_c' ab = A_s f_s$$

$$\Rightarrow 0.85 \times 7 \times a \times 18 = 2.75 \times 244$$

$$\Rightarrow a = 6.27 \text{ in}$$

$\therefore \phi M_n = 0.9 \times 1558 = 1402$ k-ft

Ex. 29



$$\rho = \frac{A_s}{bd} = \frac{3.67}{18 \times 31.5} = 0.00647$$

$f_u = 270$ ksi

$f_c' = 7$ ksi

$$\therefore f_s = f_u\left(1 - 0.5\rho \frac{f_u}{f_c'}\right)$$

$$= 236.3 \text{ ksi}$$

$$C = T \Rightarrow 0.85 f_c' A_c = A_s f_s$$

$$\Rightarrow A_c = 145.8 \text{ in}^2$$

$A_{flange} = 18 \times 7 = 126 \text{ in}^2$ (A_c; T-beam

$$\therefore a = 7'' + \frac{145.8 - 126}{5.5} = 10.6 \text{ in}$$

$$C_f = 0.85 f_c' \left[h_f \times (b_f - b_w)\right]$$

$$= 0.85 \times 7 \times (7 \times (18 - 5.5)) = 520.6 \text{ kip}$$

$$C_w = 0.85 f_c' \left[a b_w\right] = 346.9 \text{ kip}$$

$$\therefore M_n = 0.9\left[C_w\left(d - \frac{a}{2}\right) + C_f\left(d - \frac{h_f}{2}\right)\right]$$

$$= 21299 = 1{,}775 \text{ k-ft}$$

Chapter 08

Compression member:

$$C_c = 4.71\sqrt{\frac{E}{F_y}} \qquad F_e = \frac{\pi^2 E}{(S.R)^2}$$

$$SR = \frac{KL}{r}$$

$S.R. > C_c \rightarrow$ slender $\rightarrow F_{cr} = 0.877\, Fe$

$< C_c \rightarrow$ compact $\rightarrow F_{cr} = 0.658^{F_y/Fe} F_y$

Ex. 9 $L = 25'$

W14X61 $\rightarrow r_y = 2.45$ in, $A_g = 17.9\ in^2$

A992 $\rightarrow F_y = 50$ ksi

$$SR = \frac{KL}{r} = \frac{0.8 \times 25 \times 12}{2.45} = 97.96 \text{ in}$$

$$C_c = 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.4 \text{ in}$$

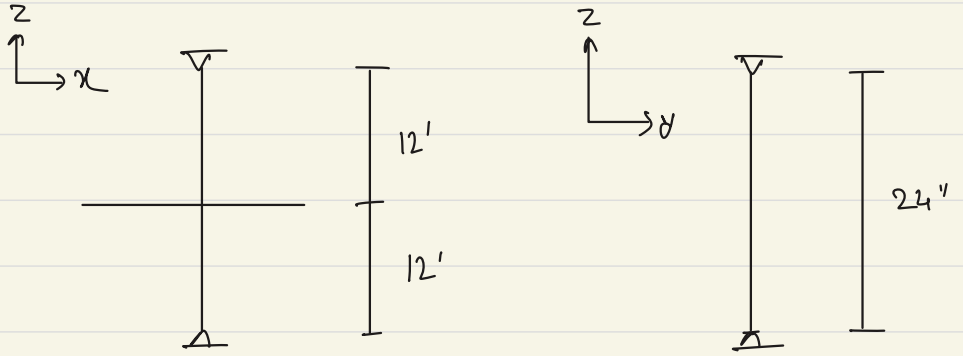
∴ Stocky col

$$F_e = \frac{\pi^2 E}{(SR)^2} = 29.83 \text{ ksi}$$

$$F_{cr} = 0.658^{\frac{F_y}{Fe}} F_y = 24.79 \text{ ksi}$$

$$\therefore \phi P_n = \phi A_g F_{cr} = 400 \text{ kip}$$

Ex 10



$A_g = 11.5\ in^2$; $r_x = 4.27\ in$; $r_y = 1.98\ in$

$$SR = \max\left[\frac{KL}{r_x}, \frac{KL}{r_y}\right] = \left[\frac{1 \times 24 \times 12}{4.27}, \frac{1 \times 12 \times 12}{1.98}\right]$$

$$= 67.45,\ 72.73$$

$\therefore 72.73$

$C_c = \ldots$

$- - -$

Ex. 11 $L = 25'$

$DL = 135^k$, $LL = 145^k$

A992

Demand: $P_u = 1.2DL + 1.6LL = 394^k$

$$C_c = 4.71\sqrt{\frac{E}{F_y}} = 113.4 \text{ in}$$

$$KL = 0.8 \times 25 = 20'$$

Trial 1: $F_{cr} = \frac{2}{3} 50 = 33.3 \text{ ksi}$

$$\therefore A_g = \frac{P_u}{\phi F_{cr}} = 13.15 \text{ in}^2$$

$\phi P_n = \phi F_{cr} A_g$

$\Rightarrow A_g = \frac{P_u}{\phi F_{cr}}$

W14x48; $A_g = 14.1 \text{ in}^2$, $r_y = 1.91 \text{ in}$

$$SR = \frac{KL}{r} = \frac{20 \times 12}{1.91} = 125.65 > C_c \rightarrow \text{slender}$$

$$F_e = \frac{\pi^2 E}{SR^2} = 18.13 \text{ ksi}$$

$$\therefore F_{cr} = 0.877 F_e = 15.9 \text{ ksi}$$

$$\phi P_n = \phi A_g F_{cr} = 201.8 < P_u$$

Trial 2: $F_{cr} = \frac{15.9 + 33.3}{2} = 24.6 \text{ ksi}$

$$\therefore A_g = \frac{P_u}{\phi F_{cr}} = 17.8 \text{ in}^2$$

W14x61 $\rightarrow$ $A_g = 17.9 \text{ in}^2$; $r_y = 2.45 \text{ in}$

$SR = 97.96$

$\therefore F_{cr} = 0.658^{\frac{F_y}{F_e}} F_y$ $\qquad F_e = \frac{\pi^2 E}{SR^2} = 29.83 \text{ ksi}$

$\therefore 24.79 \text{ ksi}$

$\therefore \phi P_n = 399 \text{ ksi} > P_u$

Ex. 8 $L = 15'$

$DL = 35^k$, $LL = 70^k$, $W_L = 60^k$

A36

$3/4''$ bolt dia

$t_{min} = 1/4$ in

Demand: $P_u = 1.2DL + 1.6LL = 154^k$

$P_u = 1.2DL + 0.5LL + 1W_L = 137^k$

Area:

Based on yielding:

$$P_u = \phi F_y A_g$$

$$\Rightarrow A_g = \frac{P_u}{\phi F_y} = 4.75 \text{ in}^2$$

Based on fracture:

$$P_u = \phi F_u A_e$$

$$\Rightarrow 154 = 0.75 \times 58 \times 0.8 [A_g - 1.75t]$$

$$\Rightarrow 154 = 34.8 [A_g - 1.75t]$$

$$\Rightarrow A_g = 4.43 + 1.75t$$

$$A_e = U A_n$$

$$A_n = A_g - 2t\left[\frac{3}{4} + \frac{1}{8}\right]$$

$$= A_g - 1.75t$$

$U = 0.8$ for 4 or more bolts in a row

$$r_{min} = \frac{L}{300} = 0.6$$

t	A_{g1}	A_{g2}	r_2	Section
1/4	4.75	4.87		
5/16	4.75	4.98		
3/8	4.75	5.09		
7/16	4.75	5.20	0.867	L 8x4x 7/16

Fast review

Ex. 1 $A_g = 5.77\ in^2$; $\bar{x} = 1.67\ in$

$A_n = A_g - 2 \times \frac{1}{2}\left[\frac{5}{8} + \frac{1}{8}\right] = \cdots$

$A_e = U A_n$ $\qquad U = 1 - \frac{\bar{x}}{\ell}$

$= 1 - \frac{1.67}{6}$

Ex. 2 $A_g = 1/2 \times 5$

$A_n = A_g - 2\frac{1}{2}\left[\frac{5}{8} + \frac{1}{8}\right] \cdots$

$A_e = 1 \times A_n$

$F_y = 0.9 \times F_y A_g$ ← 36

$F_u = 0.75 \times F_u A_e$ ← 58

Staggered fastener

Ex 4: $A_n = A_g - \sum A_n + \sum \frac{s^2}{4g} t$

Line abe: $A_n = A_g - 1 \times \frac{7}{16}[\cdots] \cdots$

Line abed: $A_n = A_g - 2 \times \frac{7}{16}[\cdots] + \frac{2^2}{4 \times 3} \times \frac{7}{16}$

Ex 5

Line abcd: $A_n = A_g - 2x \cdots$

Line debc: $A_n = A_g - 3x[\cdots] + \frac{1.5^2}{4x3.94} x[\,]$

$g = 2 + 2.25 - \frac{5}{16}$

$= 3.94$

$=$

Ex. 8 $P_u = \cdots$

$P_u = \cdots$

Based on yielding: $P_u = \phi F_y A_g$

" " fracture: $P_u = \phi F_u A_e$

$[\,] = 0.75 \times 58 \times 0.8 (A_g - 1.75t)$

$\therefore A_g = \cdots$

$A_e = U A_n$

$A_n = A_g - 2t\left[\frac{3}{4} + \frac{1}{8}\right]$

$= A_g - 1.75t$

$U = 0.8$

$n_{min} = \frac{L}{300}$

$= \cdots$

Ex 9: $SR = \dots$ $\quad C_c = 4.71\sqrt{\frac{E}{F_y}}$

$C_c = \dots$ $\quad F_e = \frac{\pi^2 E}{(SR)^2}$

F_e

F_{cr} $\quad F_{cr} = 0.658^{F_y/F_e} F_y$ → compact

$P = \dots$ $\quad 0.877 F_e$ → slender

Ex. 11

$KL = \dots$ $\quad C_c = \dots$

$P_u = \dots$

$f_{cr} = 2/3 F_y$

$A_g = \dots$

Plastic moment:

$M_y = F_y S$; $\quad S = \frac{I}{c}$

$M_p = F_y Z$; $\quad Z = A_c a$

Ex 15 $\quad \bar{y}_1 = \dots$

$a = 2\bar{y}_1$

$M_p = F_y \times A_c a$ $\quad A_c = 6 \times 0.5 + 8 \times 1$

Ex. 16 $\bar{y} = \cdots$

$\bar{y}_1 = \cdots$

$\bar{y}_2 = \cdots$

$a = \bar{y}_1 + \bar{y}_2$

$A_c = \cdots$

$M_p = F_y A_c a$

Local buckling:

$\lambda < \lambda_p$ — compact

$\lambda_p < \lambda < \lambda_r \rightarrow$ non

$\rightarrow$ slender

	λ_p	λ_r
Flange	$0.38\sqrt{\cdots}$	$1\sqrt{\cdots}$
web	$3.76\sqrt{\cdots}$	$5.7\sqrt{\cdots}$

Ex. 20/ $M_p = F_y Z_x$

$$\phi M_n = \phi \left[M_p - (M_p - 0.7 F_y S_x) \frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right]$$

Moment gradient!

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

Ex. 24

$w_u = \ldots$

$M_u = \frac{w_u L^2}{8}$

Trial . . .

Live ld: $\Delta_{max} = \frac{L}{360}$ $\qquad \Delta = \frac{5}{384} \frac{w_u L^4}{EI}$

LL+DL: $\Delta_{max} = \frac{L}{240}$

$V_u = \frac{w_u L}{2}$

$\phi V_n \longrightarrow \ldots$

Bolt: $R_n = 1.2 F_u l_c t \leq 2.4 F_u d t$

$l_c \geq 2.67 d$

$l_e \geq d + 0.25$

$R_n = n F_n A_b$ ← 48 ksi