Concepts of Steel Design

Companion Course Notes for CES 4605 AISC Fifteen Edition Manual of Steel Construction (LRFD only)

Professor Mullins

10th adaptation from the notes originally penned by Dr. William Carpenter (circa 1985)

Advantages of steel

* High Strength
* Uniformity
* Elasticity
* Permanence
* Ductility (ability to withstand extensive deformation without failure)

    relieves stress concentrations
    permits large deflections before failure

* Toughness (ability to absorb large amounts of energy)

    allows fabrication without fracture

* Easy connections

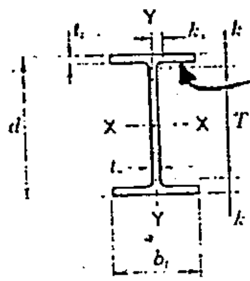
    bolts
    welds
    rivets

* Prefabrication
* Speed erection
* Rolled into special shapes
* Possible reuse if disassembled
* Scrap Value

Disadvantages

* Maintenance Cost
* Fireproofing
* Buckling

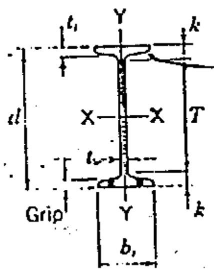
Structural Shapes



MAXIMUM slope of 1 to 20

W SHAPES Dimensions

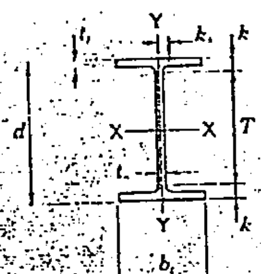
Designation	Area A	Depth d		Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Distance k_1
	In.^{2}	In.		In.		In.	In.		In.		In.	In.	In.
W 40x328	96.4	40.00	40	0.910	15/16	1/2	17.910	17 7/8	1.730	1 3/4	33 3/4	3 1/2	1 11/16
x298	87.6	39.69	39 3/4	0.830	13/16	7/16	17.830	17 7/8	1.575	1 9/16	33 3/4	3	1 5/8
x268	78.8	39.37	39 3/8	0.750	3/4	3/8	17.750	17 3/4	1.415	1 7/16	33 3/4	2 13/16	1 9/16
x244	71.7	39.06	39	0.710	11/16	3/8	17.710	17 3/4	1.260	1 1/4	33 3/4	2 5/8	1 9/16
x221	64.8	38.67	38 5/8	0.710	11/16	3/8	17.710	17 3/4	1.065	1 1/16	33 3/4	2 7/16	1 9/16
x192	56.5	38.20	38 1/4	0.710	11/16	3/8	17.710	17 3/4	0.830	13/16	33 3/4	2 1/4	1 9/16



Slope 1 to 6.

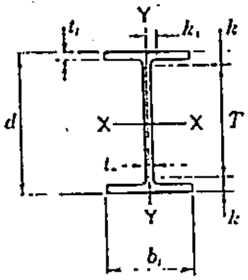
S SHAPES Dimensions

Designation	Area A	Depth d		Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Grip	Max. Flge. Fastener
	In.^{2}	In.		In.		In.	In.		In.		In.	In.	In.	In.
S 24x121	35.6	24.50	24 1/2	0.800	13/16	7/16	8.050	8	1.090	1 1/16	20 1/2	2	1 1/8	1
x106	31.2	24.50	24 1/2	0.620	5/8	5/16	7.870	7 7/8	1.090	1 1/16	20 1/2	2	1 1/8	1



M SHAPES Dimensions

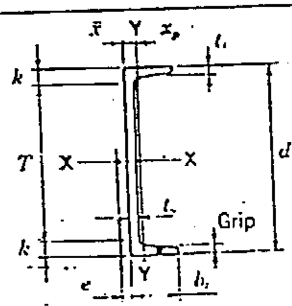
Designation	Area A	Depth d		Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Grip	Max. Flge. Fastener
	In.^{2}	In.		In.		In.	In.		In.		In.	In.	In.	In.
M 14x18	5.10	14.00	14	0.215	3/16	1/8	4.000	4	0.270	1/4	12 3/4	5/8	1/4	3/4



HP SHAPES
Dimensions

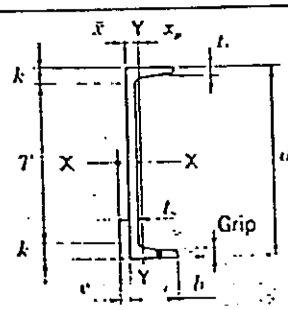
Bearing piles
thick flanges
& webs

Designation	Area A	Depth d		Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Distance k_1
	In.^{2}	In.		In.		In.	In.		In.		In.	In.	In.
HP 14x117	34.4	14.21	14¼	0.805	13/16	7/16	14.885	14⅞	0.805	13/16	11¼	1½	1 1/16
x102	30.0	14.01	14	0.705	11/16	⅜	14.785	14¾	0.705	11/16	11¼	1⅜	1
x 89	26.1	13.83	13⅞	0.615	⅝	5/16	14.695	14¾	0.615	⅝	11¼	1 5/16	15/16
x 73	21.4	13.61	13⅝	0.505	½	¼	14.585	14⅝	0.505	½	11¼	1 3/16	⅞



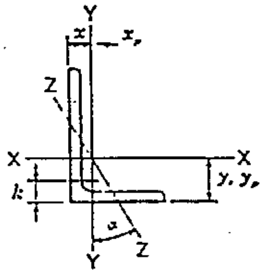
CHANNELS
AMERICAN STANDARD
Dimensions

Designation	Area A	Depth d	Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Average thickness t_f		Distance T	Distance k	Grip	Max. Flge. Fastener
	In.^{2}	In.	In.		In.	In.		In.		In.	In.	In.	In.
C 15x50	14.7	15.00	0.716	11/16	⅜	3.716	3¾	0.650	⅝	12⅛	1 7/16	⅝	1
x40	11.8	15.00	0.520	½	¼	3.520	3½	0.650	⅝	12⅛	1 7/16	⅝	1
x33.9	9.96	15.00	0.400	⅜	3/16	3.400	3⅜	0.650	⅝	12⅛	1 7/16	⅝	1



CHANNELS
MISCELLANEOUS
Dimensions

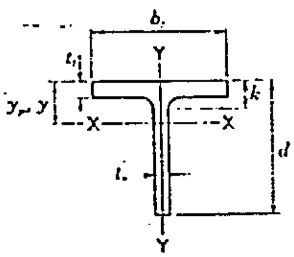
Designation	Area A	Depth d	Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Average thickness t_f		Distance T	Distance k	Grip	Max. Flge. Fastener
	In.^{2}	In.	In.		In.	In.		In.		In.	In.	In.	In.
MC 18x58	17.1	18.00	0.700	11/16	⅜	4.200	4¼	0.625	⅝	15¼	1⅜	⅝	1
x51.9	15.3	18.00	0.600	⅝	5/16	4.100	4⅛	0.625	⅝	15¼	1⅜	⅝	1
x45.8	13.5	18.00	0.500	½	¼	4.000	4	0.625	⅝	15¼	1⅜	⅝	1
x42.7	12.6	18.00	0.450	7/16	¼	3.950	4	0.625	⅝	15¼	1⅜	⅝	1



ANGLES

Equal legs and unequal legs
Properties for designing

Size and Thickness	k	Weight per Ft	Area	Axis X-X					
				I	S	r	y	Z	y_p
In.	In.	Lb.	In.^{2}	In.^{4}	In.^{3}	In.	In.	In.^{3}	In.
L 9x4x ⅝	1⅛	26.3	7.73	64.9	11.5	2.90	3.36	19.7	2.81
9/16	1 1/16	23.8	7.00	59.1	10.4	2.91	3.33	17.9	2.78
½	1	21.3	6.25	53.2	9.34	2.92	3.31	16.0	2.75
L 8x8x1⅛	1¾	56.9	16.7	98.0	17.5	2.42	2.41	31.6	1.05
1	1⅝	51.0	15.0	89.0	15.8	2.44	2.37	28.5	0.938
⅞	1½	45.0	13.2	79.6	14.0	2.45	2.32	25.3	0.827
¾	1⅜	38.9	11.4	69.7	12.2	2.47	2.28	22.0	0.715
⅝	1¼	32.7	9.61	59.4	10.3	2.49	2.23	18.6	0.601
9/16	1 3/16	29.6	8.68	54.1	9.34	2.50	2.21	16.8	0.543
½	1⅛	26.4	7.75	48.6	8.36	2.50	2.19	15.1	0.484



STRUCTURAL TEES

Cut from W shapes
Dimensions

Designation	Area	Depth of Tee d		Stem Thickness t_w		$\frac{t_w}{2}$	Area of Stem	Flange Width b_f		Flange Thickness t_f		Distance k
	In.^{2}	In.		In.		In.	In.^{2}	In.		In.		In.
WT 18 x179.5	52.7	18.70	18 11/16	1.120	1⅛	9/16	20.9	16.730	16¾	2.010	2	3⅛
x164	48.2	18.54	18 9/16	1.020	1	½	18.9	16.630	16⅝	1.850	1⅞	3
x150	44.1	18.370	18⅜	0.945	15/16	½	17.4	16.655	16⅝	1.680	1 11/16	2 13/16
x140	41.2	18.260	18¼	0.885	⅞	7/16	16.2	16.595	16⅝	1.570	1 9/16	2 11/16
x130	38.2	18.130	18⅛	0.840	13/16	7/16	15.2	16.550	16½	1.440	1 7/16	2 9/16
x122.5	36.0	18.040	18	0.800	13/16	7/16	14.4	16.510	16½	1.350	1⅜	2½
x115	33.8	17.950	18	0.760	¾	⅜	13.6	16.470	16½	1.260	1¼	2⅜

Stress - Strain Curves

Steel = alloy of iron (apx 98% or more) & cabon & other

low carbon	high carbon
steel softer	high hardness
more ductile	high strength
weaker	brittle
	poor weldability

Alloyed steel with chromium, silicon, nickel } high strengths, expensive, Not easy to fabricate

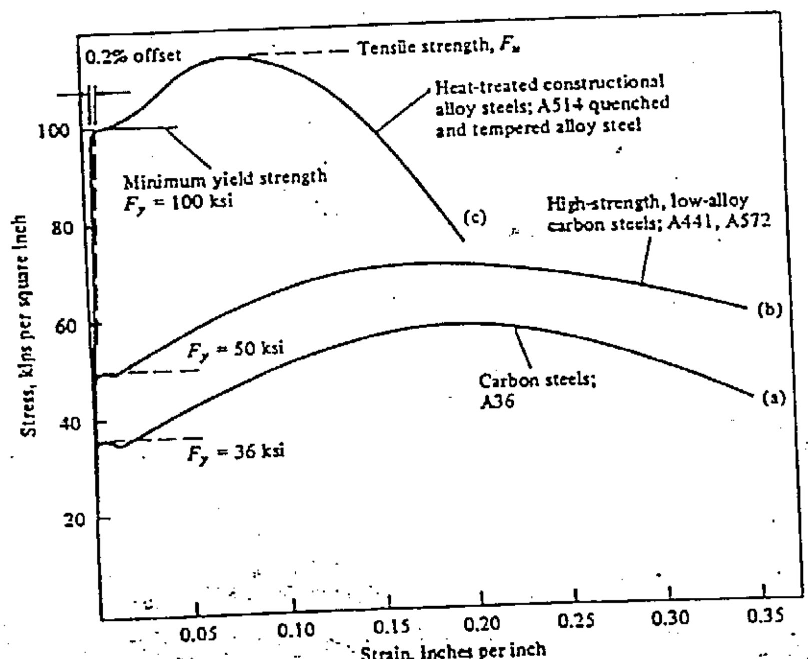


Figure 1-4 Stress-strain curves. (From C.G. Salmon and J.E. Johnson; *Steel Structures*, 2nd ed., Copyright © 1980, Harper & Row, Publishers, Inc.)

ILE 1-1 PROPERTIES OF STRUCTURAL STEELS

ASTM designation	Type of steel	Forms	Recommended uses	Minimum yield stress F_y, ksi[a]	Specified minimum tensile strength F_u, ksi[b]
A36	Carbon	Shapes, bars, and plates	Bolted, welded, or riveted buildings and bridges and other structural uses	36 but 32 if thickness >8 in.	58–80
A529	Carbon	Shapes, plates to ½ in.	Similar to A36	42	60–85
A441	High-strength low-alloy	Shapes, plates, and bars to 8 in.	Similar to A36	40–50	60–70
A572	High-strength low-alloy	Shapes, plates, and bars to 6 in.	Bolted, welded, and riveted construction. Not for welded bridges for F_y grades 55 and above	42–65	60–80
A242	Atmospheric corrosion-resistant high-strength low-alloy	Shapes, plates, and bars to 4 in.	Bolted, welded, or riveted construction; welding technique very important	42–50	63–70
A588	Atmospheric corrosion-resistant high-strength low-alloy	Plates and bars	Bolted and riveted construction	42–50	63–70
A514	Quenched and tempered alloy	Plates only to 4 in.	Welded structures with great attention to technique, discouraged if ductility important	90–100	100–130

[a] F_y values vary with thickness and group (see Tables 1 and 2, Part 1, LRFD Manual).
[b] F_u values vary by grade and type.

Terms to note

F_y = yield strength = yield stress

F_u = tensile strength

A36 most common steel (> 50% of all steel)

A572 next most common

Loads

Dead Load – loads of constant magnitude that remain in one position

Live Loads – change in position & magnitude

Floor loads

Snow and Ice

Rain

Wind

Earthquake

Blast

Thermal forces

⋮

Do we want to design for all these loads at once?

Design for

$$\left.\begin{array}{l} DL + LL \\ DL + W \\ DL + BL \\ \text{etc} \end{array}\right\} \text{with appropriate factors of safety}$$

i.e. certain combinations of loadings

Typical LOADS

Floor Loads

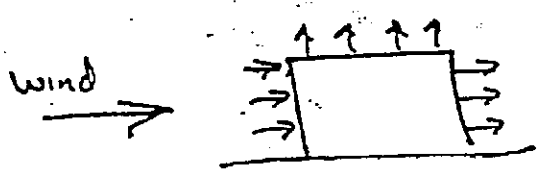
TABLE 2-1 TYPICAL MINIMUM UNIFORM LIVE LOADS FOR DESIGN OF BUILDINGS

Type of building	LL (psf)
Apartment houses	
Apartments	40
Public rooms	100
Dining rooms and restaurants	100
Garages (passenger cars only)	50
Gymnasiums, main floors, and balconies	100
Office buildings	
Lobbies	100
Offices	50
Schools	
Classrooms	40
Corridors first floor	100
Corridors above 1st floor	80
Storage warehouses	
Light	125
Heavy	250
Stores (retail)	
First floor	100
Other floors	75

Wind LOADS

$p = .002558\, C_s V^2$ = basic wind pressure

loads on structure depend on p



etc

Specifications

Muncipal & State government have building codes = laws about how structures are to be built.

Several Organizations have recommended practices that are embodied in many building codes

AISC = American Institute of Steel Construction

AASHTO = American Association of State Highway & Transportation Officials

are examples

Codes are to protect public

Represent state of the art design

Protect engineer in case of failure (if structure designed by code)

Factor of Safety

Do we want to have strength of structure = that required to support expected loads — Do we want extra strength —

We want a factor of safety

Factor of Safety entered in <u>two</u> ways.

LOADS $= Q_i$

λ_i = safety factor

LOADS are increased to $\lambda_i Q_i$

Strength $= R_n$ = nominal strength

Strength is decreased by ϕ

$\therefore$ ϕR_n = usable strength

$\left.\right\}$ ϕ takes into account imperfection in analysis theory, variations in material properties imperfect dimensions

We want

$$\Sigma \lambda_i Q_i \leq \phi R_n$$

$$U = 1.4D \qquad \text{(LRFD Formula A4-1)}$$

$$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \qquad \text{(LRFD Formula A4-2)}$$

Impact loads will only be included in the second of these combinations. Should wind (W) or earthquake (E) forces be involved, the following combinations need to be considered:

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \qquad \text{(LRFD Formula A4-3)}$$

$$U = 1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R) \qquad \text{(LRFD Formula A4-4)}$$

$$U = 1.2D + 1.5E + (0.5L \text{ or } 0.2S) \qquad \text{(LRFD Formula A4-5)}$$

It is necessary to consider impact loading only in combination A4-5 of this group. There is a change in the value of the load factor for L for combinations A4-3, A4-4, and A4-5 for garages, public assembly areas, and all areas where the live load exceeds 100 psf. For such cases 1.0 is to be used.

To account for possibilities of uplift, another load combination is given in the LRFD Specification. This condition is included to cover cases where tension forces develop owing to overturning moments. It will govern only for tall buildings where high lateral loads are present. In this combination the dead loads are reduced by 10 percent to take into account situations where they may have been overestimated.

$$U = 0.9D - (1.3W \text{ or } 1.5E) \qquad \text{(LRFD Formula A4-6)}$$

The magnitudes of the loads (D, L, L_r, etc.) should be obtained from the governing building code or from ANSI 58.1-1982 (*Minimum Design Loads for*

[13] *American National Standard Minimum Design Loads for Buildings and Other Structures* (New York: American National Standards Institute, Inc.), ANSI A58.1-1982.

EXAMPLE 2-1

The various axial loads for a building column have been computed according to the applicable building code with the following results: dead load = 200 k, load from roof = 50 k (live load or snow or rainwater), live load from floors (has been reduced as applicable for large floor area and multistory columns) = 250 k, wind = 80 k, and earthquake = 60 k. Determine the critical design load using the six LRFD load combinations.

Solution

A4-1	$U = (1.4)(200) = 280$ k
A4-2	$U = (1.2)(200) + (1.6)(250) + (0.5)(50) = 665$ k ←
A4-3(a)	$U = (1.2)(200) + (1.6)(50) + (0.5)(250) = 445$ k
A4-3(b)	$U = (1.2)(200) + (1.6)(50) + (0.8)(80) = 384$ k
A4-4	$U = (1.2)(200) + (1.3)(80) + (0.5)(250) + (0.5)(50) = 494$ k
A4-5	$U = (1.2)(200) + (1.5)(60) + (0.5)(250) = 455$ k
A4-6(a)	$U = (0.9)(200) - (1.3)(80) = 76$ k
A4-6(b)	$U = (0.9)(200) - (1.5)(60) = 90$ k

The critical factored load combination or design strength required for this column is 665 k as determined by LRFD Formula A4-2. It will be noted that the results of Formula A4-6 do not indicate an uplift problem.

TABLE 2-2 TYPICAL RESISTANCE FACTORS

Resistance or ϕ factors	Situations
1.00	Bearing on the projected areas of pins, web yielding under concentrated loads, slip-resistant bolt shear values
0.90	Beams in bending and shear, fillet welds with stress parallel to weld axis, groove welds-base metal
0.85	Columns, web crippling, edge distance and bearing capacity at holes
0.80	Shear on effective area of full-penetration groove welds, tension normal to effective area of partial-penetration groove welds
0.75	Bolts in tension, plug, or slot welds, fracture in the net section of tension members
0.65	Bearing on bolts (other than A307)
0.60	Bearing on A307 bolts, bearing on concrete foundations

ϕ values such that designs apx 99.7% reliable

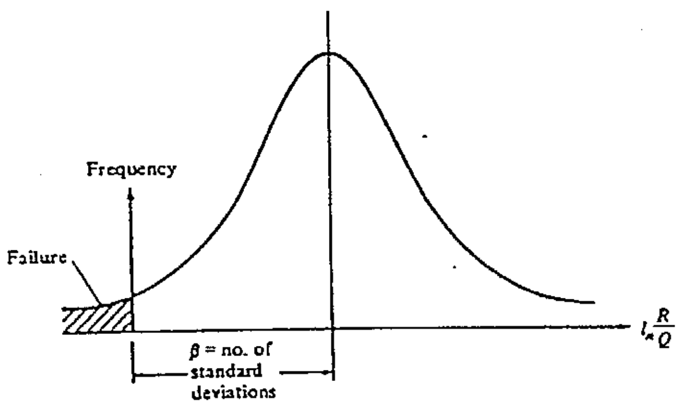


Figure 2-2

LRFD vs Allowable Stress Method

LRFD

$$\Sigma \lambda_i Q_i \leq \phi R_n$$

Allowable Stress

$$\Sigma Q_i \leq \frac{R_n}{F.S.}$$

SAME F.S. on DL as LL

if $\frac{LL}{DL} < 3$ savings with LRFD

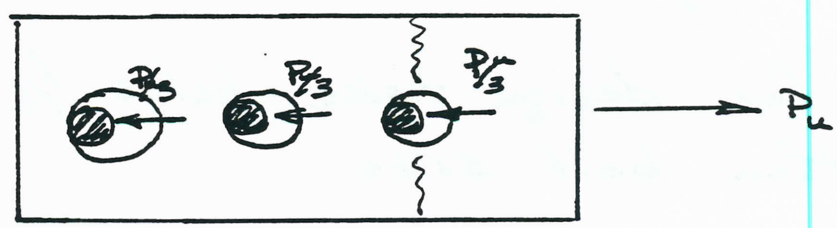
LRFD good in that it gives more uniform reliability for all steel structures

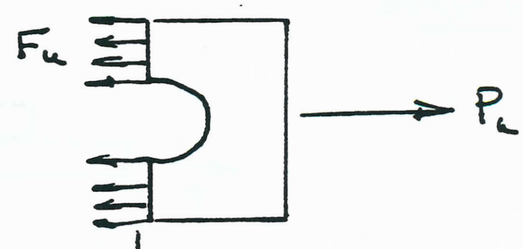
Rod fails when stress reaches F_y because it keeps elongating

$\therefore P_u = \phi_t F_y A_g$ $\qquad \phi_t = .9$ *

(16.1-28)

We could also get fracture through 1^{st} row of bolt holes





assume stress uniform = F_u

$P_u = \phi_t F_u A_e$ $\qquad \phi_t = .75$ **

A_e = Net effective area

(16.1-28)

P_u = smaller of * or **

Net Area

A_g = gross area

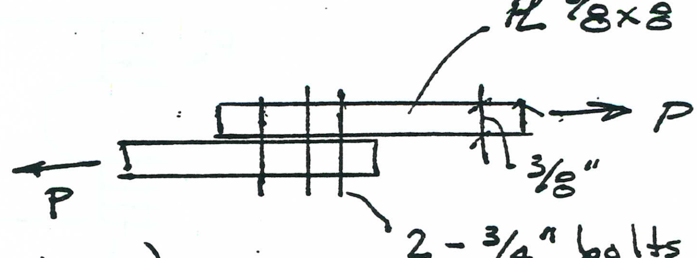
A_N = Net area = A_g − holes

Need to make holes 1/16" bigger than bolt size

When punch holes, we assume we destroy 1/16" d of material (16.1-20)

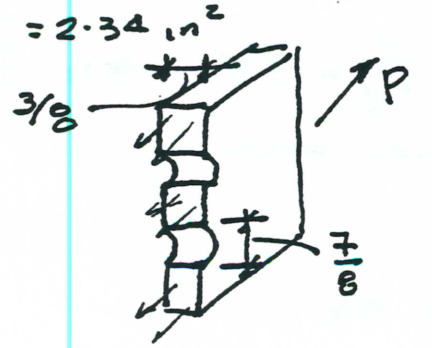
∴ for design take holes 1/8" bigger than bolt size

Example

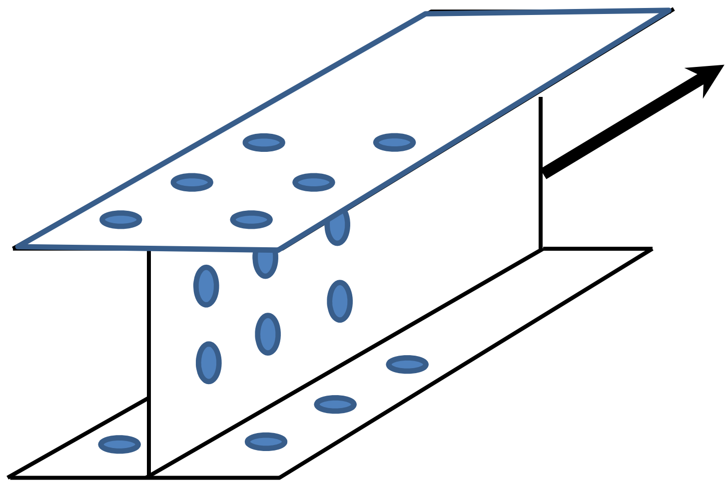


$$A_N = \frac{3}{8}'' \times 8'' - 2\left(\frac{3}{4}'' + \frac{1}{8}''\right)\left(\frac{3}{8}''\right)$$

$= 2.34 \text{ in}^2$



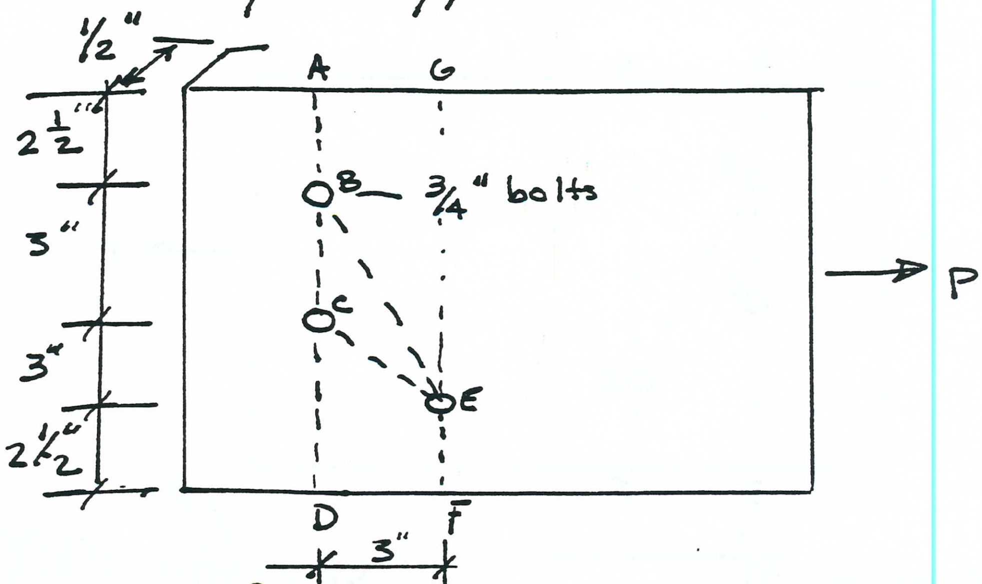
<u>Example</u>



W18x71 *AISC Code Page 1-22* $A = 20.9in^2$; $t_w = 0.495''$; $t_f = 0.810''$

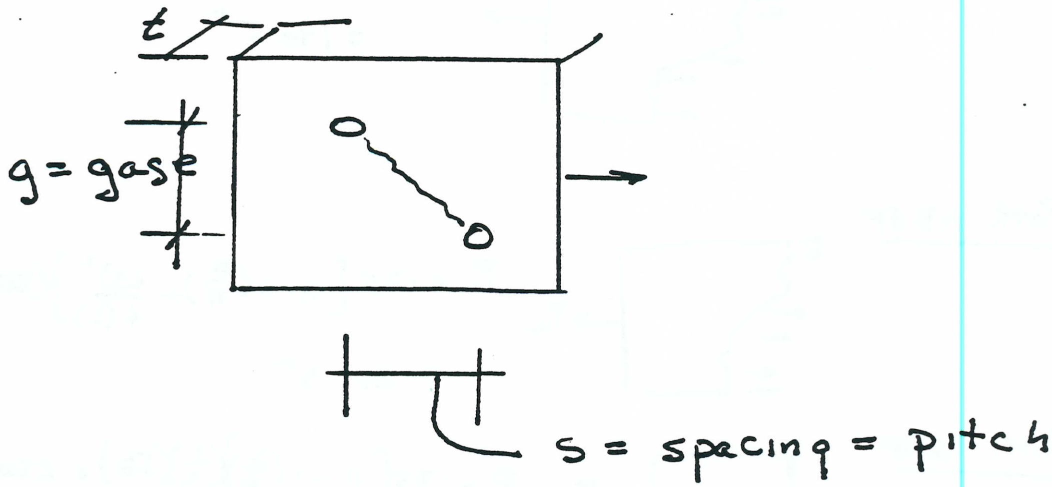
7/8" Bolts

Effect of Staggered holes



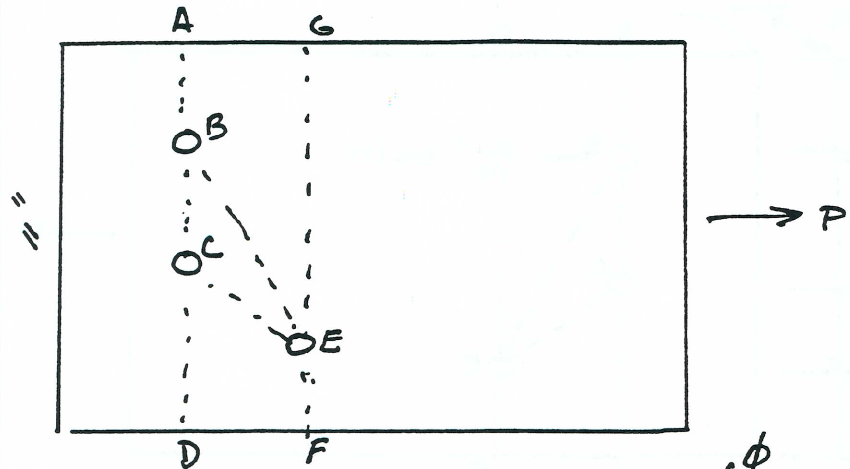
Possible failure paths ABCD
ABEF
ABCEF

Definition

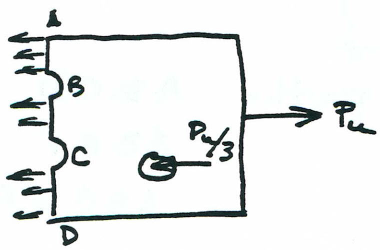


If failure along diag line add correction of $\frac{ts^2}{4g}$ to Net area

(16.1-20)



Path ABCD

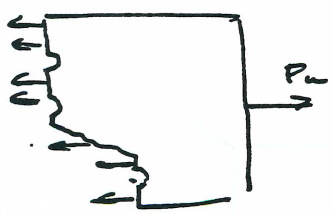


ϕ

$\frac{2}{3}P_u = (.75)\left[11-2\left(\frac{7}{8}\right)\right]\left(\frac{1}{2}\right)\left(58\frac{k}{in^2}\right)$

$P_u = 301.8^k$

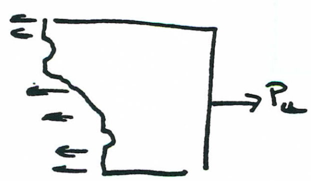
Path ABCEF



$P_u = .75\left[11-3\left(\frac{7}{8}\right)+\frac{(3)^2}{4(3)}\right]\left(\frac{1}{2}\right)(58)$

$= 198.5^k$

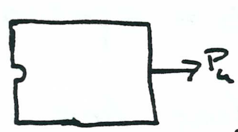
Path ABEF



$P_u = .75\left[11-2\left(\frac{7}{8}\right)+\frac{(3)^2}{4(6)}\right](58)\left(\frac{1}{2}\right)$

$= 209.3^k$

Path GEF



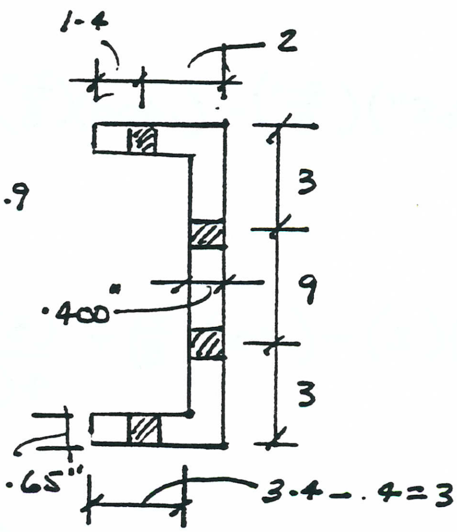
$P_u = .75\left[11-1\left(\frac{7}{8}\right)\right]\left(\frac{1}{2}\right)(58) = 220.2^k$

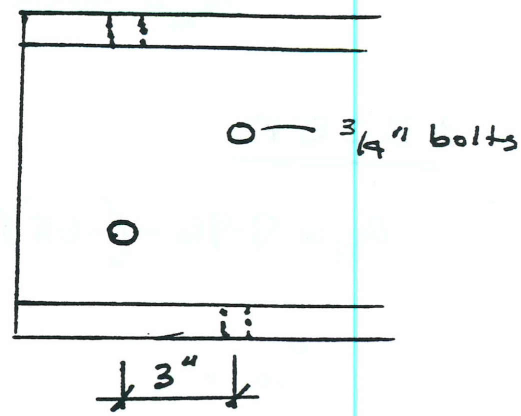
ϕ

stress on gross area $= .9(11)\left(\frac{1}{2}\right)(36) = 178.2^k$ ← controls

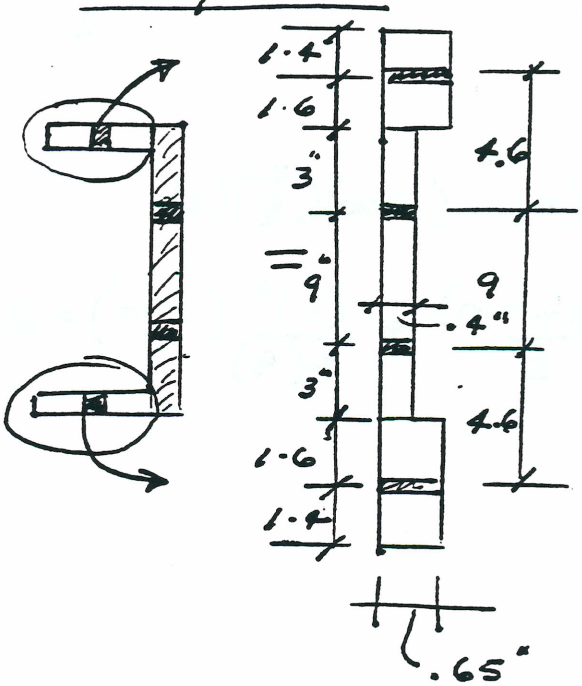
Staggered holes in web & flange

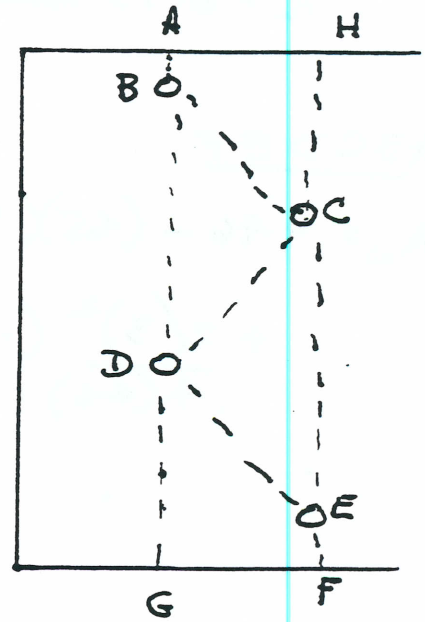
C15×33.9





To do problem





Path

ABDG or HCEF

$$A_N = 9.96 - (.65'')\left(\tfrac{7}{8}''\right) - (.400)\left(\tfrac{7}{8}''\right) = 9.04$$

ABCEF

$$A_N = 9.96 - (.65)\left(\tfrac{7}{8}\right)(2) - (.400)\left(\tfrac{7}{8}\right) + \frac{(3'')^2}{4(4.6)}\left(\quad\right)$$

$$\frac{.65 + .400}{2}$$

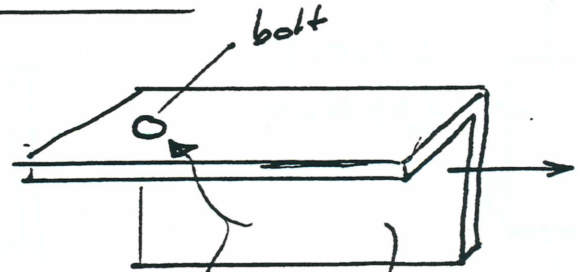
$$A_n = 8.73$$

ABCDEF

$$A_N = 9.96 - (.65)\left(\tfrac{7}{8}\right)(2) - (.400)\left(\tfrac{7}{8}\right)(2)$$

$$+ \frac{2(3)^2}{4(4.6)}\left(\frac{.650 + .400}{2}\right) + \frac{(3)^2}{4(9)}(.400) = 8.74 \text{ in}^2$$

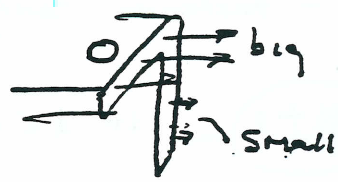
Net effective Area



out here stress ≈ uniform over crossection

Stress in leg must run up to bolt

∴ stress near bolt looks like



to take into account Non uniform stress distribution

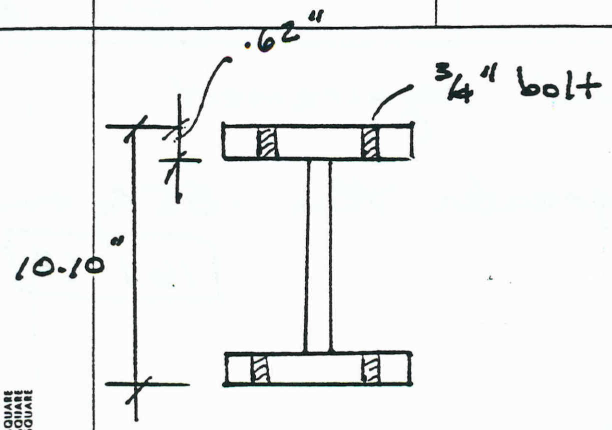
$$A_e = \text{Net effective area} = U A_N$$

(16.1-29)

Shear Lag Factor (16.1-30 and 300)

EXAMPLE A36

What is $P_{u_{MAX}}$



W10×45, A36

$$P_u = \phi F_y A_g = .9\left(36\frac{k}{in^2}\right)(13.3\,in^2) = 430.9^k \leftarrow \text{controls}$$

$$A_N = 13.3\,in^2 - 4\left(\frac{7}{8}"\right)(.620\,in) = 11.13\,in^2$$

(16.1-30) Case 7

Is $b_f > \frac{2}{3}d$

$$8.02" \overset{?}{>} \frac{2}{3}(10.10) = 6.7 \quad \text{yes}$$

$$U = .9$$

$$A_e = U A_N = (.9)(11.13\,in^2) = 10.02\,in^2$$

$$P_u = \phi F_u A_e = (.75)\left(58\frac{k}{in^2}\right)(10.02\,in^2)$$

$$= 435.9^k$$

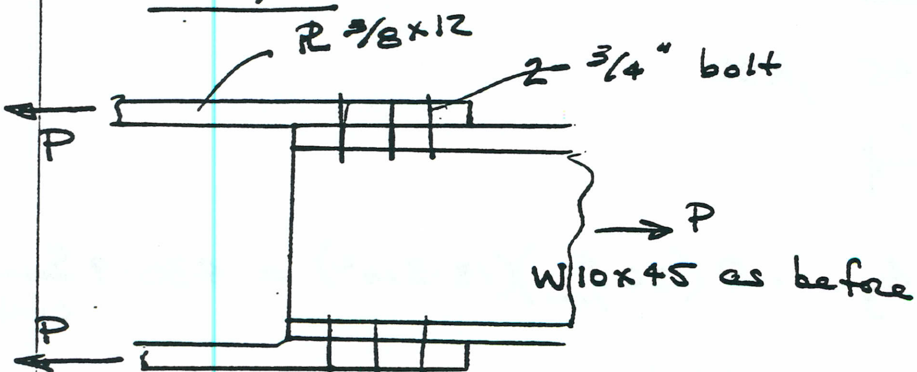
$$\therefore P_u = 430.9^k$$

Connecting PLs - special requirement

A_N can not be greater than $.U_i A_g$

(16.1-30)

Example



U,

Check Plates

$P_u = \phi_t F_y A_g = .9\left(36\frac{k}{in^2}\right)\left(2 \times \frac{3}{8}'' \times 12''\right) = 291.6^k$ ← controls

$A_N (2\text{ PLs}) = \left(\frac{3}{8} \times 12 - \frac{7}{8}(2)\left(\frac{3}{8}\right)\right)2 = 7.69 \text{ in}^2$

$.85 A_g = .85\left(2 \times \frac{3}{8}'' \times 12''\right) = 7.65 \text{ in}^2$ ← use

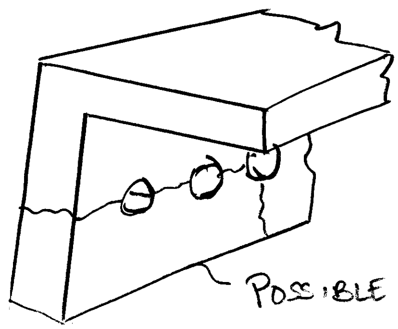
$P_u = \phi A_e F_u$

$A_e = A_n$

$P_u = .75\left(7.65 \text{ in}^2\right)\left(58\frac{k}{in^2}\right) = 332.7^k$

$P_u = 291.6^k$

BLOCK SHEAR



(16.1-138)

CODE CHECKS TWO CASES:

(1) $R_N = 0.6 F_u A_{NV} + U_{bs} F_u A_{Nt}$

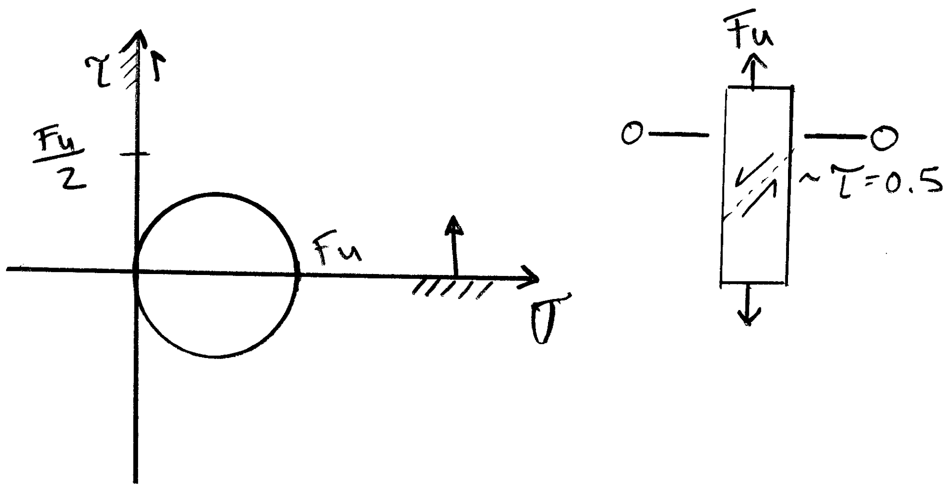
WHICH MUST BE EQUAL TO

(2) $R_N \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

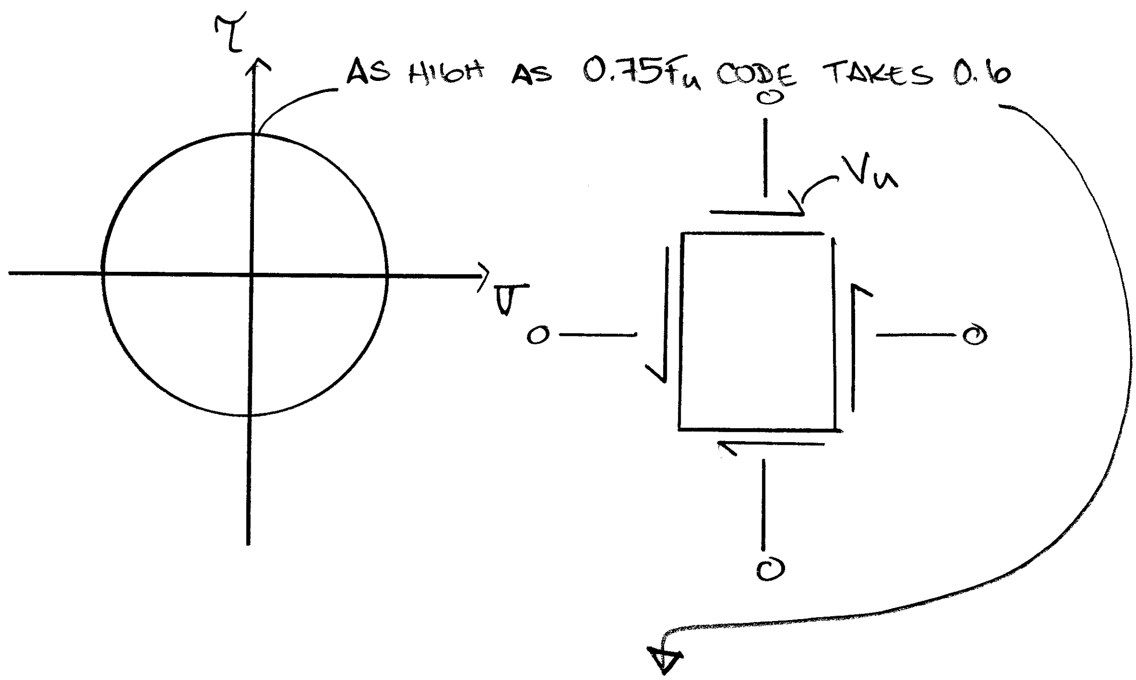
FAILURE STRENGTH AND YEILD STRENGTH IN SHEAR

WHEN IN PURE TENSION:

MOHR'S CIRCLE



WHEN PURE SHEAR:

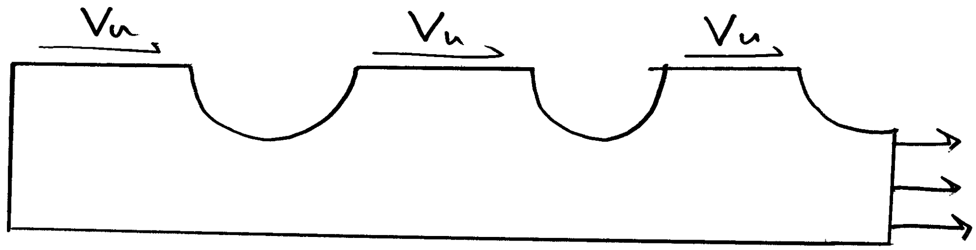


V_u OR $F_u|_{SHEAR} = 0.6 F_u|_{TENSION}$

V_y OR $F_y|_{SHEAR} = 0.6 F_y|_{TENSION}$

CASE 1

ULTIMATE SHEAR + ULTIMATE TENSION



$V_u A_{nv} + F_u A_{nt}$

ULTIMATE SHEAR STRENGTH (0.6Fu)

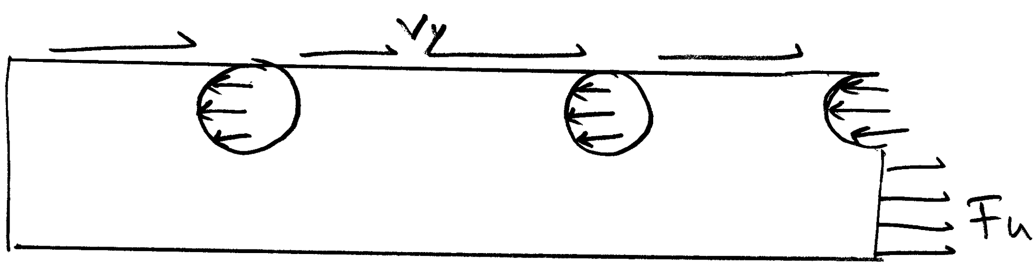
NET AREA

NET AREA TENSION

ULTIMATE TENSION STRENGTH

CASE 2

YIELD IN SHEAR + ULTIMATE TENSION



$V_y A_{gr} + F_u A_{nt}$

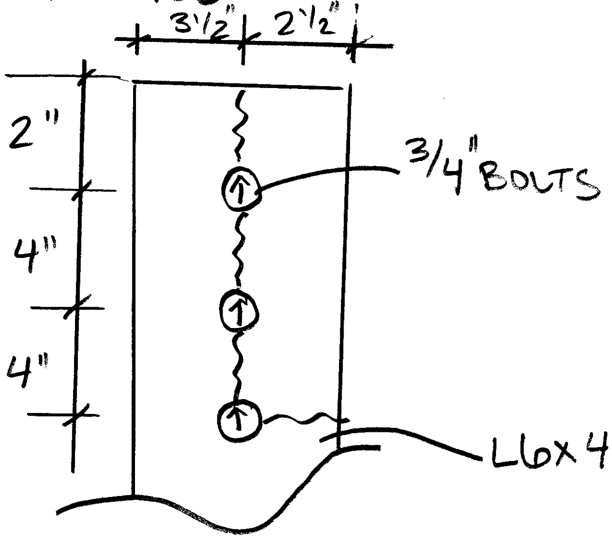
0.6Fy

GROSS AREA IN SHEAR

SAME

National Brand 42-381 50 SHEETS EYE-EASE® - 5 SQUARES 42-382 100 SHEETS EYE-EASE® - 5 SQUARES 42-389 200 SHEETS EYE-EASE® - 5 SQUARES

EXAMPLE



A572 GR. 50
(50ksi F_y;
65ksi F_u)

$$\phi P_{BS_1} = 0.75\left[0.6(65)\left(\overset{226}{8'' - 2\tfrac{1}{2}(7/8)}\right) + (65)\left(\overset{134}{2\tfrac{1}{2}'' - \tfrac{7/8}{2}}\right)\right] \Rightarrow$$

ϕ F_u A_{nv} A_{nt}

$= 270 \text{ kips}$

$$\phi P_{BS_2} = 0.75[.6(50)(8'') + 134]$$

$= 280.5 \text{ kips}$

$P_{BS_1} \geq P_{BS_2}$

$\phi P_{BS} = 270 \text{ kips}$

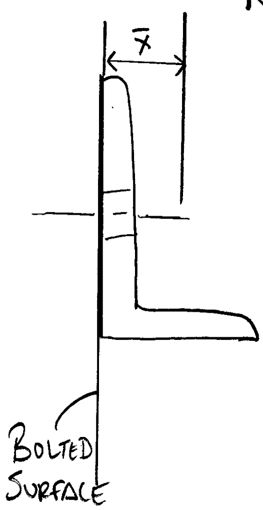
CHECK GROSS & NET AREA CALCS

GROSS AREA

$$\phi P_{N_{GROSS}} = 0.9 A_g f_y = 0.9\,(4.75\,\text{IN}^2)(50\,\text{ksi}) = 214\,\text{k}$$

NET AREA

$$A_N = 4.75 - (1)(\underbrace{7/8}_{3/4 + \frac{1}{16} + \frac{1}{16}})(1/2) = 4.31\,\text{IN}^2$$



$$\bar{x} = 0.981''$$

$$U = 1 - \bar{x}/\ell = 1 - \frac{0.981''}{8''} = 0.88$$

$$A_e = 0.88(4.31) = 3.793$$

$$\phi P_{N_{NET}} = 0.75(4.31)(.88)(65) = 184.89\,\text{k}$$

→ NET AREA CONTROLS ←

$$\boxed{\phi P_N = 185\,\text{k}}$$

42-381 50 SHEETS EYE-EASE® - 5 SQUARES
42-382 100 SHEETS EYE-EASE® - 5 SQUARES
42-389 200 SHEETS EYE-EASE® - 5 SQUARES
National® Brand

Design of Tension Members

$$P_u \leq \phi P_n$$

$\phi_t P_n = \phi_t * F_y * A_g$

$\phi_t = 0.90$

$$P_u = \phi P_n$$

$P_u = \phi_t * F_y * A_g$

Step 1: Solve for $A_{g\,min} = \dfrac{P_u}{0.9F_y}$

$P_u = \phi_t * F_u * A_e$

Step 2: Check $A_{e\,min} = \dfrac{P_u}{0.75F_u}$

Step 3: Check Slenderness Ratio

Slenderness Ratio

Provide minimum compressive strength in case load reverses direction due to wind, earthquake, etc…

$$\frac{L}{r} < 300 \qquad \text{(AISC 16.1-26)}$$

L = Unsupported Length of Member

r = Smallest Radius of Gyration of the Member

$$r = \sqrt{I/_A}$$

I = Least Moment of Inertia

A = Cross-sectional Area

$$r_{min} = L/_{300}$$

Example

$L = 30'$

A36

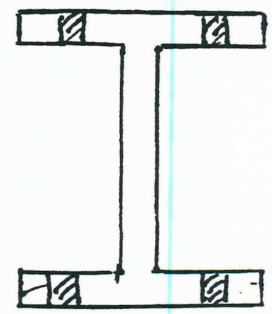
$P_D = 140^k$

$P_L = 80^k$

at least 3 bolts in line

$\frac{7}{8}$" bolts

get W12 section



Get P_u

$$P_u = 1.4D = 1.4(140^k) = 196^k$$

$$P_u = 1.2D + 1.6L = 1.2(140) + 1.6(80) = 296 \leftarrow \text{controls}$$

Get Min A_g

$$\text{Min } A_g = \frac{P_u}{\phi_t F_y} = \frac{296^k}{.9(36\frac{k}{in^2})} = 9.14 in^2$$

$\frac{7}{8}" + \frac{1}{8}"$

$$\text{min } A_g = \frac{P_u}{\phi_t U F_u} + \text{est holes} = \frac{296^k}{.75(.9)(58\frac{k}{in^2})} + 4(1in)(.52in)$$

Assume – must check after member selected

4 holes

est flange thickness

$= 9.64 in$

Get Preferable min r

$$r_{min} = \frac{L}{300} = \frac{30ft(\frac{12in}{ft})}{300} = 1.2 in$$

try W 12x35

$A_g = 10.3 in^2$

$d = 12.50''$

$b_f = 6.56''$

$t_f = .52''$

$r_y = 1.54$

<u>check</u>

$P_u = \phi_t F_y A_g = (.9)(36 \frac{k}{in^2})(10.3 in^2) = 333.7^k > 296^k$ ok

$P_u = \phi_t F_u A_e$

$A_n = 10.3 in^2 - 4(1.0'')(.52'') = 8.22 in^2$

$A_e = U A_n$

$\frac{b_f}{d} = \frac{6.56}{12.50} = .52 < 2/3$

$\therefore U = .85$

$\therefore A_e = .85(8.22 in^2) = 6.99 in^2$

$P_u = \phi_t F_u A_e = .75(58 \frac{k}{\cancel{in^2}})(6.99 \cancel{in^2}) = 303.9^k > 296^k$ ok

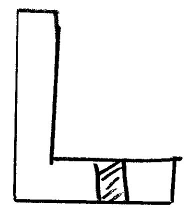
$\frac{L}{r} = \frac{30 \cancel{ft}}{1.54 in} \left(\frac{12 in}{\cancel{ft}}\right) = 234 < 300$ OK

Use <u>W12x35</u>

National® Brand
42-381 50 SHEETS EYE-EASE® - 5 SQUARES
42-382 100 SHEETS EYE-EASE® - 5 SQUARES
42-389 200 SHEETS EYE-EASE® - 5 SQUARES

EXAMPLE

$L = 9\text{FT}$
$P_{DL} = 30k$
$P_{LL} = 40k$



$7/8$" BOLT (3 IN LINE)

BOLTED ONE FLANGE ONLY

FIND LIGHTEST L SECTION

GET LOADS

$P_u = 1.4D = 1.4(30k) = 42k$

$P_u = 1.2D + 1.6L = 1.2(30k) + 1.6(40k) = 100k$ ←

GET MIN A_g

$$\text{MIN } A_g = \frac{P_u}{\Phi_t f_y} = \frac{100k}{.9(50k)} = 2.22\text{IN}^2$$

GET MIN r

$$\text{MIN } r = \frac{L}{300} = \frac{9'(12\text{IN})}{300} = .36\text{IN}$$

8 MINUTES

GROUP WORK - SELECT SECTION

& CHECK A_{NET} & BLOCK SHEAR

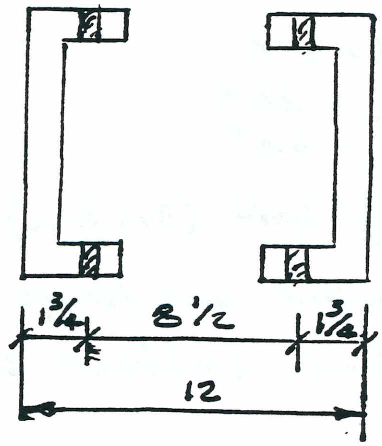
Example

$P_{DL} = 120^k$
$P_{LL} = 240^k$

$L = 30\text{ ft}$

$\frac{7}{8}$" bolts (At least 3 in line)

~~Select 2 Ts~~
Check 2 C12×30

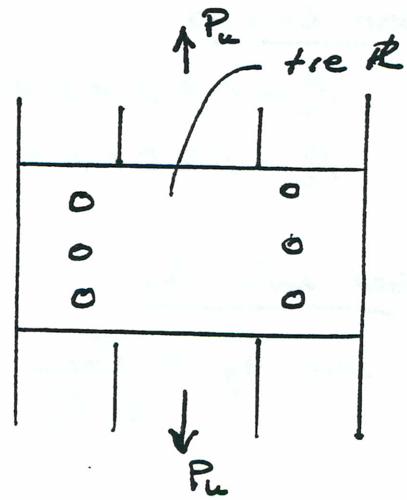


Get P_u

$$P_u = 1.4(120^k) = 168^k$$

$$P_u = 1.2(120^k) + 1.6(240^k)$$

$$P_u = 528^k = (P_u)_{required}$$



Design Strengths

$$P_u = \phi_t F_y A_g$$

$$= .9(36\tfrac{k}{in^2})(2)(8.82\,in^2)$$

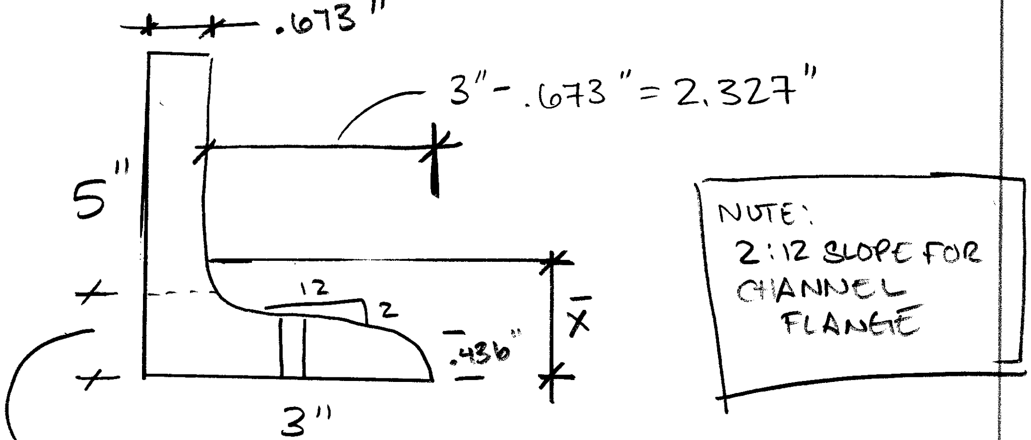
$$= 571.5^k > 528^k \quad \text{ok}$$

$\frac{7}{8}" + \frac{1}{8}"$ 2C's

$$A_n = [8.82\,in^2 - 2(.501\,in)(1\,in)]2 = 15.64\,in^2$$

$A_e = 15.64\,\text{in}^2\,U$

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



$2.327(2/12) + .436'' = .82''$

$U = 1 - \frac{.82}{8''} = .9$

$A_e = 15.64\,\text{in}^2(.9) = 14.1\,\text{in}^2$

$P_u = \phi F_u A_e = .75(65\,\text{k/in}^2)(14.1\,\text{in}^2) = 687.38\,\text{k} > 528\,\text{k}$ OK

National® Brand 42-381 50 SHEETS EYE-EASE® - 5 SQUARES 42-382 100 SHEETS EYE-EASE® - 5 SQUARES 42-389 200 SHEETS EYE-EASE® - 5 SQUARES

42-381 50 SHEETS 5 SQUARE
42-382 100 SHEETS 5 SQUARE
42-389 200 SHEETS 5 SQUARE
NATIONAL

Check slenderness ratio

$$I_x = 2(162\ in^4) = 324\ in^4$$

two C's

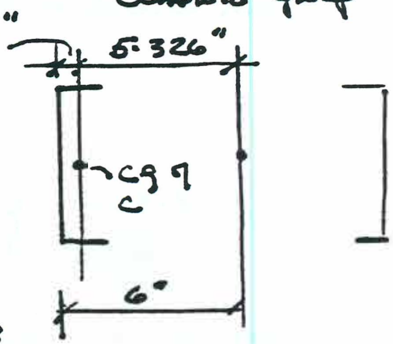
$$I_y = 2(5.14\ in^4) + 2(8.82\ in^2)(5.33\ in)^2$$

I_y about own centroid

Area

dist of centroid to centroid group

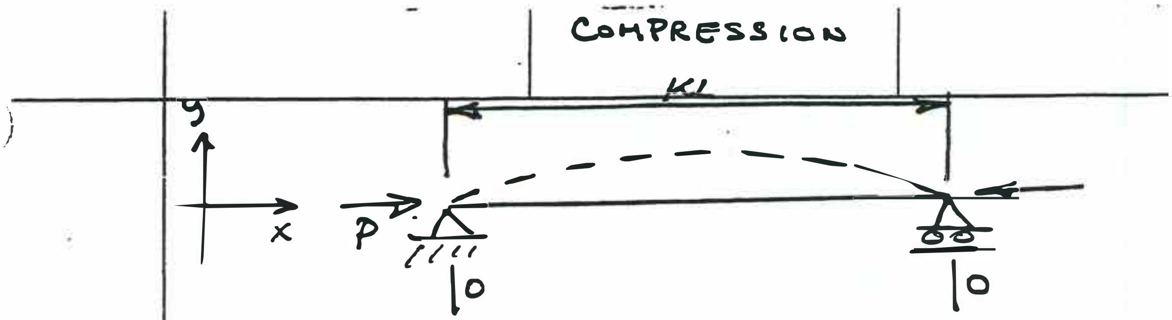
$$= 511\ in^4$$

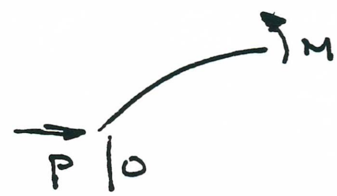


$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{324\ in^4}{2(8.82\ in^2)}} = 4.286$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{511\ in^4}{2(8.82\ in^2)}} = \cancel{4.29}\ 5.38\ in$$

$$\frac{L}{r} = \frac{30ft}{4.29\ in}\left(\frac{12\ in}{ft}\right) = 83.9 < 300 \quad OK$$





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

$$EI\frac{d^2y}{dx^2} + Py = 0$$

$$\frac{d^2y}{dx^2} + \frac{P}{EI}y = 0$$

$$D^2y + \frac{P}{EI}y = 0$$

$$\left(D^2 + \frac{P}{EI}\right)y = 0$$

$$D^2 = -\frac{P}{EI}$$

$$D = \pm\sqrt{\frac{P}{EI}}\,i$$

$$y = A\cos\sqrt{\frac{P}{EI}}\,x + B\sin\sqrt{\frac{P}{EI}}\,x$$

<u>B.C.</u> at $x=0$, $y=0$

$x = KL$, $y=0$

$$y = A\cos\sqrt{\frac{P}{EI}}\,x + B\sin\sqrt{\frac{P}{EI}}\,x$$

at $x=0$, $y=0$

$$0 = A(1) + B(0)$$

$$A=0$$

at $x = KL$, $y=0$

$$0 = B\sin\left(\sqrt{\frac{P}{EI}}\;KL\right)$$

either $B=0$

or $\sqrt{\frac{P}{EI}}\;KL = m\pi$ take $m=1$

$$\sqrt{\frac{P}{EI}}\;KL = \pi$$

$$\sqrt{\frac{P}{EI}} = \frac{\pi}{KL}$$

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

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

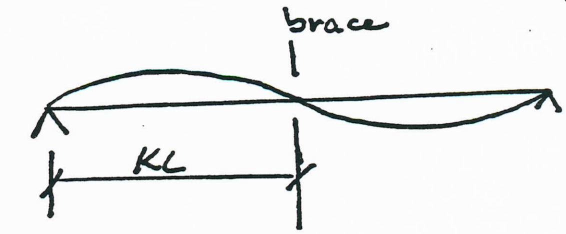
Now $I = Ar^2$

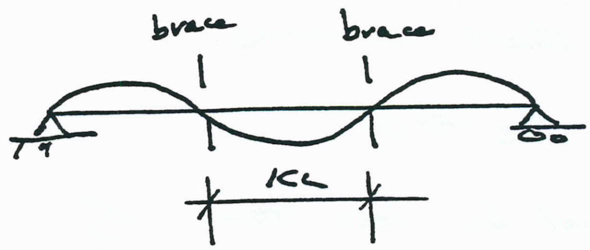
$$P = \frac{\pi^2 E A r^2}{(KL)^2}$$ — *Euler Buckling load*

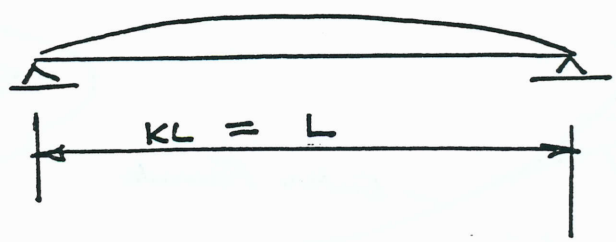
$$\boxed{\frac{P}{A} = Fe = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}}$$ — *Euler Buckling stress*

Examine KL

KL = distance between pts of inflection

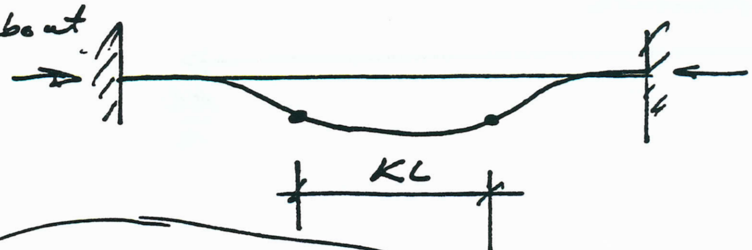






K = 1

How about

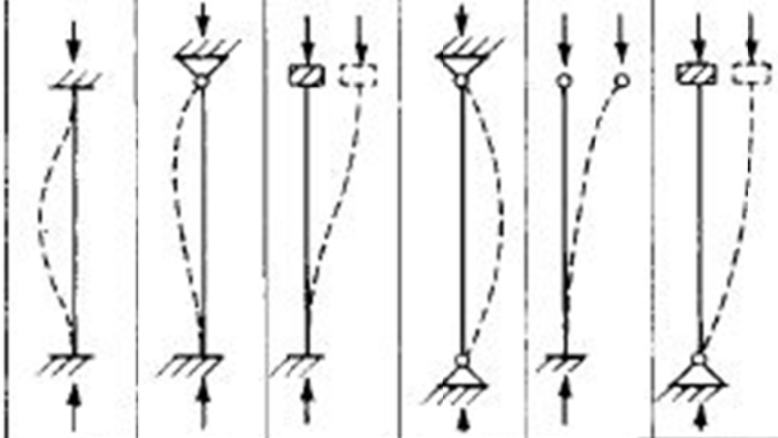


K = .5

See table C-C2.1

(16.1-570)

Tablle C-A-7.1 (AISC 16.1-570)

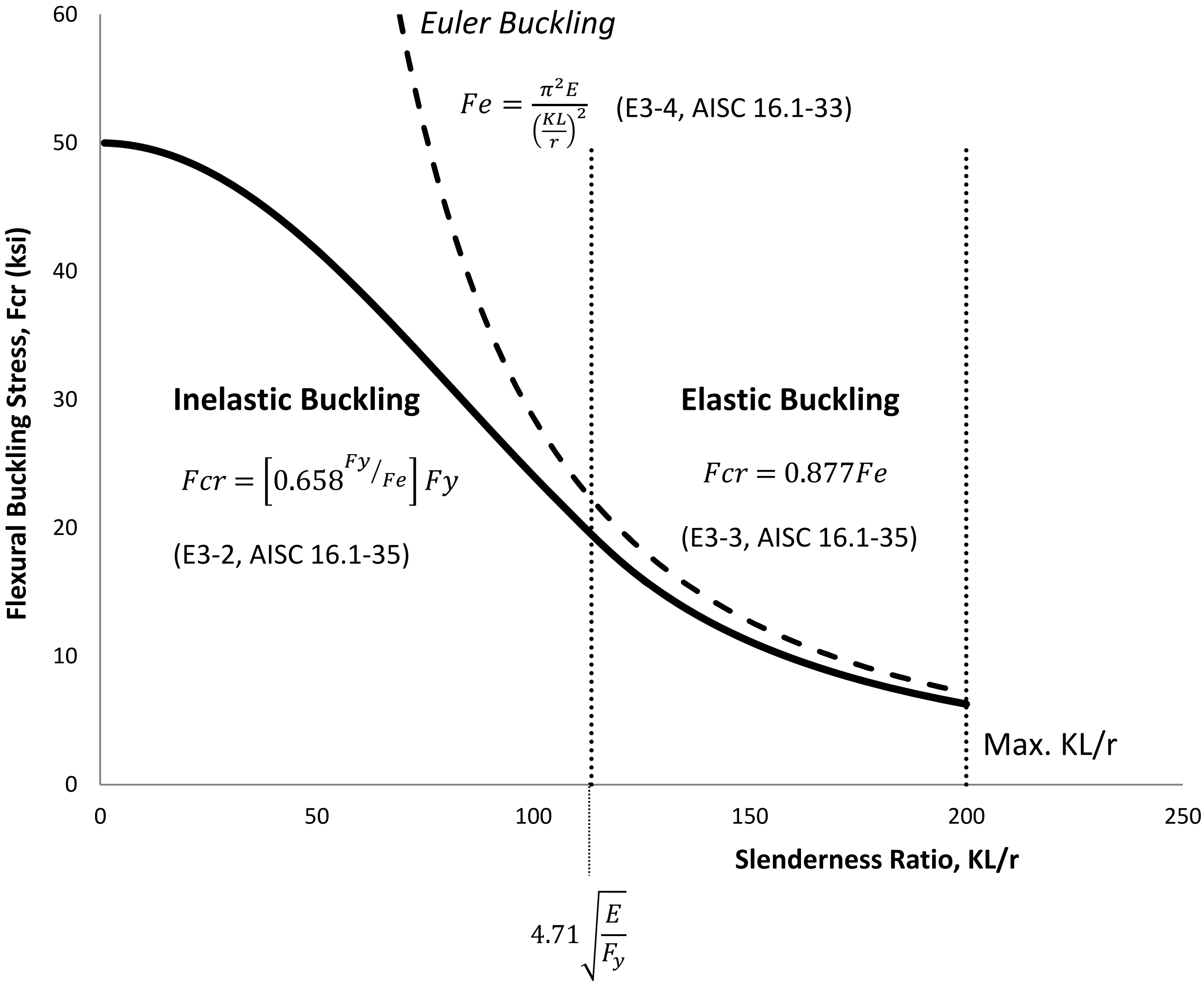
	(a)	(b)	(c)	(d)	(e)	(f)
BUCKLED SHAPE OF COLUMN IS SHOWN BY DASHED LINE						
THEORETICAL K VALUE	0.5	0.7	1.0	1.0	2.0	2.0
RECOMMENDED DESIGN VALUE WHEN IDEAL CONDITIONS ARE APPROXIMATED	0.65	0.80	1.2	1.0	2.10	2.0
END CONDITION CODE	ROTATION FIXED AND TRANSLATION FIXED					
	ROTATION FREE AND TRANSLATION FIXED					
	ROTATION FIXED AND TRANSLATION FREE					
	ROTATION FREE AND TRANSLATION FREE					

LRFD Compression Limit State $P_u \leq \phi_c P_n$ *(B3-1, AISC 16.1-1H)*

LRFD Compression Design Strength $\phi_c P_n = \phi_c F_{cr} A_g$ *(E3-1, AISC 16.1-3p) LRFD*

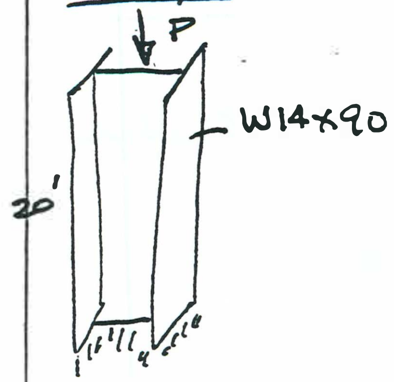
Compression Strength Reduction Factor $\phi_c = 0.9$ *(AISC 16.1-3o)*

Nominal flexural buckling stress F_{cr} :



$\phi_c F_{cr}$ is tabulated in Table 4-22 (AISC 4-229).

Example $KL = 2.1(20') = 42'$

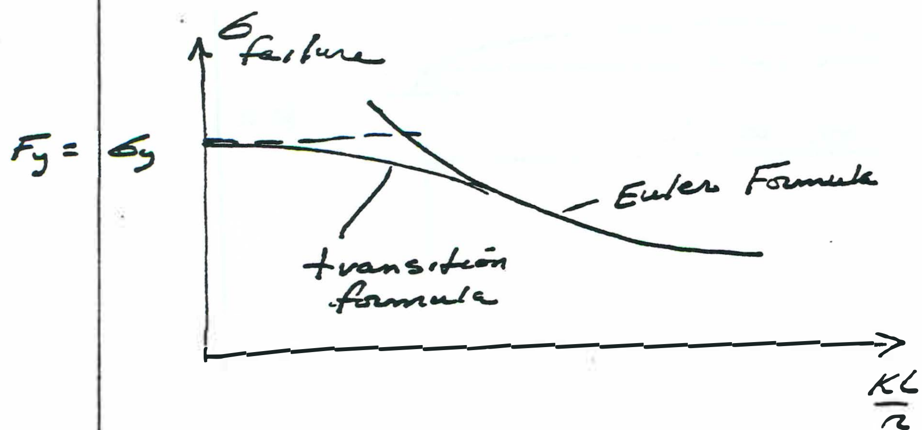


Code now refers to effective length (KL) with the term Lc

How about short columns

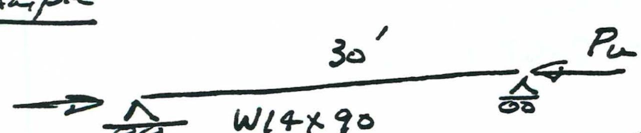
$P_e \to \infty$, $\sigma_e \to \infty$

But σ must be less than σ_y



(4-229)
table 4-14

Example



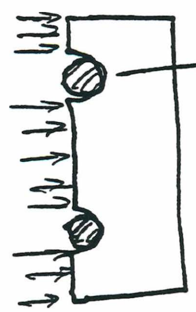
$\left(\frac{KL}{r}\right)_x = \frac{30\,ft}{6.14\,in}\left(\frac{12\,in}{ft}\right) = 58.6$

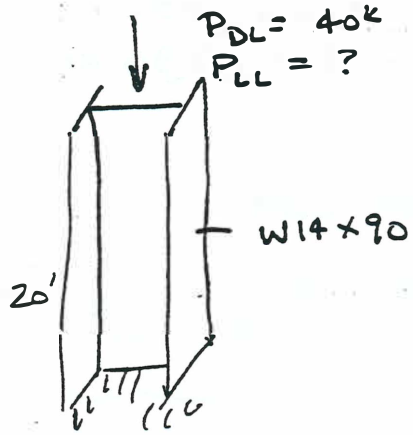
$\left(\frac{KL}{r}\right)_y = \frac{30\,ft}{3.70\,in}\left(\frac{12\,in}{ft}\right) = 97.3 \leftarrow$ Controls

$\phi_c F_{cr} = \phi F_{cr}(98) = 18.46 \frac{k}{in^2}$ ($\phi \sigma_{failure}$)

$P_u = (18.46 \frac{k}{in^2})(26.5\,in^2) = 489\,k$

Note: No need to reduce area for holes when in compression





$KL = 2.1(20') = 42'$

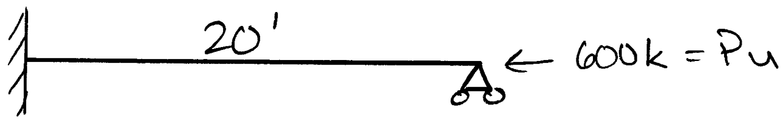
$\frac{KL}{r} = \frac{42\,ft\,(12\,in/ft)}{3.70\,in} = 136.22$ use 137

$\phi_c F_{cr} = \phi_c F_{cr}(137) = 12$ ksi

$\phi_c P_n = 12$ ksi $(26.5\,in^2) = 318$ k

$1.2(40^k) + 1.6\,P_{LL} = 318$

$P_{LL} = 162.75$



$KL = 20'(.8) = 16'$

TRY W14×82

$A = 24.0\ in^2$
$r_x = 6.05\ in$
$r_y = 2.48\ in$

$$\frac{KL}{r_y} = \frac{16'(12''/ft)}{2.48''} = 77.4$$

50ksi $\phi_c F_{cr} = 29.2 - 0.4(29.2 - 28.8) = 29.04$ (4-229)

$$\phi P_N = (24.0)(29.04) = \boxed{696.96k} \quad \text{OK} \checkmark$$

ALTERNATE 1 BASED ON KL_y CONTROL

$KL_y = 16 \quad \phi_c P_N =$ 697 (4-17)

ALTERNATE 2 $4.71\sqrt{\frac{E}{F_y}} = 113$ ($\frac{KL}{r}$ CUTOFF BETWEEN ELASTIC AND INELASTIC BUCKLING)

$77.4 < 113$

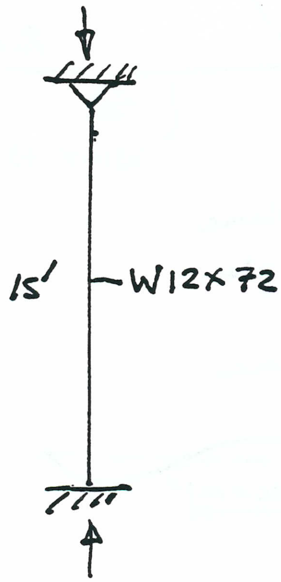
$$\therefore F_{cr} = (0.658)^{F_y/F_e} F_y \qquad F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi(29000)}{(77.4)^2}$$

$= 32.2654ksi \qquad = 47.776ksi$

$$\phi P_N = 0.9(32.3)(24)$$

$$= \boxed{696.93k}$$

Example



$A = 21.1\ in^2$

$r_x = 5.31''$

$r_y = 3.04''$

$P_{U_{MAX}} = ?$

$K = .8$ (16.1-570)

36 ksi

$$\frac{KL}{r} = \frac{.8(15ft)}{3.04\ in}\frac{(12in)}{ft} \doteq 47.37$$

$$\phi_c F_{cr} = 27.19 \frac{k}{in^2}$$

$$P_{u_{MAX}} = 27.19\frac{k}{in^2}(21.1\ in^2) = 573.7^k$$

50 ksi (4-229)

$$\frac{KL}{r} = 48$$

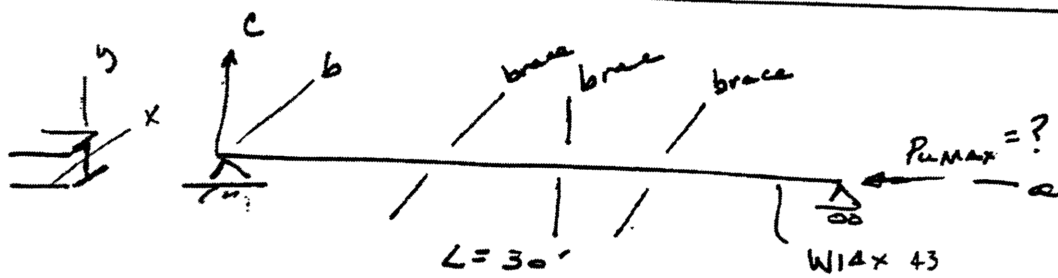
$$\phi_c F_{cr} = 35.9$$

$$P_{U_{MAX}} = \phi P_N$$

$$\phi P_N = (35.9)(21.1) = 757.5$$

or Alternately

$$KL_y = .8(15\ ft) = 12\ ft \qquad P_u = 761k$$



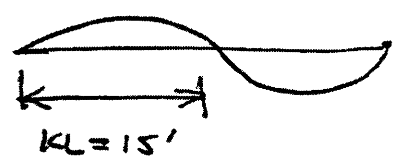
Bm braced at 1/3 pts in ab plane
" " " 1/2 " " bc plane

For buckling in the ab plane
Bm bends thus

Bm bends about y = c axis

$$\left(\frac{KL}{r}\right)_y = \frac{10\ ft}{1.89\ in}\left(\frac{12\ in}{ft}\right) = 63.5 \leftarrow \text{controls}$$

For buckling in the ac plane
Bm bends thus



Bm bends about the b = x axis

$$\left(\frac{KL}{r}\right)_x = \frac{15\ ft}{5.82\ in}\left(\frac{12\ in}{ft}\right) = 30.93$$

$\boxed{50ksi}$

$\frac{KL}{r}$	$\phi_c F_{cr}$
63	33.7
$\boxed{63.5}$	$\boxed{33.55}$
64	33.4

$$\phi_c F_{cr} = 24.83 - .5(24.83 - 24.67) = 24.75 \frac{k}{in^2}$$

$$(P_u)_{possible} = \cancel{24.75}\ 33.55 \frac{k}{in^2}\left(12.6\ in^2\right) = \cancel{311.9}\ 422.7\ k$$

Alternately

$(KL)_x = 15\text{ft}$

$(KL)_y = 10\text{ft}$

For column tables

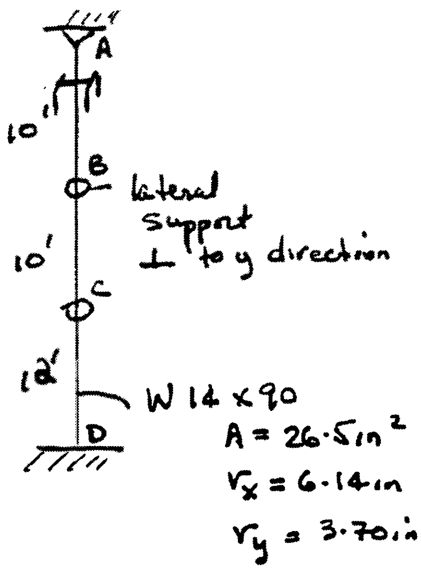
$$(KL)_y = (KL)_y\Big|_{actual} = 10\text{ ft} \leftarrow \text{controls}$$

OR

$$\frac{(KL)_x}{\frac{r_x}{r_y}} = \frac{15\text{ft}}{3.08} = 4.87\text{ ft}$$

bottom (4-17)

$(P_u)_{possible} = 422k$



$KL_x = .8(32ft) = 25.6 ft$

$(KL_y)_{AB} = (KL_y)_{BC} = 1(10ft) = 10 ft$ ← controls buckling about y axis

$(KL_y)_{CD} = .8(12ft) = 9.6 ft$

$\frac{KL}{r_x} = \frac{25.6\,\cancel{ft}\,(12\,in)}{6.14\,\cancel{in}\,\cancel{ft}} = \cancel{50.03}\ 51$ ← controls

$\frac{KL}{r}$	ksi
50	37.5
$\boxed{50.03}$	$\boxed{37.49}$
51	37.2

$\frac{KL}{r_y} = \frac{10ft\,(12in)}{3.70in\ ft} = 32.43$

W/O INTERPOLATION

$\phi P_N = \phi_c F_{cr}(51) A_g$

$= (37.2ksi)(26.5in^2) = 985.8$

$\phi P_N = (37.49ksi)(26.5)$

$= 993.5k$

W/ INTERPOLATING

Alternately (for 50 ksi)

$(KL)_x = 25.6$ ft

$(KL)_y = 10$ ft

For column tables

Controlling $(KL)_y$ is $(KL)_y = 10$ ft

OR

$$\frac{(KL)_x}{\frac{r_x}{r_y}} = \frac{25.6 \text{ ft}}{1.66} = 15.42' \leftarrow \text{controls}$$

P (4-16)

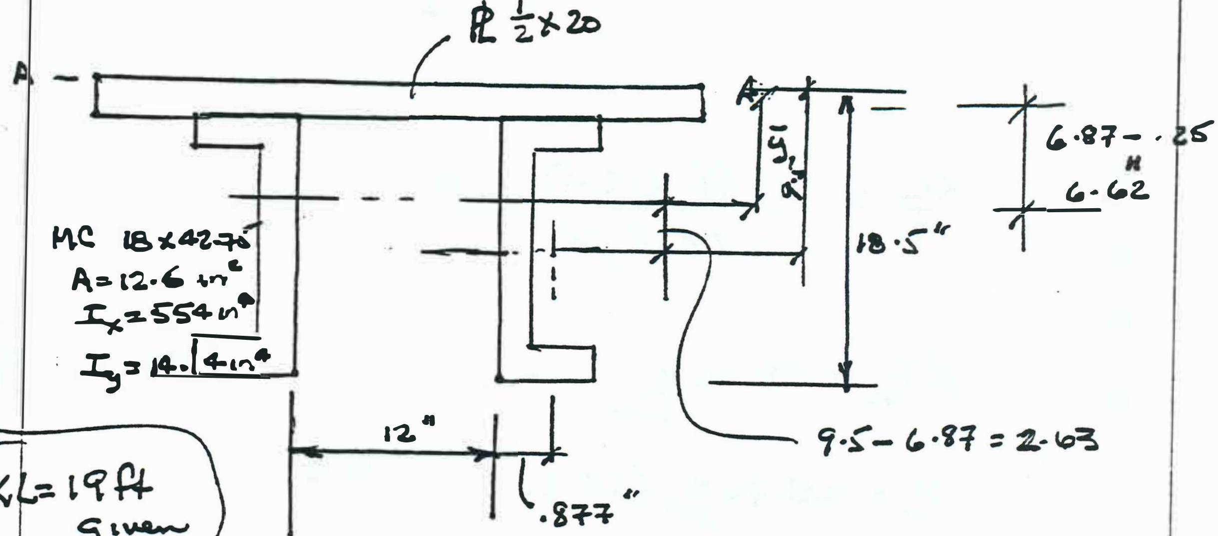
$\phi_c P_N (15') =$ 1000

ROUND $KL_{y\,equiv} = \cancel{15.42} \rightarrow 16$

$\phi_c P_N (16') =$ 979 $\leftarrow$

Interpolating for 15.42 results in 991kips

Example



KL = 19 ft
given

9.50
6.87
2.63

Get A, Ixx, Iyy

$$A = 20''\left(\tfrac{1}{2}\right)'' + (2)(12.6\ in^2) = 35.2\ in^2$$

get $\bar{y}$. (use A-A as ref line)

$$\bar{y} = \frac{(20\ in)\left(\tfrac{1}{2}''\right)(.25\ in) + 2(12.6\ in^2)(9.5\ in)}{35.2\ in^2} = 6.87\ in$$

$$I_x = 2(554\ in^2) + 2(12.6\ in^2)(2.63\ in)^2 + \tfrac{1}{12}(20'')(.5'')^3 + \left(\tfrac{1}{2}in\right)(20\ in)(6.62\ in)^2 = 1721\ in^4$$

$$I_{yy} = 2(14.4\ in^4) + 12.6(6.877\ in)^2(2) + \tfrac{1}{12}\left(\tfrac{1}{2}''\right)(20'')^3$$
$$= 1554\ in^4$$

Get least r

$$r_{Min} = \sqrt{\frac{1554\ in^4}{35.2\ in^2}} = 6.64\ in$$

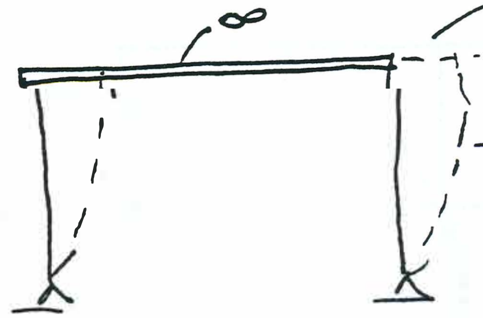
$$\frac{KL}{r} = \frac{19\ \cancel{ft}\ (12\ \cancel{in})}{6.64\ \cancel{in}\ \cancel{ft}} = 34.34$$

$$\phi F_{cr} = 28.76\ \frac{k}{in^2}$$

$$P_u = 28.76\ \frac{k}{\cancel{in^2}}\ (35.2\ \cancel{in^2}) = 1012^k$$

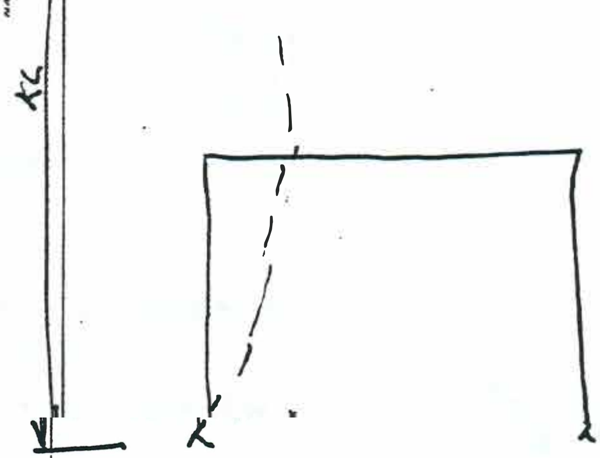
Getting K in rigid Frames

unbraced frame



shape when buckles

But what if girder not ∞ stiff



KL could be near ∞

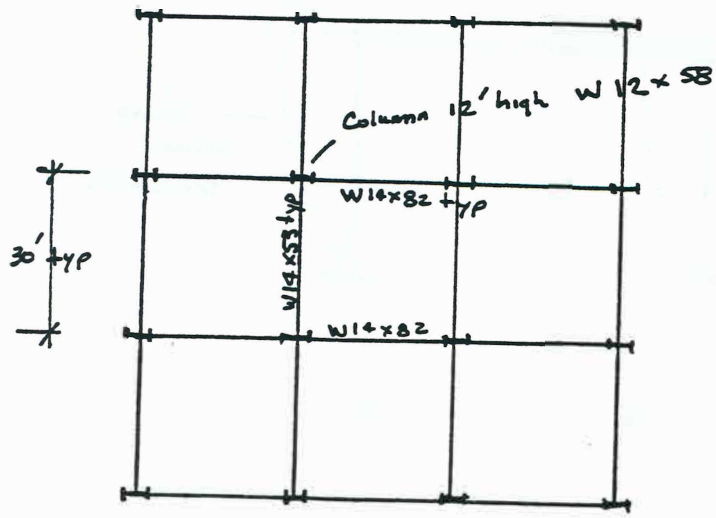
Therefore we have charts on page (16.1-570-572)

$$G = \frac{\Sigma I_c/L_c}{\Sigma I_G/L_G}$$

G=0 then G=1 take

G=∞ then take G=10

(16.1-571)

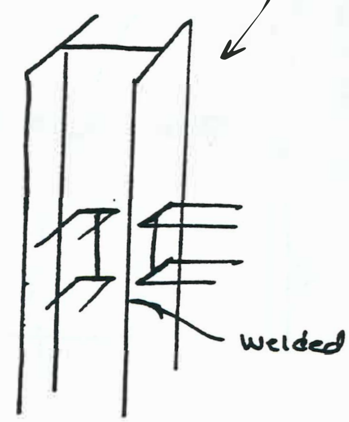


Plan view

20' typ

Sidesway permitted both directions

Connections are welded Moment connections



Examine Column (Interior)

Get $\left(\frac{KL}{r}\right)_x$

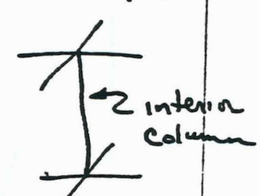
$$G = \frac{\Sigma(I/L)_{col}}{\Sigma(I/L)_{Gir}} = \frac{\cancel{\Sigma}\left(\frac{475\ in^4}{12\ ft}\right)}{\cancel{\Sigma}\left(\frac{882\ in^4}{40\ ft}\right)}$$

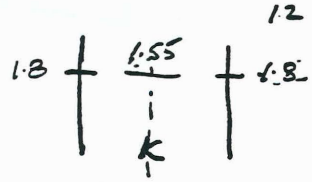
$$= \frac{475}{12} \cdot \frac{40}{882} = 1.80$$

W14×82 $I_x = 882\ in^4$
$I_y = 148\ in^4$

W14×53 $I_x = 541\ in^4$
$I_y = 57.7\ in^4$

W12×58 $I_x = 475\ in^4$
$I_y = 107\ in^4$
$r_x = 5.28\ in$
$r_y = 2.51\ in$
$A = 17.0\ in^2$



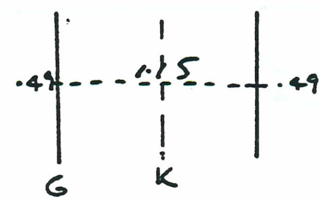


sidesway uninhibited (16.1-572)

$$\left(\frac{KL}{r}\right)_x = \frac{1.55(12ft)(12in)}{5.28in\ \ ft} = 42.3$$

Get $\dfrac{KL}{R_y}$

$$G = \frac{\Sigma (I/L)_{col}}{\Sigma (I/L)_{Gir}} = \frac{\cancel{2}\left(\frac{107\,in^4}{12\,ft}\right)}{\cancel{2}\left(\frac{541\,in^4}{30\,ft}\right)} = \frac{107}{12} \cdot \frac{30}{541} = .49$$



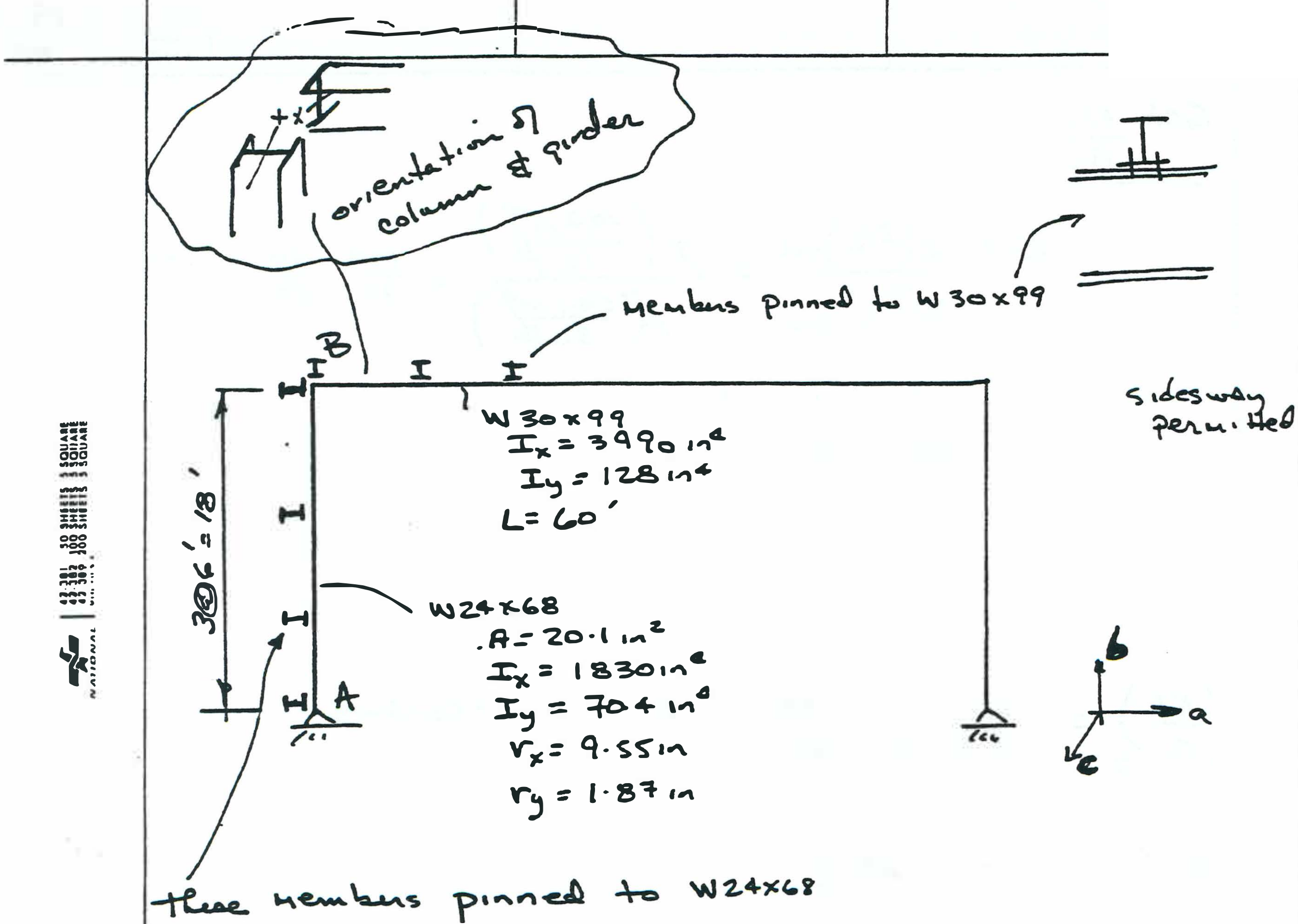
Sidesway uninhibited

$$\left(\frac{KL}{r}\right)_y = \frac{1.15\,(12\,ft)(12\,in/ft)}{2.51\,in} = 66.0 \longleftarrow \text{Controls}$$

$$\phi_c F_{cr}(66) = 32.7\ \frac{k}{in^2}$$

$$(P_u)_{possible} = 32.7\ 3\frac{k}{in^2}\,(17.0\,in^2) = 555.9\ k$$

This type of construction not used much as one would have to do lots of field welding – See next example for a more practical situation



For buckling in the ab plane, column & girder bend about $\underline{c = x}$ axis

Get G_B

$$G_B = \frac{\sum \frac{I_c}{L_c}}{\sum \frac{I_G}{L_G}} = \frac{\frac{1830 \text{ in}^4}{18 \text{ ft}}}{\frac{3990 \text{ in}^4}{60 \text{ ft}}} = 1.53$$

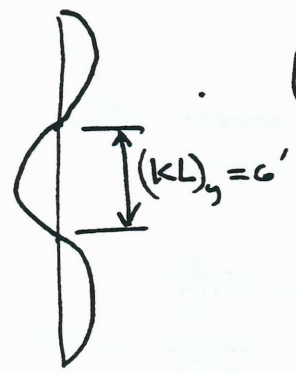
$$G_A = 10$$

$$K = 1.9$$

$$\left(\frac{KL}{r}\right)_x = \frac{1.9(18 \text{ ft})\ (12 \text{ in}/\text{ft})}{9.55 \text{ in}} = 45.6 \leftarrow \text{Controls}$$

For buckling in the bc plane,
column bends about the $\underline{a=y}$ axis

Buckled shape



$$\left(\frac{KL}{r}\right)_y = \frac{6\ \text{ft}\ (12\ \text{in/ft})}{1.87\ \text{in}} = 38.5$$

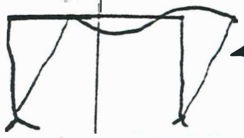
$\phi_c F_{cr}(46) =$ 38.5ksi

$P_u = (20.1\ \text{in}^2)$(38.5ksi) = 773.9k

~~Sidesway~~

Alignment Charts based on a number of assumptions

Behavior elastic



$\left.\begin{array}{l}\text{Girders have reverse} \\ \text{curvature bending}\end{array}\right\}$

others – see, (16.1-570)

If we do not meet these conditions we use corrections.

If Behavior inelastic $G = \dfrac{\sum \frac{I_c}{L_c}}{\sum \frac{I_G}{L_G}} * SRF$

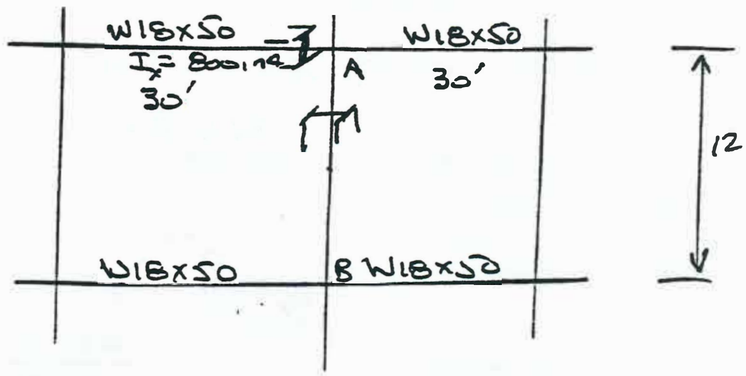
If Far end girder hinged, take $\left(\frac{I}{L}\right)$ of that girder times $\frac{1}{2}$ to get G.

See examples

Select column AB

$P_u = 1240\,k$

Assume Column braced about Weak axis



Assume Column in Elastic Range

Try W12x170 $(A = 50\,in^2)$ $I_x = 1650\,in^4$ $r_x = 5.74\,in$

$$G_A = G_B = \frac{\Sigma I_c/L_c}{\Sigma I_b/L_b} = \frac{2\left(\frac{1650\,in^4}{12\,ft}\right)}{2\left(\frac{800\,in^4}{30\,ft}\right)} = 5.16$$

$K = 2.3$ alignment Chart (16.1-571)

$$\frac{KL}{r_x} = \frac{2.3(12\,ft)}{5.74\,in}\left(\frac{12\,in}{ft}\right) = 57.7$$

$\phi F_{cr} =$ 35.2ksi

$P_u = ($ 35.2ksi $)(50\,in^2) =$ 1760k $> 1240^k$

TRY LIGHTER SECTION

INELASTIC SOLUTION

try W12×152 ($A = 44.7 in^2$, $I_x = 1430 in^4$, r_x 5.66 in)

$$\frac{P_u}{\phi A} = \frac{1240k}{(.85)\,44.7} = 32.6\ k/in^2$$

SRF = 0.244

$$G_A = G_B = \frac{\sum I_c/L_c}{\sum I_G/L_G}\ SRF = \frac{2\left(\frac{1430 in^4}{12ft}\right)}{2\left(\frac{800 in^4}{30ft}\right)}(.244) = 1.09$$

(16.1-570-572)

K = 1.35

$$\frac{KL}{r} = \frac{1.35(12)}{5.66}\left(\frac{12 in}{ft}\right) = 34.4$$

ϕF_{cr} = 28.69 k/in

P_u = (28.69)(44.7 in^2) = 1282 > 1240 k

USE W12×152

National® Brand 42-381 50 SHEETS EYE-EASE® - 5 SQUARES 42-382 100 SHEETS EYE-EASE® - 5 SQUARES 42-389 200 SHEETS EYE-EASE® - 5 SQUARES

Design of Compression Members

Step 1: Determine the load, including an estimated self-weight of the column.

Step 2: Calculate the effective length about the minor axis.

$$(KL)_y = K_y L_y$$

Step 3: Enter Tables 4-1 through 4-20 and find a section that will support the load with an effective length of $(KL)_y$.

Step 4: Check for buckling about the major axis by calculating an "equivalent" effective length with respect to the major axis:

$$(KL)_{x,eq} = \frac{(KL)_x}{r_x/r_y}$$

If $(KL)_{x,eq} < (KL)_y$

Section is good

If $(KL)_{x,eq} > (KL)_y$

Check capacity of section at $(KL)_{x,eq}$

If capacity is still greater than load, section is good.

If not, choose a new section and repeat process.

Leaning Columns

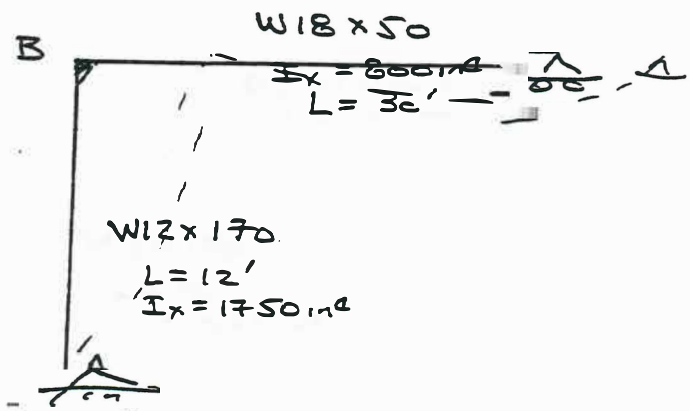
Large single story buildings often have pinned end columns in the middle that are designed with a K = 1.

However, the outer columns must be overdesigned to provide global lateral support to afford the low K in the middle.

Design the center columns with K = 1 and the load as predicted/shown.

Design outer columns by summing all center loads, and splitting between the two outer columns (e.g. left column design load = 100k + 650/2; right column design load = 150k + 650/2). Compute K as appropriate for the end conditions.

If center columns are fixed to the beam, then no leaning column effects should be considered and all loads are resisted locally.



$G_A = 10$

Get G_B

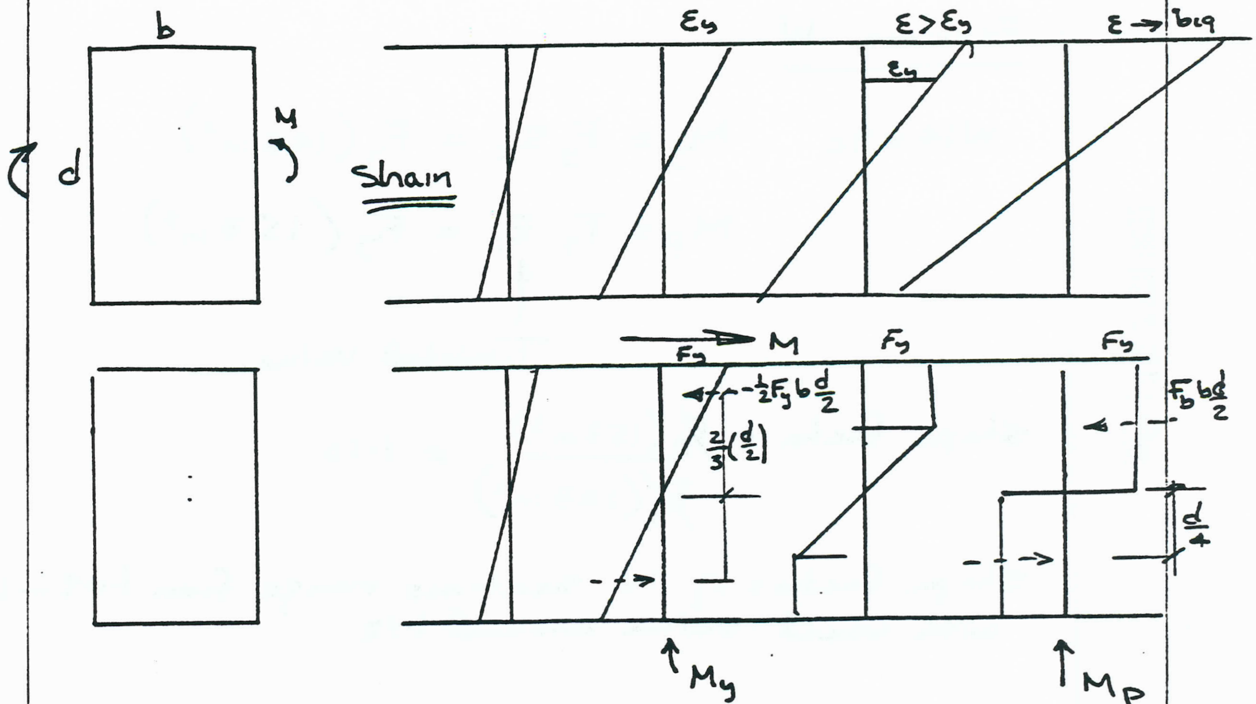
$$G_B = \frac{\Sigma \frac{I_c}{L_c}}{\Sigma \frac{I_G}{L_G}} = \frac{\frac{1750 \text{ in}^4}{12 \text{ ft}}}{\frac{800 \text{ in}^4}{30 \text{ ft}}(.5)}$$

(16.1-570-572)

corrects for fact that we don't have reverse curvature bending

etc

Examine a Rect Steel Bm



What is M at First Yield $= M_y$

$$M_y = F_y S = F_y \frac{bd^2}{6}$$

$$\text{or } M_y = \underbrace{\frac{1}{2} F_y (b) \frac{d}{\cancel{2}}}_{\text{Force}} \underbrace{\left(\frac{\cancel{2}d}{3}\right)}_{\text{l.a}} = F_y \frac{bd^2}{6}$$

What is $M_P =$ full plastic Moment

$$M_P = \underbrace{F_y b \frac{d}{2}}_{\text{force}} \underbrace{\left(\frac{d}{2}\right)}_{\text{l.a}} = F_y \frac{bd^2}{4}$$

$$\text{Shape Factor} = \frac{M_P}{M_y} = \frac{F_y bd^2/4}{F_y bd^2/6} = 1.5$$

Full Plastic Moment - Zone I

OK rectangles have 50% more strength after 1^{st} yield

Examine W

W14 x 90 $\quad M_y = F_y S_x = F_y(143\ in^3)$

$M_p = F_y Z_x = F_y(157\ in^3)$

Tabulated Value

$$\text{shape factor} = \frac{\cancel{F_y}(157\ in^3)}{\cancel{F_y}(143\ in^3)} = 1.10$$

Shape factors of W sections range from 1.09 – 1.18 with usual value around 1.12

If bm adequately braced so that it does not

a) buckle laterally  (twisting instability)

b) does not buckle locally  local instability

(we can prevent local buckling by having fat flanges & webs)

then $\quad M_n = \text{strength} = M_p$

$\phi_b M_n = \phi_b M_p = \text{usable strength}$

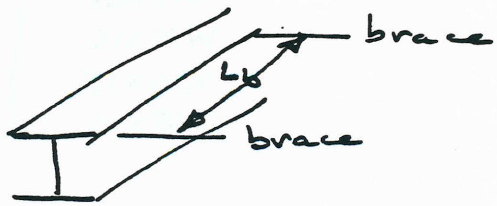
$\uparrow$ .9

See Next page for (a) & (b) discussion

$M_n = \phi_b M_p$ (16.1-46)

$\uparrow$ .9

(a) To prevent buckling of the compression flange brace beam every L_b distance



(3-19)

To be in Zone 1 $\quad L_b \le L_p = \dfrac{300 r_y}{\sqrt{F_{yf}}}$

(16.1-48)

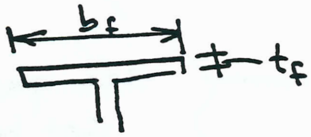
(b) If ~~the~~ the beam can reach M_p without local buckling it is SAID to be compact. If not, it is Non compact or slender

$$\underset{\qquad\qquad\qquad\lambda_p}{\text{compact}\ |}\ \underset{\qquad\qquad\qquad\lambda_r}{\text{Non-compact}\ |}\ \text{slender} \longrightarrow \lambda$$ (16.1-16)

$\lambda = \dfrac{\text{width}}{\text{Thickness}}$ of comp elements

for W

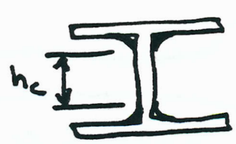
for flange



tabulated

$\dfrac{b_f}{2t_f} \le \dfrac{65}{\sqrt{F_y}}$ to be compact

for web



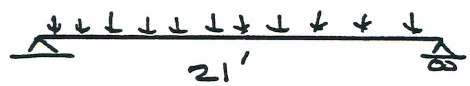
$\dfrac{h_c}{t_w} \le \dfrac{640}{\sqrt{F_y}}$ to be compact

tabulated

1 section	Non-compact A36	W6x15	See (3-28) Manual indicates what sections Non-compact
6 " " "	$F_y=50$	W14x99 W14x90 W12x65 W10x12 W6x15 W8x10	

Design of Bms – Zone 1

$D = 1\ ^k/_{ft}$, $L = 3\ ^k/_{ft}$ $\qquad L_b = 3'$ given



Assume bm wt = 55 lbs/ft

$$w_u = 1.2\left(1.055\,\frac{k}{ft}\right) + 1.6\left(3\ ^k/_{ft}\right) = 6.07\ ^k/_{ft}$$

$$M_u = \frac{wL^2}{8} = 6.07\,\frac{k}{ft}\,\frac{(21\,ft)^2}{8} = 334.6\ k\text{-}ft$$

$$\xrightarrow{50\ ksi}\ \frac{334.6}{.9\,(50)} = 89.2\ in^3$$

$$Z_{req'd} = \frac{M_u}{\phi F_y} = \frac{334.6\ \cancel{k\text{-}ft}\ in^2}{(.9)(36\ \cancel{k})}\left(\frac{12\,in}{\cancel{ft}}\right) = 123.9\ in^3$$

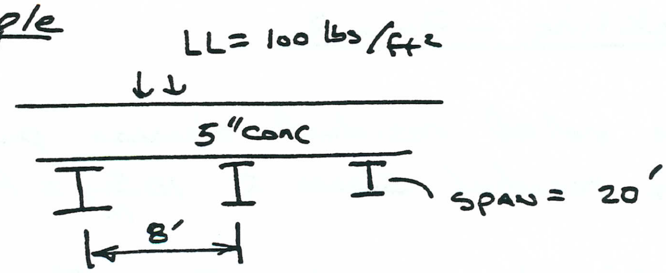
use W24x55 — See page (3-24)
Get bm with Z > 123.9
Bold Sections most economical

Alternately
Get beam with $\phi M_p > 334.6$ k-ft — Also tabulated (3-24)

36 ksi → Use W24x55 — Note wt assumption conservative

50 ksi → USE W21x44 — NOTE WT ASSUMPTION CONSERVATIVE ✓

Example



unbraced length = 0'

Get Loads

$$DL\ conc = 5\cancel{in}\left(150\frac{lbs}{ft^3}\right)(8ft)\left(\frac{ft}{12\cancel{in}}\right) = 500\frac{lbs}{ft}$$

est bm $\quad 15\frac{lbs}{ft}$

Total DL $\quad 515\frac{lbs}{ft}$

$$w_u = 1.2\left(515\frac{lbs}{ft}\right) + 1.6\left(100\frac{lbs}{ft^2}\right)(8ft) = 1898\frac{lbs}{ft}$$

$$M_u = 1.898\frac{k}{ft}\frac{(20ft)^2}{8} = 94.9\ k\text{-}ft$$

try W12×26 ← 36ksi $\qquad$ 50ksi → W14x22, W12x22, W10x22 } ALL OK ✓

Note wt assumption made is unconservative we will have to check & see if this bm OK

RE CHECK DL (36ksi) $\qquad$ (INSERT 22 FOR 50ksi ALTERNATE)

$$w_u = 1.2(500+26)\frac{lbs}{ft} + 1.6\left(100\frac{lbs}{ft^2}\right)(8ft) = 1911\frac{lbs}{ft}$$

$$M_u = \left(1.911\frac{k}{ft}\right)\frac{(20ft)^2}{8} = 95.6\ k\text{-}ft < 100\ k\text{-}ft$$

wt assumption OK

use <u>W12x26</u>

Inelastic Buckling – Zone 2

When bm rolled residual stresses put in bm – MAX residual stress $\approx 10 \frac{k}{in^2} = F_r$

When bending stress reaches $F_{yw} - F_r$ we

yield stress web = F_y

get 1st yielding in W. The moment corresponding to this stress state is M_r. The usealbe moment is $\phi_b M_r$.

EXAMPLE

W14x22

$M_p = Z_x F_y = (33.2)(50ksi) = 1660$ k-in/12 $= 138.3$ k-ft

$M_r = S_x(.7F_y) = (29.0)(0.7)(50ksi) = 1015$ k-in/12 $= 84.5$ k-ft

$\phi M_p = 0.9(138.3) = 124.5$ k-ft

$\phi M_r = 0.9(84.5) = 76.1$ k-ft

SAME

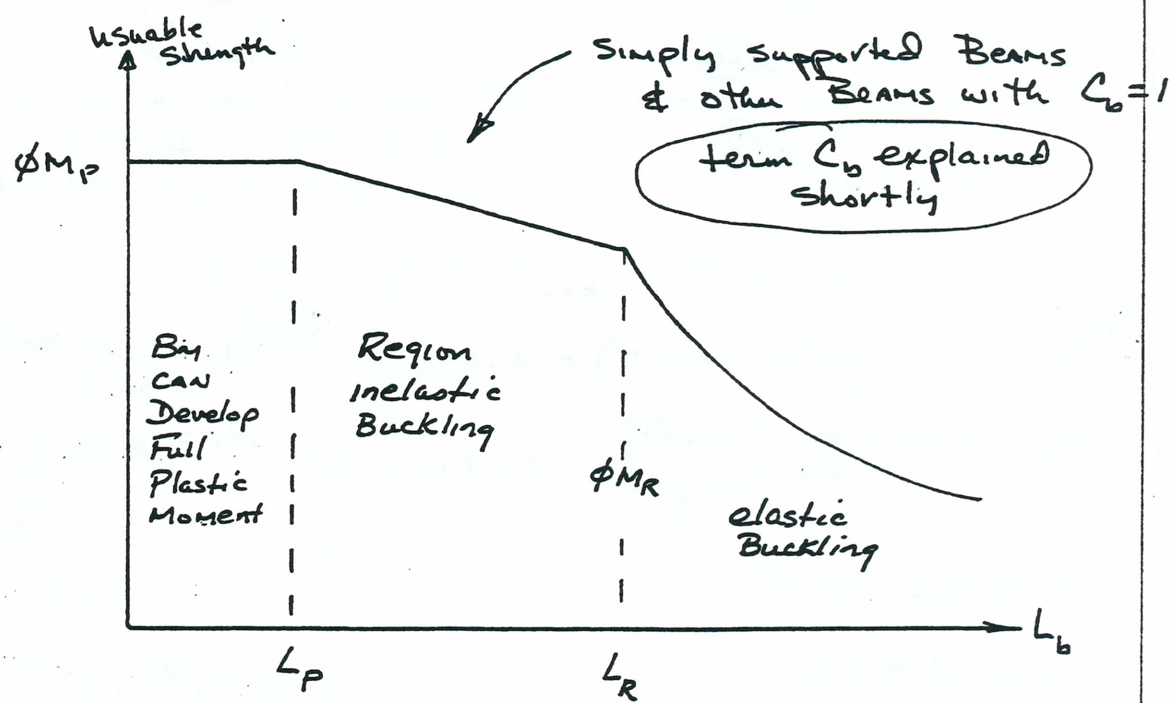
VALUES TABULATED

(3-28) $\phi_b M_p = 125$ k-ft

$\phi_b M_r = 76.1$ k-ft

Zone 2,3 –

If we increase distance between pts of lateral bracing of compression flange, the moment required to fail a beam decreases (because Beam now failing by inelastic buckling. If we further increase distance between bracing beam fails by elastic buckling.

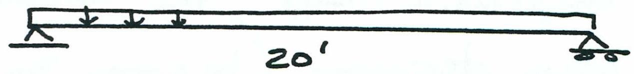


See (3-91)
for beam curves

Example

$w_{DL} = 1\ ^{k}/_{ft}$
$w_{LL} = 2\ ^{k}/_{ft}$

unbraced against lateral Movement



$\therefore L_b = 20'$

Since I don't have any idea what the beam DL is, ignore it in the moment. I will put it in after I get a trial section

$w_u = 1.2\left(1\frac{k}{ft}\right) + 1.6\left(2^{k}/_{ft}\right) = 4.4\ \frac{k}{ft}$

$M_u = \dfrac{4.4\frac{k}{ft}\,(20\text{ ft})^2}{8} = 220\ \text{k-ft}$

try (3-119) get solid line above pt on p 3-70 – Dotted lines Not most economical

W12 x 53 (50)

we now know $w_{DL}\big|_{W12\times 60} = 60\ \frac{lbs}{ft} = .06\ \frac{k}{ft}$

(50)

$w_u = 1.2(1.053) + 1.6(2)$
$= 4.46$

$M_u = \dfrac{4.46\frac{k}{FT}\,(20\text{ FT})^2}{8}$

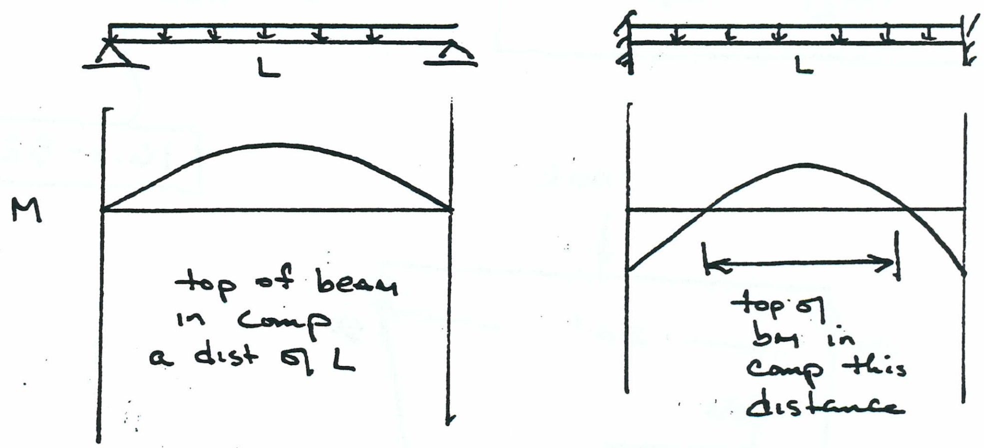
$= 223\ \text{k-FT}$

$\phi_b M_N @ L_b = 20 = 238\ \text{k-FT}$

$238 > 223$ OK ✓

The Parameter C_b (16.1-46)

How a beam is loaded affects the torsional buckling of a beam.



Think of the top flange of a beam as a column. The longer column wants to buckle at a lower load (stress) than a shorter column. When the flanges buckle they cause the beam to twist

To modify the buckling strength of a beam the code uses a factor called C_b

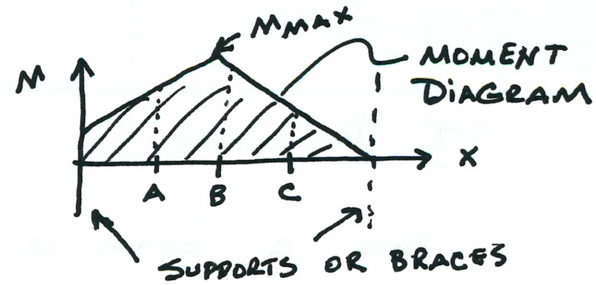
OLD CODE (1ST ed)

$\frac{M_1}{M_2} = (+)$ FOR REVERSE CURV.

$(-)$ FOR SINGLE

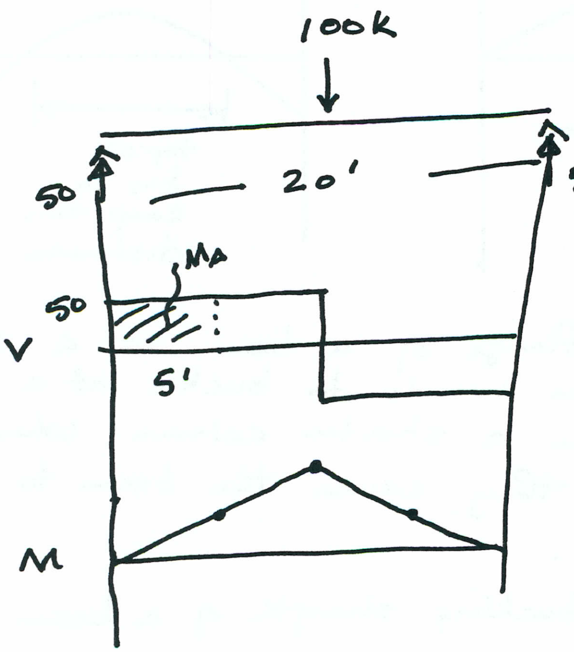
$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + .3\left(\frac{M_1}{M_2}\right)^2 < 2.3$$

NEWER CODES



$$C_b = \frac{12.5\, M_{MAX}}{2.5 M_{MAX} + 3M_A + 4M_B + 3M_C} \le 2.3$$

(16.1-46)



$M_{MAX} = \frac{PL}{4} = \frac{(100)(20)}{4} = 500$

$M_A = 250$ K-FT

$M_B = M_{MAX} = 500$

$M_C = M_A = 250$

$$C_b = \frac{12.5(500)}{2.5(500) + 3(250) + 4(500) + 3(250)}$$

$= 1.3158$

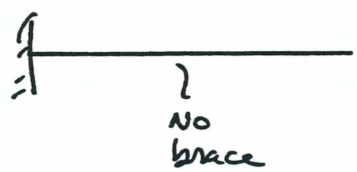
Table 3-1 (3-18)

Further discussion of C_b

$C_b = 1$ for unbraced cantilever beams

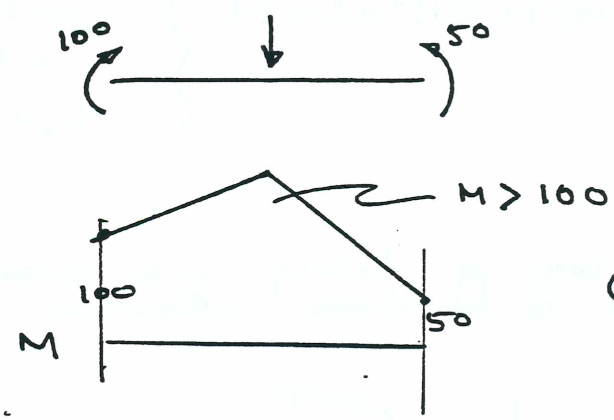
$= 1$ for beams with moment between ends greater than end moment

ex



$C_b = 1$

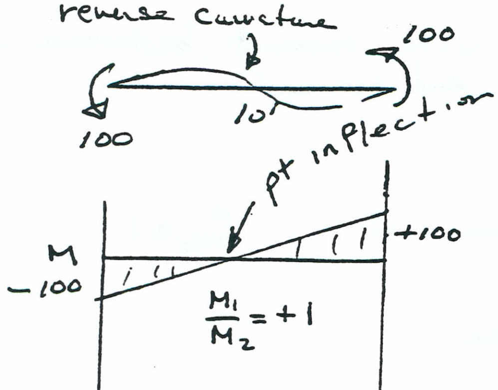
ex



$C_b = 1$

Further discussion of C_b & L_b

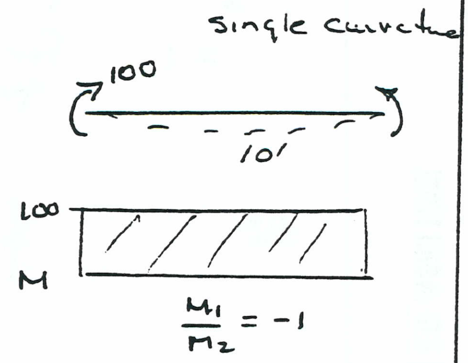
C_b takes into account the Moment Gradient in the beam



$C_b = 1.75 + 1.05(1) + .3(1)^2 = 3.1 > 2.3$

$C_b = 2.3$

$L_b = 10'$

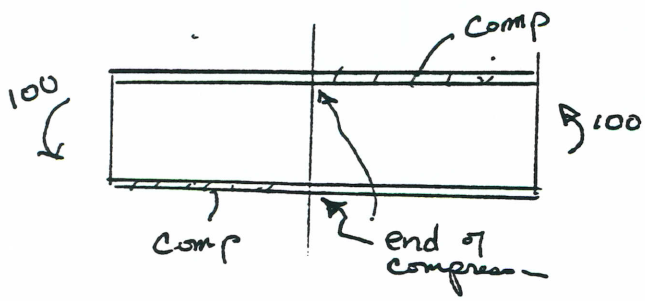


$C_b = 1.75 + 1.05(-1) + .3(-1)^2$

$= 1.0$

$L_b = 10'$

Another way of taking into account moment gradient is to take inflection pt as braced point



could use $C_b = 1$

$L_b = 5'$

However we would not want to take

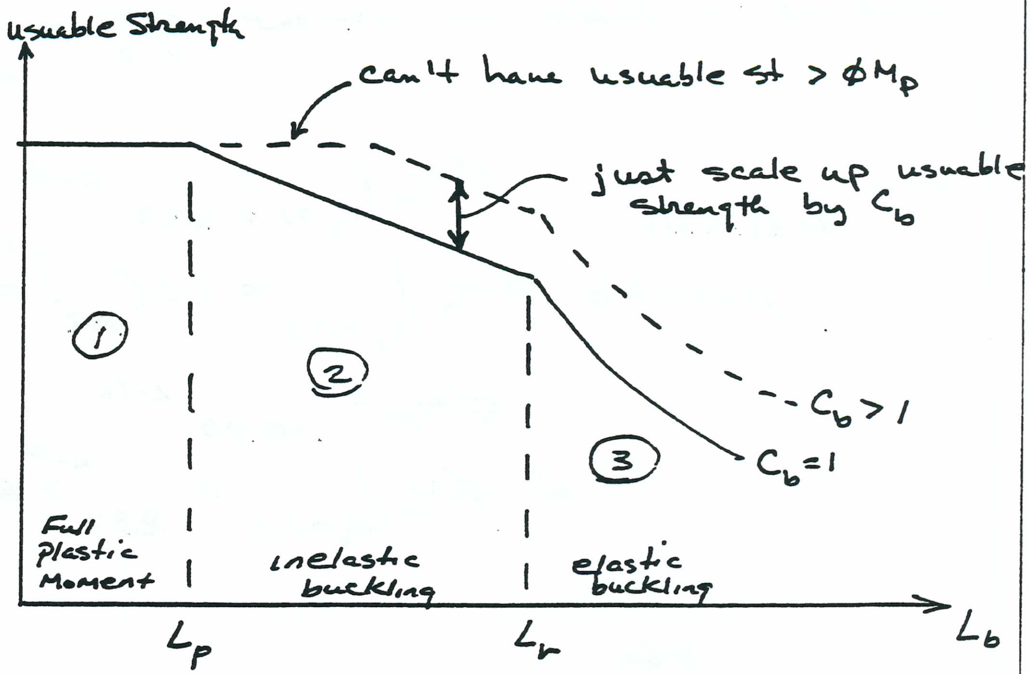
~~$C_b = 2.3$~~ → NO

~~$L_b = $~~

this takes moment gradient into account twice

Zone 2 & 3

Bm Strengths in this region controlled by buckling. Buckling Strength increased by C_b. However you cannot have usable strength greater than ϕM_p



Example

$M_u = 800\ k\text{-}ft$

$L_b = 30\ ft$

$C_b = 1.2$

Look up in Tables for moment $= \frac{800}{1.2} = 667\ k\text{-}ft$

$L_b = 30\ ft$

try W 21 x 111 $\quad \phi M_n \big|_{C_b=1} = 692\ k\text{-}ft$

possible $\phi M_n \big|_{C_b=1.2} = 1.2(692) = 830\ k\text{-}ft$

$\phi M_p = 1050\ k\text{-}ft$

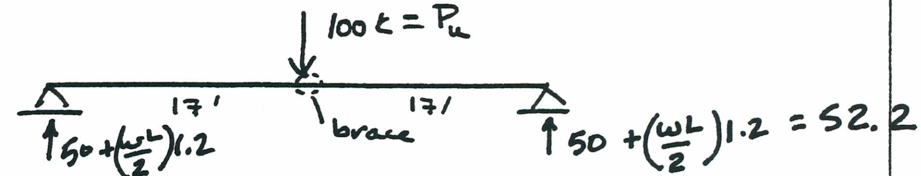
$\therefore \phi M_n \big|_{C_b=1.2} = 830\ k\text{-}ft > 800\ k\text{-}ft$ OK

use

W 21 x 111

NOTE: BOLD CURVES DON'T NECESSARILY APPLY TO DESIGNS USING $C_B > 1$.

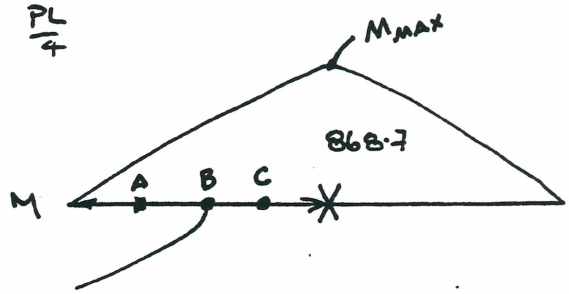
Example



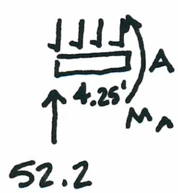
Assume bm wt = 108 lbs/ft

$$M_u = \frac{100^k(34 ft)}{4} + \overbrace{1.2\left(.108 \frac{k}{ft}\right)}^{\text{factored load}}\frac{(34 ft)^2}{8} = 868.7 \text{ k-ft}$$

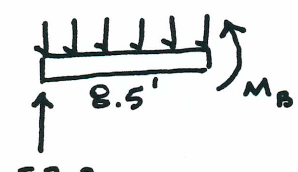
$\uparrow$
$\frac{PL}{4}$



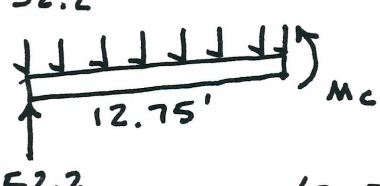
examine this part of beam between bracing points



$M_A = 220.7$



$M_B = 439.0$



$M_c = 655$

$$C_b = \frac{12.5(868.7)}{2.5(868.7) + 3(221) + 4(439) + 3(655)} = 1.66$$

NON-COMPACT Sections

Bm Tables (3-20s)

&

Bm Curves (3-90s)

Have usable strengths adjusted if beam is NON-COMPACT. Therefore, we don't have to worry about compactness.

Compact Section

LRFD

A section is compact if no local buckling at M_p

to be compact

1) webs continuously connected to flange
2) width-thickness ratios of compression elements don't exceed limiting values

Members classified as follows

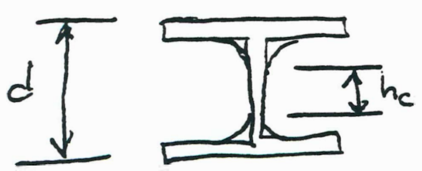
Compact | Non Compact | Slender $\rightarrow \lambda$

λ_r

$\frac{b_f}{2t_f}$

$\frac{b}{t} \le \frac{65}{\sqrt{F_y}}$ ← Same as allowable stress

$\frac{h_c}{t_w} \le \frac{640}{\sqrt{F_y}}$ ← for allowable stress $\frac{d}{t_w} \le \frac{640}{\sqrt{F_y}}$



$\frac{h_c}{t_w}$ is Tabulated

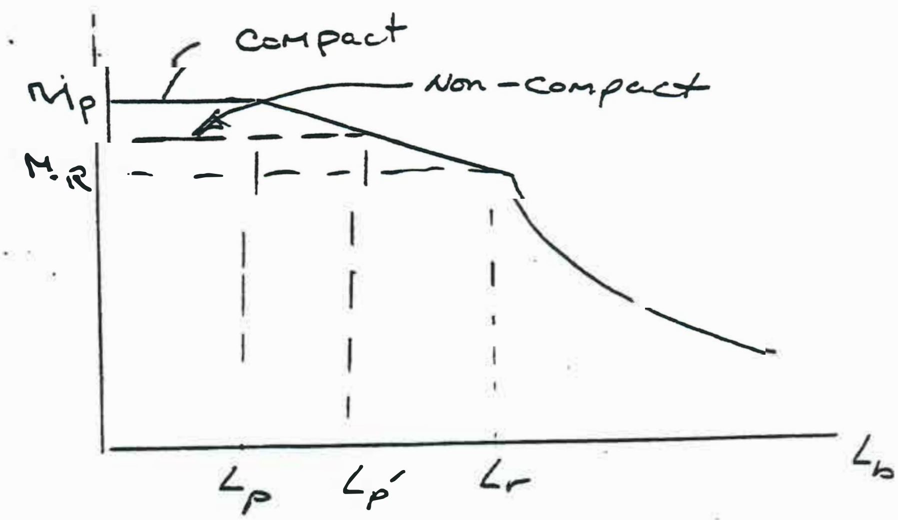
LRFD

Non-compact Sections $\lambda_p \le \lambda \le \lambda_r$

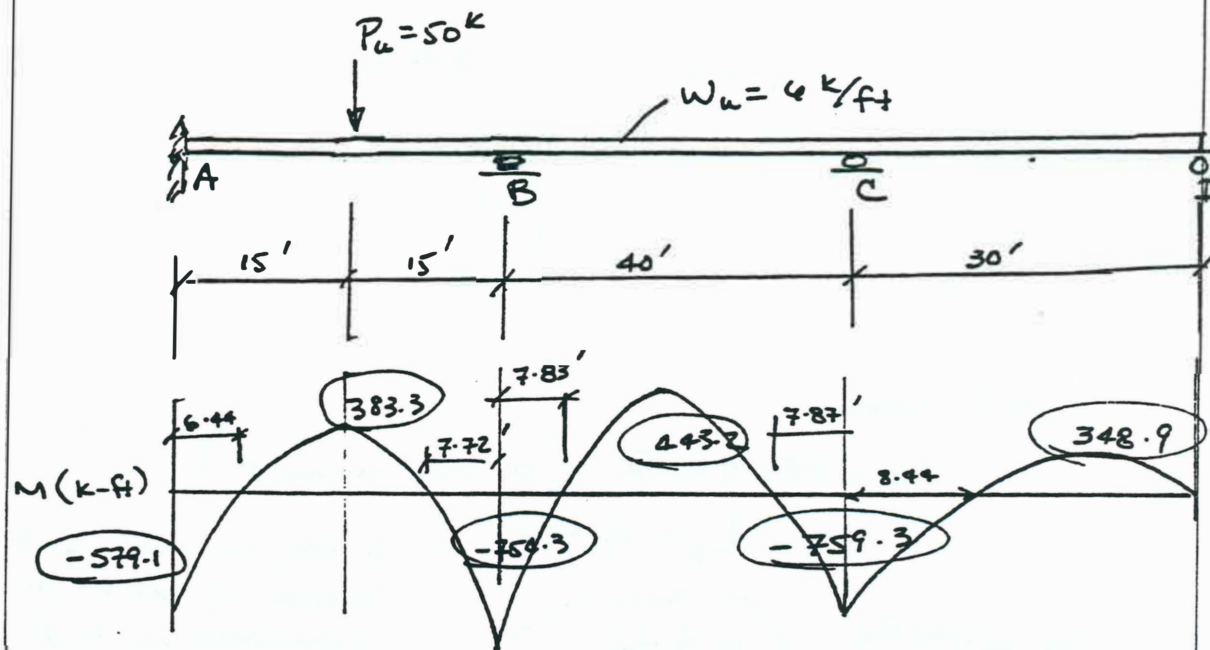
M_n = smaller of

$$M_n = C_b\left[M_p - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \le M_p$$

$$M_n = M_p - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad (16.1\text{-}49)$$



Design of Continuous Beams



Assume Top Flange Braced – Bottom is not

Before Continuous Beam Collapses, a number of plastic hinges form which redistribute moments in beam. One can account for this redistribution by using for design

$$M_{\substack{neg \\ design}} = .9\, M_{\substack{neg \\ analysis}}$$

if $$M_{\substack{pos \\ design}} = M_{\substack{pos \\ Analysis}} + \frac{1}{10}\left(\frac{M_{\substack{neg \\ end\,1}} + M_{\substack{neg \\ end\,2}}}{2}\right)$$

we don't have to do this procedure but we are allowed to.

Example – EXAMINE SPAN BC

$M_{\substack{Neg \\ design}} = .9(759.3) = 683.4$ k-ft

$M_{\substack{pos \\ design}} = 443.2 + \frac{1}{10}\left(\frac{754.3 + 759.3}{2}\right) = 518.9$ k-ft

select beam

$M_u = 683.9$ k-ft (a neg moment)

$L_b = 7.87'$ — A neg moment puts bottom of beam in compression but only top flange braced

use W24x94

as $L_b < L_p$, $\phi M_n = \phi M_p$ ✓?

(3-90s)

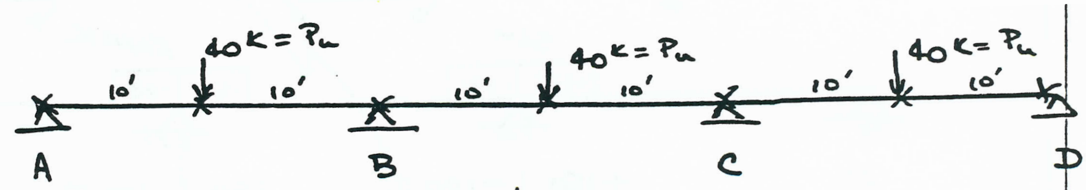
If we didn't use .9 rule

$\left.\begin{array}{l} M_u = 759.3 \\ L_b = 7.87' \end{array}\right\} \Rightarrow$ W30x99

Here we are taking moment gradient into account by taking inflection pt as brace pt. ∴ use $C_b = 1$ & $L_b = 7.87'$

Design Continuous Bm

ignore DL beam



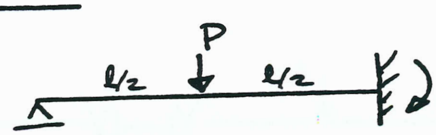
X = brace $\qquad L_b = 10'$

Get relative stiffnesses

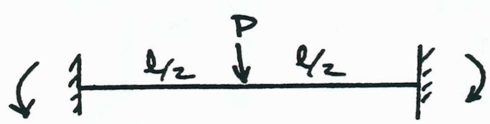
Mem	k	rel $k = \frac{k}{\Sigma k}$
BA	$\frac{3EI}{L} = \frac{3EI}{20}$	$\frac{3EI}{20} \cdot \frac{20}{7EI} = \frac{3}{7} = .429$
BC	$\frac{4EI}{L} = \frac{4EI}{20}$	$\frac{4EI}{20} \cdot \frac{20}{7EI} = \frac{4}{7} = .571$
	$\Sigma k = \frac{7EI}{20}$	1.000 ✓

GET FEM

Table 3.23 (3-212)



$$\frac{3Pl}{16} = \frac{3(40^k)(20ft)}{16} = 150\ k\text{-}ft$$

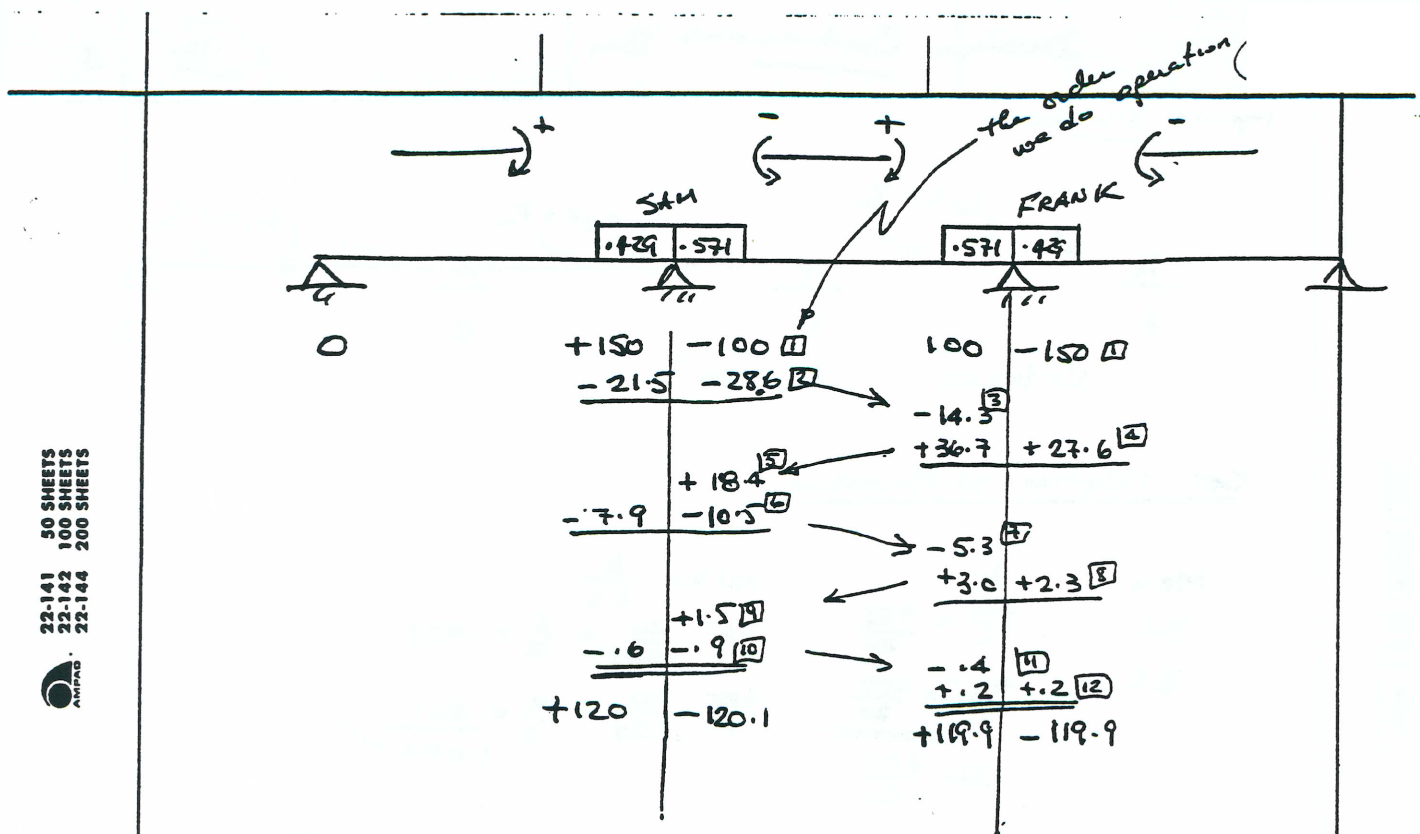


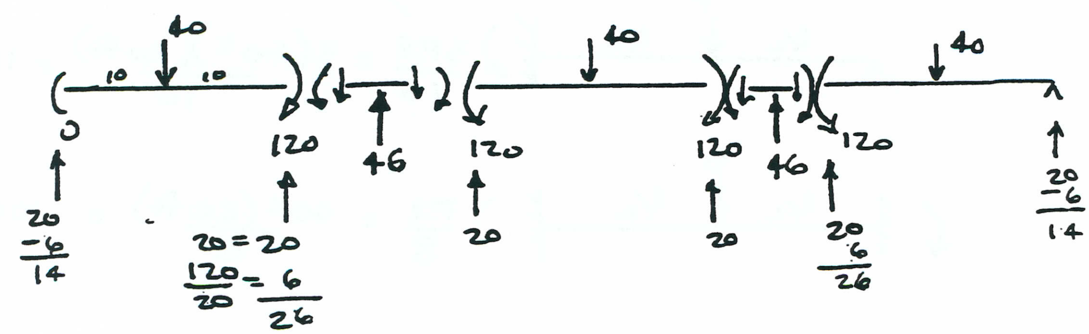
$$\frac{Pl}{8} = \frac{40^k(20ft)}{8} = 100\ k\text{-}ft$$

Sign Convention $\qquad$ cw Moments on member $\oplus$

42-381 50 SHEETS 5 SQUARE
42-382 100 SHEETS 5 SQUARE
42-389 200 SHEETS 5 SQUARE
NATIONAL

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS



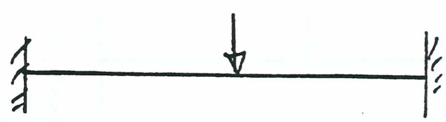


Alternate procedure

Get relative stiffnesses

Mem	k	rel $k = \frac{k}{\Sigma k}$
BA	$\frac{4EI}{L} = \frac{4EI}{20}$	$\frac{4EI}{20} \cdot \frac{20}{8EI} = .5$
BC	$\frac{4EI}{L} = \frac{4EI}{20}$	$\frac{4EI}{20} \cdot \frac{20}{8EI} = .5$
	$\Sigma k = \frac{8EI}{20}$	1.0 ✓

Use FEM



$$\frac{P\ell}{8} = \frac{40^k(20\,ft)}{8} = 100\,k\text{-}ft$$

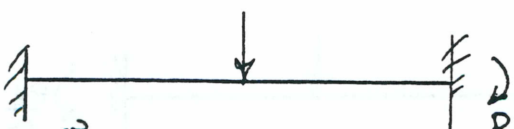
JANE	SAM	SAM	FRANK	FRANK	HARRY
	.5	.5	.5	.5	
-100 [1]	+100 [1]	-100 [1]	100 [1]	-100 [1]	100 [1]
+100 [4]	+50 [5]	-25 [6]	-12.5 [7]	-50 [3]	-100 [3]
-12.5 [7]	-25 [6]	15.6 [9]	+31.3 [8]	+31.3 [8]	+15.6 [9]
+12.5 [10]	+6.3 [4]	-11.0 [12]	-5.5 [13]	-7.8 [15]	-15.6 [14]
-5.5 [13]	-11.0 [12]	+3.3 [17]	+6.7 [16]	+6.7 [16]	+3.3 [17]
+5.5 [18]	+2.8 [19]	-3.1 [20]	-1.5 [21]	-1.7	-3.3 [22]
-1.5 [21]	-3.1 [20]	+.8 [23]	+1.6 [22]	+1.6 [22]	+.8 [23]
+1.5 [22]	+.8 [25]	-.8 [26]	-.4 [27]	-.4 [29]	-.8 [28]
-.4 [27]	-.8 [26]	+.2 [31]	+.4	+.4 [30]	+.2 [31]
+.4 [32]	+.2 [33]	-.2 [34]			-.2 [35]
	-.2 [34]				
0	+120.0	-120.2	120.1	-119.9	0

Alternate procedure 2

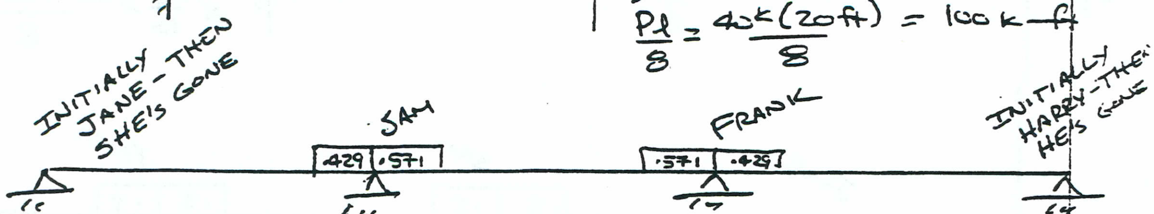
Get relative k

Mem	k	rel k
BA	$\frac{3EI}{L}$	$3/7 = .429$
BC	$\frac{4EI}{L}$	$4/7 = .571$
	$\Sigma k = \frac{7EI}{L}$	

Get FEM

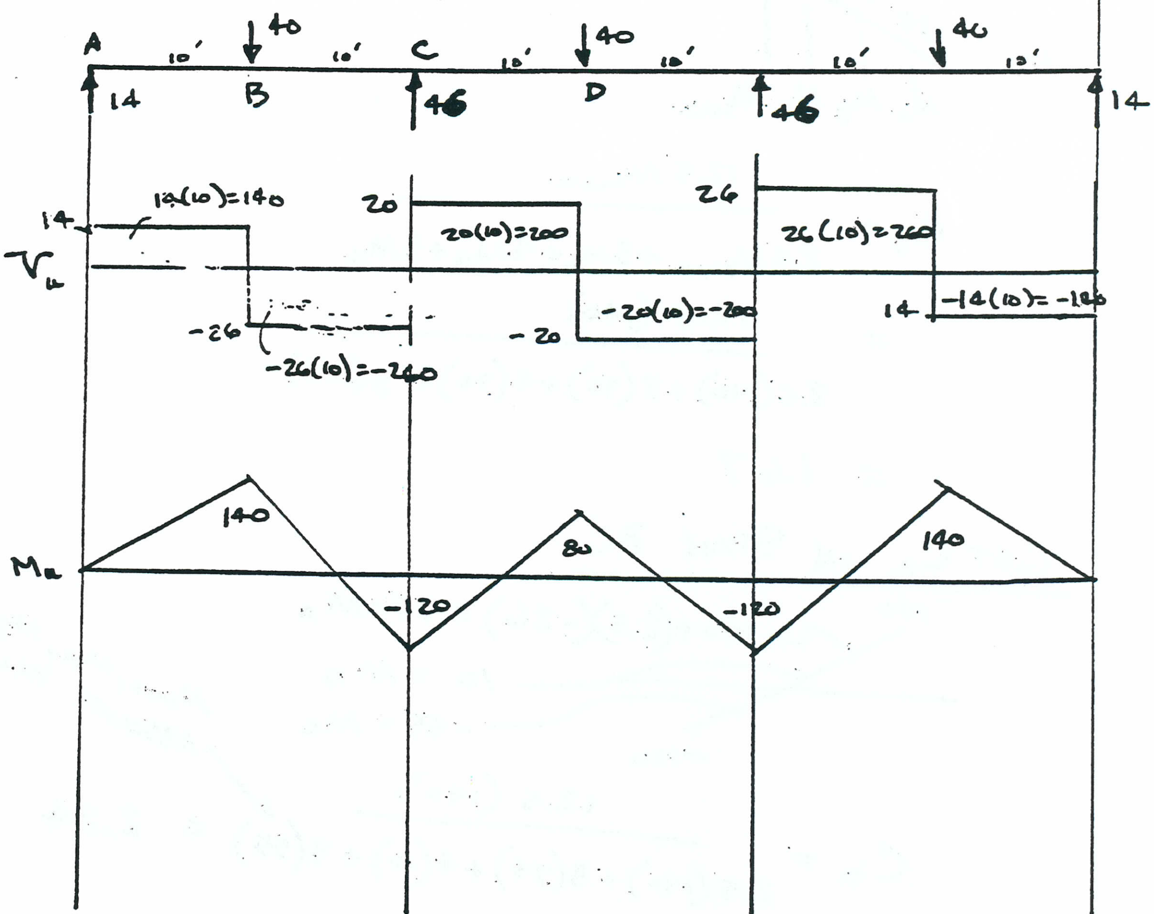


$$\frac{Pl}{8} = \frac{40^k(20ft)}{8} = 100\,k\text{-}ft$$



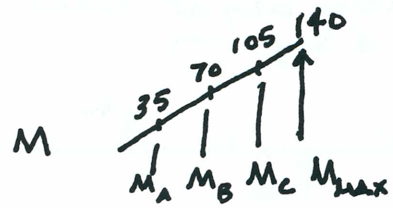
A	B (BA)	B (BC)	C (CB)	C (CD)	D
−100 [1]	100 [1]	−100 [1]	100 [1]	−100 [1]	100 [1]
+100 [2]	+50 [3]			−50 [5]	−100 [4]
	−21.5 [6]	−28.6 [7]	−14.3 [8]		
			+36.7 [9]	+27.6 [9]	
		18.4 [10]			
	−7.9	−10.5 [11]	−5.3 [12]		
			+3.0 [13]	+2.3 [13]	
		+1.5 [14]			
	−.6	−.9 [15]	−.5 [16]		
			+.3 [17]	+.2	
		+.2 [18]			
	−.1	−.1 [19]			
	119.9	−120.2	119.9	−119.9	

Bracing only at Loads & supports



Here we don't WANT TO reduce M_{Neg} & increase M_{pos}

GET C_B IN SPAN AB (FROM SYMMETRY) BOTH ENDS

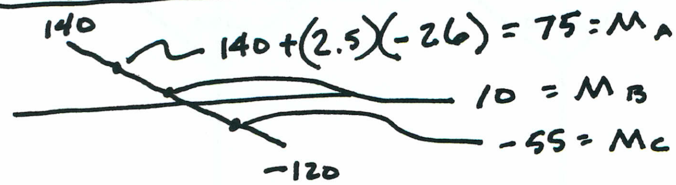


$$C_b = \frac{12.5\,M_{MAX}}{2.5M_{MAX} + 3M_A + 4M_B + 3M_C}$$

$$= \frac{12.5\,(140)}{2.5(140) + 3(35) + 4(70) + 3(105)}$$

$$= 1.67$$

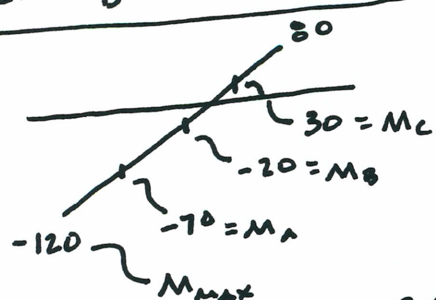
GET C_B IN SPAN BC



M_{MAX}, M_A, M_B & M_C ABSOLUTE VALUES

$$C_b = \frac{12.5\,(140)}{2.5(140) + 3(75) + 4(10) + 3(55)} = 2.24$$

GET C_B IN SPAN CD



$$C_b = \frac{12.5\,(120)}{2.5(120) + 3(70) + 4(20) + 3(30)} = 2.2$$

Span	C_B	$(M_u)_{max}$	$\frac{(M_u)_{max}}{C_b}$
A B	1.67	140	$\frac{140}{1.67} = 83.8$ ← controls
B C	2.24	140	$\frac{140}{2.24} = 62.5$
C D	2.2	120	$\frac{120}{2.2} = 54.5$

Select bm with $L_b = 10'$

$$\phi M_n > 83.8 \text{ K-FT}$$

try W 10×26, W 12×26, W 14×26 — (3-69)

W12×26 → $\phi M_n \Big|_{\substack{C_b = 1.67 \\ L_b = 10'}} = 117.8 \text{ K-FT} > 83.8 \text{ K-FT}$ ok ✓

CHECK

IS $\phi M_p > 140$?

$\phi M_p = 139.5$ TOO CLOSE

CHECK W14×26

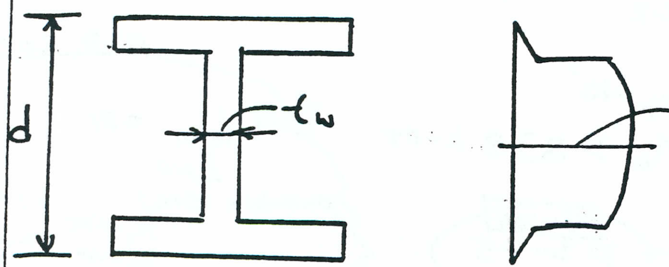
$$\phi M_N \Big|_{\substack{C_b = 1.67 \\ L_b = 10'}} = 107.5 > 83.8 \checkmark$$

$$\phi M_p = 157 > 140 \checkmark$$

USE W14×26

Beam Shear

Examine Shear Stress in W



$$\tau_{max} \approx \frac{V}{d t_w}$$

$$\tau = \frac{VQ}{It}$$

$$V_n = \text{strength in shear} = \overbrace{.6F_y}^{\text{yield stress in shear}}\, d t_w$$

$$\phi_v V_n = \text{usable strength} = \underset{.9}{\phi_v}(.6F_y)\, d t_w$$

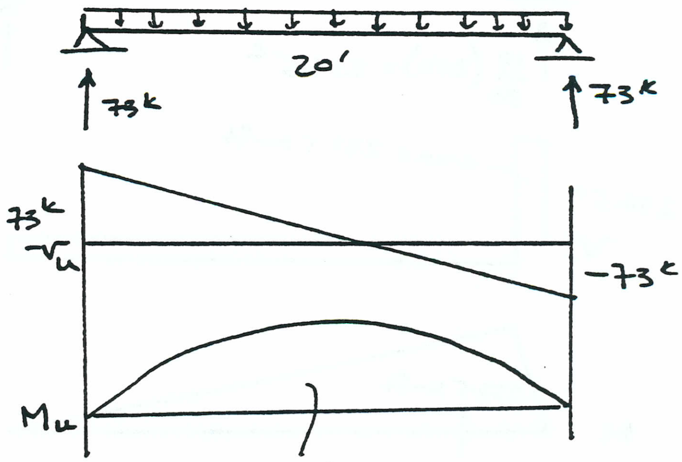
(16.1-70)

Shear strength seldom controls except when heavy loads & short spans or heavy load near support (giving big shear force but small moment.

NATIONAL 42-381 50 SHEETS 5 SQUARE 42-382 100 SHEETS 5 SQUARE 42-389 200 SHEETS 5 SQUARE

Example – typical case – bending Controls

$w_u = 7.3^k/ft$ (includes bm wt)



Assume Comp flange fully braced

$$M_u\big|_{max} = \frac{wL^2}{8} = 7.3\frac{k}{ft}\frac{(20ft)^2}{8} = 365\ k\text{-}ft$$

try W21x62 ($d = 20.99in$)($t_w = .40$)

$$\phi_v V_n = (.9)(.6)\left(36\frac{k}{in^2}\right)(20.99in)(.40in) = 163.2^k > 73^k$$

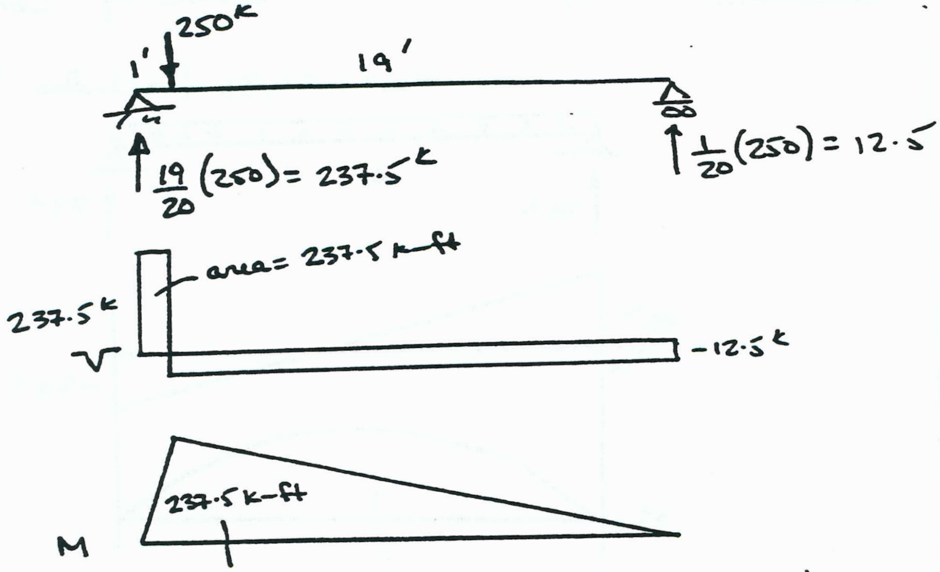
shear OK

use W21x62

Abnormal Case – Shear Controls

Neglect DL bm for now

$L_b = 0$



Note bending moment not very big

W21x44 to carry moment

$$\phi_v V_n \Big|_{W21\times44} = .9(.6)(36\tfrac{k}{in^2})(20.66'')(.35'') = 140.6^k < 237.5^k$$

NO good

We will have to get a beam with a bigger web area – How much area?

$$237.5^k = .9(.6)(36\tfrac{k}{in^2})\,A_w$$

$$A_w = 12.217\ in^2$$

if we stay with $d \cong 21\ in$, what t_w?

$$(21in)\,t_w = 12.217\ in^2$$

$$t_w = .58\ in$$

try W 21x93

$$\phi_v V_n = .9(.6)(36\tfrac{k}{in^2})(21.62in)(.58in) = 243.8^k > 237.5^k$$

OK – now check with bm D.L. etc.

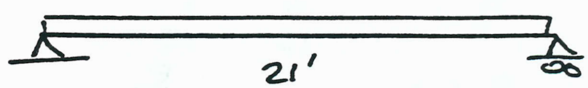
Deflections

LRFD does not specify exact maximum permissibe deflections

STANDARD American Practice : $\delta\Big|_{\substack{\text{live}\\\text{load}\\\text{service}}} < \frac{\text{SPAN}}{360}$

Example

$D = 1\ k/ft$ $\quad L = 3\ k/ft$ $\quad L_b = 0$



21'

est bm wt = 55 lb/ft

$$w_u = 1.2\left(1.055\,\frac{k}{ft}\right) + 1.6\left(3\,\frac{k}{ft}\right) = 6.066\,\frac{k}{ft}$$

$$M_u = 6.066\,\frac{k}{ft}\,\frac{(21\ ft)^2}{8} = 334.4\ k\text{-}ft$$

try W 24x55 ($I_x = 1350\ in^4$)

check deflection (3-208)

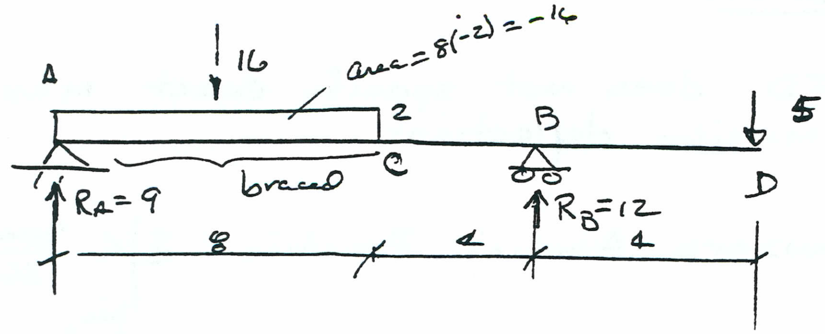
$$\delta_{MAX} = \frac{5wL^4}{384EI}$$

$$= \frac{5\left(3\frac{\cancel{k}}{\cancel{ft}}\right)(21\ \cancel{ft})^4\ in^2}{384(29\times10^6)\ \cancel{lbs}\ (1350\ \cancel{in^4})}\ \frac{1000\ \cancel{lbs}}{\cancel{k}}\ \frac{(12)^3\ in^3}{\cancel{ft^3}}$$

$$= \frac{5(3)(21)^4(12)^3(1000)}{384(29\times10^6)(1350)} = .335''$$

$$\frac{L}{360} = \frac{21\ ft\left(\frac{12\ in}{ft}\right)}{360} = .70'' > .335''$$ ok no defl prob

Review Shear & Bending Moment Diagrams



$\Sigma M_A = 0$

$64 = 16(4)$

$80 = 5(16)$

$144 = R_B(12)$

$R_B = 12$

$\Sigma V = 0$

↓	↑
16	12
5	R_A

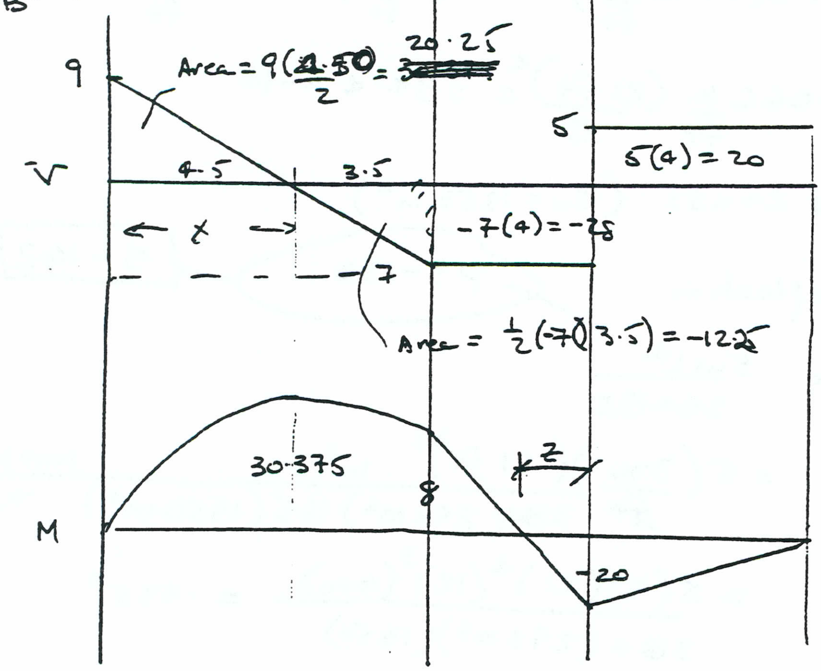
$R_A = 9$

$\Sigma M_B = 0$ check

$108 = 9(12)$ $16(8) = 128$ ✓

$20 = 5(4)$

128 ✓



$\frac{16}{8} = \frac{9}{x}$

$x = \frac{8(9)}{16}$

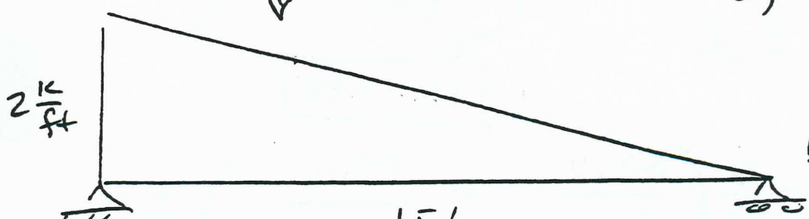
$x = 4.50$

$2\frac{k}{ft}(15\,ft)(\tfrac{L}{2}) = 15^k$

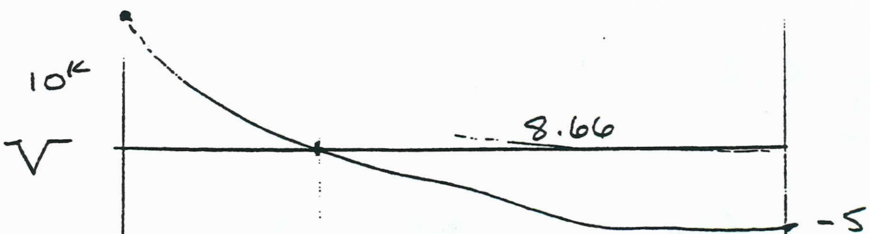
a) Continuously braced

Ignore DL

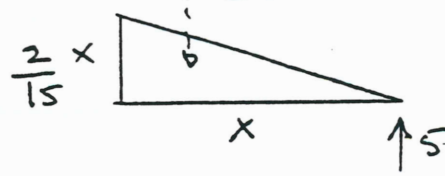
b) unbraced

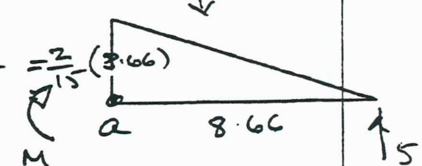


$\uparrow \tfrac{2}{3}(15) = 10^k$ $\qquad$ $\uparrow \tfrac{1}{3}(15^k) = 5^k$



$\frac{1}{2}(\frac{2}{15}x)(x) = 5$ $\qquad$ $\frac{1}{2}(8.66)(1.154) = 5$





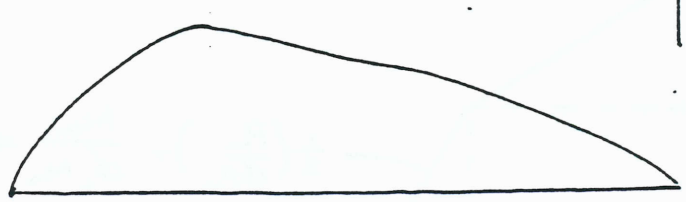
$\frac{x^2}{15} = 5$

$x^2 = 75$

$x = 8.66$

$\Sigma M_c = 0$

$M \qquad 5\left(\frac{8.66}{3}\right) \qquad 5(8.66)$



$M + 14.43 = 43.3$

$M = 28.87$

use M12×11.8 braced

W8×18 unbraced

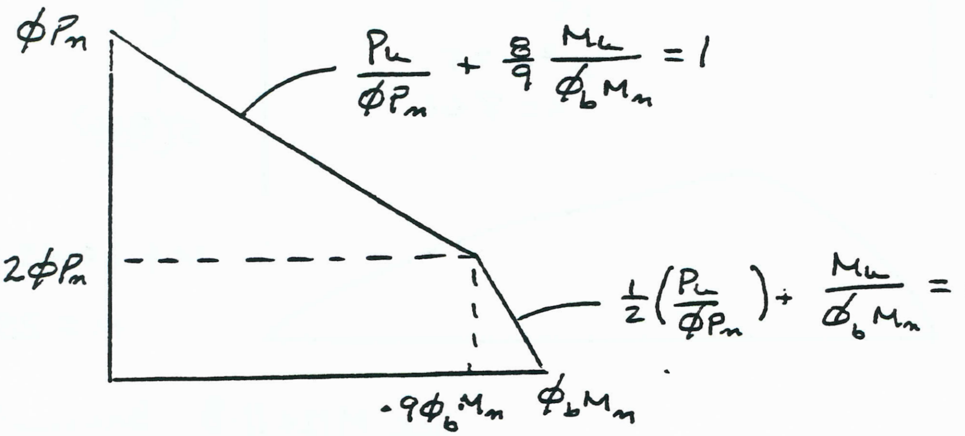
(16.1-77

If $\dfrac{P_u}{\phi_t P_n} \geq .2$

$$\frac{P_u}{\phi_t P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1$$

If $\dfrac{P_u}{\phi_t P_n} < .2$

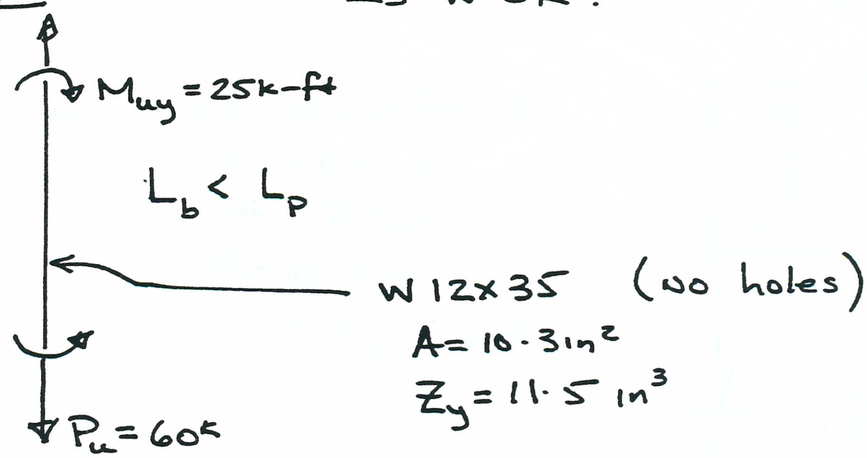
$$\frac{P_u}{2\phi_t P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1$$



NATIONAL 42-381 50 SHEETS 5 SQUARE 42-382 100 SHEETS 5 SQUARE 42-389 200 SHEETS 5 SQUARE

Example

Is W ok?



$$\phi_t P_n = \phi_t F_y A_g = .9\left(36\frac{k}{in^2}\right)(10.3\,in^2) = 333.7^k$$

$$\frac{P_u}{\phi_t P_n} = \frac{60^k}{333.7^k} = .18 < .2$$

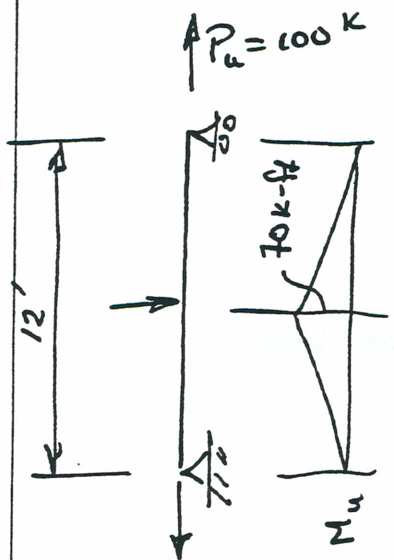
$\therefore$ use $\dfrac{P_u}{2\phi P_n} + \left(\dfrac{M_{ux}}{\phi_b M_{nx}} + \dfrac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1$

p1-33

$$\phi_b M_{ny} = \phi_b F_y Z_y = .9\left(36\frac{k}{in^2}\right)(11.5\,in^3)\left(\frac{ft}{12\,in}\right) = 31.05\text{k-ft}$$

$$\frac{P_u}{2\phi_t P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \overset{?}{\leq} 1$$

$$\frac{60^k}{2(333.7)} + \left(0 + \frac{25\,k\text{-}\cancel{ft}}{31.05\,k\text{-}\cancel{ft}}\right) = .895 < 1 \quad \text{OK}$$



Is W 10x30 OK? No holes

$A = 8.84\ in^2$

$L_p = 5.7$ ft

$L_r = 20.3$ ft

take $L_b = 12'$

$$\phi_t P_n = .9\left(36\frac{k}{in^2}\right)(8.84\ in^2) = 286.4^k$$

$$\frac{P_u}{\phi_t P_n} = \frac{100}{286.4} = .349 > .2$$

$$\therefore \frac{P_u}{\phi_t P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \cancel{\frac{M_{uy}}{\phi_b M_{ny}}}\right) \le 1$$

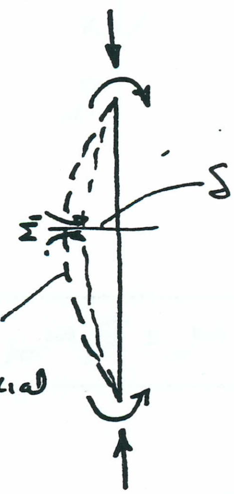
$\phi_b M_{nx} = 83$ k-ft — from curves p 3-75
$L_b = 12'$

$$\therefore \frac{P_u}{\phi_t P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}\right) = .349 + \frac{8}{9}\left(\frac{70}{83}\right) = 1.095$$ **No good**

COMPRESSION + BENDING

Apply moments to a beam
Beam bends

Now apply axial (compressive load)
Bm bends more



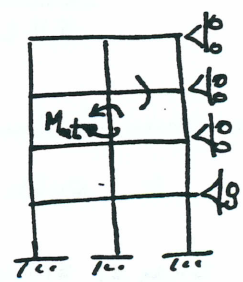
extra defl due to axial force

$$M_{nt}^* = M_{nt} + P_u \delta$$

$$\boxed{M_{nt}^* = B_1 M_{nt}}$$

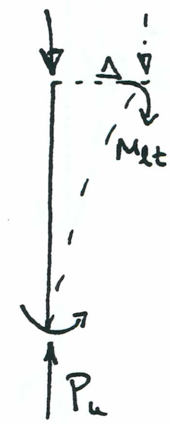
↑ Amplification factor

M_{nt} = moment with no joint translation



NATIONAL 42-381 50 SHEETS 5 SQUARE 42-382 100 SHEETS 5 SQUARE 42-389 200 SHEETS 5 SQUARE

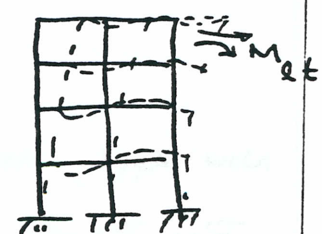
We can also get additional moments put in beam when ends of member has joint translation



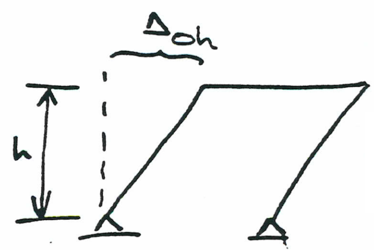
$$M^*_{lt} = M_{lt} + P_u \Delta$$

$$M^*_{lt} = B_2 M_{lt}$$

M_{lt} = Moments due to joint translation



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



$$B_1 = \frac{C_m}{1 - \frac{P_u}{P_e}} \geq 1$$

(16.1-249

$$B_2 = \frac{1}{1 - \frac{\Sigma P_u \Delta_{oh}}{\Sigma H L}} \geq 1 \text{ or } B_2 = \frac{1}{1 - \frac{\Sigma P_u}{\Sigma P_e}} \geq 1$$

ΣP_u = sum factored loads of all columns on floor in question

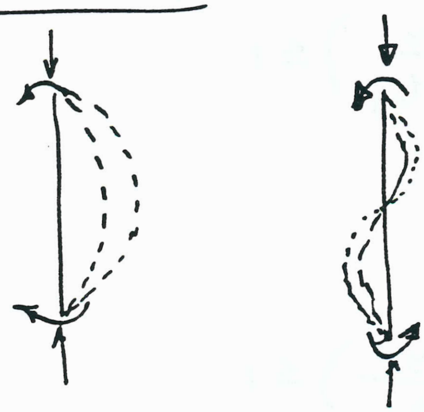
ΣP_e = " euler buckling load " " " " " "

ΣH = sum of story horiz. forces producing Δ_{oh}

$\Delta_{oh} = .0015$ to $.003$ for comfort of occupants

$\cong .004$ at ult. load

Examine C_m



C_m reflects that how moments applied to column affect $P\delta$ effects

(16.1-250)

$$C_m = .6 - .4\frac{M_1}{M_2}$$

M_1 = smaller Mom't

M_2 = larger

$\frac{M_1}{M_2} = +$ if

reverse curvature bending

$\frac{M_1}{M_2} = -$ if

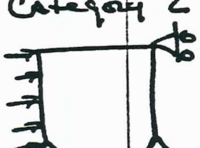
single curvature bending

Category 1

No joint translation & no load along column

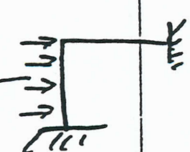
No load along member

Category 2

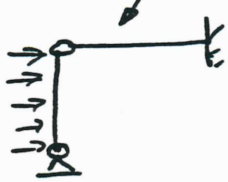


or

Members with restrained ends $C_m = .85$



Members with unrestrained ends $C_m = 1$



OR

Table 11.1

Interaction Equations Moment + Axial Compression Force

If $\dfrac{P_u}{\phi_c P_n} \geq .2$

$\phi_c = 0.9$
$\phi_b = .9$

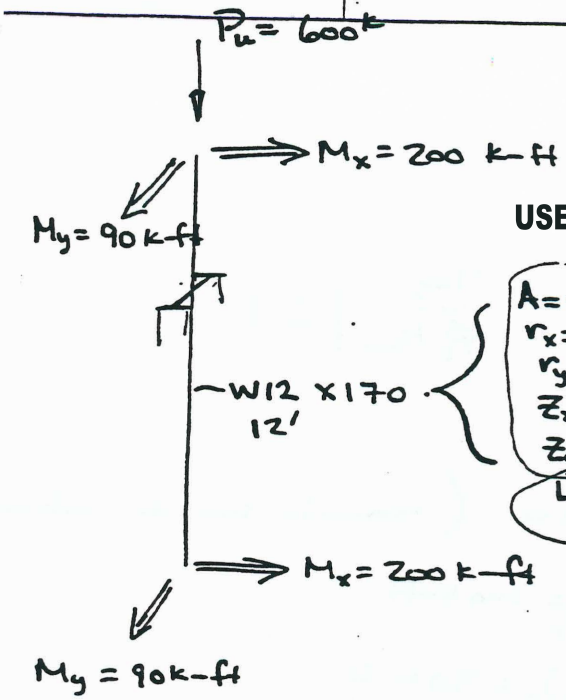
$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1 \qquad (6\text{-}3)$$

If $\dfrac{P_u}{\phi_c P_n} < .2$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1$$

$$M_{ux} = (B_1)_x (M_{nt})_x + (B_2)_x (M_{\ell t})_x$$

$$M_{uy} = (B_1)_y (M_{nt})_y + (B_2)_y (M_{\ell t})_y$$



USE A36 Steel

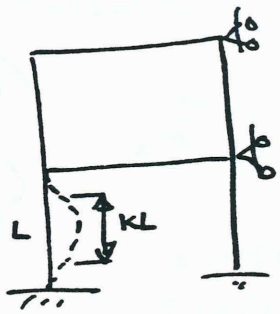
Moments from braced frame

$A = 50\ in^2$
$r_x = 5.74''$
$r_y = 3.22''$
$Z_x = 275\ in^3$
$Z_y = 126\ in^3$

$L_p = 13.4'$ — bottom page 4-19

In braced frame

$K \leq 1$



HOWEVER
code says in braced frame use $K = 1$

$$\left(\frac{KL}{r}\right)_x = \frac{(1)(12\ ft)}{5.74\ in}\left(\frac{12\ in}{ft}\right) = 25.09$$

$$\left(\frac{KL}{r}\right)_y = \frac{(1)(12\ ft)}{3.22\ in}\left(\frac{12\ in}{ft}\right) = 44.72$$

$\phi_c F_{cr} =$ 29.1ksi

$\phi_c P_n =$ (29.1ksi)($50\ in^2$) = 1455 k

$$\frac{P_u}{\phi P_n} = \frac{600^k}{1455\text{ k}} > .2$$

$$\therefore \text{ use } \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1$$

NATIONAL 42-381 50 SHEETS 5 SQUARE 42-382 100 SHEETS 5 SQUARE 42-389 200 SHEETS 5 SQUARE

In braced frame $M_{lt} = 0$ (moments due to sidesway)

$(M_{nt})_x = 200$ k-ft

$(M_{nt})_y = 90$ k-ft

$$C_{mx} = C_{my} = .6 - .4\frac{M_1}{M_2}$$

reverse curvature bending

$$\frac{M_1}{M_2} = +1$$

$$\therefore C_{mx} = .6 - .4(1) = .2$$

$$C_{my} = .6 - .4(1) = .2$$

Get P_{ex} & P_{ey}

Note P_{ex} ($\left(\frac{KL}{r}\right)_x$ only)

P_{ey} ($\left(\frac{KL}{r}\right)_y$ only)

CUT FROM OLD CODE

$$\boxed{\begin{aligned} P_{ex}(25.09) &= 454.84\ \tfrac{k}{in^2}(50\,in^2) = 22742^k \\ P_{ey}(44.72) &= 143.16\ \tfrac{k}{in^2}(50\,in^2) = 7158^k \end{aligned}}$$

OR

$$P_{ex}(KL)^2/10^4 = 47200$$

$$P_{ex} = \frac{47200 \times 10^4}{\left[(12')(12''/_1)\right]^2} = 22762\text{ k}$$

OR

$$\frac{\pi^2 EI}{(KL)^2} = \frac{3.14^2(29000)(1650)}{(144)^2} = 22775\text{ k}$$

Get B_{1x} & B_{1y}

$$B_{1x} = \frac{C_{mx}}{1-\frac{P_u}{P_{ex}}} = \frac{.2}{1-\frac{600}{22742}} = .204 \qquad \text{use } 1$$

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

$$B_{1y} = \frac{C_{my}}{1-\frac{P_u}{P_{ey}}} = \frac{.2}{1-\frac{600}{7158}} = .218 \qquad \text{use } 1$$

$$\therefore M_{ux} = B_1 M_{ntx} + B_2 \cancel{M_{ltx}}^{0} = (1)(200) + 0 = 200 \text{ k-ft}$$

$$M_{uy} = B_1 M_{nty} + B_2 \cancel{M_{lty}}^{0} = 1(90) + 0 = 90 \text{ k-ft}$$

since $L_b < L_p$

$$\phi_b M_{nx} = \phi_b F_y Z_{px} = .9\left(36\frac{k}{in^2}\right)(275 in^3)\left(\frac{ft}{12 in}\right) = 742.5 \text{ k-ft}$$

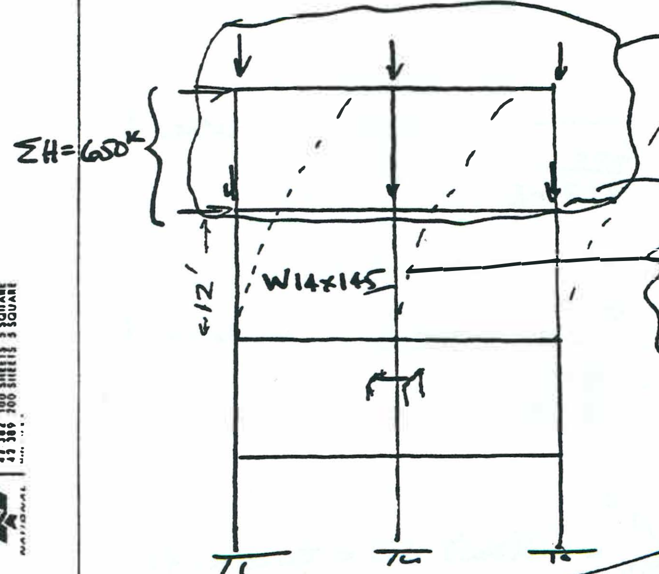
Also

$$\phi_b M_{ny} = \phi_b F_y Z_{py} = .9\left(36\frac{k}{in^2}\right)(126 in^3)\left(\frac{ft}{12 in}\right) = 340.2 \text{ k-ft}$$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = \frac{600}{1455k} + \frac{8}{9}\left(\frac{200}{742.5} + \frac{90}{340.2}\right)$$

$$= 0.89 < 1.00 \quad \text{OK}$$

Example — Rigid Frame 2 directions



No moments due to DL & LL

$M_{ux} = 290$ k-ft due to wind

$P_u = 800^k$

We could get K by getting G at top & bottom of column & using nomograph

Given:

$KL_x = KL_y = 14'$ $\qquad P_u = 800^k$

$\dfrac{\Delta_{oh}}{L} = .0025$ $\qquad M_{ux} = 290$ k-ft

From computer analysis of structure. (If no data available, .004 may be a reasonable value)

Get $\phi_c P_n$

$\phi_c P_n = 1190^k$ — for 36 ksi

$$\frac{P_u}{\phi_c P_n} = \frac{800}{1190} > .2$$

use

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1$$

Only Moments are due to joint translation

$\therefore M_{ntx} = M_{nty} = 0$

No moments about the y axis $\therefore M_{lty} = 0$

$$B_2 = \frac{1}{1 - \frac{\Sigma P_u}{\Sigma H}\frac{\Delta_{oh}}{L}} = \frac{1}{1 - \frac{20000k}{650k}(.0025)} = 1.08$$

$$M_{ux} = \cancel{B_1 M_{ntx}} + B_2 M_{ltx} = 1.08(290k\text{-}ft) = 313.2\ k\text{-}ft$$

Note $L_p = 16.6'$

$L_p = \frac{300 r_y}{\sqrt{F_{yf}}}$ or $1.76 r_y \sqrt{\frac{E}{F_y}}$

$$\therefore M_{nx} = \phi_b F_y Z_{px} = .9\left(36\frac{k}{in^2}\right)(260\ in^3)\left(\frac{ft}{12\ in}\right) = 702\ k\text{-}ft$$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = \frac{800}{1190} + \frac{8}{9}\left(\frac{313.2}{720} + 0\right) = 1.07 > 1$$

No good

Group Work Rework in 50ksi

Design of Beam-Columns

To get a satisfactory trial section without design aides may be a killer because it takes so long to check interaction formula. To get a trial section use: Table 6-2

Table 6-2 gives axial and bending strength that can be directly used in interaction equations. But only for 50ksi. If working with a different grade of steel you will need to bounce between chapters 3 and 4 to find all the info you need. Chapter 6 puts it all in one place.

This is a trial and error procedure where the solution and number of iterations will depend on the amount of bending and axial load that is required.

Start by assuming about 50% axial usage and therefore the 8/9 form of the interaction equation applies. This makes the first term 0.5.

So if Pu = 1000k, look for a capacity of 2000k using the blue lettered columns on the left side of Table 6-2. Remember you must first assume KLy buckling and scan down to the appropriate effective length value in the middle of the table. You are looking always looking for the lightest section that can safely carry the design, factored load.

To use this table, Lb and KLy can be taken from different lines depending on the actual bracing conditions.

Then get the bending capacity from the right side of the table and check values in the interaction equation(s).

SELECT A COLUMN in 50ksi (A572) steel

$L = 14'$

$M_{ntx} = 200\ k\text{-}ft \quad P_u = 400k$

$M_{nty} = 80\ k\text{-}ft$

$M_{ltx} = M_{lty} = 0$

$(KL)_x = (KL)_y = 14'$

$C_m = 0.85$

ASSUME INITIALLY $B_{1x} = 1$, $B_{1y} = 1$ $\therefore$ $M_{ux} = 200\ k\text{-}ft$, $M_{uy} = 80\ k\text{-}ft$

$\frac{P_u}{\phi P_n}$ = 0.5 (assumed), so we are looking for 800k where KL = 14ft

There are multiple sections that can satisfy this load, so let's restrict it to common column sections, try a W14 section.

Try W14x90: $\varphi_c P_n$ = 1030k $\varphi_b M_{nx}$ = 714k-ft $\varphi_b M_{nx}$ = 273k-ft page 6-77

400/1030 + 8/9 [200/714 + 80/273] = 0.9   Section OK

CHECK $B_{1x} = B_{1y} = 1$ ASSUMPTION

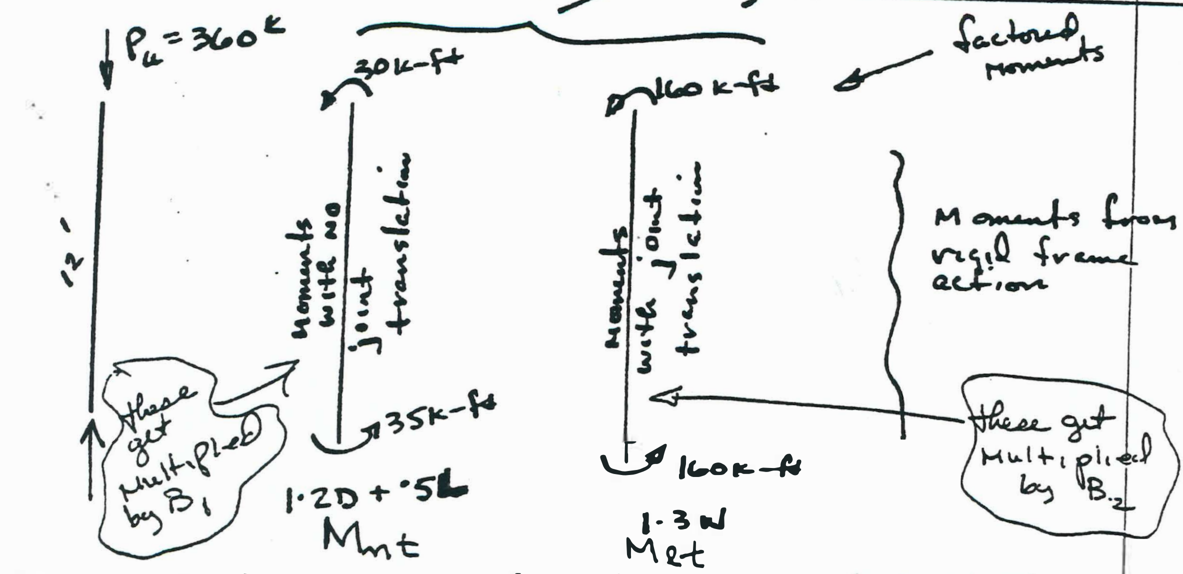
$$B_{1x} = \frac{C_m}{1 - \frac{P_u}{P_{ex}}} = \frac{0.85}{1 - \frac{400}{10133}} \qquad B_{1y} = \frac{C_m}{1 - \frac{P_u}{P_{ey}}} = \frac{0.85}{1 - \frac{400}{3685}}$$

$B_{1y} = 0.95$
$B_{1x} = 0.88$ } OK TO USE 1.0

Bottom of page 4-37

Example

M_x moments
$M_y = 0$



Assume 1.2D + .5L + 1.3W controls

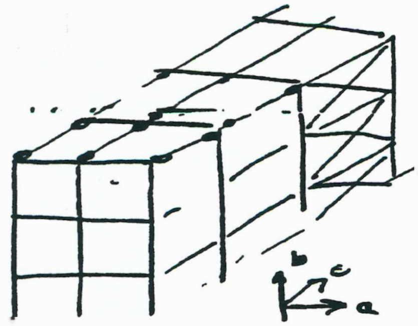
get lightest W12

$K_x = 1.5$ given $K_y = 1$ given

Column from building then

rigid unbraced frame in the ab plane

braced frame in the bc plane



ASSUMPTION #1

Assume for moment $B_1 = B_2 = 1$. We will get trial section & then we can calculate B_1 & B_2

Get trial section size

$KL_x = 1.5(12) = 18'$

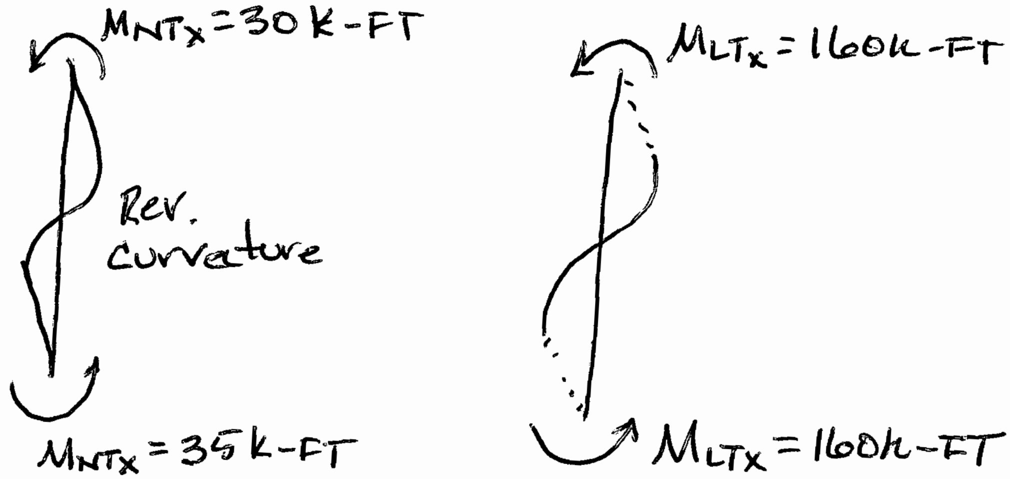
$KL_y = 1(12) = 12'$

$M_{ux} = B_1 M_{NT} + B_2 M_{LT}$

$= 1(35) + 1(160)$

$= 195$ K-FT

$P_u = 360K$



$$M_{r_x} = B_1 M_{NT_x} + B_2 M_{LT_x}$$

NEW CODE TERMINOLOGY

<u>ASSUME</u> $B_{1_x} = B_{2_x} = 1$ ✳

$$M_r = 1(35) + 1(160) = 195 \text{ k-FT}$$

<u>ASSUME</u> $\frac{KL_x}{r_x} < \frac{KL_y}{r_y}$ y axis controls ✳

FIND SECTION WITH Roughly 2 × 360K capacity

TRY W12×65 (6-87)

$\phi P_N = 728K$ $\quad \phi_b M_{Nx}(L_b = 12') = 356$ k-FT

$$\frac{360}{728} + \frac{8}{9}\left(\frac{195}{356} + \cancel{\frac{M_{uy}}{\phi_b M_{ny}}}^{\,0}\right) = 0.98 \quad \underline{\text{OK}}$$

CHECK ASSUMPTIONS

CHECK $B_1 = 1$

$$B_1 = \frac{C_m}{1 - \frac{P_u}{P_{ex}}} \quad (K = 1 \text{ for } B_1)$$

$$C_m = 0.6 \pm 0.4 \frac{M_1}{M_2}$$

$$= 0.6 - 0.4 \frac{30}{35} = 0.257$$

$$P_{ex} = \frac{15300 \times 10^4}{(144")^2} = 7378.5 \text{ kip}$$

$$B_1 = \frac{0.257}{1 - \frac{360}{7378.5}} = 0.257 < 1.0 \text{ OK}$$

$$B_2 = \frac{1}{1 - \frac{\Sigma P_{ux}}{\Sigma P_{ex}}}$$

entire floor ratio ≈ single column

$k = 1.5$ for sway moments

$$P_{ex} = \frac{15300 \times 10^4}{(18 \times 12")^2} = 3279.3 \text{k}$$

$$B_2 = \frac{1}{1 - \frac{360}{3279.3}} = 1.12 > 1.0 \text{ No good}$$

MUST GO BACK TO INTERACTION EQUN.

$$M_r = 1(35) + 1.12(195) = 253.4 \text{ k-FT}$$

$$\frac{360}{728} + \frac{8}{9}\left(\frac{253.4}{356} + 0\right) = 1.13 \text{ No good}$$

TRY LARGER SECTION

W12 × 72

$\phi P_N = 806k \quad \phi_b M_{NX} = 398\text{ k-FT}$

$$\frac{360}{806} + \frac{8}{9}\left(\frac{195}{398} + 0\right) = 0.88 \quad \text{OK}$$

NO SURPRISE!

B_1 STILL OK (P_{ex} SLIGHLY LARGER)

CHECK B_2 $\quad P_{ex}\ (KL = 18') = \dfrac{17100 \times 10^4}{(18 \times 12)^2} = 3665.1$

$$B_2 = \frac{1}{1 - \dfrac{360}{3665.1}} = 1.11$$

RECHECK INTERACTION EQUATION

$$M_r = 1(35) + 1.11(195) = 251.5\text{ k-FT}$$

$$\frac{360}{806} + \frac{8}{9}\left(\frac{251.5}{398} + 0\right) = 1.007$$

VERY CLOSE
BUT TECHNICAL NO GOOD

C_b COULD BE REVIEWED TO INCREASE $\phi_b M_N$

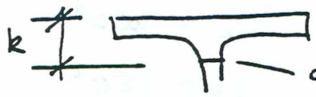
ALSO MUST CHECK ASSUMPTION 2

$$\frac{KL_x}{\dfrac{r_x}{r_y}} \le KL_y \qquad \frac{(1.5)(12)}{1.75} = 10.3 \quad \text{OK}$$

BOTTOM (P 4-40)

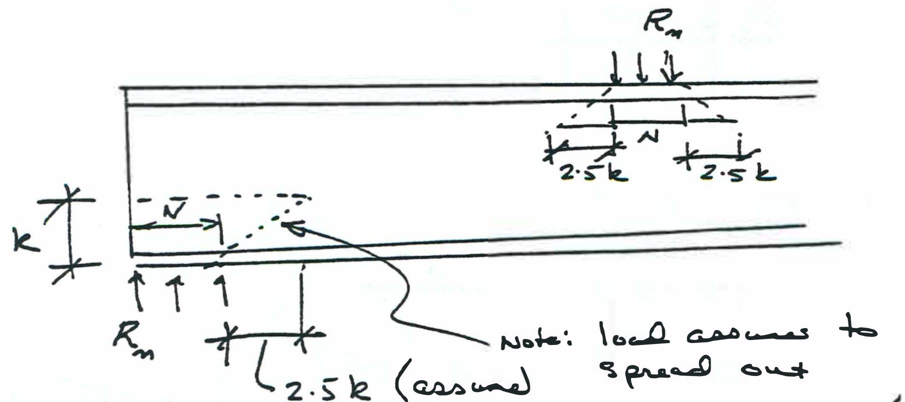
Web Yielding

<u>Examine W</u>



at dist down from top flange of k, web is thinest

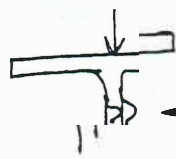
(161-136)



At support $\quad R_n = (N + 2.5k)\, t_w F_{yw}$

At interior $\quad R_n = (N + 5k)\, t_w F_{yw}$

We don't want local buckling of web under load



don't want local buckling

a) for load $> \frac{d}{2}$ from end mem (16.1-136)

$$R_n = 135 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right] \sqrt{F \ldots t_f}$$

$\phi = .75$

b) for loads located $< d/2$ from end of mem

* case 1 $N/d \le 0.2$

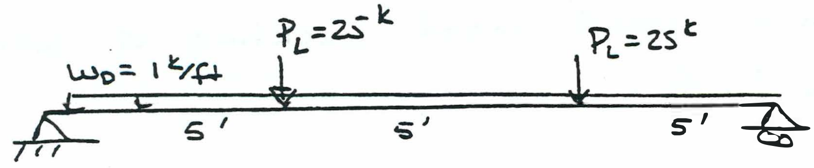
$$R_n = 68 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right] \quad t_w \quad t_f$$

* case 2 $N/d > 0.2$

$$R_n = 68 t_w^2 \left[1 + \left(\frac{N4}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right] \sqrt{\frac{F_{yw} t_f}{t_w}}$$

(a)

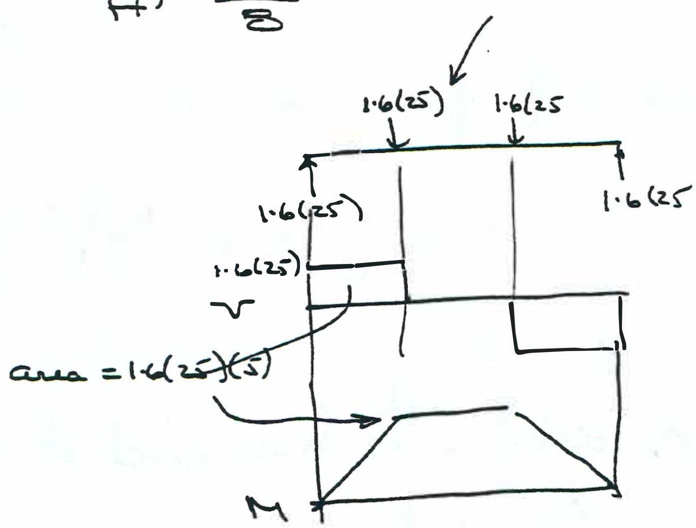
Example – Web Yielding & Web Crippling



Bm Braced at end & loads

est bm wt = $44 \frac{lbs}{ft}$

$$M_u = 1.2\left(1.044\frac{k}{ft}\right)\frac{(15ft)^2}{8} + 1.6(25^k)(5ft) = 235.2\,k\text{-}ft$$



Use W 21×44 $\quad L_p = 5.3' > 5'$

$d = 20.66''$

$t_w = .350''$

$t_f = .450''$

$k = 1\frac{3}{16}''$

Get reaction at support

$$R_u = 1.2\left(1.044\frac{k}{ft}\right)\left(\frac{15ft}{2}\right) + 1.6(25^k) = 49.4^k$$

Get bearing length to prevent web yielding at supports

$$R_n = (2.5k + N)\,F_{yw}\,t_w$$

$$\phi = 1$$

$$\phi R_n = R_u$$

$$(2.5k + N)\,F_{yw}\,t_w = 49.4^k$$

$$\left(2.5\left(\frac{19}{16}\right) + N\right)\left(36\frac{k}{in^2}\right)(.350\,in) = 49.4^k$$

$$N = .95''$$

Get bearing length to prevent web crippling at supports

$$R_n = 68t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw}t_f}{t_w}}$$

$$\phi = .75$$

$$\phi R_n = R_u$$

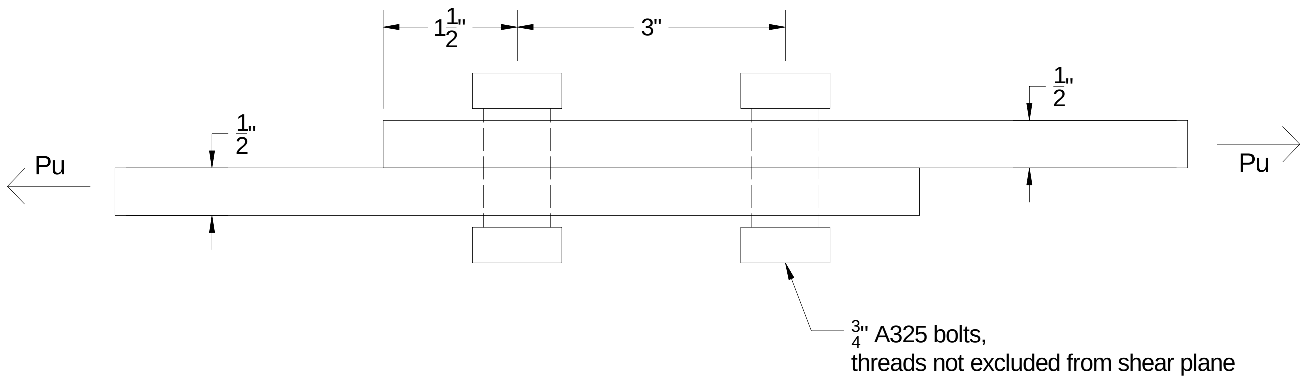
$$.75(68)(.350)^2\left[1 + 3\left(\frac{N}{20.66}\right)\left(\frac{.350}{.450}\right)^{1.5}\right]\sqrt{\frac{36(.450)}{.350}} = 49.4^k$$

$$N = 1.63$$

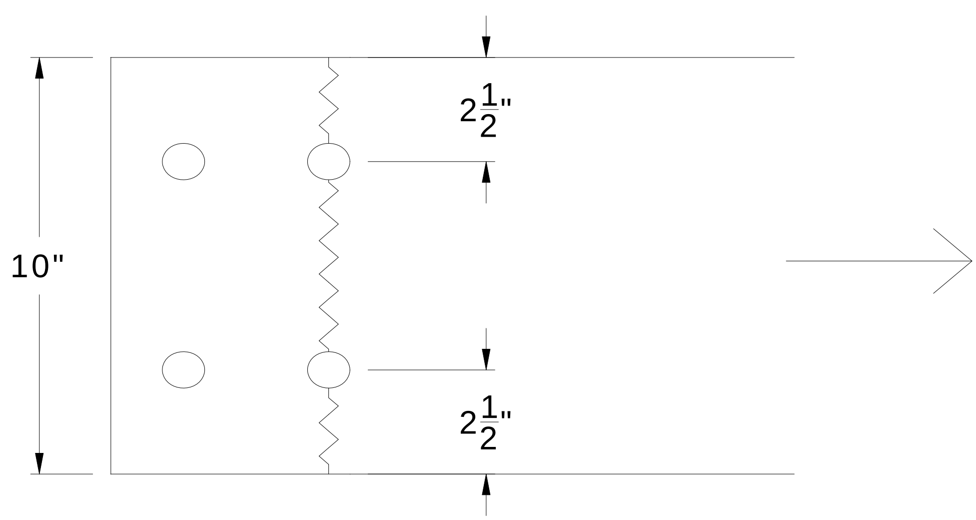
use 4" min

Bolted Connections

Consider Lap Joint:



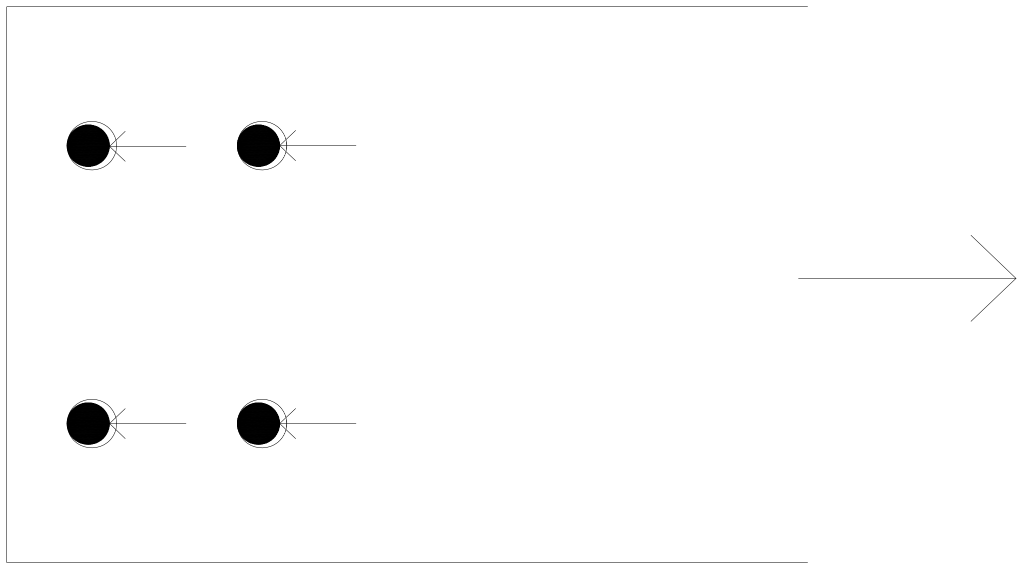
Examine Tensile Failure:



Gross: $P_u = \phi P_n = \phi F_y A_g = 0.9(36ksi)(10in)(0.5in) = 162\ k$

Net Eff.: $P_u = \phi P_n = \phi F_u A_e = 0.75(58ksi)\left\{\left[10 - 2\left(\frac{7}{8}\right)\right] in\right\}(0.5in) = 179.4\ k$

Examine Bearing Failure:



AISC Section J3.10 (16.1-136)

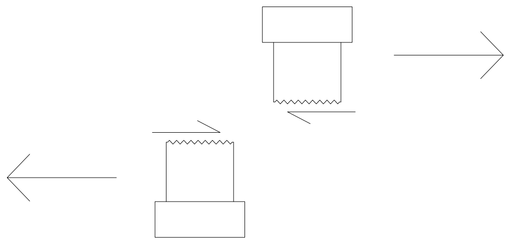
For standard bolts spacing.

(when spacing less, use $1.2L_ctF_u$*)*

$$P_u = \phi P_n = [\phi(\overbrace{2.4dtF_u})](4\ bolts)$$

$$= 0.75(2.4)\left(\frac{3}{4}in\right)\left(\frac{1}{2}in\right)(58ksi)(4) = 156.6\ k$$

Examine Bolt Shear Failure



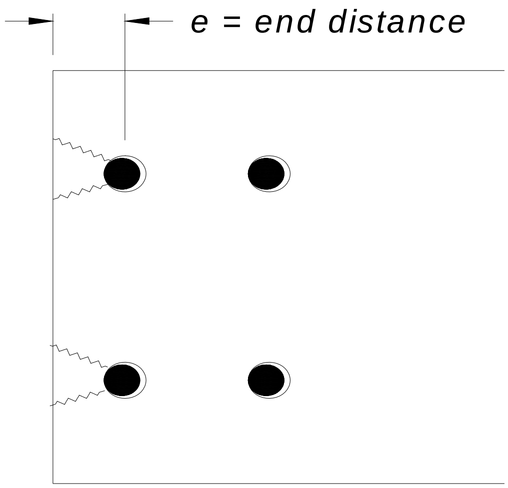
$$P_u = \phi F_{nv} A_{bolt}(4\ bolts) \qquad \text{(AISC 16.1-131)}$$

Shear strength of bolt,

Table J3.2 (AISC 16.1-129)

$$= 0.75(54ksi)\left[\pi\left(\frac{3}{8}\right)^2\right](4) = 71.3\ k$$

Check End Failure



$$e_{provided} = 1.5"$$

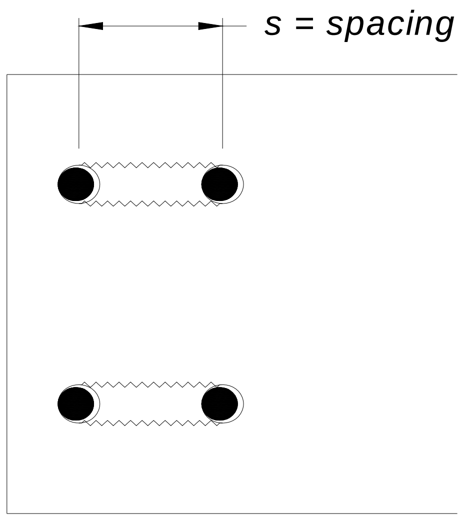
From Table J3.4 (AISC 16.1-131):

$$e_{minimum} = 1.25" \qquad OK$$

Also acceptable:

$$e_{min} = 1.5d = 1.125" \qquad OK$$

Check Spacing Failure



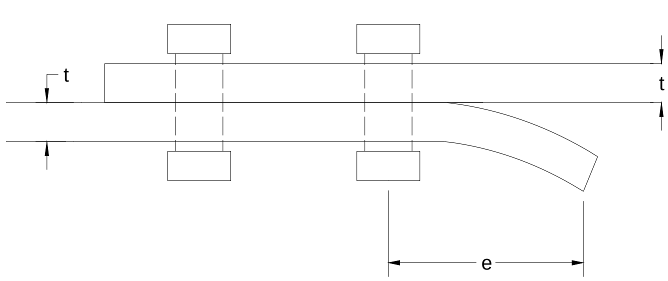
$$s_{provided} = 3"$$

From Section J3.3 (AISC 16.1-130):

$$s_{min} = 3d = 3\left(\frac{3}{4}\right) = 2.25" \quad OK$$

Check Maximum Edge Distance

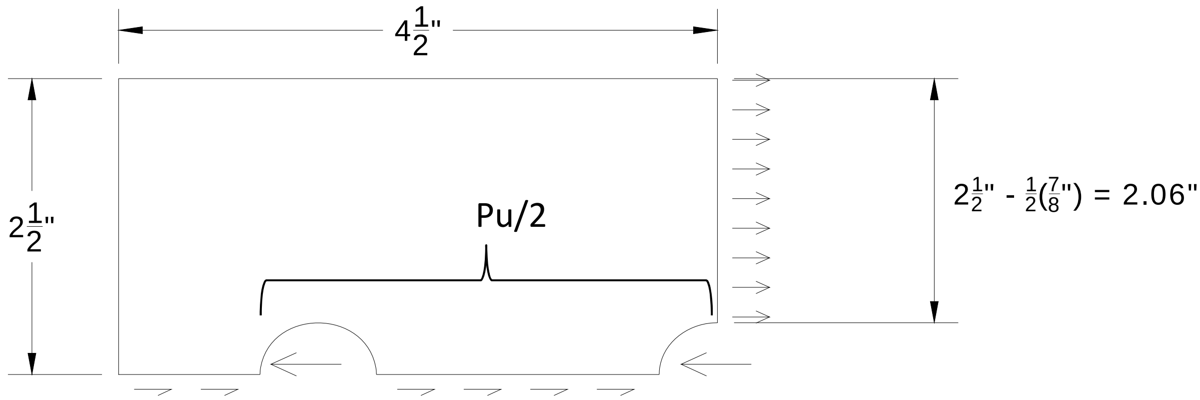
From Section J3.5 (AISC16.1-131):



$$e_{max} = 12t < 6"$$

$$= 6" \qquad OK$$

Examine Block Shear - through 2 holes



tension fracture & shear fracture:

$$\frac{P_u}{2} = 0.75\left\{\left(\frac{1}{2}in\right)(2.06in)(58ksi) + \left[4.5in - 1.5\left(\frac{7}{8}in\right)\right]\left(\frac{1}{2}in\right)(0.6)(58ksi)\right\}$$

$$P_u = 172.8\ k$$

1 ½ bolt holes

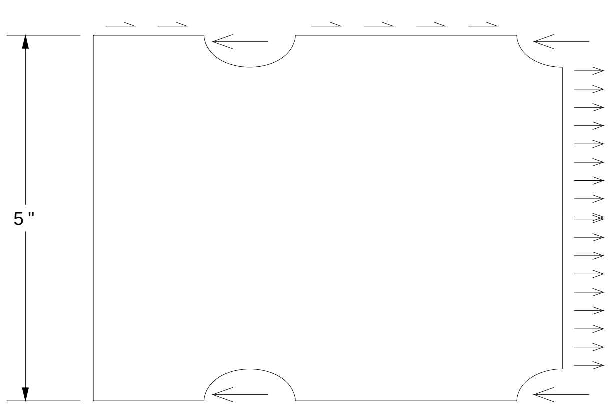
tension fracture & shear yielding:

$$\frac{P_u}{2} = 0.75\left\{\left(\frac{1}{2}in\right)(2.06in)(58ksi) + [4.5in]\left(\frac{1}{2}in\right)(0.6)(36ksi)\right\}$$

$$P_u = 162.5\ k$$

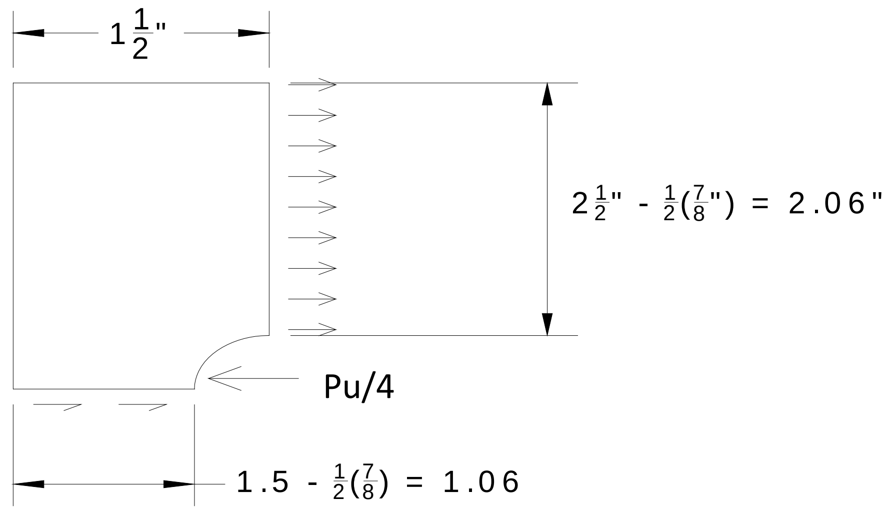
smaller controls for block shear

Examine Block Shear – through 4 holes



Load and surface area are both doubled from above, so Pu is the same as for 2 holes.

Examine Block Shear – through 1 hole



tension fracture & shear fracture:

$$\frac{P_u}{4} = 0.75\left\{\left(\frac{1}{2}in\right)(2.06in)(58ksi) + [1.06]\left(\frac{1}{2}in\right)(0.6)(58ksi)\right\}$$

$$P_u = 234.6\ k$$

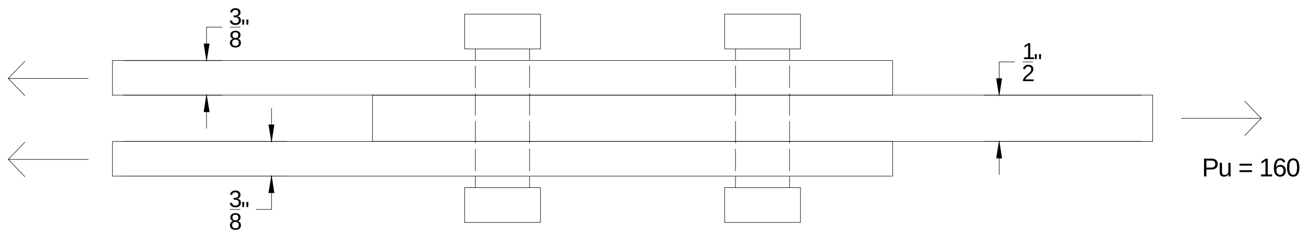
tension fracture & shear yielding:

$$\frac{P_u}{4} = 0.75\left\{\left(\frac{1}{2}in\right)(2.06in)(58ksi) + [1.5in]\left(\frac{1}{2}in\right)(0.6)(36ksi)\right\}$$

$$P_u = 227.8\ k$$

Considering all possible failure modes, we see that shear in the bolts controls, so the usable strength of this connection is P_u = 63.6 k.

Example – bearing connection, double shear



Design for the lap length and number of bolts. Use 7/8" A325 bolts, threads excluded.

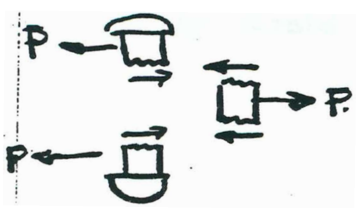
Assume edge distance & spacing ok – assume PLs ok

Check bearing

ult load carried by 1 bolt bearing on 2 - 3/8" PL $= \overset{.75}{\cancel{\phi}}(2.4 \overset{58\frac{k}{in^2}}{\cancel{F_u}})\underbrace{(2)(\tfrac{3}{8}")(\tfrac{7}{8}")}_{\text{bearing area}} = 68.5^k$

ult load carried by 1 bolt bearing on 1 - 1/2" PL $= (.75)(2.4)(58\frac{k}{in^2})(\tfrac{1}{2}")(\tfrac{7}{8}") = 45.7^k$

Check shear



(16.1-129)

ult load in shear $= \phi\left(\text{shear strength bolt}\right) \underbrace{A_{bolt}\,(2)}_{\text{two failure planes — double shear}}$

$= .75(68\frac{k}{\cancel{in^2}})\left(\frac{\pi(\frac{7}{8})^2 \cancel{in^2}}{4}\right)(2)$ double shear

= 62.1k/bolt

bolts required – $\dfrac{150^k}{22.5\frac{k}{bolt}}$ = 2.45 bolts   use 3 bolts

Slip Critical Connections

Even though the available strength of a bearing type connection may be adequate, in many instances we want to limit slippage in the connection, usually for serviceability reasons. For example:

- Connections subject to load reversals
- Connections subject to fatigue
- Connections with oversized or slotted holes
- Connections where even small displacements are undesirable (e.g. causing architectural damage)
- Connections with few bolts (i.e. greater probability of not having any bolt with initial bearing)

These are typically serviceability concerns, designed for using the service limit state, meaning that we want to keep them at bay for the day-to-day "expected" loads, but we are willing to allow slippage in holes for the rare higher loads as long as structural failure does not occur. There are, however, some cases in which a serviceability failure, often with oversized or slotted holes, could inadvertently cause an increase in load beyond that of the strength limit state. In these cases, the slip resistance of a connection should be designed for the strength limit state. Examples include:

- High aspect braced frames where displacement due to slip could result in large P-Δ effects.
- Long-span, flat roof trusses where displacement due to slip could result in ponding, creating greater loads.
- Built-up compressions members where slip could increase the effective length of the member, thus reducing buckling strength.
- Any condition where potential displacement due to slip is larger than that accounted for in structural analysis.

In any circumstance where slip is a concern, design should be done in one of two ways:

1. Design the connection to resist slip under service loads, then include the effects of the deformed structure in structural analysis.
2. Design the connection to resist slip under all loads.

Section J3.8 (AISC 16.1-134)

0.30 Class A (mill scale)

0.50 Class B (blast cleaned)

Factored Load

Min. bolt pretension Table J3.1 16.1-127

$$P_u \leq \phi \mu D_u h_f T_b n_s [n_b]$$

\# of bolts

\# of slip planes

standard holes 1.0

1.13

oversized or short slots 0.85

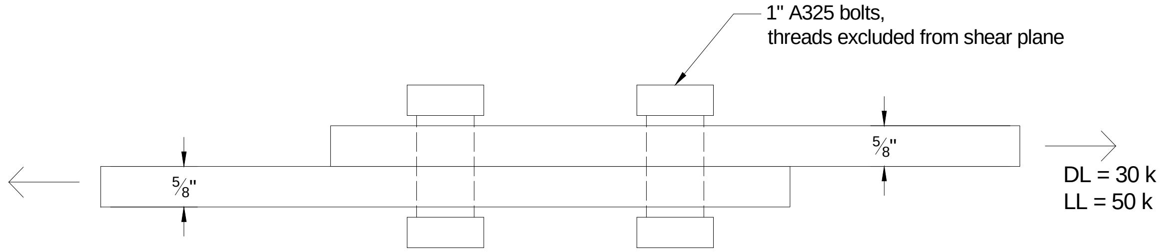
long slots 0.70

1.0 no fillers OR bolts added

1.0 one filler, no bolts added

0.85 two or more fillers, no bolts added

Example

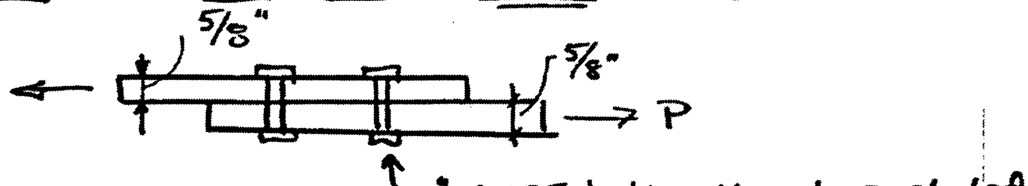


How many bolts for a bearing type connection? Slip critical?

Slip Critical Connections

Design Strength as bearing type connection must be greater than factored forces

Example – Slip Critical Connection



$P_D = 30^k$

$P_L = 50^k$

Examine Slipping

Get strength of 1 bolt

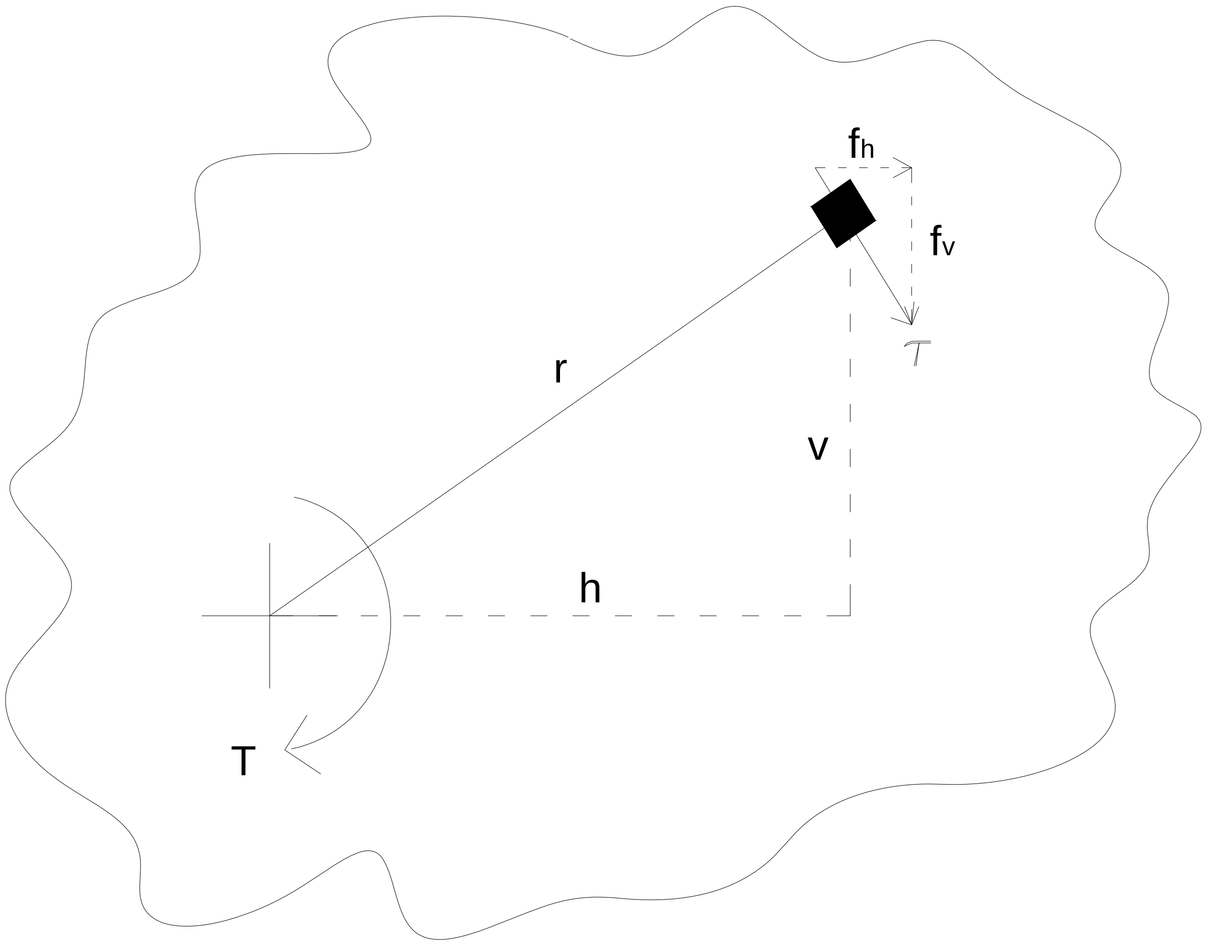
$\varphi\mu D_u T_b n_s$ = single bolt slip critical capacity

φR_n = 0.75 (0.3)(1.13)(51k)(1) = 12.97k/bolt slip critical capacity

no bolts = 112k / 12.97 = 8.9 bolts (use 9 or 10 depending on pattern)

fewer bolts are needed if blasted surface condition is provided (μ = 0.5)

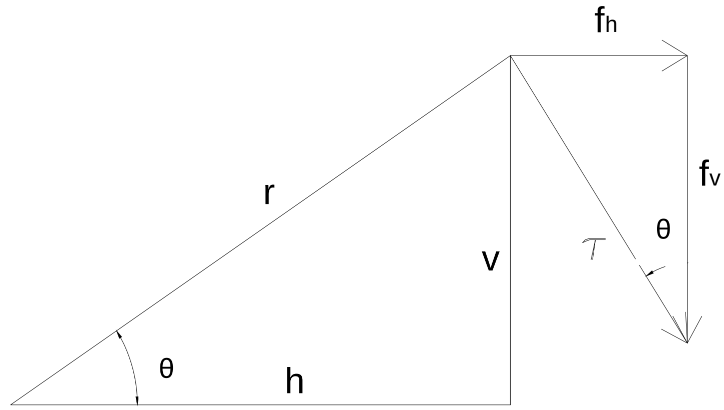
Torsional Bolt Groups



$J = I_x + I_y$

$$\tau = \frac{Tr}{J}$$

Examine angles

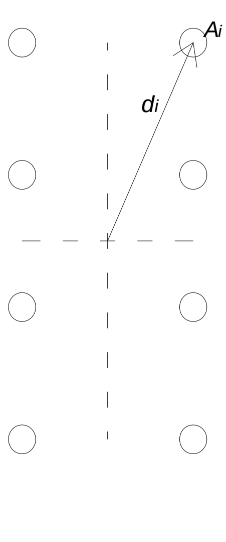


$$f_v = \tau \cos\theta = \frac{Tr}{J}\frac{h}{r} = \frac{Th}{J} \qquad f_h = \tau \sin\theta = \frac{Tr}{J}\frac{v}{r} = \frac{Tv}{J}$$

$$f_v = \frac{Th}{J}$$

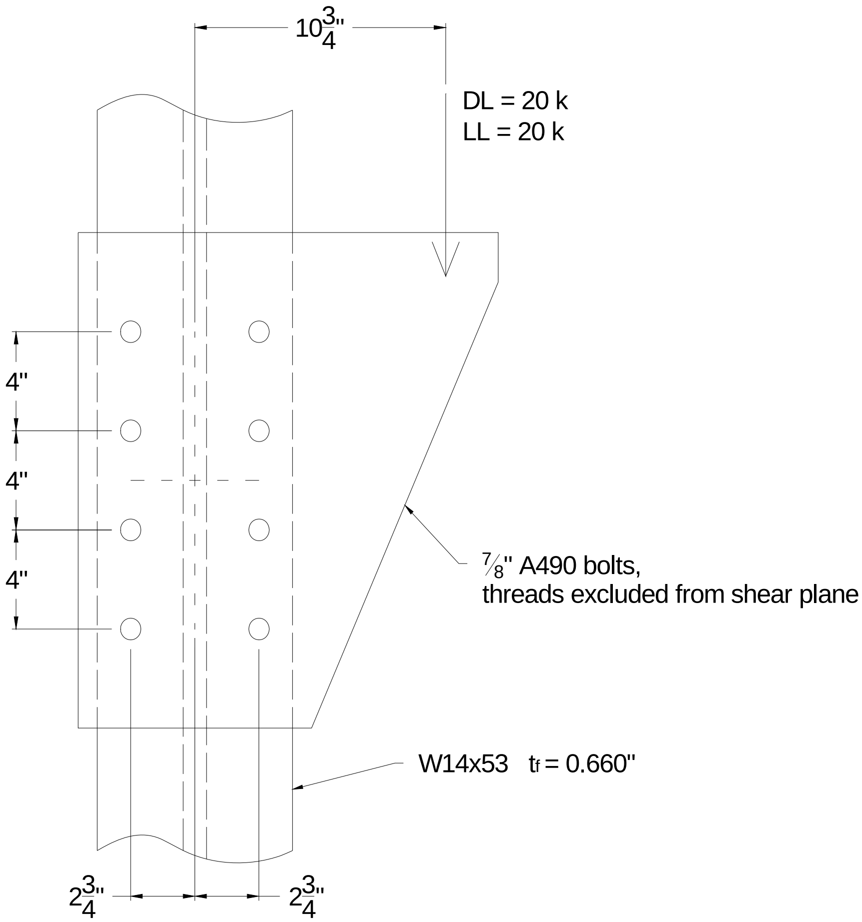
$$f_v = \frac{Th}{A\sum {d_i}^2}$$

$$f_v A = V = \frac{Th}{\sum {d_i}^2} \qquad likewise, \qquad H = \frac{Tv}{\sum {d_i}^2}$$



$$J = \sum A_i {d_i}^2$$
$$= \sum A_i({x_i}^2 + {y_i}^2)$$

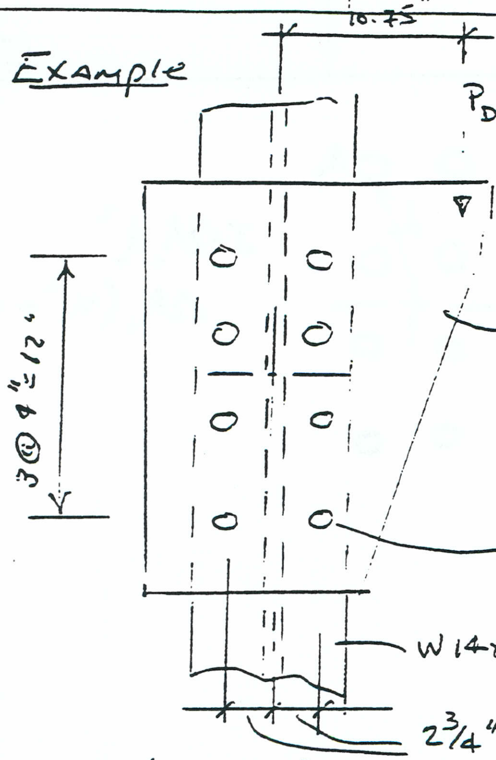
Example



10 3/4"
DL = 20 k
LL = 20 k
4"
4"
4"
7/8" A490 bolts,
threads excluded from shear plane
W14x53 tf = 0.660"
2 3/4"
2 3/4"

Example

$P_D = 20^k$, $P_L = 20^k$, $P_u = 1.2(20) + 1.6(20) = 56^k$



$\frac{7}{16}$" PL = .4375"

$\frac{7}{8}$" A490 bolts threads excluded from shear plane

W14×53 $t_f = .660$ in

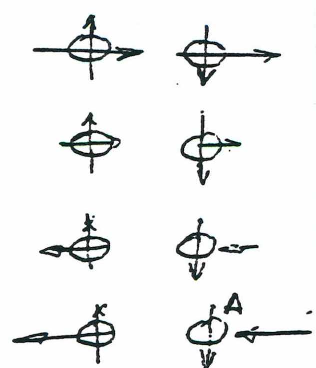
$T = 56^k(10.75\,in) = 602^{k}\text{-}in$

$\Sigma d_i^2 = \Sigma x_i^2 + \Sigma y_i^2 = (2.75\,in)^2(8) + 4(2\,in)^2 + 4(6\,in)^2$

$= 220.5\,in^2$

$(V)_{\text{Torque}} = \dfrac{Th}{\Sigma d_i^2} = \dfrac{602\,k\text{-}in(2.75\,in)}{220.5\,in^2} = 7.51^k$

$H_{MAX} = \dfrac{T\,V_{MAX}}{\Sigma d_i^2} = \dfrac{602\,k\text{-}in\,(6\,in)}{220.5\,in^2} = 16.38^k$

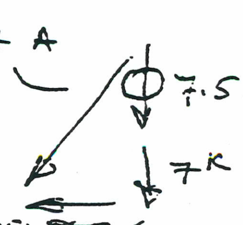


due to torque

$(V)_{\text{due to Shear}} = \dfrac{56^k}{8} = 7^k$

Get worst resultant $= \sqrt{(16.38)^2 + (7.51 + 7)^2} = 21.88^k$

bolt A



Check Shear bolt

Strength 1 - $\frac{7}{8}$" A490 bolt = φ Ab Fv

= 0.75(0.6in^2)(84ksi) = 37.8k > 21.88k OK

Check Bearing

Bearing of plate critical $\frac{7}{16}" < (t_f)_W = .660 in$

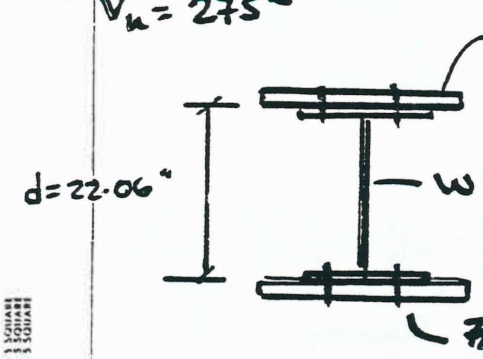
Strength one bolt = $\underset{.75}{\phi}$(contact area)(2.4 F_u)

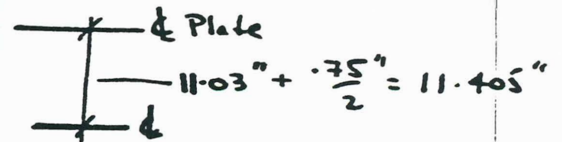
$= .75\left(\frac{7}{16}"\right)\left(\frac{7}{8}"\right)(2.4)\left(58\frac{k}{in^2}\right) = 39.97^k$

$= 39.97^k > 21.88^k$ OK

Bearing Connection

$V_u = 275^k$





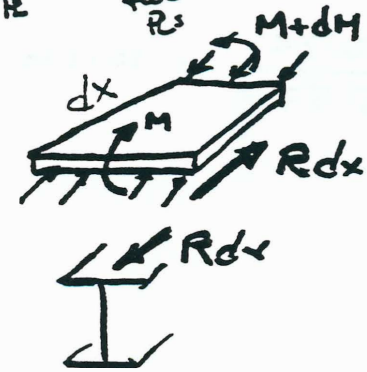
$I_x = 3630 \text{ in}^4$

$d = 22.06 \text{ in}, \; t_f = 1.15"$

$$I = 3630 \text{ in}^4 + \underbrace{\left(\tfrac{3}{4}\right)(16)\text{ in}^2}_{\text{area}} \underbrace{(11.405 \text{ in})^2}_{\text{dist}} \underset{\uparrow \; \text{PL top \& bottom}}{(2)} = 6753 \text{ in}^4$$

$$+ \underbrace{\tfrac{1}{12}(16 \text{ in})\left(\tfrac{3}{4}\text{ in}\right)^3}_{I_{PL}} \underset{\uparrow \; \text{two PLs}}{(2)}$$

Review

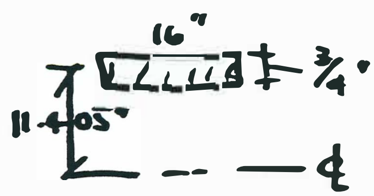


$$Rdx = \int_y^{y_{top}} \frac{dM \, y \, b \, dy}{I}$$

$$R = \int_y^{y_{top}} \frac{dM}{dx} \frac{y \, b \, dy}{I}$$

$$R = \frac{VQ}{I}, \quad Q = \int_y^{y_{top}} b \, y \, dy$$

Get R at interface conn PL & W



$Q = 16 \text{ in}(.75 \text{ in})(11.405")$

$= 136.86 \text{ in}^3$

$$R = \frac{VQ}{I} = \frac{275^k (136.86 \text{ in}^3)}{6753 \text{ in}^4} = 5.57 \frac{k}{\text{in}}$$

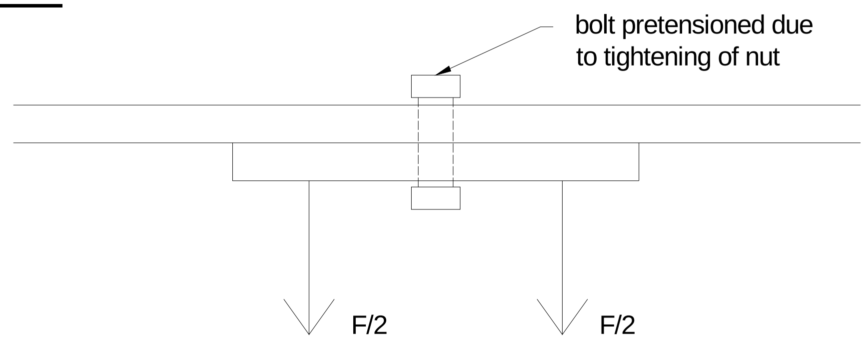
Design strength 2 bolts in shear $= \underbrace{2}_{\text{two bolts}}\underbrace{\left(\pi\frac{\left(\frac{7}{8}\right)^2}{4}\,in^2\right)}_{A_b}\left(\underset{(16.1\text{-}129)}{68ksi}\right)\left(\underset{\phi}{0.75}\right)$ = 61.2k (controls)

Design strength 2 bolts in bearing $= \underbrace{2}_{\text{two bolts}}\underbrace{\left(\frac{3}{4}''\right)\left(\frac{7}{8}''\right)}_{\text{contact areas}}\underbrace{(2.4)\left(58\frac{k}{in^2}\right)}_{\text{Bearing strength}}\underbrace{(.75)}_{\phi} = 137.0^k$

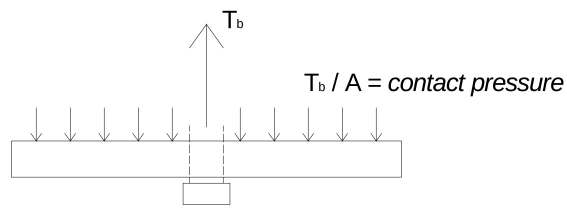
spacing bolts $= \dfrac{\text{strength provided}}{\text{strength required per inch}}$

$= \dfrac{61.2k}{5.57\frac{k}{in}}$ = 10.99 bolts use 12 (in pairs)

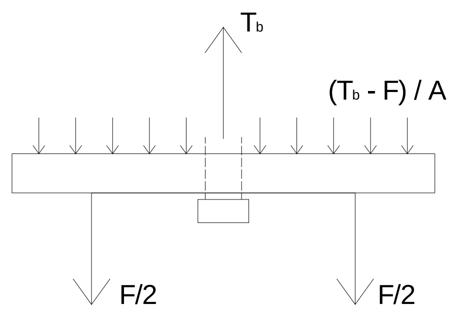
Bolts in Tension



Before F applied:

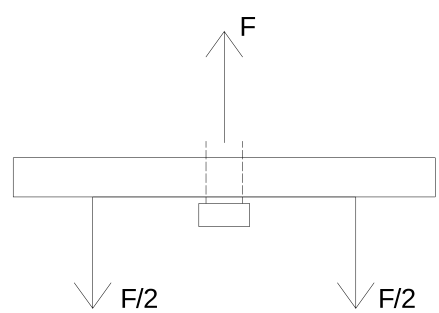


After F applied ($F < T_b$):



Contact stress reduced. Force in bolts stays approx. same.

For $F > T_b$:

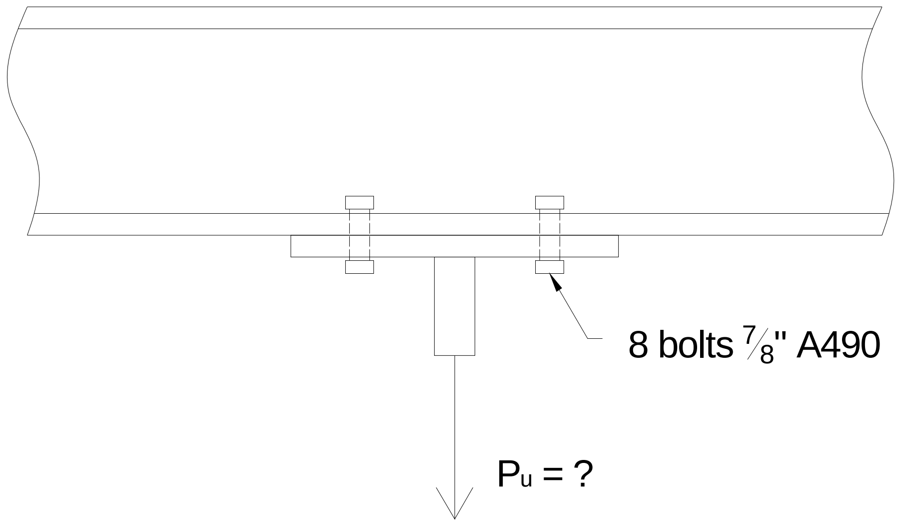


Only when $F > T_b$ does applying tensile load actually change the force in a bolt.

When bolts are tightened, tensile stress induced is near F_y. Some engineers are afraid to load bolts in tension for this reason. From above we see that this is not necessarily a problem though. Following code procedure will give a safe design.

(AISC 16.1-131) $P_u \le \varphi F_{nt} A_{bolt} n_{bolts}$ *ϕ=0.75* *(F_{nt} AISC 16.1-129)*

Example



$$P_u = \phi A_b (\text{strength})(\underset{\uparrow \atop \#\text{ bolts}}{8})$$

$$= .75\left(\pi\frac{\left(\frac{7}{8}\right)^2}{4}\right)\left(112.5\frac{k}{in^2}\right)(8) = 405^k$$

Combined Tension and Shear in Bearing-Type Connections

Combined stresses in bolt from shear and tension may overload the bolts.

Check shear same as before, but reduce tensile capacity based on the amount of imposed shear.

- Find imposed shear in bolts, f_{rv}
- Then find usable tensile strength (F'_{nt}) modified to include the effects of shear. *J3.7 (AISC 16.1-134)* :

$$P_u \leq \phi F'_{nt} A_b \qquad\qquad \phi = 0.75$$

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt}$$

$$\left.\begin{array}{l} F_{nt} = nominal\ tensile\ strength \\ F_{nv} = nominal\ shear\ strength \end{array}\right\} \textit{Table J3.2}$$

$$f_{rv} = imposed\ shear\ stress\ from\ load\ combinations$$

Combined Tension and Shear in Slip Critical Connections

Tension force will reduce clamping force which in turn will reduce friction.

Code applies reduction to the slip-critical capacity *J3.9 (AISC 16.1-135)*

Slip Capacity (same as before)

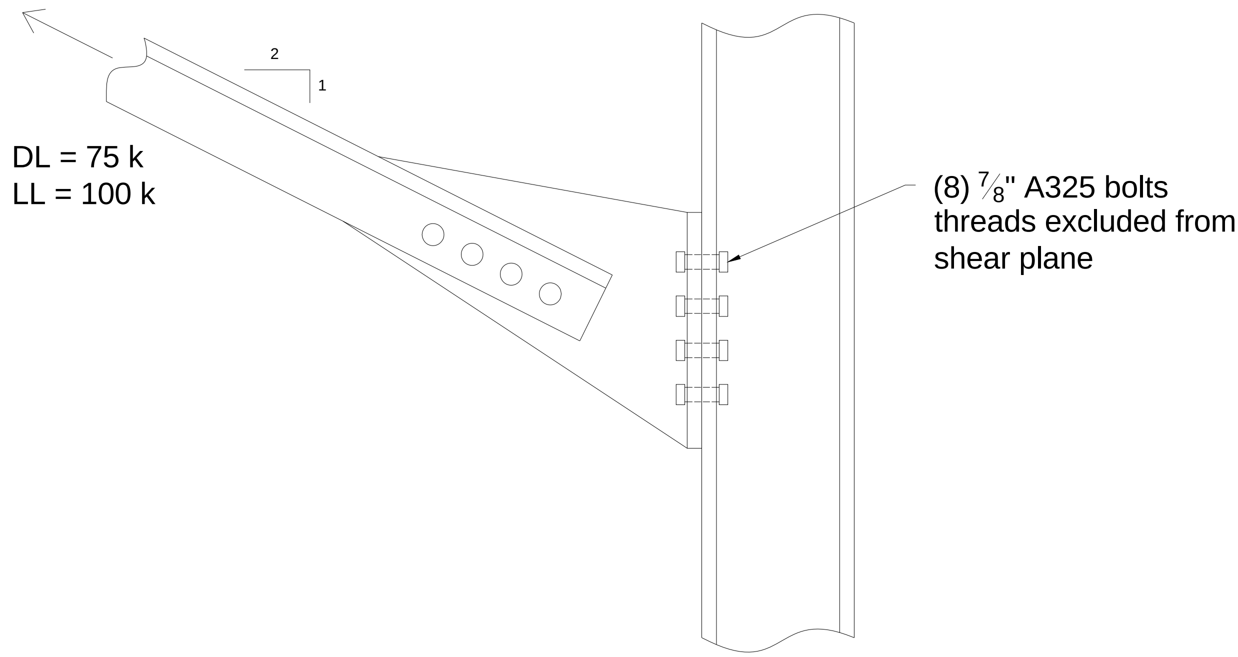
$$P_u \leq k_{sc}\phi P_n$$

Factored tension force in bolts from load combinations

$$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b}$$

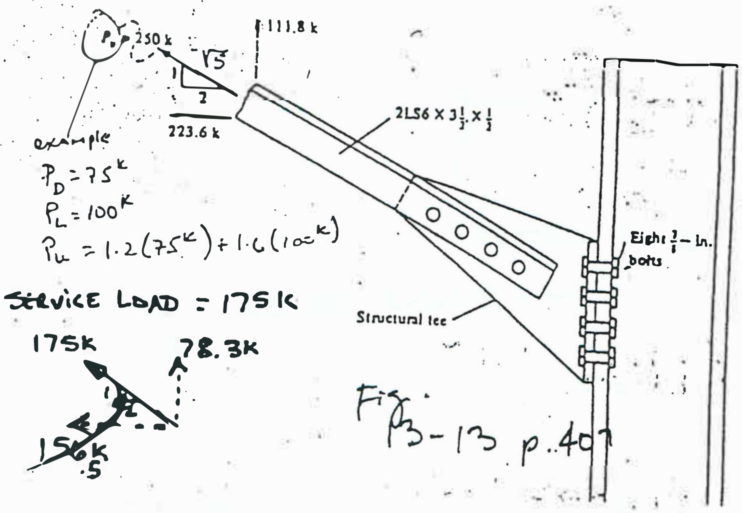
# of bolts carrying the applied tension

Example



Check shear, tension, and slip critical capacity of bolts (*to check bearing you would also need flange thicknesses, etc.*).

BOLTS SUBJECTED TO SHEAR & TENSION (INSERT)



$P_D = 75^k$

$P_L = 100^k$

$P_u = 1.2(75^k) + 1.6(100^k)$

SERVICE LOAD = 175 K

Fig. B3-13 p. 407

7/8" A325 BOLTS

THREADS EXCLUDED FROM SHEAR PLANE

BEARING CONNECTION

GET SHEAR STRESS (f_v)

$$A_b = \frac{\pi (7/8)^2}{4} = .60 \text{ in}^2$$

$$f_v = \frac{111.8 \text{ k}}{.60(8)} = 23.29 \text{ ksi} < \phi \text{ STRENGTH}$$

SHEAR OK ✓

$.75\,(68\text{ksi}) = 51$ ksi

GET TENSILE STRESS (f_t)

$$f_t = \frac{223.6 \text{ k}}{.60(8)} = 46.58 \text{ ksi}$$

(16.1-134)

$$F'_{nt} = 1.3F_{nt} - F_{nt}(f_{rv}/\varphi F_{nv})$$

$$F'_{nt} = 1.3\,(90) - (90)(23.29/51) = 75.9\text{ksi}$$

75.9 ksi > 46.58

TENSION OK

SLIP CRITICAL REDUCTION FACTOR 16.1-135

$$k_{sc} = 1 - \frac{T_u}{1.13\,T_b\,n_b}$$

$$= 1 - \frac{223.6\,k}{1.13(39_k)(8)}$$

16.1-127

$$k_{sc} = 0.36$$

SHEAR COMPONENT

$$P_u \le \overset{1.0}{\phi}\ \underset{\text{ASSUME } 0.5}{\mu}\ 1.13\ \overset{39}{T_b}\ \overset{1}{n_s}\ \overset{8}{n_b}\ \overset{0.36}{k_{sc}}$$

$111.8_k \le 63.5_k$ NO GOOD EVEN WITH HIGHER μ

Bearing connection

A325X – 3/4" bolts, $A_b = .44\ in^2$

threads excluded from shear plane

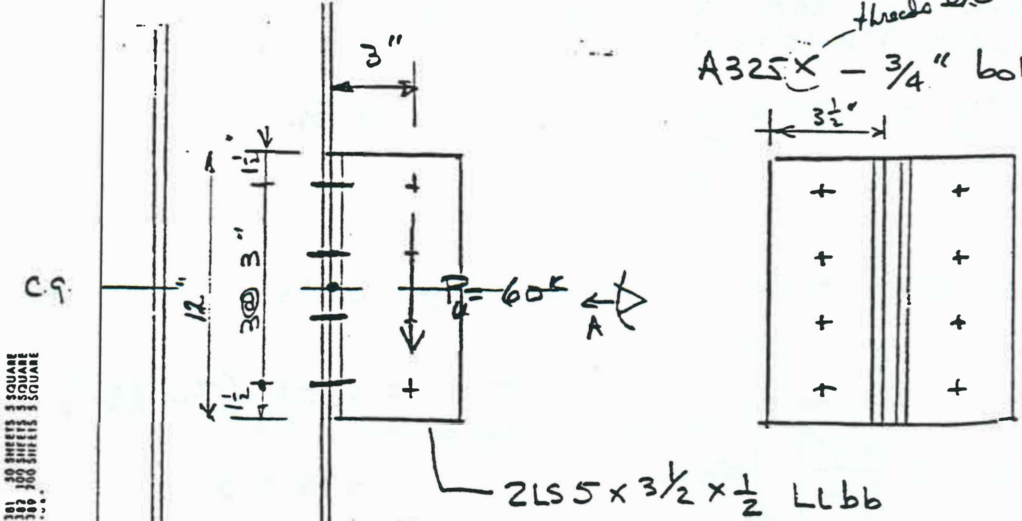
$P_L = 30^k$

$P_D = 10^k$

$P_u = 1.2(10) + 1.6(30)$

$= 60^k$

$P_{service} = 40^k$



a) Check bearing connection

Check initial bearing pressure $\quad T_i = 28^k$

$$f_{bi} = \frac{8(28^k)}{7in\,(12in)} = 2.67\ ^k/_{in^2}$$

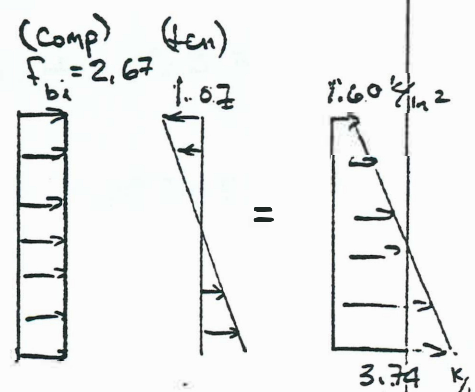
Check stress top plate f_{tp}

$$M = 60^k(3in) = 180\ ^k\text{-}in$$

$$S = \tfrac{1}{6}bd^2 = \tfrac{1}{6}(7in)(12in)^2 = 168\ in^3$$

$$f_{tp} = \frac{M}{S} = \frac{180\ ^k\text{-}in}{168\ in^3} = 1.07\ \frac{k}{in^2}$$

contact stress looks thus



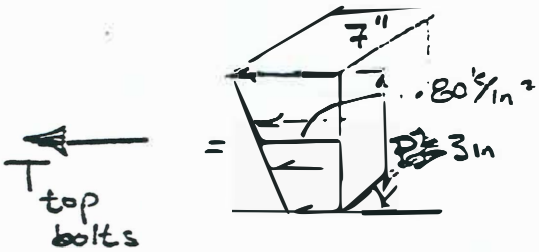
OK – contact maintained

Get tension in top bolt due to M

$I = \frac{1}{12}(7\text{in})(12\text{in})^3 = 1008\,\text{in}^4$

$$f_{tb} = \frac{My}{I} = \frac{180\,\text{k-in}\,(4.5\,\text{in})}{1008\,\text{in}^4} = .80\ \text{k/in}^2$$

this stress must go into top bolts



$$T_{\text{top bolts}} = .80\,\frac{\text{k}}{\text{in}^2}(7\text{in})(3\text{in}) = 16.80^{\text{k}}$$

$T_{\text{top bolts}}$ carried by 2 bolts with stress f_t

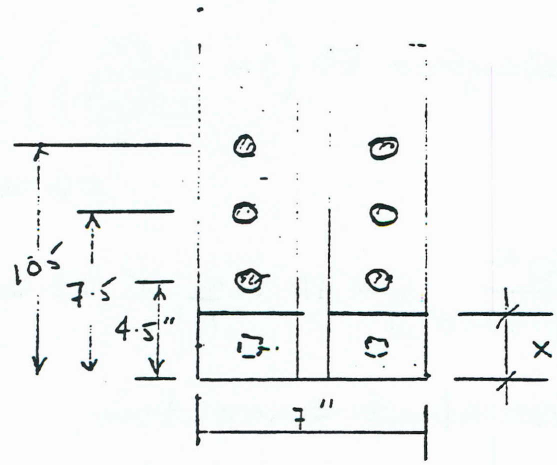
$$f_t = \frac{16.80^{\text{k}}}{2(.44\,\text{in}^2)} = 19.09\ \text{k/in}^2$$

$$f_v = \frac{60^{\text{k}}}{8(.44\,\text{in}^2)} = 17.05\ \text{k/in}^2$$

c) Do for case when bolts not tightened properly, where machine bolts used, or rivets used.

In this case can not assume prestressing Ls to column.

In particular, use 3/4" A502 Grade 1 rivets



(assume between 1st & 2nd bolt)

$\underline{\Sigma A_y = 0}$

$$\underbrace{7\,in\,(x)}_{\text{area}}\ \underbrace{\frac{x}{2}}_{\text{e.a}} = 2(.44\,in^2)(4.5-x) + 2(.44)(7.5-x) + 2(.44)(10.5-x)$$

$$3.5x^2 = .88(4.5+7.5+10.5) - .88(3x)$$

$$3.5x^2 = 19.8 - 2.64x$$

$$3.5x^2 + 2.64x = 19.8$$

$$x^2 + .754x = 5.657$$

$$x^2 + .754x + .142 = 5.657 + .142 = 5.799$$

$$(x + .377)^2 = 5.799$$

$$x = -.377 \pm 2.408$$

$x = 2.031$ assumption OK

$$I = \frac{1}{12}(7_{in})(2.031_{in})^3 = 4.887$$

$$7_{in}(2.031)\left(\frac{2.031}{2}\right)^2 = 29.322$$

$$2(.44)(4.5-2.031)^2 = 5.364$$

$$2(.44)(7.5-2.031)^2 = 26.321$$

$$2(.44)(10.5-2.031)^2 = 63.117$$

$$129.011 \text{ in}^4$$

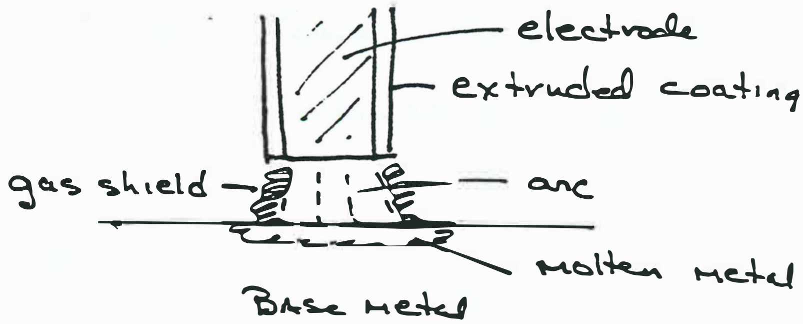
$$f_t = \frac{180 \text{ k-in } (10.5-2.031_{in})}{129.011 \text{ in}^4} = 11.816 \frac{k}{in^2}$$

etc.

Welds

How welding performed (16.1-116)

(SMAW) Shielded Metal Arc Welding



Most commonly used

this is our default weld

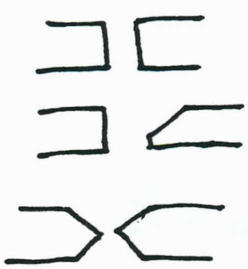
(SAW) Submerged Arc Process

1st put down fusible powered flux

Then weld with bare electrode

Types of welds

Butt or groove



No design calculations needed

Fillet

Plug

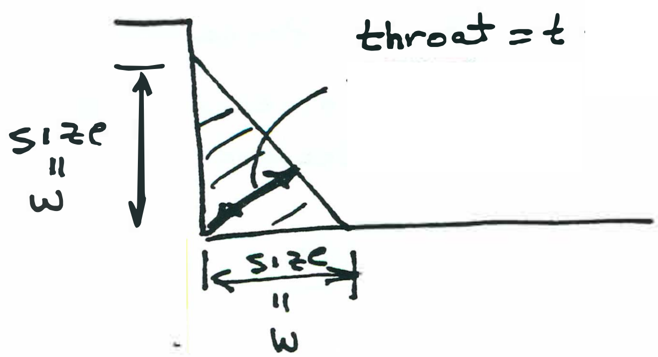
Slot

Electrode material used is always stronger than base metal – strength of weld determined by strength of electrode

E70 electrode used with A36

$\underbrace{70}$ St of electrode material

In welds critical section through throat of weld



(16.1-118)

For SMAW: $t = .707w$

For SAW: $t = w$ for $w \le 3/8"$

$.707w + .11$ for $w > 3/8$

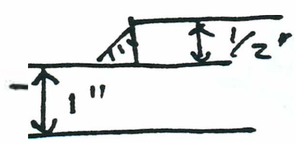
Limitations on size

Minimum size

Weld must be big enough to get metal hot (16.1-118)

Table J2.4

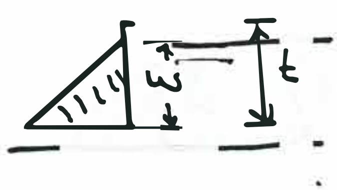
ex



thicker part = 1", $\therefore$ $w_{MIN} = 5/16"$

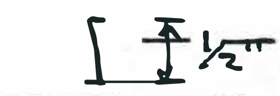
Maximum weld size

(16.1-118)



$0 \leq t \leq 1/4$ $\quad w_{MAX} = t$

$t > \frac{1}{4}$ $\quad w_{MAX} = t - \frac{1}{16}"$

ex 

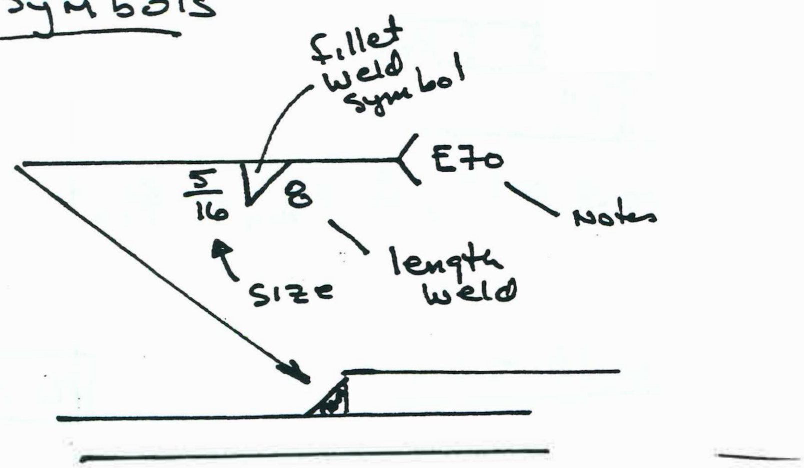
$w_{MAX} = \frac{1}{2} - \frac{1}{16} = \frac{7}{16}"$

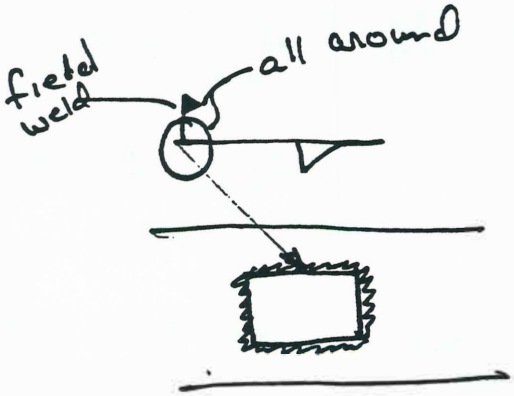
Strength weld material = $.6F_u$ of weld material

(we are using shear strength of weld MAT)

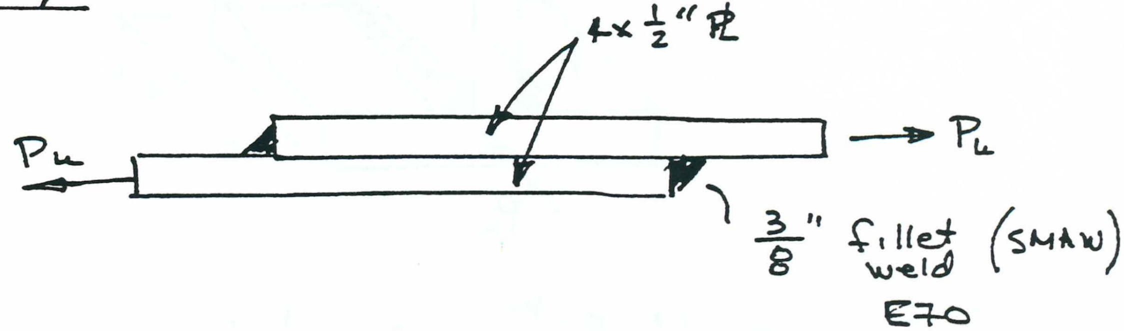
$$\text{usable Strength of weld} = \underset{.75}{\phi}\ \underbrace{\left(\begin{array}{c}\text{Strength}\\ \text{of}\\ \text{weld Mat}\end{array}\right)}_{.6F_u}\left(\begin{array}{c}\text{Throat}\\ \text{area}\end{array}\right)$$

Weld Symbols





Example



effective throat $= .707 \left(\frac{3}{8}''\right)$

$$P_u\Big|_{\substack{\text{weld}\\\text{controls}}} = \phi\,(\underset{\text{Mat}}{\text{St weld}})(\text{area weld throat})$$

$$= .75\underbrace{(.6)\left(70\,\tfrac{k}{in^2}\right)}_{.6\,F_{u\,\substack{\text{weld}\\\text{Mat}}}}(.707)\left(\tfrac{3}{8}\,in\right)\underbrace{(8'')}_{\substack{\text{length}\\\text{weld}}}$$

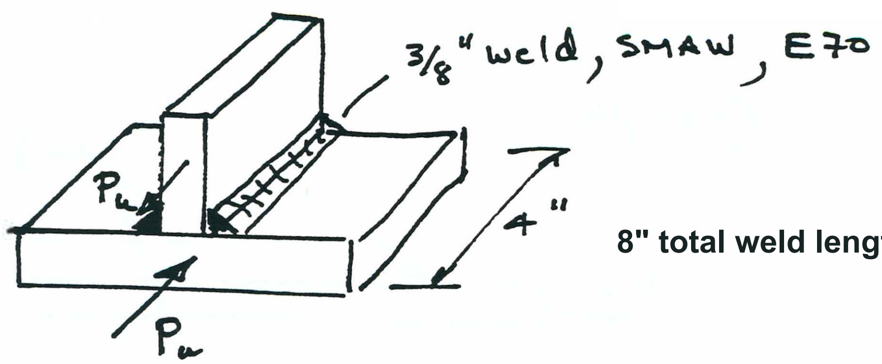
$$= 66.81^{k}$$

$$P_u\Big|_{\text{plate}} = .9\left(36\,\tfrac{k}{in^2}\right)(4\,in)\left(\tfrac{1}{2}\,in\right) = 64.8^{k} \leftarrow \text{controls}$$

on this problem

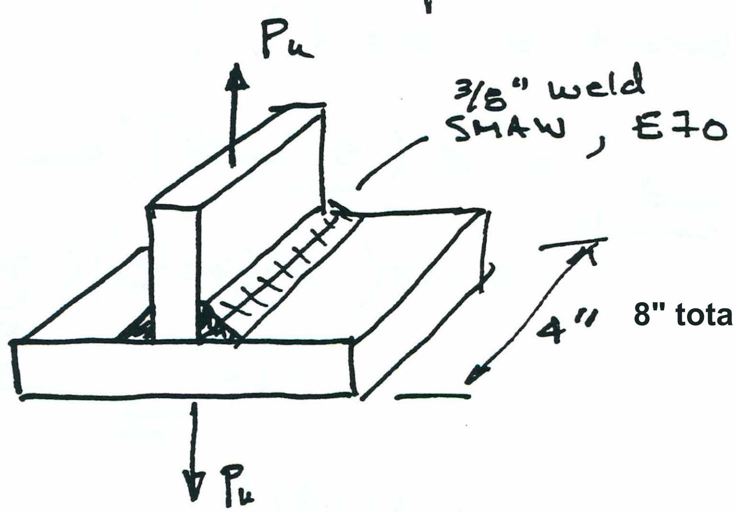
$w_{MAX} = \frac{1}{2}'' - \frac{1}{16}'' = \frac{7}{16}''$ **(16.1-119, 2b)**

$w_{MIN} =$ **3/16"** **(16.1-119, 2a; Table J2.4)**



8" total weld length

$P_u|_{weld} = 66.81^k$ as before

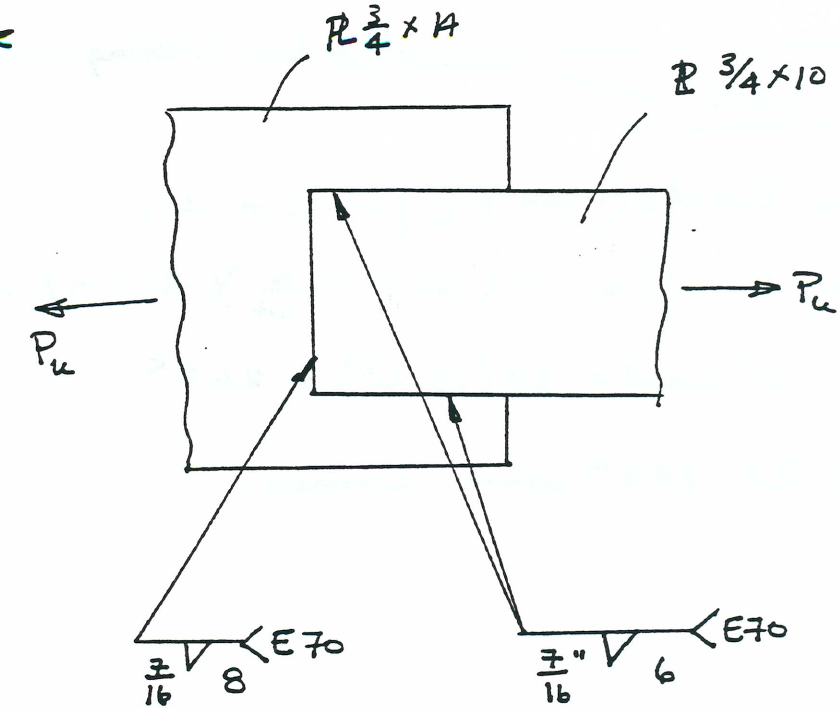


8" total weld length

$P_u|_{weld} = 66.81^k$

Dirrection of load doesn't effect calculations

Example



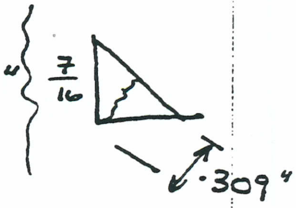
Examine stress in PL

$P_u = .9(36 \frac{k}{in^2})(\frac{3}{4}'')(10 in) = 243^k$

Shielded Metal Arc Welding

Examine weld. (SMAW)

- Effective throat $= .707(\frac{7}{16}'') = .309''$



Capacity 1 in of weld $= \underbrace{\phi}_{.75}(\underbrace{.6F_u}_{\text{of weld}})$ Throat area

$= .75(.6)(70\frac{k}{in^2})(.309 in)(1 in) = 9.73^k$

Capacity of $6+6+8=20''$ weld $= 20(9.73^k) = 194.65^k$ controls

If welds SAW — Submerged Arc Welding

Effective throat $= (.707)\left(\frac{7}{16}\right) + .11 = .419$

Capacity 1" weld $= .75\left(.60 \times 70 \frac{k}{in^2}\right)(.419\ in)(1\ in) = 13.2^k$

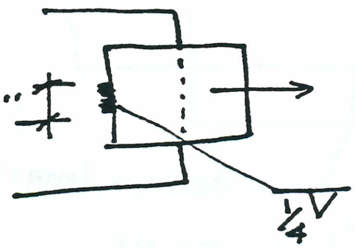
Capacity 20" weld $= 20(13.2^k) = 264^k$

Strength $R = 243^k$ ← Controls.

Additional LRFD requirements

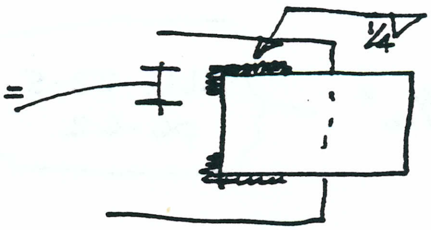
1. Min length weld = 4 * leg size = $4w$

$l_{min} = 4(\frac{1}{4}) = 1''$

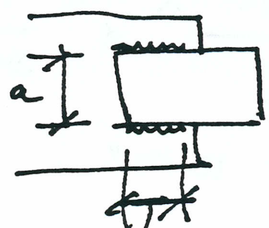


2. End returns should be used. Min length of returns = $2w$

Min return = $2(\frac{1}{4}) = \frac{1}{2}''$

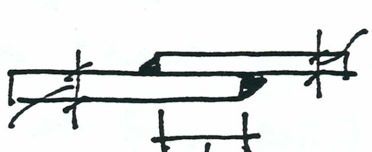


3. min length longitudinal weld = dist between welds



Min $l = a$

4

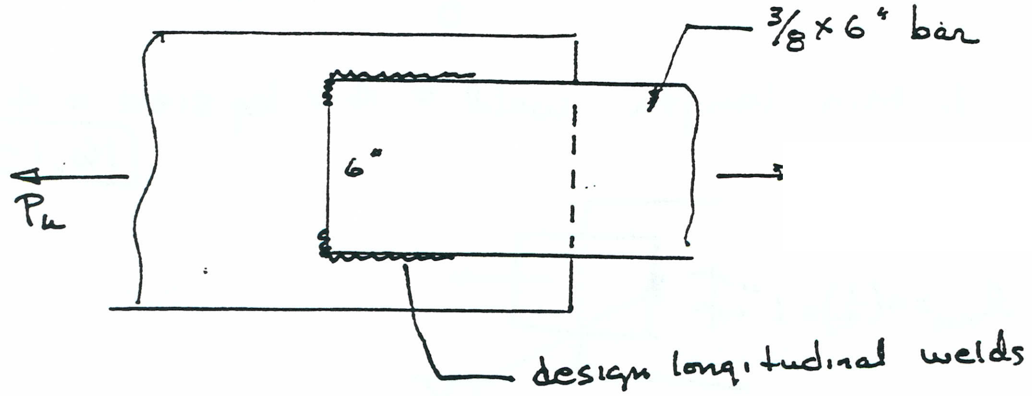


t_1

$t_2 > t_1$

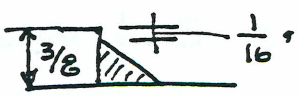
$5t_1$ = min lap

Design Example – Design for full capacity bar
SMAW, E70



$P_u\big|_{bar} = .9\left(36\,\frac{k}{in^2}\right)\left(\frac{3}{8}\,in\right)(6\,in) = 72.9^k$

Maximum weld size $= \frac{3}{8} - \frac{1}{16} = \frac{5}{16}"$



Minimum weld size $= 3/16"$

use $5/16"$ weld

Effective throat $= .707\left(\frac{5}{16}"\right) = .221"$

Capacity 1" weld $= \underbrace{(.221\ in)(1\,in)}_{\text{throat area}}\underbrace{(.75)}_{\phi}\underbrace{(.6)\left(70\,\frac{k}{in^2}\right)}_{\text{Design st.}} = 6.96\,k$

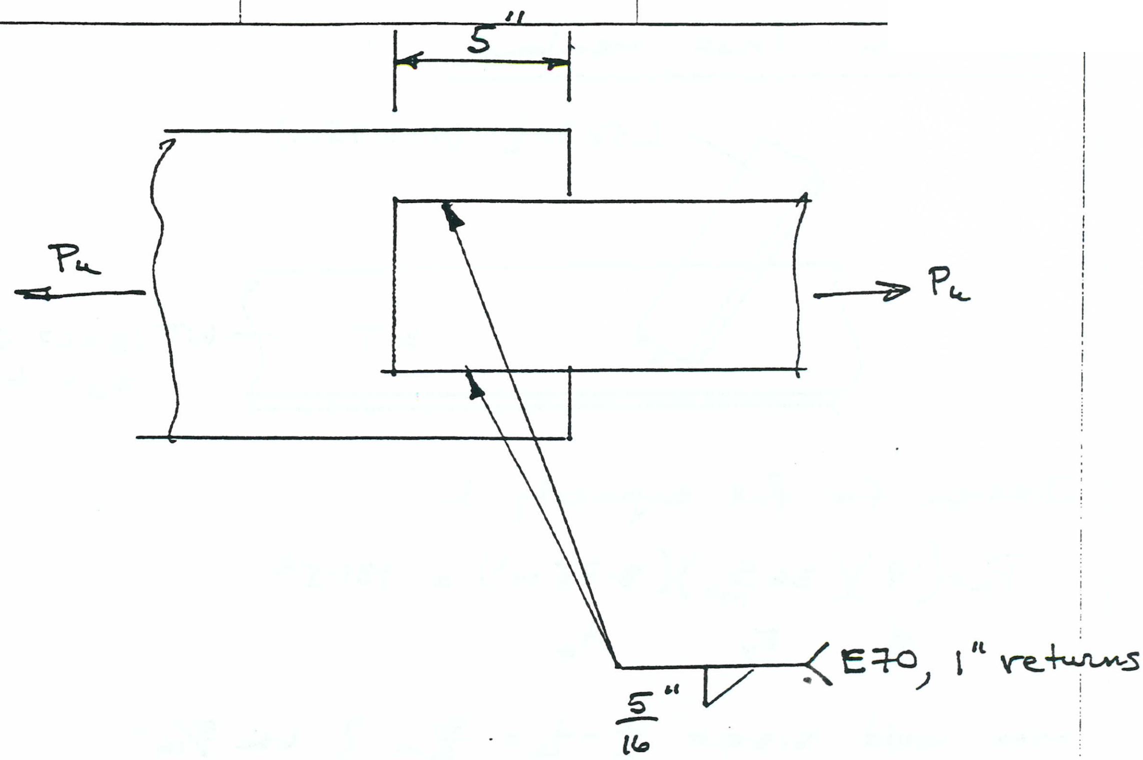
length weld req'd $= \frac{72.9\,k}{6.96\,k} = 10.47"$

Use end returns not less than $2 \times \frac{5}{16}" = \frac{10}{16}"$ (use 1")

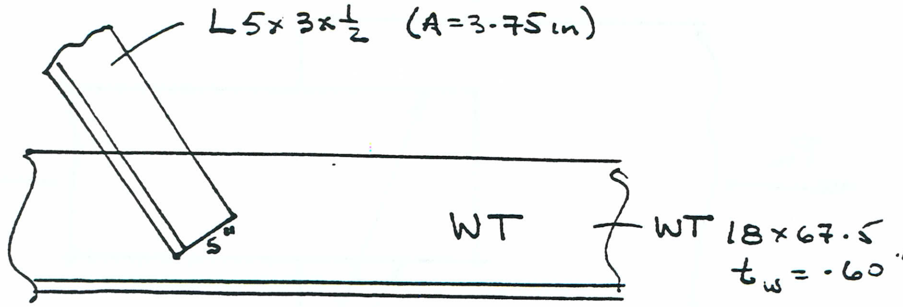
Leaving $10.47 - 2 = 8.47$ in or $4.24"$ ea side

Must have length of fillet weld ea side at least = dist between welds (in this case 6")

∴ use $6" - 1"$ returns $= 5"$ ea. side


5"
Pu
Pu
5/16"
E70, 1" returns

Welds on Truss Members



Design for full capacity L

$$P_u = (.9)(36 \tfrac{k}{in^2})(3.75\ in^2) = 121.5^k$$

ϕ F_y A_G

MAX weld size $= \frac{1}{2} - \frac{1}{16} = \frac{7}{16} in$ } use 5/16"

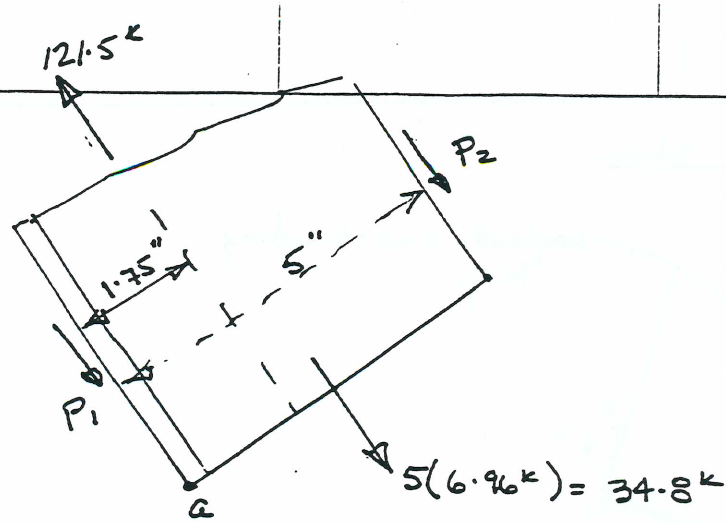
MIN weld size = 1/4"

Effect throat of weld $= .707(\frac{5}{16}") = .221 in$

Capacity 1" weld $= \underbrace{.221 in (1 in)}_{area} \underbrace{(.6)(70 \tfrac{k}{in^2})}_{st\ weld} \underbrace{(.75)}_{\phi} = 6.96k$

length weld req'd $= \frac{121.5^k}{6.96^k} = 17.46 in$

Balance welds – would want to do if we have oscillating loads – Otherwise no restriction where you put weld



$\underline{\Sigma M_a = 0}$

$121.5^k(1.75'')$ $P_2(5'')$

$34.8^k(2.5\text{ in})$

$P_2 = 25.12^k$

$\underline{\Sigma F = 0}$

121.5^k 34.8^k

$P_2 = 25.12^k$

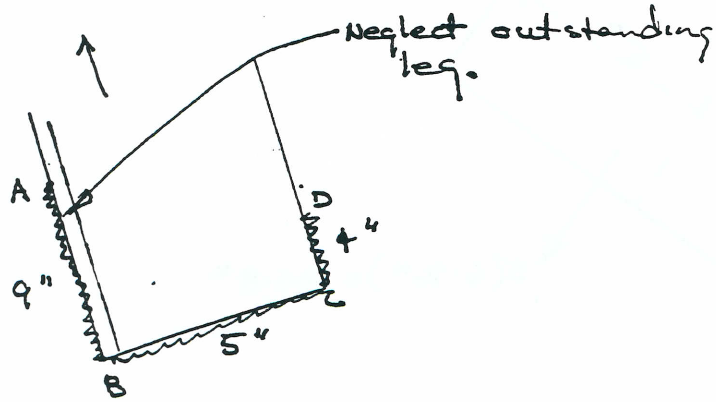
P_1

$P_1 = 61.58^k$

$L_1 = \dfrac{61.58^k}{6.96^k} = 8.85\text{ in}$ use 9"

$L_2 = \dfrac{25.12^k}{6.96^k} = 3.61\text{ in}$ use 4"

check block shear



Could have shear failure along AB & CD & tension failure along BC

check tension yielding & shear fracture

$$P_{bs} = .75\left[\left(36\tfrac{k}{in^2}\right)(5\,in)\left(\tfrac{1}{2}''\right) + .6\left(58\tfrac{k}{in^2}\right)(13\,in)\left(\tfrac{1}{2}\,in\right)\right]$$

$\uparrow$
ϕ

no holes for block shear

$$= 237.2^{k} > 121.5^{k} \text{ OK}$$

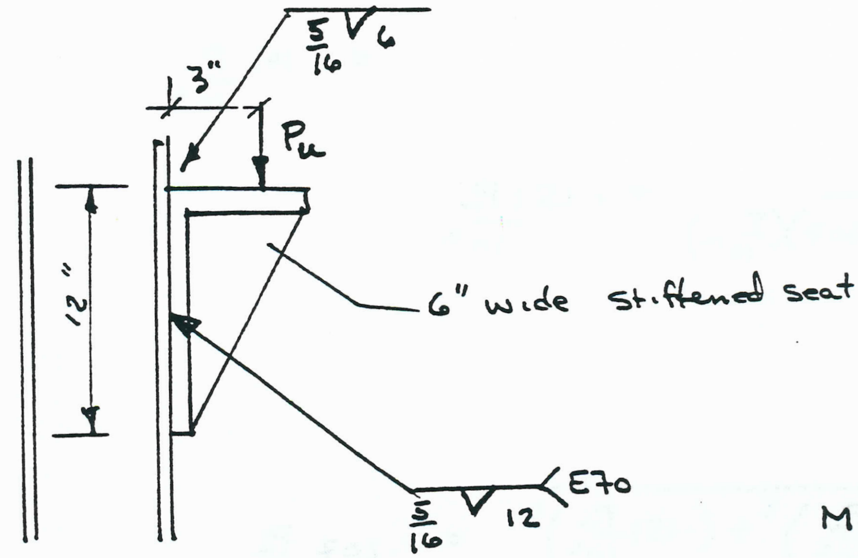
check tension fracture & shear yielding

$$P_{bs} = .75\left[\left(58\tfrac{k}{in^2}\right)(5\,in)\left(\tfrac{1}{2}\,in\right) + .6\left(36\tfrac{k}{in^2}\right)(13\,in)(.5\,in)\right]$$

$$= 214.1\ k$$

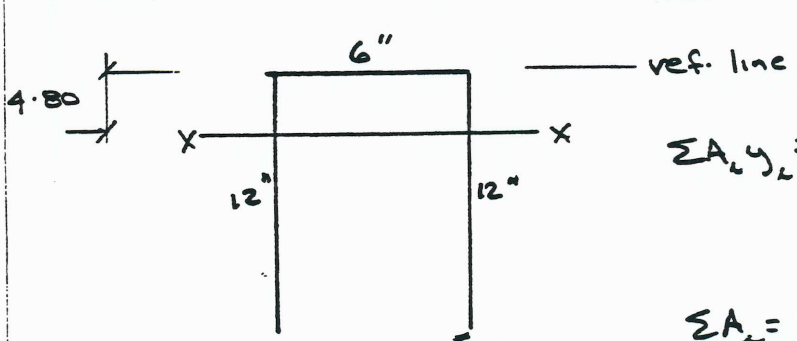
42-381 50 SHEETS 5 SQUARE
42-382 100 SHEETS 5 SQUARE
42-389 200 SHEETS 5 SQUARE
NATIONAL

What load can the following connection carry



$M = P(3\text{ in})$

Find centroid weld



$$\Sigma A_i y_i = \underbrace{(12'')(.707)\left(\tfrac{5}{16}\right)}_{\text{area}}\underbrace{(6'')}_{\text{dist to. centroid}}\underbrace{(2)}_{\text{two 12" welds}}$$

$$\Sigma A_i = (12+12+6)\text{in}\,(.707)\left(\tfrac{5}{16}\text{in}\right)$$

$$\bar{y} = \frac{12(6)(2)\cancel{(.707)}\cancel{\left(\tfrac{5}{16}\right)}}{30\,\cancel{(.707)}\cancel{\left(\tfrac{5}{16}\right)}} = 4.800$$

Get I_{xx} weld

$$I_{xx} = \tfrac{1}{12}(.707)\left(\tfrac{5}{16}''\right)(12\text{in})^3(2) + (12\text{in})(.707)\left(\tfrac{5}{16}\text{in}\right)(6-4.80)^2\,\text{in}^2\,(2)$$
$$+\ 6\text{in}(.707)\left(\tfrac{5}{16}\text{in}\right)(4.80)^2\,\text{in}^2$$

$$= 31.815(2) + 3.818(2) + 30.542 = 101.81\text{ in}^4$$

MAX tension stress top connection $= \dfrac{P(3\cancel{in})(4.80\cancel{in})}{101.81 \quad in^{2}}$

$= .141 \dfrac{P}{in^2}$

$$f_v = \frac{P_u}{\underbrace{30in}_{\text{total length weld}}(.707)(\frac{5}{16}in)} = .151 \frac{P_u}{in^2}$$

$$\sigma_r = \sqrt{\left(.141 \frac{P_u}{in^2}\right)^2 + \left(.151 \frac{P_u}{in^2}\right)^2} = .207 \frac{P_u}{in^2}$$

usable weld st $= \phi(.6\, F_{u\ weld\ MAT}) = .75(.6)(70\frac{k}{in^2}) = 31.5\frac{k}{in^2}$

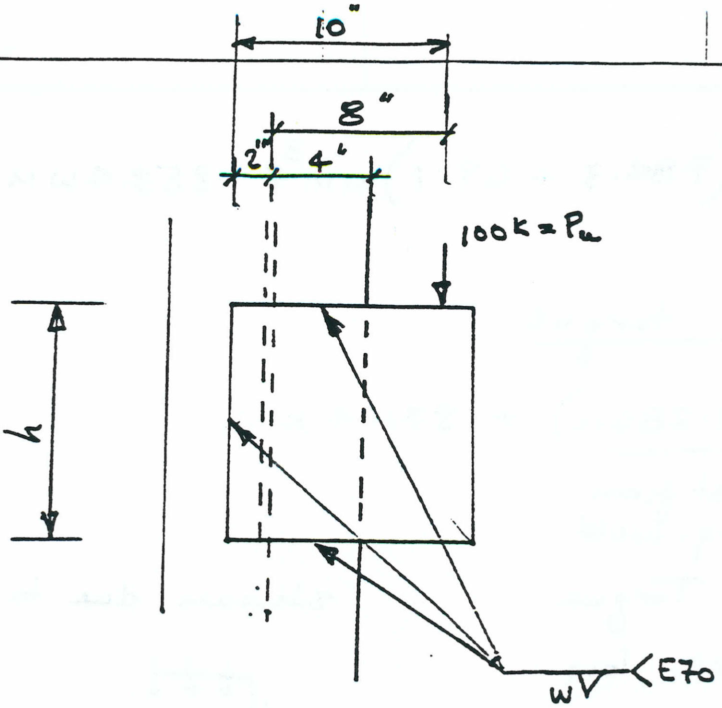
$$31.5\frac{k}{\cancel{in^2}} = .207 \frac{P_u}{\cancel{in^2}}$$

$$P_u = 152.17^k$$

Note MAX comp stress bot weld $= \dfrac{P_u(3in)(12-4.80)in}{101.81 \quad in^4}$

$= .212 \dfrac{P_u}{in^2}$

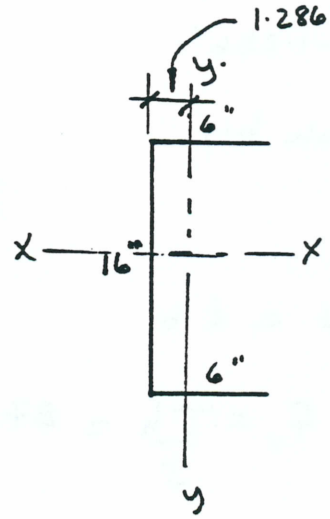
However at bottom seat bears against column & carries some of the compression, thus assume bottom not critical.



Select weld size & dimensions

Guess h = 16 in

Find c.g. of weld



$A_L x_L = 6in(.707w)(3in)(2)$

$A_L = (6+6+16in)(.707w)$

$$\bar{x} = \frac{6(3)(2)(\cancel{.707w})\, in^2}{28\, \cancel{in}\,(\cancel{.707w})}$$

$\bar{x} = 1.286''$

Get I_{xx}, I_{yy}, J

$$I_{xx} = \underbrace{\tfrac{1}{12}(.707w)(16in)^3}_{\frac{1}{12}bh^3} + \underbrace{6in(.707w)}_{area}\underbrace{(8in)^2}_{dist^2}(2) =$$

top & bottom

$$= (341.3 + 768)(.707w)\, in^3 = 784.3\, w\, in^3$$

$$I_{yy} = \Big[\underbrace{\tfrac{1}{12}(.707w)(6in)^3}_{\frac{1}{12}bh^3} + \underbrace{(.707w)(6in)}_{area}\underbrace{(3'' - 1.286'')^2}_{dist^2}\Big](2)$$

top & bottom

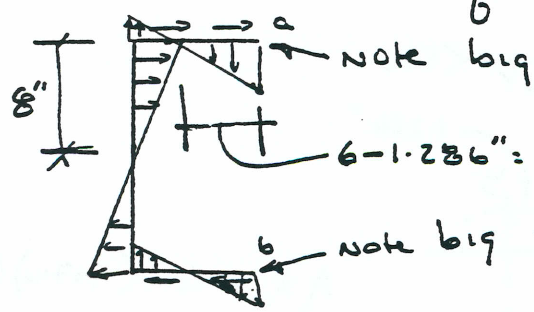
$$+ \underbrace{(16in)(.707w)}_{area}\underbrace{(1.286in)^2}_{dist^2} = 69.08\, in^3\, w$$

$$J = I_{xx} + I_{yy} = (784.3 + 69.1)\,w\,in^3 = 853.4\,w\,in^3$$

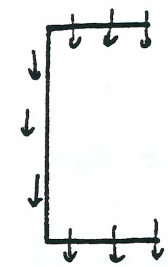
get stresses due to torque

$$T = 100^k(\underbrace{10in - 1.286in}_{\text{dist from c.g. weld}}) = 871.4\ k\text{-}in$$

stresses due to Torque



stresses due to shear



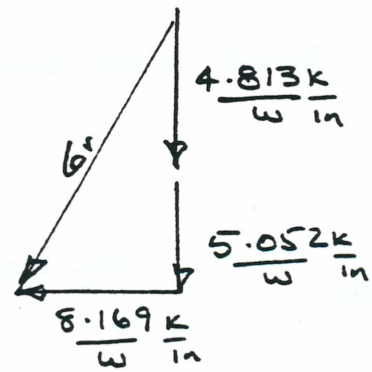
$\therefore$ worst stress at a & b

Due to torque $$f_v = \frac{Th}{J} = \frac{871.4\ k\text{-}in(4.714\,in)}{853.4\ in^3 w} = 4.813\ \frac{k}{in\ w}$$

$$f_h = \frac{Tv}{J} = \frac{871.4\ k\text{-}in\,(8\,in)}{853.4\ in^3 w} = 8.169\ \frac{k}{in\ w}$$

Due to shear $$\sigma_v = \frac{100^k}{(6+6+16\,in)(.707w)} = 5.052\ \frac{k}{in\ w}$$

Add stresses vectorially



$$\sigma_r = \sqrt{\left(\frac{5.052}{w} + \frac{4.813}{w}\right)^2 + \left(\frac{8.169}{w}\right)^2} = \frac{12.81}{w}\frac{k}{in}$$

usable st. weld MAT $= \phi(.6 F_{u\, weld\, MAT}) = .75(.6)(70 \frac{k}{in^2}) = 31.5 \frac{k}{in^2}$

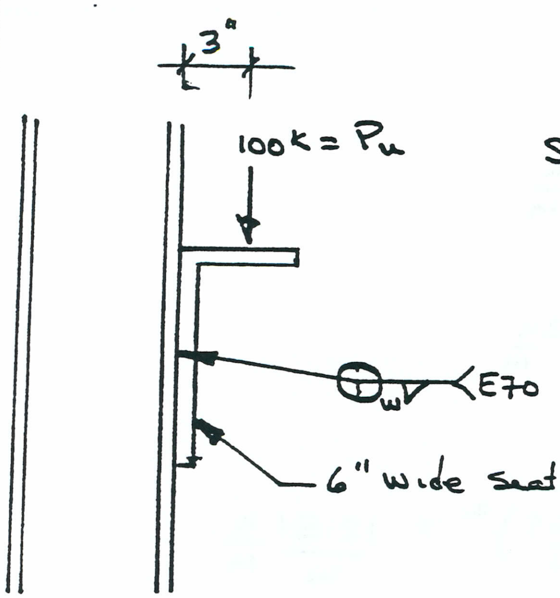
Set weld stress to usable strength

$$\frac{12.81}{w}\frac{k}{\cancel{in}} = 31.5 \frac{k}{in^{\cancel{2}}}$$

$$w = \frac{12.81\,in}{31.5} = .41 \quad \text{or } 7/16'' \text{ weld}$$

7/16" weld would require more than 1 pass of welder try redesigning connection so that one can use 5/16" weld which can be put down in one pass

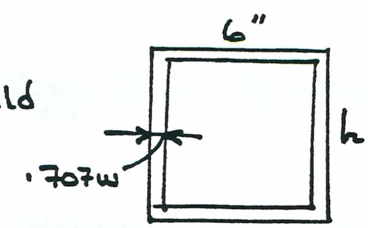
⋮



Select seat size & weld size

$$M_u = 100^k(3\,in) = 300\text{ k-in}$$

Examine I weld



$$I = \tfrac{1}{12}(.707w)h^3(2) + 6\,in(.707w)\left(\tfrac{h}{2}\right)^2(2)$$

$$I = .1178\,wh^3 + 2.1212\,wh^2(in)$$

$$\sigma_t = \frac{300\text{ k-in}}{I}\left(\frac{h}{2}\right) = \frac{150\text{ k-in}\,(\cancel{h})}{.1178\,w\,h^{\cancel{3}\,2} + 2.1212\,w\,h^{\cancel{2}}(in)}$$

$$\sigma_v = \frac{100^k}{(2h + 12\,in)(.707w)}$$

$$\sigma_r = \sqrt{\sigma_t^2 + \sigma_v^2}$$

for $h = 10\,in$ (guess)

$$\sigma_t = \frac{150\,k\text{-}in}{.1178\,w\,(10)^2 in^2 + 2.1212\,w\,(10)\,in^2} = \frac{4.5466}{w}\ \frac{k}{in}$$

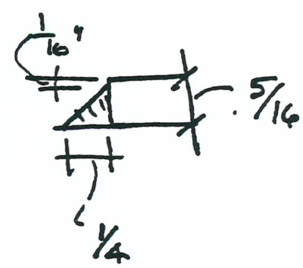
$$\sigma_v = \frac{100^k}{[2(10in) + 12in](.707)w} = \frac{4.4201}{w}\ \frac{k}{in}$$

$$\sigma_r = \sqrt{\left(\frac{4.5466}{w}\right)^2 + \left(\frac{4.4201}{w}\right)^2} = \frac{6.3410}{w}\ \frac{k}{in}$$

$$\sigma_r = \frac{6.3410}{w}\ \frac{k}{in} = \underbrace{.75(.6)(70\tfrac{k}{in^2})}_{\text{usuable St}} = 31.5\frac{k}{in^2}$$

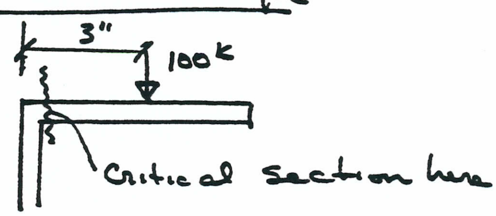
$$w = \frac{6.341}{31.5}\,in = .20 \quad \text{use } \frac{1}{4}''$$

Min size angle $= \frac{1}{4} + \frac{1}{16}'' = \frac{5}{16}$



try using 3/8" thick L

Check stress in angle



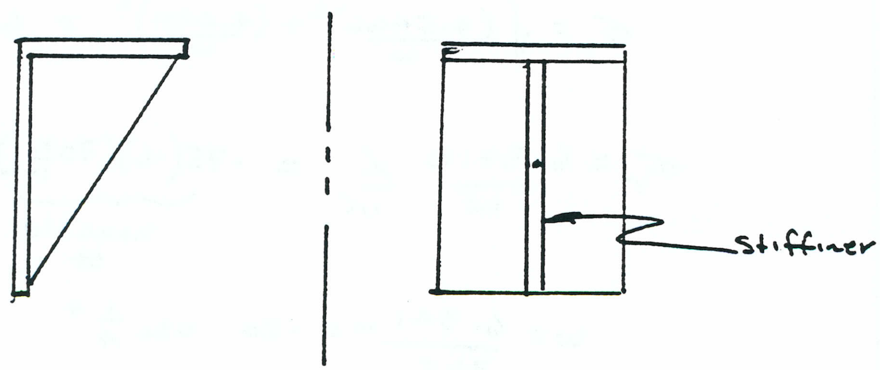
$$M_{u\ \text{at critical section}} = 100^k\left(3 - \frac{3}{8}''\right) = 262.5\,k\text{-}in$$

42 381 50 SHEETS 5 SQUARE
42 382 100 SHEETS 5 SQUARE
42 389 200 SHEETS 5 SQUARE
NATIONAL

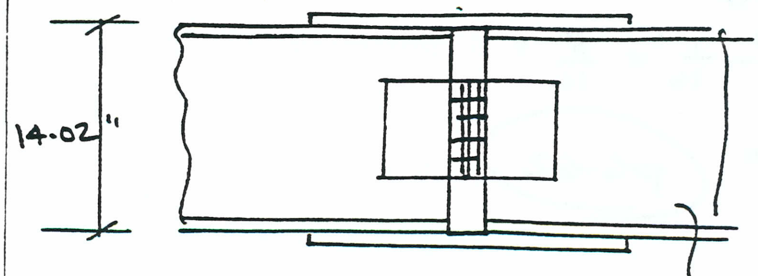
$$\phi M_p\Big|_{seat} = .9\left(36\frac{k}{in^2}\right)(6\,in)\frac{\left(\frac{3}{8}\,in\right)^2}{4} \doteq 6.83\ k\text{-}in \ll 262.5\ k\text{-}in$$

$$M_p\ rect = F_y\frac{bd^2}{4}$$

use stiffened seat (use stiffener)



We will stop here & not design stiffener



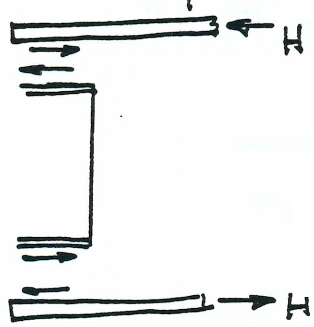
W14×90

$b_f = 14.52 \text{ in}$

$t_f = .710 \text{ in}$

Assume shear connection OK

design ℞ top & bottom so connection can carry Moment of 100 k-ft = M_u



$$H(14.02\text{ in}) = 100\text{ k-ft}\left(\frac{12\text{ in}}{\text{ft}}\right)$$

$$H = 85.59^k$$

Get ℞

$$\phi F_y A_{℞} = 85.59^k$$

$$A_{℞} = \frac{85.59^k}{.9(36 \frac{k}{in^2})} = 2.64 \text{ in}^2$$

for bottom & top ℞ . use 8 in ℞

$$(8\text{ in})\, t_p = 2.64 \text{ in}^2$$

$$t_p = .33 \text{ in} \quad \text{use } 3/8\text{" ℞}$$

Find weld required

$$w_{MAX} = \frac{3}{8}'' - \frac{1}{16}'' = \frac{5}{16}''$$

$$w_{Min} = \frac{1}{4}''$$ p6-62

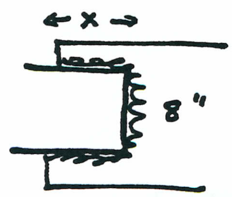
use $\frac{5}{16}''$ weld

st of 1" weld = $\phi(.6F_u)$(throat area)

$$= .75(.6)(70\tfrac{k}{in^2})(.707)(\tfrac{5}{16}\text{in})(1\text{in}) = 6.96\text{k}$$

length weld req'd = $\dfrac{85.59^k}{6.96^k} = 12.3''$

use welds thus

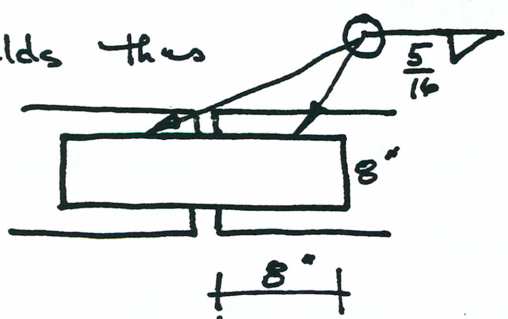


$$2x + 8 = 12.3$$
$$x = 2.15''$$

However code limits min length of longitudinal welds to dist between welds

$\therefore$ use $x = 8''$

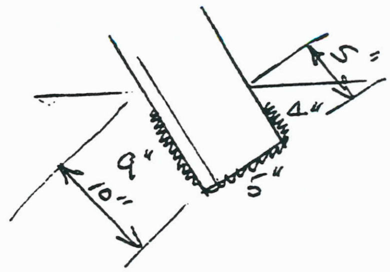
$\therefore$ Make welds thus



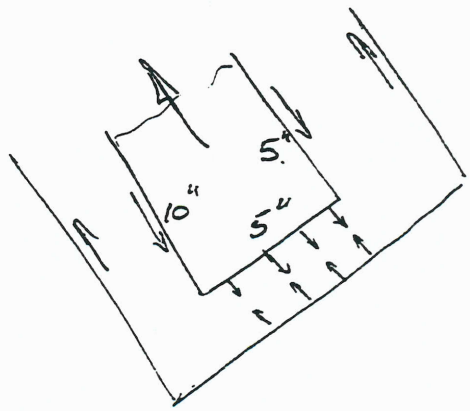
top view

Check Web WT

$t_w = .60 in$



Freebody



check tension yield & shear fracture

$$P_{bs} = .75 \left[36\tfrac{k}{in^2}(5in)(.6") + .6(58\tfrac{k}{in^2})(15in)(.6in) \right]$$

$$= 315.8 \text{ k}$$

check tension fracture & shear yield

$$P_{bs} = .75 \left[58\tfrac{k}{in^2})(5)(.6) + .6(36\tfrac{k}{in^2})(15)(.6) \right]$$

$$= 276.3^{k}$$

$$P_{bs} = 315.8^{k}$$

load from each angle

load coming in $= 121.5^{k}(2) = 243^{k} < 315.8$ OK