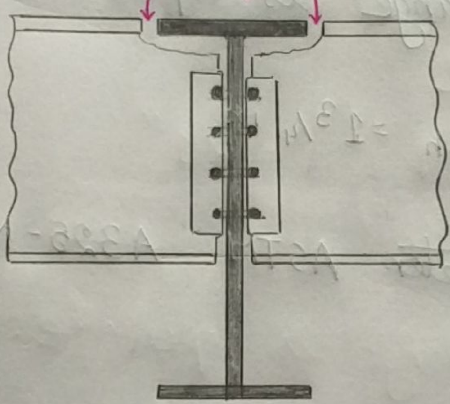
Chapter 13

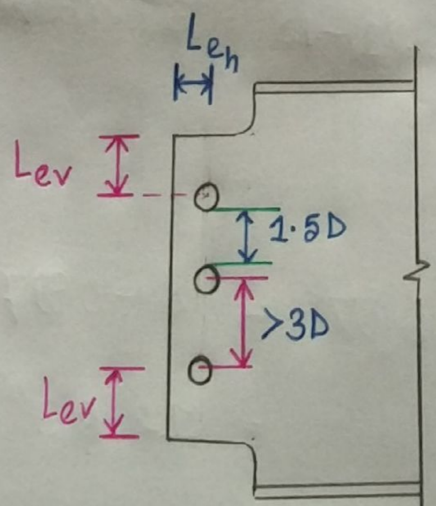
Connections

Simple Shear Connections:

* Moment transfers by flange
* Shear transfers by web
* (copping is used to keep both top surface at a plane)

Cope flange ($\frac{1}{2}$" clearance at copping)





D = bolt dia

if not given $\begin{cases} L_{eh} = 1.5'' \sim 2'' \\ L_{ev} = 1.25'' \end{cases}$

Problem 1

All Bolted Double Angle

Select an all bolted double angle connection between a W18×50 beam and a W21×62 girder web to support the following beam end reactions:

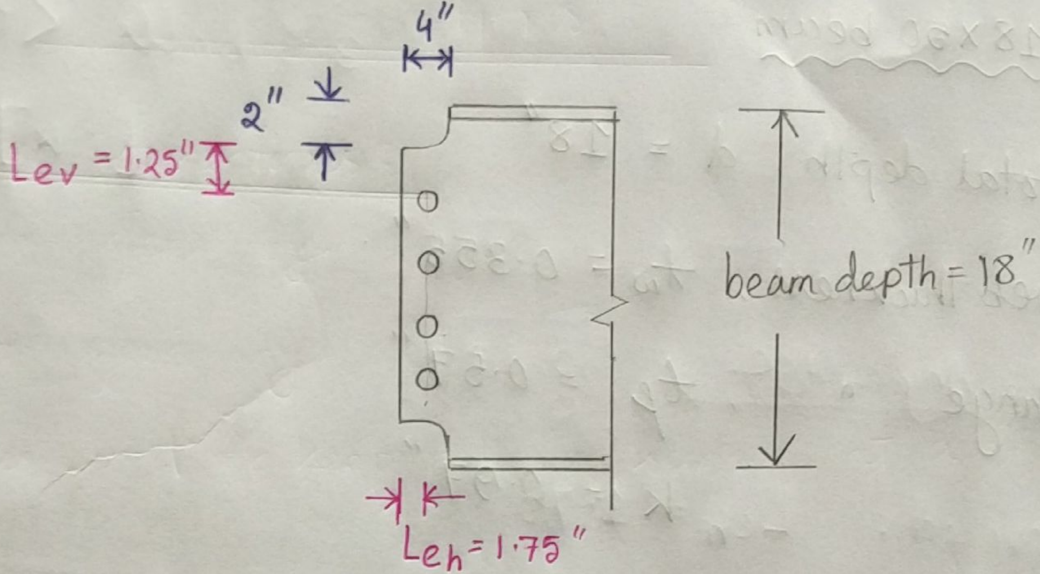
$$R_D = 30\,k \;;\; R_L = 40\,k$$

The beam top flange is coped 2 inch deep by 4 inch long.

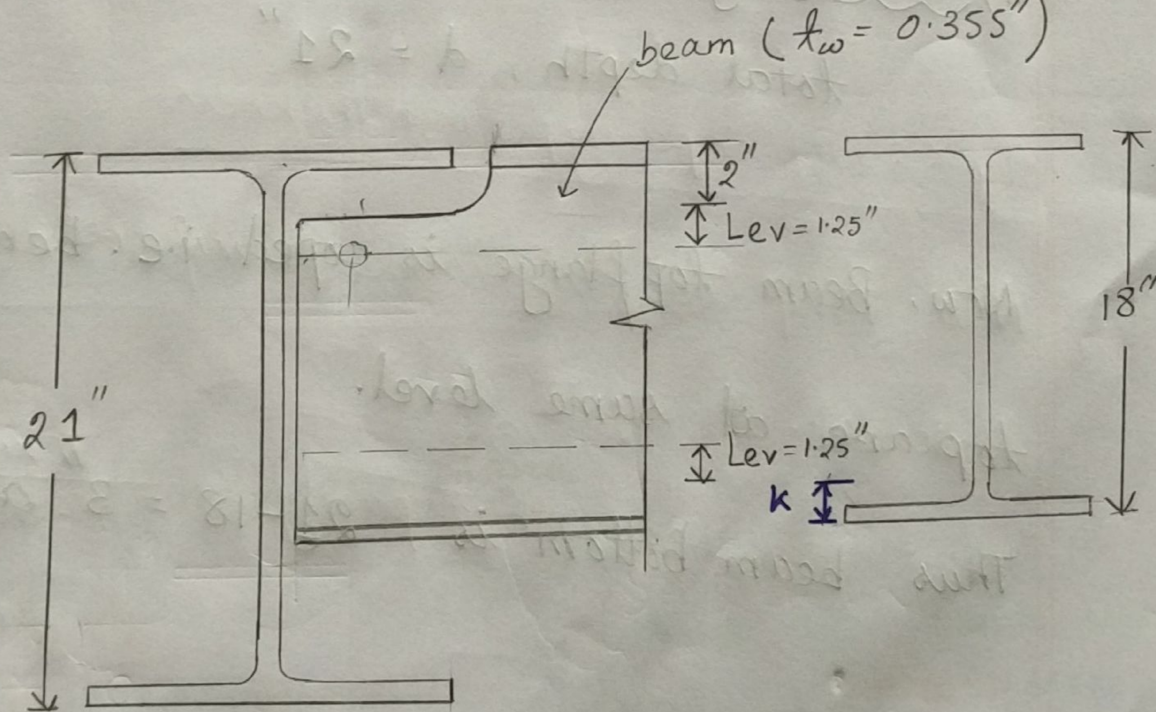
$L_{ev} = 1\frac{1}{4}$ in ; $L_{eh} = 1\,3/4$ in

Use $\frac{3}{4}$ in diameter ASTM A325-N bolts in standard holes.

Beams are A572 grade 50 material.



4"
2"
Lev = 1.25"
beam depth = 18"
Leh = 1.75"



beam (t_w = 0.355")
2"
Lev = 1.25"
21"
18"
Lev = 1.25"
k

Step 1: From AISC Manual table 1:

i) W18 X 50 beam

total depth, $d = 18''$

web thickness, $t_w = 0.355''$

flange " , $t_f = 0.57''$

$k = 0.97''$

ii) W21 X 62 girder

total depth, $d = 21''$

Now, Beam top flange is coped i.e. beam ~~top~~ and girder top are at same level.

Thus beam bottom is $21 - 18 = 3''$ above girder bottom.

Step 2: Now,

Space available for bolts $= 18 - 0.97 - 2 \times 1 - 1.25 \times 2$

$= 12.53''$

Let us assume,

bolt spacing $= 3''$ $\left(> 3D = 3 \times 3/4 = 2.25'' \right)$

$\therefore$ Max^m number of bolts $= \frac{12.53}{3} + 1 = 5.17$

$= 5$ bolts

Step 3: According to ASD principle,

Total load to be transferred $= R_D + R_L$

$= 30 + 40$

$= 70$ kips

Step 4: Allowable bolt shear capacity –

A325-N bolts

Threads included in shear plane.

$$R_n = \frac{m F_v A_b}{\Omega}$$

$$= \frac{2 \times (0.4 \times 120) \times \left(\pi/4 \times \left(\frac{3}{4}\right)^2 \right)}{2.0}$$

$= 21.2$ kip

$m = 2$ (for double shear)

$F_v = 0.4 F_u$ $\rightarrow$ 120 ksi (A325 bolts) $\rightarrow$ 150 ksi (A490 bolts)

A_b = bolt area

$\Omega = 2.0$

$\therefore$ No. of bolts required $= \dfrac{70}{21.2}$

$= 3.3$ bolts < 5 bolts

[OK]

Step 5: Angle size

Provided, $L_{eh} = 1.75''$

$\therefore$ Angle size shall be $= 1.75 \times 2 = 3.5''$

Step 6: Web capacity per inch thickness

Now, 70kip load is to be transferred by 0.355 inch thick web.

Thus, required web capacity $= \dfrac{70\text{kip}}{0.355\text{in}} = 197.2$ kips per inch thickness

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Now, Considering web capacity, we choose page 10-20 of AISC manual with 5-rows of bolts.

→ From page 10-20, lower part of AISC manual,

For coped at top flange only, $L_{ev} = 1\frac{1}{4}"$ and $L_{eh} = 1\frac{3}{4}"$

for ASD,

Beam web available strength per inch thickness

= 216 kip/inch

∴ Beam web capacity = 216 X 0.355 = 76.68 kip > 70 kip

[ok]

→ From page 10-20, upper part of AISC manual,

For A325 bolts, Thread cond N (threads included) and for

Bolt and angle available strength = 83.3 kip in ASD

Angle thickness = $\frac{1}{4}$ inch

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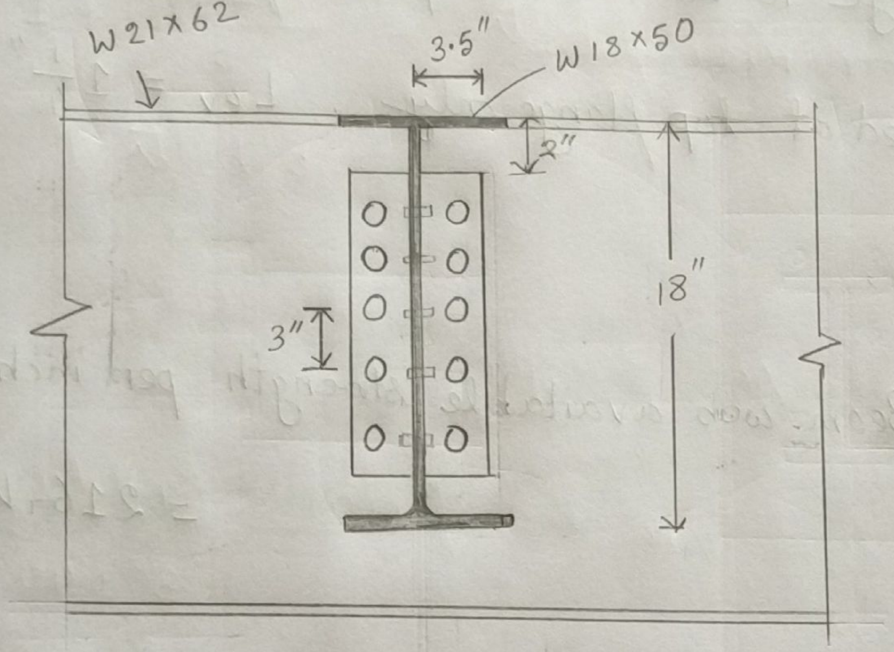
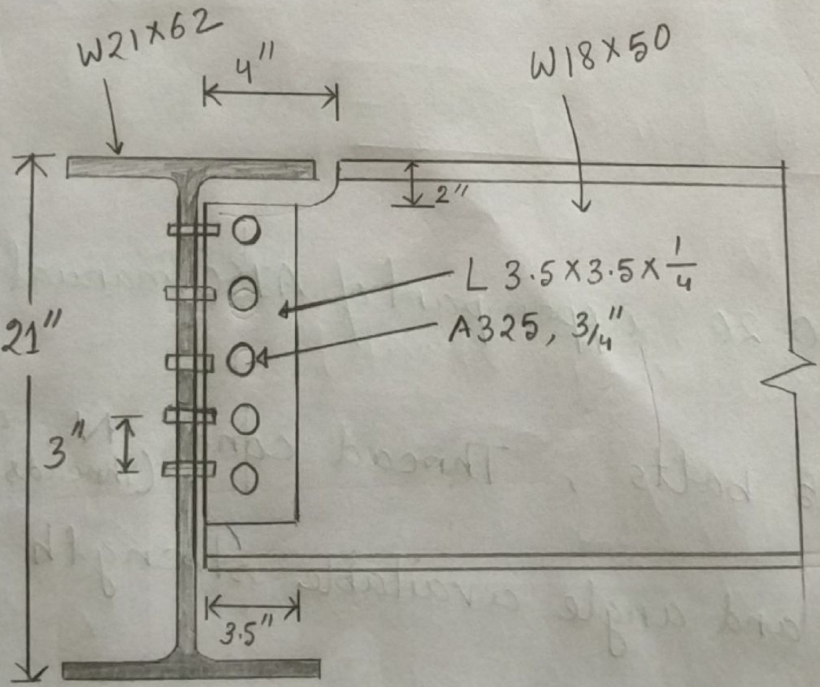
Chosen angle $L\ 3.5 \times 3.5 \times \frac{1}{4}$

$F_y = 36$ ksi (A36)

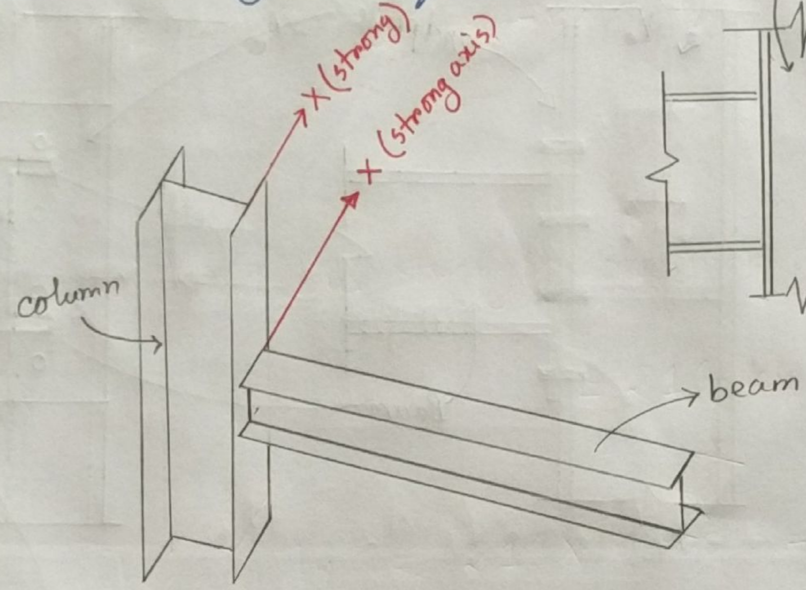


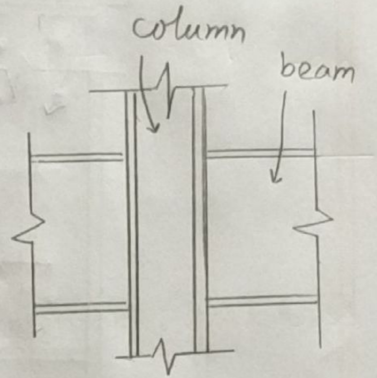


Rigid Connections:

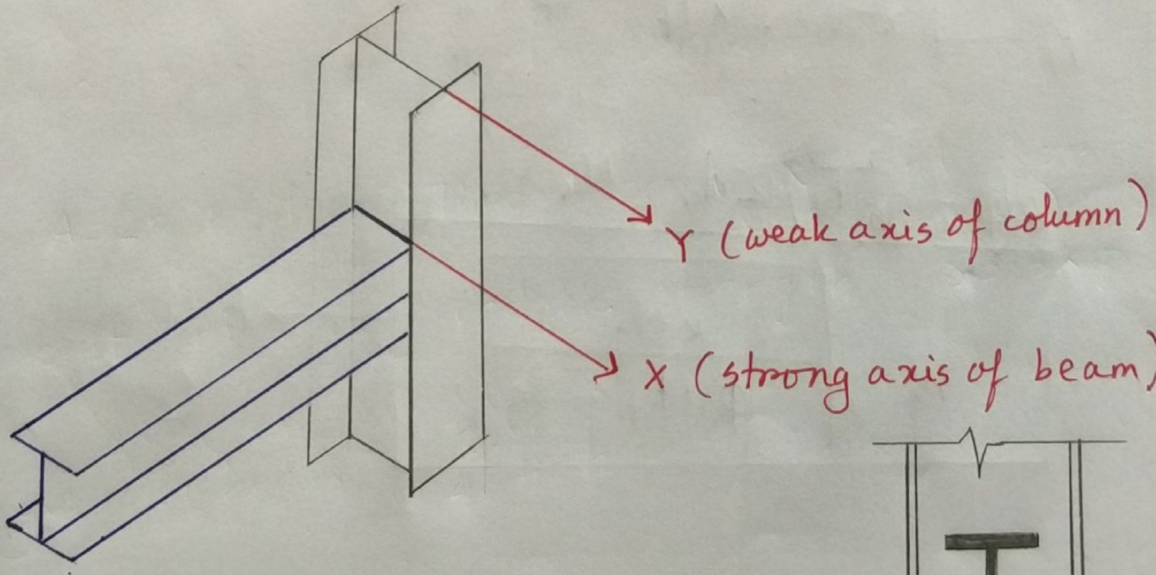
Continuous beam to column connection

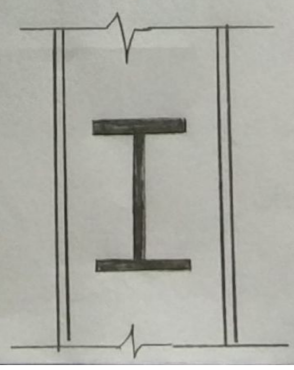
1) Beam connected to strong axis of column



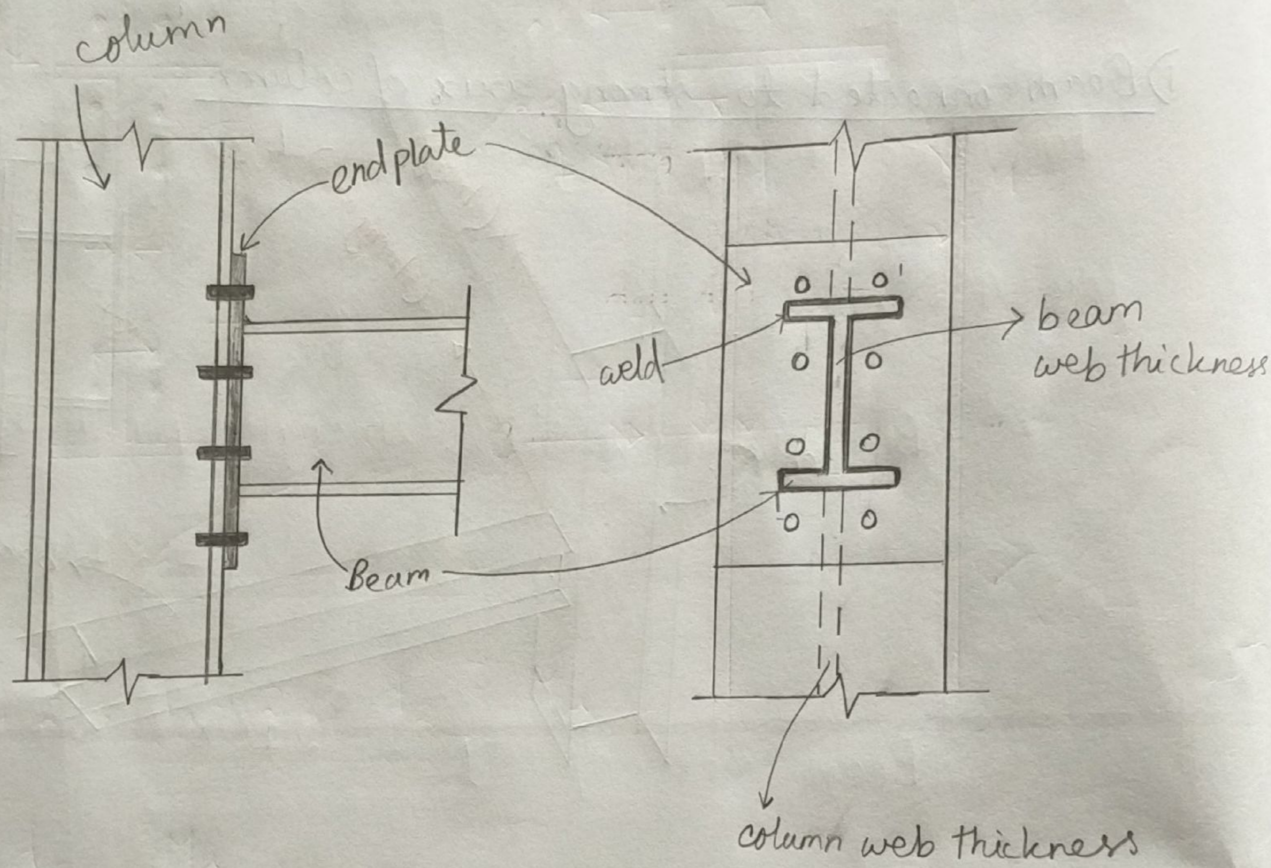


2) Beam connected to weak axis of column



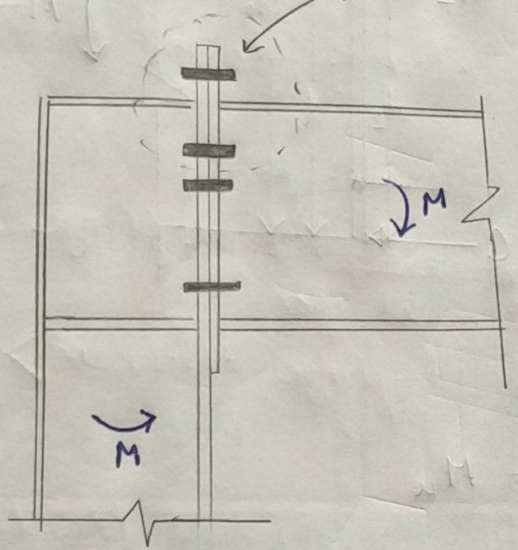


Extended End Plate moment Connection:



Endplate is thick and rigid enough to cause contact separation by elongation of bolt.

Tension zone



as M clockwise $\longrightarrow$ Tension zone in upper part

M ↺ $\longrightarrow$ " " " lower part

$P_t \longrightarrow$ force of every bolt

$F_t \longrightarrow$

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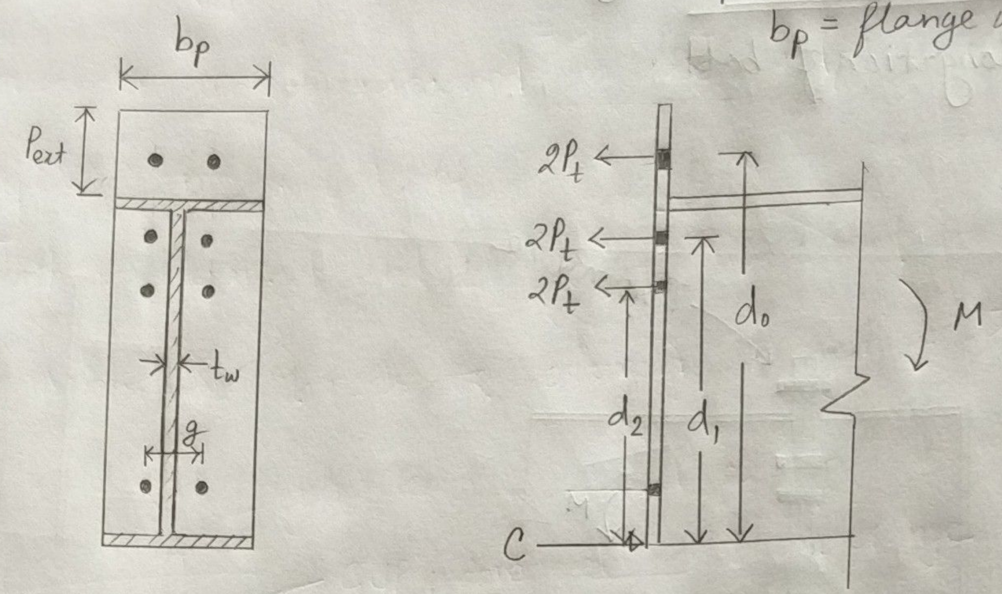
1) Determination of bolt diameter

end plate width,
b_p = flange width of beam



$$2P_t \cdot d_0 + 2P_t d_1 + 2P_t \cdot d_2 = M_u$$

$$2P_t \Sigma d_n = M_u$$

$$P_t = \phi \times \text{Area} \times F_t = \phi \times \frac{\pi}{4} d_b^2 \times F_t$$

$$\therefore 2\left(\phi \frac{\pi}{4} d_b^2 F_t\right) \Sigma d_n = M_u$$

$$\boxed{d_b = \sqrt{\frac{2M_u}{\phi \pi F_t \Sigma d_n}}} \longrightarrow \text{Bolt diameter}$$

(will be given in exam)

11) Determination of endplate thickness

Endplate thickness,

$$t_p = \sqrt{\frac{1.11\ \gamma_r\ \phi M_{np}}{\phi_b\ F_{py}\ Y}}$$

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$\phi_b = 0.90$ (for bending)

$\gamma_r = 1.0$ (for extended endplate)

F_{py} = endplate yield strength

Y = Yield line mechanism parameter

ϕM_{np} = connection strength based on bolt tension limit state

$$\phi [2 P_t \Sigma d_n]$$

$$P_t = \frac{\pi}{4} d_b^2 F_t \quad ; \quad \phi = 0.75$$

** $F_t = 90$ ksi ⟶ A325 bolts

$F_t = 113$ ksi ⟶ A490 bolts

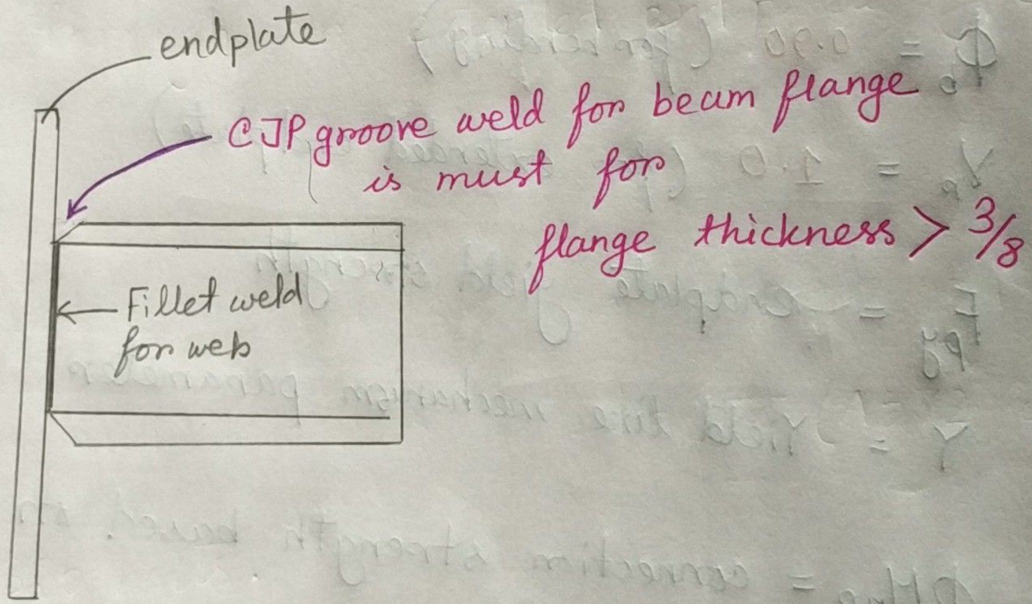
** ASTM bolt diameters : $\frac{1}{2}, \frac{5}{8}, \frac{3}{4}, \frac{7}{8}, 1\frac{1}{8}, 1\frac{1}{4}, 1\frac{3}{8}, 1\frac{1}{2}$ inch

III) Welding of Endplate with Beam

Beam forces are transferred to the endplate through welding.

which in turn,

transfers the forces to column through bolts.



Problem 2

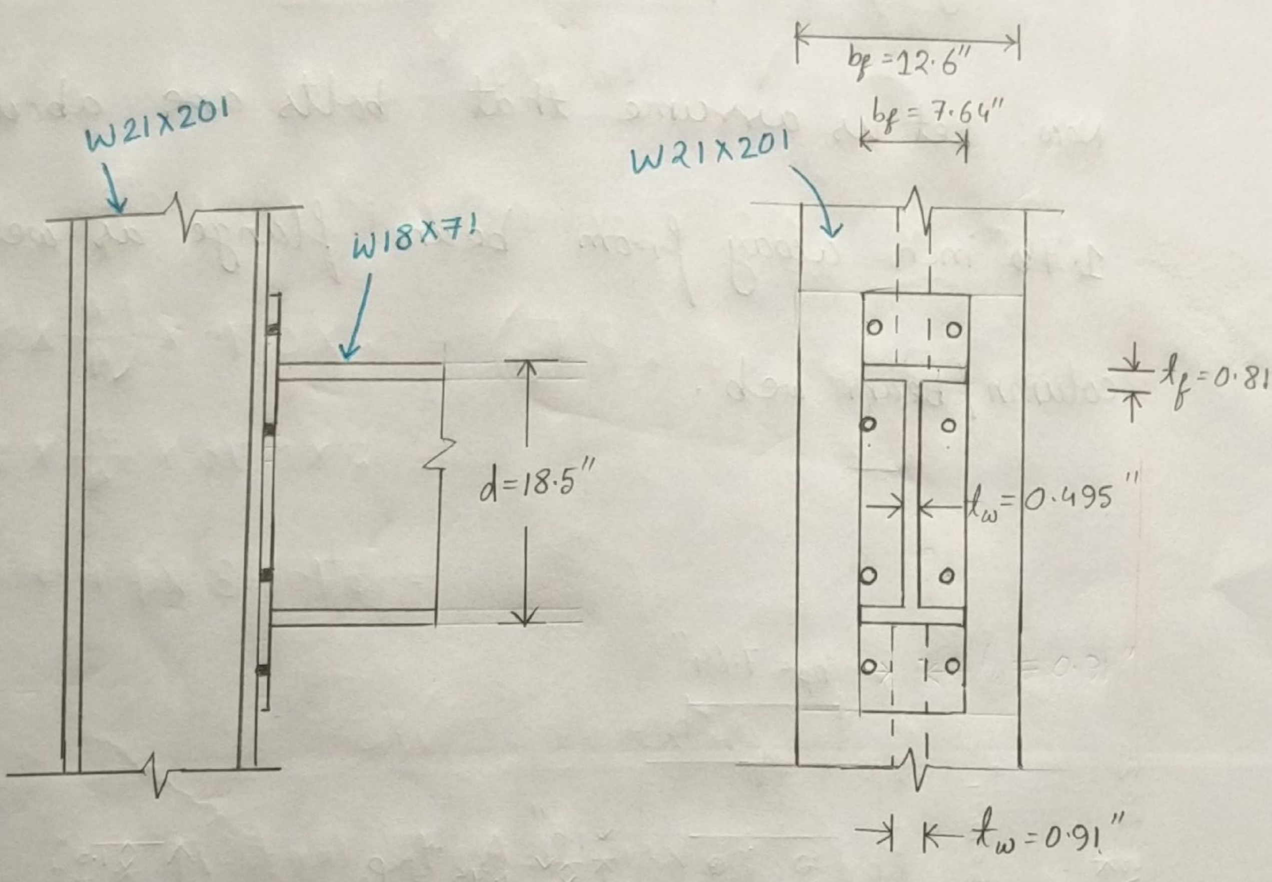
Non seismic Application: Extended unstiffened moment Connection

A W18 X 71 beam (A572 Gr. 50) has to transfer 75 k-ft dead load and 140 k-ft live load moment on to a W21 X 201 (A572 Gr. 50) column on its strong axis through an extended end plate type connection.

Determine Suitable dimension of the end plate and the bolt diameter and thickness of end plate (A572 Gr. 50)

Use ASTM A325 bolts.

Dimensions of W18X71 beam and W21X201 column obtained from AISC manual are shown in the figure below –



Now, Initially let us assume "two row four bolt" configuration. Bolts shall be placed closed to the beam flange and column web in such a way that enough clear space around the bolt exists for it to be easily placed and tightened.

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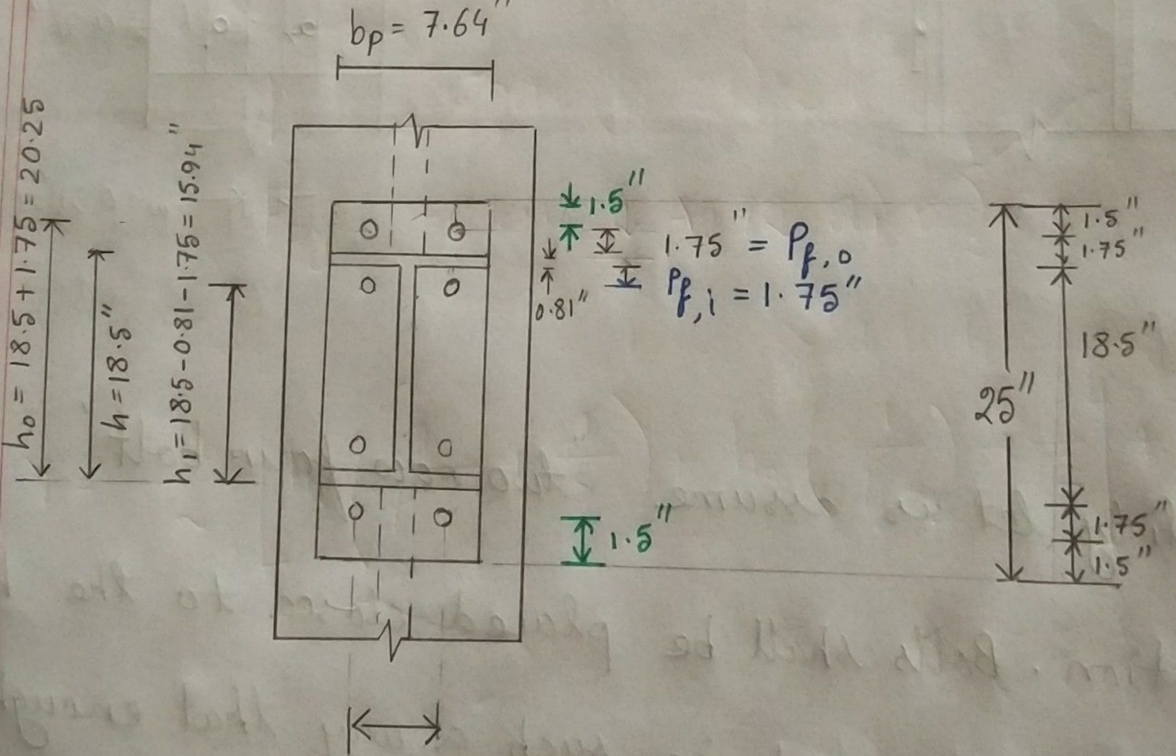
Also there shall be at least $1.5 d_b$ distance from free edge of end plate to bolt centre.

Now, Let us assume that bolts are about 1.75 inch away from beam flange as well as column/beam web.



$g = 0.91 + 1.75 \times 2 = 4.41''$

Bolt diameter :

$$d_b = \sqrt{\frac{2 M_u}{\phi \pi F_t \Sigma d_n}}$$

Now, $M_u = 1.2(75) + 1.6(140) = 314 \text{ k-ft} = 3768 \text{ k-inch}$

$\phi = 0.75$ (for shear)

$F_t = 90$ ksi (for A325 bolt)

$\Sigma d_n = d_0 + d_1$

Now, $d_0 = h_0 - t_f/2 = 20.25 - 0.81/2 = 19.845''$

$d_1 = h_1 - t_f/2 = 15.94 - 0.81/2 = 15.535''$

$\therefore \Sigma d_n = 19.845 + 15.535 = 35.38''$

So, $d_b = \sqrt{\frac{2 \times 3768}{0.75 \times \pi \times 90 \times 35.38}}$

$$\therefore \boxed{d_b = 1.0 \text{ inch}}$$

End plate thickness

$$t_p = \sqrt{\frac{1.11\, \gamma_r\, \phi M_{np}}{\phi_b\, F_{py}\, Y}}$$

Now, $\gamma_r = 1.0$ for extended end plate

$$\phi M_{np} = \phi\left(2 P_t \Sigma d_n\right) = 2 \times 0.75 \times \left(\frac{\pi}{4} \times d_b^2 \times 90\right) \times 35.38$$

$$= 2 \times 0.75 \times \frac{\pi}{4} \times 1^2 \times 90 \times 35.38$$

$$= 3751.3 \text{ kip-inch}$$

$\phi_b = 0.90$

$F_{py} = 50$ ksi (A572 Gr.50)

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{P_{f,i}} + \frac{1}{s}\right) + h_o\left(\frac{1}{P_{f,o}}\right) - \frac{1}{2}\right] + \frac{2}{g}\left[h_1\left(P_{f,i} + s\right)\right]$$

use $P_{f,i} = s$ if $P_{f,i} > s$

$$s = \frac{1}{2}\sqrt{b_p g}$$

Now, $s = \frac{1}{2}\sqrt{b_p g} = \frac{1}{2}\sqrt{7.64 \times 4.41} = 2.9''$

$P_{f,i} = 1.75''$

Now, $Y = \frac{7.64}{2}\left[15.94\left(\frac{1}{1.75} + \frac{1}{2.9}\right) + 20.25\left(\frac{1}{1.75}\right) - \frac{1}{2}\right]$

$+ \frac{2}{4.41}\left[15.94\left(1.75 + 2.9\right)\right]$

$\therefore Y = 131.7$ inch

So, $t_p = \sqrt{\frac{1.11 \times 1.0 \times 3751.3}{0.90 \times 50 \times 131.7}} = 0.84'' \approx 7/8''$

$\boxed{t_p = 7/8 \text{ inch}}$

Dimensional and other checks:

Let us choose, 1.5" ($1.5\,d_b$) extension of endplate beyond the centre of outer bolts.

So, the total size of endplate 7.64" x 25"

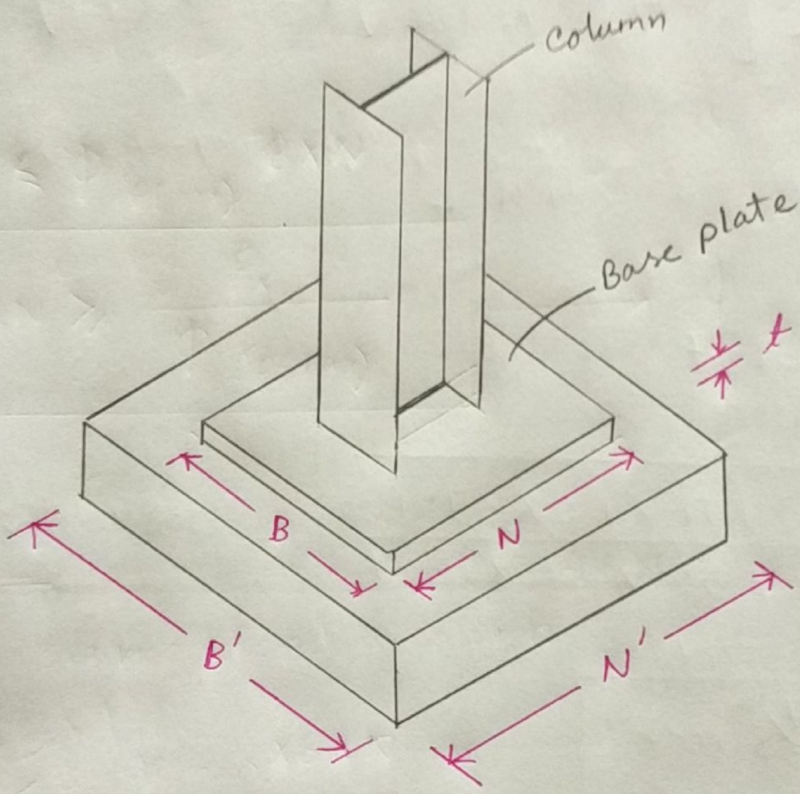
$t_p = 7/8$" with 1" dia A325 bolts

Horizontal distance of bolt centre from vertical edge of endplate $= (b_p - g)/2$

$= (7.64 - 4.41)/2$

$= 1.62'' > 1.5\,d_b$ [ok]

Column Bases : Concentric Compressive Axial Loads



A_1 = area of base plate = $B \times N$

A_2 = max^m area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area.

1) Nominal Bearing Strength of Concrete Surface

Nominal bearing strength of concrete surface,

$$P_p = (0.85 f_c' A_1)\left(\sqrt{\frac{A_2}{A_1}}\right) \leq 1.7 f_c' A_1$$

* If $\sqrt{\frac{A_2}{A_1}} > 2.0$ then take $\sqrt{\frac{A_2}{A_1}} = 2.0$

So, $\left(\sqrt{\frac{A_2}{A_1}}\right)_{max} = 2.0$

In terms of nominal bearing stress,

$$f_{p(max)} = (0.85 f_c') \sqrt{\frac{A_2}{A_1}} \leq 1.7 f_c'$$

$$\boxed{\begin{array}{l} \phi = 0.60 \text{ (LRFD)} \\ \Omega = 2.50 \text{ (ASD)} \end{array}} \longrightarrow \text{Concrete bearing}$$

11) Determination of Base plate size :

LRFD :

P_u = factored load in column

$$P_u \le \phi f_{p(max)} \; B \times N$$

$$\boxed{B \times N \geq \frac{P_u}{\phi f_{p(max)}}} \longrightarrow \phi = 0.60$$

ASD :

P_a = service load in column

$$P_a \le \frac{f_{p(max)} \times B \times N}{\Omega}$$

$$\boxed{B \times N \geq \frac{P_a \Omega}{f_{p(max)}}} \longrightarrow \Omega = 2.5$$

f_p = actual bearing stress

iii) Base plate thickness :

$$m = \frac{1}{2}(N - 0.95d)$$

$$n = \frac{1}{2}(B - 0.80b_f)$$

l = cantilever span (m or n whichever is larger)

Now, For cantilever span l,

$$M_n = \frac{f_p l^2}{2}$$

$$\therefore F_y = \frac{M}{Z} = \frac{f_p l^2}{2} \Big/ \frac{t^2}{4}$$

$$F_y = \frac{2 f_p l^2}{t^2}$$

$$\therefore \boxed{t \geq l \sqrt{\frac{2 f_p}{F_y}}}$$

$$P_u = \phi f_p BN$$

$$P_a = \frac{f_p BN}{\Omega}$$

LRFD:

$$t \geq l \sqrt{\frac{2 P_u}{\phi F_y BN}} \rightarrow \phi = 0.90$$

ASD:

$$t \geq l \sqrt{\frac{2 \Omega P_a}{F_y BN}} \rightarrow \Omega = 1.67$$

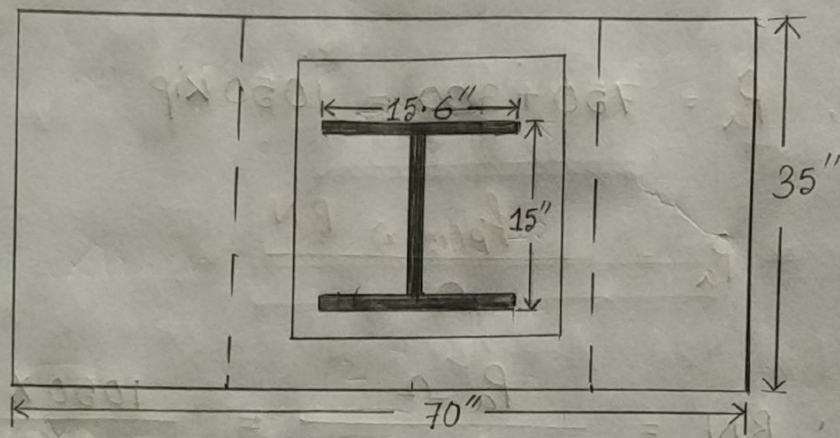
Problem-3

A W14X159 column transmits an axial compressive live load of 750 kip and dead load of 300 kip on to a concrete base having a top surface area of 35 in by 70 in.

Determine the size and thickness of base plate using A36 material.

The concrete base has $f_c' = 4\,ksi$.

[Follow ASD method]



W14 × 159 geometry :

$d = 15''$, $b_f = 15.6''$

1) Baseplate area :

Baseplate shall be smallest when allowable bearing stress is maximized .

Trial 1 :

Let us assume $\sqrt{\frac{A_2}{A_1}} = 2.0$ (max^m value)

$$f_{p(max)} = 0.85 f'_c \sqrt{\frac{A_2}{A_1}}$$

$$= 0.85 \times 4 \times 2$$

$$= 6.8 \text{ ksi}$$

Now, $P_a = 750 + 300 = 1050$ kip

and, $$P_a = \frac{f_{p(max)} \, BN}{\Omega}$$

$$\therefore BN = \frac{P_a \, \Omega}{f_{p(max)}} = \frac{1050 \times 2.5}{6.8}$$

$$\therefore \boxed{B \times N = 386.03 \text{ in}^2}$$

Now, Overall plan dimension of the column is 15×15.6 which is almost square.

So, let us choose, $B = N = \sqrt{386.03}$

$$= 19.65''$$

$$\therefore \boxed{B = N = 19.75''}$$

Now, $A_2 = 35 \times 35 = 1225$

$A_1 = 19.75 \times 19.75 = 390.0625$

$$\therefore \sqrt{\frac{A_2}{A_1}} = 1.77 < 2.0$$

[Not OK]

2nd trial :

Let us assume, $\sqrt{\frac{A_2}{A_1}} = 1.5$

So, $f_{p(max)} = 0.85 f_c' \sqrt{\frac{A_2}{A_1}} = 0.85 \times 4 \times 1.5 = 5.1 \text{ ksi}$

$$B \times N = \frac{P_a \, \Omega}{f_{p(max)}} = \frac{1050 \times 2.5}{5.1} = 514.71 \text{ in}^2$$

Let us choose, $B = N = \sqrt{514.71} = 22.687''$

$$\therefore \boxed{B = N = 22.75''}$$

So, $\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{35^2}{22.75}} = 1.54 > 1.5$

[This is OK]

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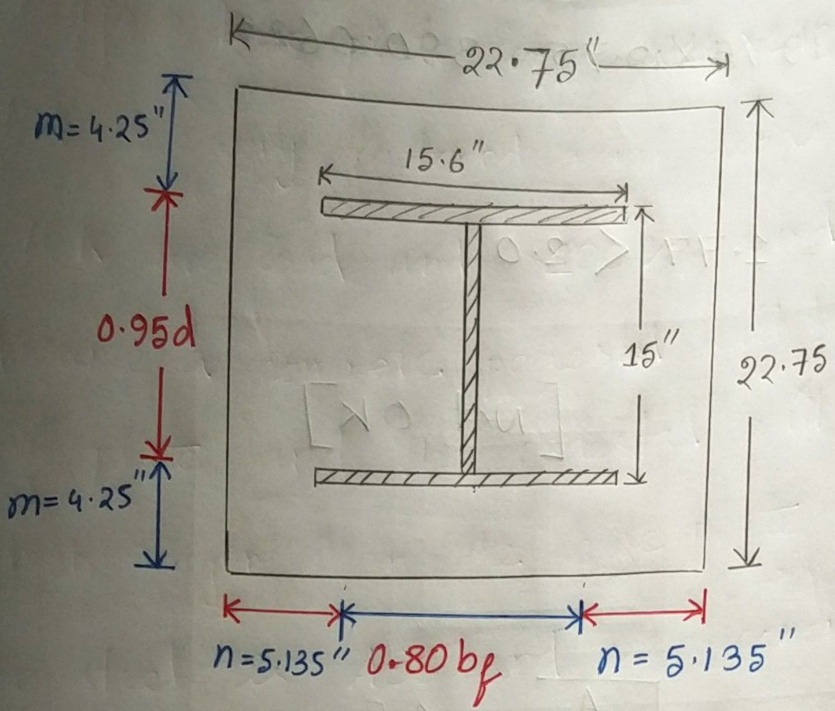


11) Baseplate thickness:

$$m = \frac{1}{2}(N - 0.95d) = \frac{1}{2}(22.75 - 0.95 \times 15) = 4.25''$$

$$n = \frac{1}{2}(B - 0.80 b_f) = \frac{1}{2}(22.75 - 0.80 \times 15.6) = 5.135''$$

Cantilever span, $l = \max^{m} \text{ of } (m, n)$

$$\therefore l = 5.135''$$

Now,

thickness of base plate,

$$F_y = \frac{F}{Z} = \frac{f_p l^2}{2} \times \frac{2}{t^2}$$

$$t = l \sqrt{\frac{2 \Omega P_a}{F_y B N}}$$

$$\therefore t = l \sqrt{\frac{2 f_p}{F_y}}$$

$$= 5.135 \sqrt{\frac{2 \times 1.67 \times 1050}{36 \times 22.75^2}}$$

ASD: $P_a = \dfrac{f_p B N}{\Omega}$

$$l \sqrt{\frac{2 P_a \Omega}{F_y B N}}$$

$$\therefore t = 2.22 \text{ in} \approx 2.25 \text{ inch}$$

So, Baseplate size $22.75'' \times 22.75''$

thickness, $t = 2.25$ inch

Example 13.9.1

Design a base plate for a W14 x 145 column of A992 steel to carry factored axial loads of 400 kip DL, 275 k live load and 100 kip wind.
Assume a concrete pedestal will be under the base plate and the pedestal will have a dimension 6 inch larger than the base plate in each direction. The steel is A36 and the concrete has $f_c' = 3\,ksi$.
[Use the AISC LRFD method]

W14 x 145 geometry:

$$d = 14.8'' \; ; \; b_f = 15.5''$$

1) Factored load

$$P_u = 1.2\,P_{DL} + 1.6\,P_{LL} = 1.2(400) + 1.6(275) = 920\,k$$

$$P_u = 1.2\,P_{DL} + 0.5\,P_{LL} + 1.6\,P_{WL}$$

$$= 1.2(400) + 0.5(275) + 1.6(100)$$

$$= 777.5\,k$$

So, the factored load, $P_u = 920K$

ii) Base plate area

1st trial

Let us assume, $\sqrt{\frac{A_2}{A_1}} = 2.0$ (max^m value)

$$f_{p(max)} = 0.85 f_c' \sqrt{\frac{A_2}{A_1}} = 0.85 \times 3 \times 2 = 5.1 ksi$$

Now, $P_u = \phi f_{p(max)} BN$

$$\therefore B \times N = \frac{P_u}{\phi f_{p(max)}} = \frac{920}{0.60 \times 5.1} = 300.65 in^2$$

Now, $0.80 b_f = 0.80 \times 15.5 = 12.4''$

$0.95 d = 0.95 \times 14.8 = 14.06''$

Let us assume, $B = 15''$ and $N = 20'' \rightarrow A_1 = 300 in^2$

So, $\left.\begin{aligned} B' &= 15 + 6 \times 2 = 27'' \\ N' &= 20 + 6 \times 2 = 32'' \end{aligned}\right\} \longrightarrow A_2 = 864 in^2$

$$\therefore \sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{864}{300}} = 1.7 < 2.0 \quad \text{[not ok]}$$

2nd trial

Let us assume, $\sqrt{\frac{A_2}{A_1}} = 1.5$

$\therefore f_{p(max)} = 0.85 f_c' \sqrt{\frac{A_2}{A_1}} = 0.85 \times 3 \times 1.5 = 3.825$

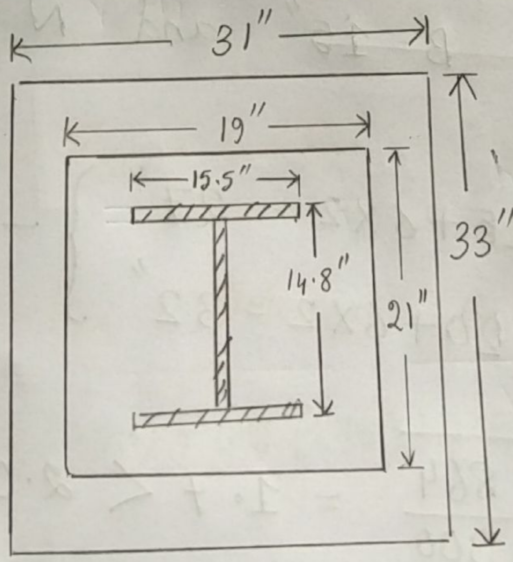
$B \times N = \frac{P_u}{\phi f_{p(max)}} = \frac{920}{0.60 \times 3.825} = 400.87 \text{ in}^2$

Let us assume, $\left.\begin{aligned} B &= 19'' \\ N &= 21'' \end{aligned}\right\} \rightarrow A_1 = 399 \text{ in}^2$

So, $\left.\begin{aligned} B' &= 19 + 6 \times 2 = 31'' \\ N' &= 21 + 6 \times 2 = 33'' \end{aligned}\right\} \rightarrow A_2 = 1023 \text{ in}^2$

So, $\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{1023}{399}} = 1.6 > 1.5$

[This is OK]



iii) Base plate thickness

$$m = \frac{1}{2}(N - 0.95d) = \frac{1}{2}(21 - 0.95 \times 14.8) = 3.47''$$

$$n = \frac{1}{2}(B - 0.80 b_f) = \frac{1}{2}(19 - 0.8 \times 15.5) = 3.3''$$

Cantilever span, $l = \max^n \text{ of } (m, n) = 3.47''$

Now, $t = l \sqrt{\frac{2 f_p}{F_y}}$ $\quad \phi = 0.90$

$l \sqrt{\frac{2 f_p}{F_y}}$

$$= l \sqrt{\frac{2 P_u}{\phi B N F_y}}$$

$F_y = 36 ksi$

[A36 steel]

$$= 3.47 \sqrt{\frac{2 \times 920}{0.90 \times 21 \times 19 \times 36}}$$

$$\therefore t = 1.31'' \approx 1\frac{1}{2}''$$

So, Base plate size $21'' \times 19''$

thickness $= 1\frac{1}{2}''$

Excercise

13.31 Design the column base plate for a W14X211 of A572 grade 50 steel subject to factored axial load P_u of 1100 kips. The plate is to be supported on a 6 ft square concrete footing having $f'_c = 3000$ psi

W14 X211 geometry:

$d = 15\ 3/4$; $b_f = 15\frac{3}{4}''$

1) Base plate area:

Given, $A_2 = 6 \times 6 = 36\ ft^2 = 864\ in^2$

Let, $\sqrt{\frac{A_2}{A_1}} = 2.0$

Now, $f_{p(max)} = 0.85 f'_c \sqrt{\frac{A_2}{A_1}} = 0.85 \times 3 \times 2 = 5.1$ ksi

$$B \times N = \frac{P_u}{\phi f_{p(max)}} = \frac{1100}{0.6 \times 5.1} = 359.5\ in^2$$

Now, $0.80\, b_f = 12.6''$

$0.95\, d = 14.96''$

Let us assume, $\left.\begin{array}{l} B = 19'' \\ N = 19'' \end{array}\right\} A_1 = 361 \text{ in}^2$

So, $\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{[illegible]}{361}}$