-1104140

Steel Structure

1	30 Aug	13	1 Nov	25	7 Dec
2	31 Aug	14	2 Nov	26	10 Dec
3	2 Sep	15	4 Nov	27	13 Dec
4	6 Sep	16	8 Nov		
5	7 Sep	17	9 Nov		
6	9 Sep	18	11 Nov		
7	14 Sep	19	15 Nov		
8	4 Oct	20	16 Nov		
9	5 Oct	21	25 Nov		
10	7 Oct	22	29 Nov		
11	11 Oct	23	30 Nov		
12	18 Oct	24	6 Dec		

CE 319 (3.00 Cr. Hr.)

Design of Steel Structures

Dr. Khan Mahmud Amanat

Main Text Book:

1) Steel Structures: Design & Behaviour 5th Edition (2009)

Salmon/Johnson/Malhas
(Pearson-Prentice Hall)

We refer to AISC (Americon Institute ~~for~~ of Steel Constrution)) for Steel construction.

* BNBC follow করে-2005 এর-edition

Design Philosophy:

Resistance/Strength/Capacity ≥ Effect of applied Loads

Limit State

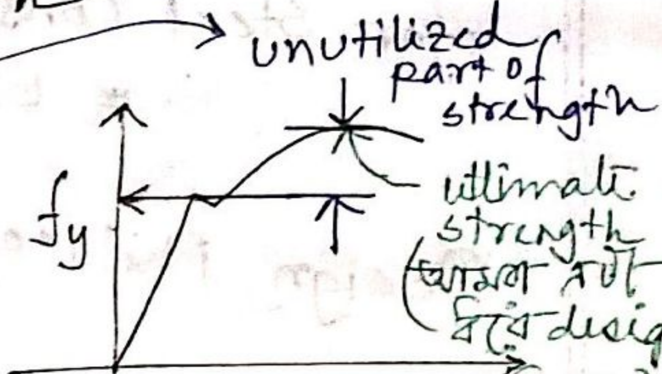
→ failure limit

এমন একটা condition যেটা আমাদের কাছে unacceptable

deflection এর limit, span length এর 360 ভাগের 1 ভাগ ধরা হয়,

- Strength Limit states
- serviceability " "
- special " "

Reserve Strength : unknown additional strength → unutilized part of strength



ASD

* load কে load এর মত রাখা হয়,
* capacity কে safety factor দিয়ে ভাগ করে allowable capacity বের করা হয়,

LRFD

* load কে factor করা হয়।

cross section এর capacity তার shape, size, material strength থেকে আসবে,

always <1 → $\phi R_n \geq R_u \; (= \sum \gamma_i Q_i)$

R_n ↑ nominal strength

γ_i → 1.2, 1.6 এগুলো হচ্ছে γ

Test থেকে যেটা পাওয়া যাচ্ছে সেটা হইল nominal strength, এটাকে কমিয়ে হিসাব করার জন্য ϕ দিয়ে গুন করা হবে।

* Advantaages & Disadvantages of Steel structure (পরীক্ষায় আসবে)

→ without fire protection steel structure design isn't complete

→ temperature কমে গেলে steel brittle হয়ে যায়।

Material

Iron ore → Pig Iron — কোনো কাজেই ব্যবহার করা যায় না,
(Ferrite)

Steel is iron in usable form

Pig Iron → Wrought Iron

Pig Iron → Cast Iron

* Steel এর composition পড়তে হবে না

Steel shapes

cold form shapes (eg. ঢেউটিন) — শুধু steel এর sheet নিয়ে normal temperature এ বাঁকা করা হয়।

hot rolled shapes
→ red-hot করে semi-solid condition এ নিয়ে করা হয়।

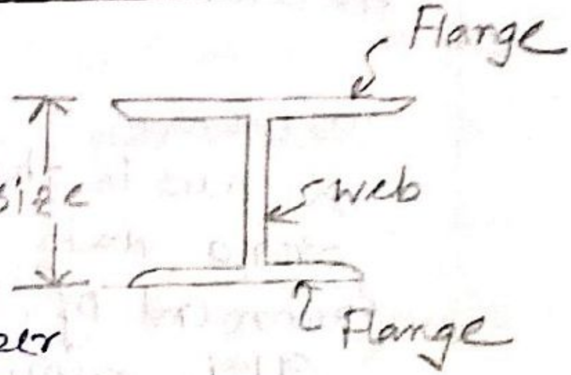
Billet → structural steel এর solid টুকরা।

AISC Steel Construction Manual

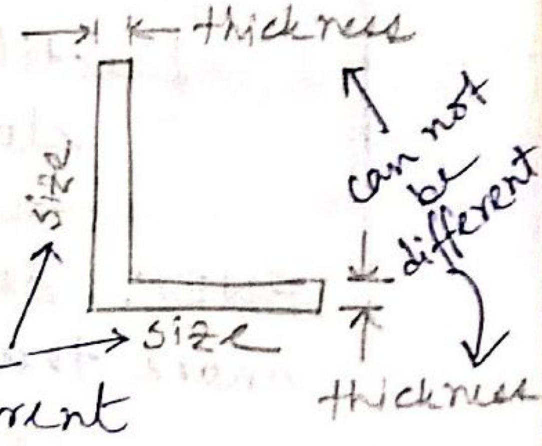
W 14 × 132

wide flange shape (W shape)

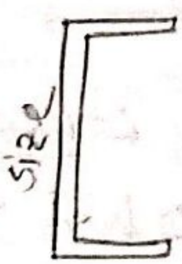
weight, lbs per linear ft.



(angle shape) L 8 × 8 × 3/4

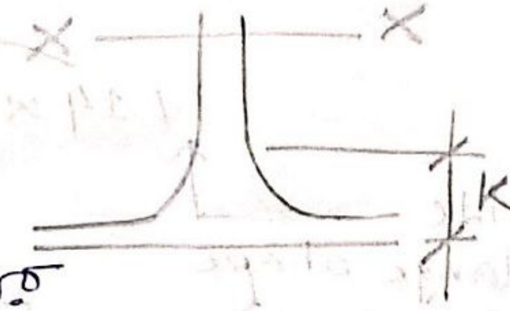


(channel shape) C 9 × 15



* অঙ্ক করার জন্য depth টা decimal fraction-এ নিতে হবে,

X-axis is the strong axis
(moment of inertia বেশি, moment নেওয়ার capacity বেশি)



Ductility → failure এর আগে elongated হওয়ার capacity

BNBC doesn't allow designing with more than 50 grade steel.

500 W
↳ weldable
(সহজে welding করা যায়)

* prestressing এর জন্য 250 ksi steel use করা হয়,

f_y → yield strength

f_u → ultimate strength

For earthquake resistant design,

$$\frac{f_u}{f_y} \geq 1.25$$

few thousand cycles at least

* <u>fatigue</u> is a design parameter for steel structures (specifically, railway bridges) only. RC design এ এটা কোনো parameter না,

Temperature ↑, yield strength ↓

Steel এর toughness ও temperature এর সাথে related.

<u>Elements of a steel building</u>

column to column beams → girder (vertical load নেয় এবং wind, earthquake load ও নেয়)

Tension Members-1

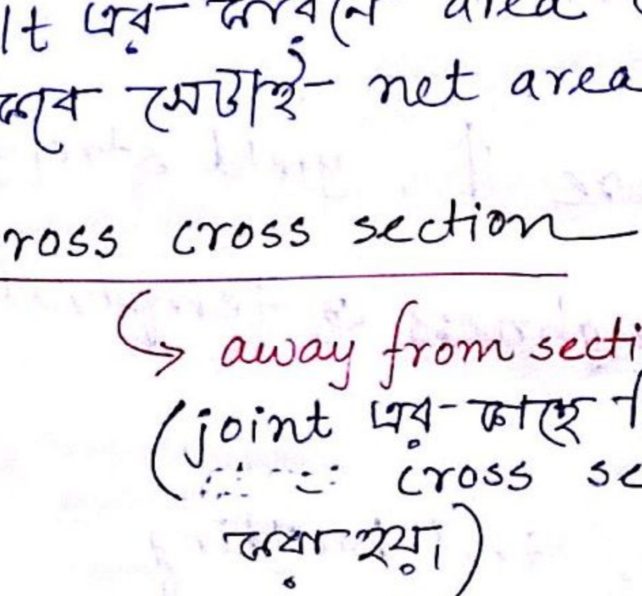
Suspension bridge এর hanger, cable tension member.

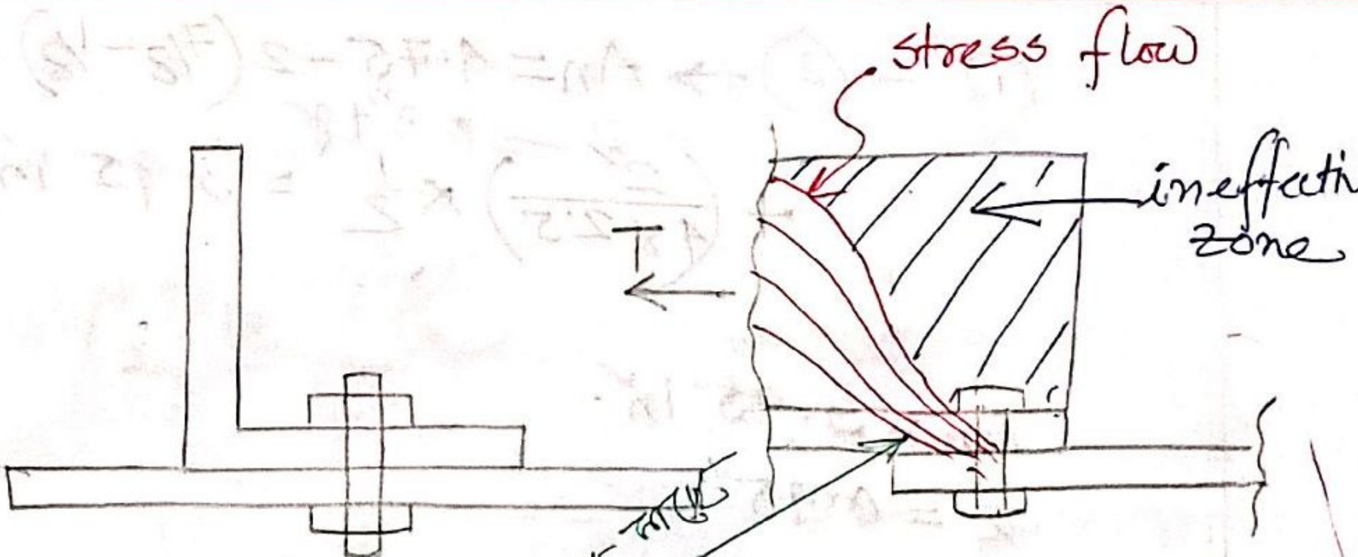
flexural member → x-section এমন হতে হবে যাতে I বেশী হয়।

Bolt এর জন্যে area কমে গিয়ে যা থাকবে সেটাই net area (A_n)

gross cross section

↳ away from section
(joint এর কাছে গিয়ে effective cross section হিসেবে ধরা হয়।)





Connection এর কাছে stress propagate হচ্ছে bolt দিয়ে।

Connection এর কাছে cross section কমে যাচ্ছে (ফলে) net area র জন্য reduction factor use করা হয়।

$T_n \rightarrow$ Nominal Strength

Grade 50 steel

$F_y = 50$ ksi

$F_u = 65$ ksi

$A_g = 4.75\ in^2$

Find net area

Along ①-① $\longrightarrow A_n = 4.75 - (7/8 + 1/8)\ 1/2$

$= 4.25\ in^2$

① - ② → $A_n = 4.75 - 2\left(\frac{7}{8} - \frac{1}{8}\right) \times \frac{1}{2} + \left(\frac{2^2}{4 \times 2.5}\right) \times \frac{1}{2} = 3.95 \text{ in}^2$

$\xrightarrow{s^2/4g}$

$A_n = 3.95 \text{ in}^2$

$\bar{x} = 0.95'$

$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.987}{14} = 0.93$

$A_e = U A_n = 0.93 \times 3.95 = 3.674 \text{ in}^2$

Yield on gross area, $T_n = F_y A_g$

$= 237.5$

ASD capacity $= \frac{T_n}{\Omega} = \frac{237.5}{1.67} = 142.2$

Net area fracture, $T_n = F_u A_e$

$= \cancel{0.}65 \times 3.674$

$= 238.81$

ASD capacity $= \frac{T_n}{\Omega} = \frac{238.81}{2.0} = \boxed{119.4}$

governs

$$P_u = 1.2D + 1.6L$$
$$= 1.2 \times 35 + 1.6 \times 70 = 154 \text{ kips}$$

$$P_u \leq \phi P_n$$

$$P_n \geq \frac{P_u}{\phi} = \frac{154}{0.90} = 171.1 \text{ (gross area yielding)}$$

Yielding on gross area:

$$P_n = F_y A_g \Rightarrow A_g = \frac{P_n}{F_y} = \frac{171.1}{36}$$
$$= 4.75 \text{ in}^2$$

on net area, $P_n = \dfrac{P_u}{\phi} = \dfrac{154}{0.75} = 205.3$

$$P_n = A_e F_u \Rightarrow A_e = \frac{P_n}{F_u} = \frac{205.3}{58}$$
$$= 3.54$$

Assume, $U = 0.8$ (অতি কম ~~too~~ conservative design)

$$A_e = UA_n \Rightarrow A_n = A_e/U = 4.43 \text{ in}^2$$

$$A_g = A_n + \text{bolt hole}$$

$$= 4.43 + 2\left(\tfrac{3}{4} + \tfrac{1}{8}\right)t \rightarrow \tfrac{1}{2}'' \text{ assume}$$

choose, $L\,6 \times 4 \times \frac{9}{16}$

Block Shear Failure

$F_y = 36$ ksi

$F_u = 58$ ksi

$A_g = 8 \times \frac{5}{8} = 5 \text{ in}^2$

1) Yielding on gross area, $T_n = F_y A_g$

$= 36 \times 5$

$= 180^k$

LRFD $\quad \phi T_n = 0.9 \times 180 = 162^k$

ASD $\quad \frac{T_n}{\Omega} = \frac{180}{1.67} = 107.8^k$

2) Fracture on Net Area

$A_n = 5 - 2\left(\frac{3}{4} + \frac{1}{8}\right)\frac{5}{8}$

$= 3.91 \text{ in}^2$, $\quad (U = 1)$

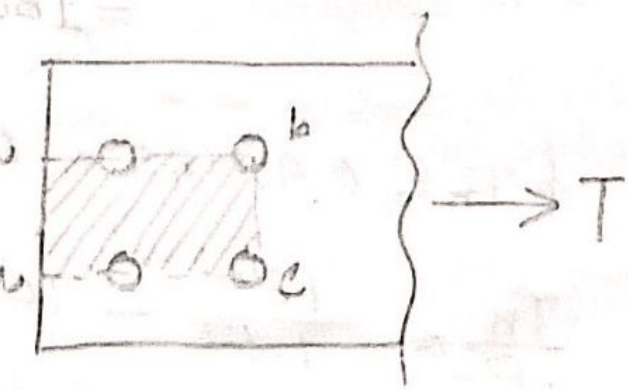
$A_e = UA_n = 3.91$

$T_n = F_u A_e = 58 \times 3.91 = 226.8^k$

<u>LRFD</u> $\phi T_n = 0.75 \times 226.8 = 170.1^k$

<u>ASD</u> $\frac{T_n}{\Omega} = \frac{226.8}{2.0} = 113.4^k$

<u>Block Shear Mode-1</u>



Tension, A_n = Area bc $= \left\{5 - 2 \times \frac{1}{2}\left(\frac{3}{4} + \frac{1}{8}\right)\right\} \frac{5}{8}$

$= 2.578 \ in^2$

Shear, A_{gv} = Area ab+de

$= \left(1\frac{1}{2} + 2\frac{1}{2}\right) \times 2 \times \frac{5}{8} = 5 \ in^2$

Shear, $A_{nv} = A_{gv}$ − bolts

$= 5 \left(1 + \frac{1}{2}\right)\left(\frac{3}{4} + \frac{1}{8}\right) \frac{5}{8} \times 2$

$= 3.36\ in^2$

Tension rupture $= F_u U_{bs} A_{nt}$

$= 58 \times 1 \times 2.578$

shear

~~Tension~~ yielding $= 0.6 F_y A_{gv}$

$= 0.6 \times 36 \times 5 = 108$

Shear rupture $= 0.6 F_u A_{nv}$

$= 0.6 \times 58 \times 3.36$

$= 116.93$

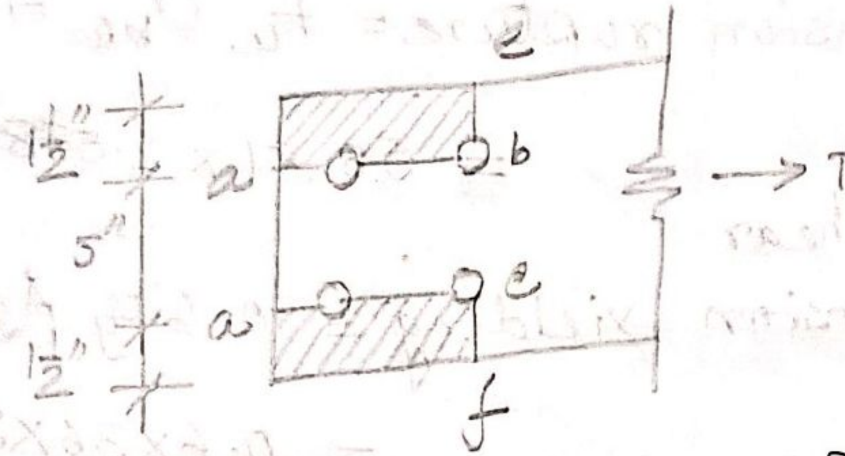
108^k

$T_n = 149.5 + 108 = 257.5^k$

$F_y = 36\ ksi$

$F_u = 58\ ksi$

Block Shear Mode - 2



$$A_{nt} = \left\{\left(1\tfrac{1}{2} + 1\tfrac{1}{2}\right) - 2 \times \tfrac{1}{2} \times \left(\tfrac{3}{4} + \tfrac{1}{8}\right)\right\} \tfrac{5}{8}$$

$$= 1.328 \text{ in}^2$$

Tension rupture $= F_u U_{bs} A_{nt}$

$$= 58 \times 1 \times 1.328 = 77.02^k$$

Shear Yield } → 108 } $T_n = 77.02 + 108$
shear rupture } $= 185^k$ → Governing Block Shear

Final

yielding on gross : LRFD : 162
ASD : 107.8

Fracture on net : LRFD : 170.1
ASD : 113.4

Block Shear : LRFD: 138.75

ASD : 92.5

Final Ans.

Steel Structure-6 9 September 2015

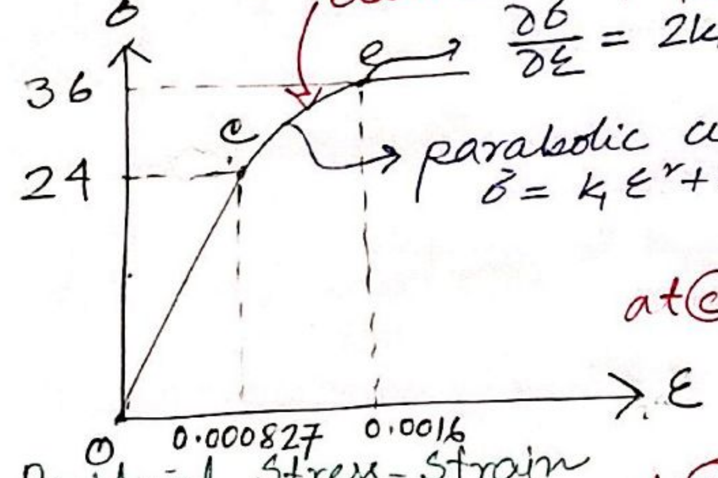
Residual Stresses in Steel section

Residual stress { * due to uneven cooling steel members develop an internal stress even before being used in a structure

cooling হলে যেকোনো body shrink করে,

→ self balancing (Force balance হবে, stress না।)

elastic এবং plastic curve এর মধ্যে সমাধিত curve থাকতে পারে,



Residual Stress-Strain Diagram

at ©, $\sigma = 24$, $\varepsilon = 0.000827$

$24 = k_1 (0.000827)^2 + k_2 (0.000827) + k_3$ — ①

at, ⓔ, $\sigma = 36$, $\varepsilon = 0.0016$

$36 = k_1 (0.0016)^2 + k_2 (0.0016) + k_3$ — ②

step-1

Elastic maximum = + 36 ksi

Stress increment $36 - 12 = 24$ ksi

Strain increment $\frac{24}{29000} = 0.000827$

at ⓔ, $\frac{d\sigma}{d\varepsilon} = 2k_1(0.0016) + k_2$

at a, $\sigma_{avg} = 24$

$\varepsilon_{avg} = 0.000827$

Steel Structure-7

14 September 2015

$F_{nv} = 0.4\, F_u^b$ (shear plane in the thread)
$\quad\;\; = 0.5\, F_u^b$ (shear plane outside the thread)

** কিছু বলা না থাকলে assume the more conservative case.

Bearing failure এর ক্ষেত্রে plate এর material property (F_u, F_y) হিসাব করতে হয়।

* L_c has a minimum (to prevent failure by shear) and a maximum 2.5d (এর চেয়ে বাড়ালেও failure একই রকম থাকবে) value.

Pg. 8-15

Example

Bolt এর limit state এর ক্ষেত্রে
(এর আগের chapter এর গুলো হল plate এর limit state এর জন্য).

Bolt A325. $\rightarrow F_u^b = 120$ ksi

plate A572 Gr. 50 $\rightarrow F_y = 50$ ksi
$F_u = 65$ ksi

Bolt shear

Threads are excluded

One bolt. $R_n = m A_b F_{nv}$

$= 1 \times \left\{ \frac{\pi}{4} \left(\frac{7}{8} \right)^2 \right\} \times 0.5 F_u^b$

$= \frac{\pi}{4} \times \left(\frac{7}{8} \right)^2 \times 0.5 \times 120$

$\boxed{R_n = 36 \text{ kip}}$ shear capacity governs

Bolt Bearing

Formula $1.2 L_c t F_u < 2.4\, d t F_u$ plate material

$2.4\, d t F_u = 2.4 \left\{ \frac{7}{8} + \frac{1}{8} \right\} \times \frac{5}{8} \times 65$

$= 91.41 \text{ kip}$

** L_c একেক bolt এর জন্য একেক রকম হতে পারে

Exterior bolt

$1.2 L_c t F_u = 1.2 \left\{ \underbrace{1\frac{1}{2} - \frac{\frac{7}{8} + \frac{1}{16}}{2}} \right\} \times \frac{5}{8} \times 65$

দেড় ইঞ্চি থেকে half of the hole diameter বাদ দিলাম

$= 50.27 \text{ kips} < 2.4\, d t F_u$

ok

Interior bolt

$$1.2 L_c t F_u = 1.2\left\{3-\left(\tfrac{7}{8}+\tfrac{1}{16}\right)\right\}\times\tfrac{5}{8}\times 65$$

$$= 100.5^k > 2.4\,d t F_u$$

not ok

For interior bolts, bearing capacity is 91.41 kips.

Considering Bolt shear

$$\text{LRFD Capacity} = 4\times 36\times \phi \quad (\phi \to 0.75)$$

$$= 108^k$$

$$\text{ASD Capacity} = \frac{4\times 36}{\Omega} \quad (\Omega \to 2)$$

$$= 72^k$$

Steel Structure - 9 5 October 2015

Steel Structure -10 7 October 2015

Shielded Metal Arc Welding (SMAW)

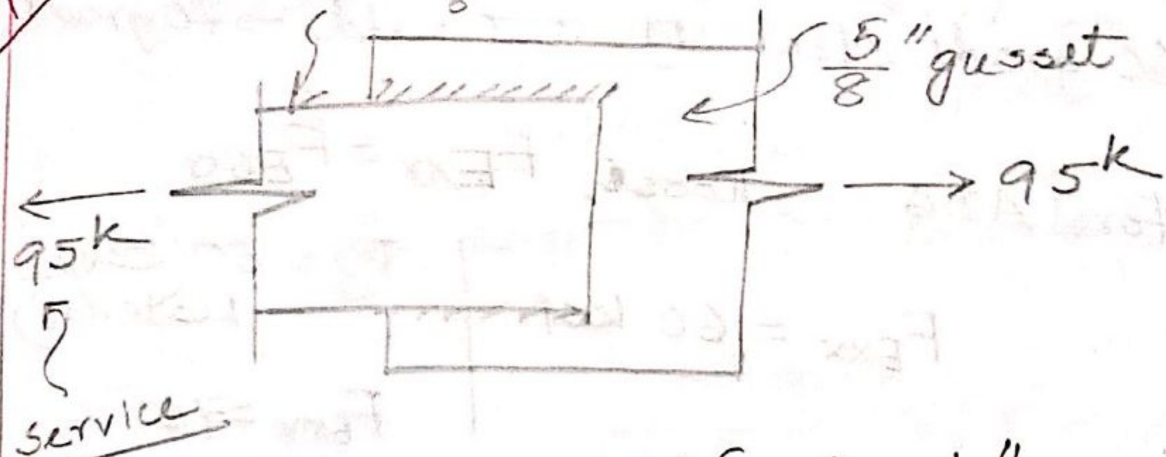
Electrode এর বাইরের element এর জন্য gas হয়, তাই metal এর oxide তৈরি হয় না।

* cast iron এ ভালো welding হয় না,
* types of welded joints and types of welds are different.

Steel Structure - 11 11 October 2015

11-15

A36 ($F_y = 36$ ksi, $F_u = 58$ ksi)



$\text{Min}^{\underline{m}}$ weld size (fillet) $= \frac{1}{4}$"

$\text{Max}^{\underline{m}}$ " " $= \frac{5}{8} - \frac{1}{16}$ (since plate thickness $> \frac{1}{4}$")

$= \frac{9}{16}$"

chosen size $= \underbrace{\frac{1}{2}}_{a}$"

Effective throat, $t_e = 0.707\,a$

$= 0.354$"

Weld: $\frac{R_n}{\Omega} = \frac{6 \times F_{EXX}\, t_e}{2.0}$ ইলেক্ট্রোড কিছু বলা নেই, তাই always ধরব, $F_{EXX} = F_{E60}$

50 grade steel এর জন্য weld → 60 grade

60 grade steel এর জন্য weld → 70 grade

For, A36, choose, $F_{EXX} = F_{E60}$

$F_{EXX} = 60$ ksi | $F_y = 50$ (Base Material)

$F_{EXX} = 70$

weld:

$$\frac{R_n}{\Omega} = \frac{0.6 \times 60 \times 0.354}{2}$$

$$= 6.4 \text{ k/"}$$

Base Material

(plate)

yield: $0.6t \frac{F_y}{\Omega} = 0.6 \times \frac{5}{8} \times \frac{36}{1.5}$

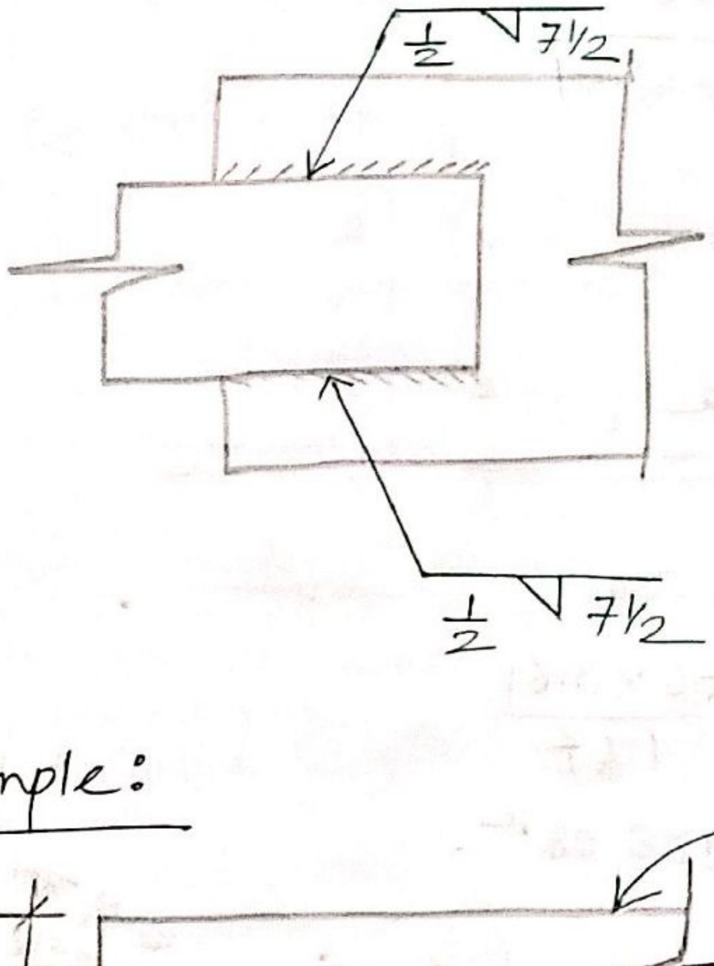
$$= 9 \text{ k/"}$$

Rupture: $0.6t \frac{f_u}{\Omega} = 0.6 \times \frac{5}{8} \times \frac{58}{2}$

$$= 10.9 \text{ k/"}$$

So, weld length $= \frac{9.5}{6.4} = 14.84''$

যদি plate thickness different হয় তাহলে minimum টাকে নিয়ে calculation করব,



11-17

Example:

Use E70 electrodes.
Follow ASD.

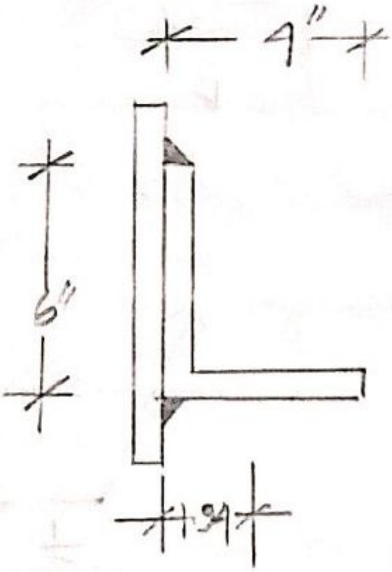
Angle Length:

yield on gross area: $\frac{T_n}{\Omega}$

$$= \frac{F_y A_g}{\Omega}$$

From chart

$$A_y = 3.61$$



Yield on Gross area

$$\frac{F_y A_g}{\Omega} = \frac{50 \times 3.61}{1.67}$$

$$= 108.08^k$$

Fracture on net Area

0.9, ধরা হয়েছে। conservative

$$\frac{T_n}{\Omega} = \frac{U F_u \times A_e}{\Omega} = \frac{0.9 \times 65 \times 3.61}{2}$$

$$= 105.6 \text{ ksi}$$

So, $\underline{105.6^k}$ governs

<u>Weld size:</u>

$a = \frac{1}{4}''$ based on max^{m} and min^{m} criteria

$t_e = 0.707 \times \frac{1}{4} = 0.177''$

weld capacity $= \frac{R_n}{\Omega} = \frac{0.6\, t_e\, F_{EXX}}{2}$

$= \frac{0.6 \times 0.177 \times 70}{2}$

$= 3.72\, k/''$

Total weld length $= \frac{105.6}{3.72} = 28.4$

12-7

welding এর size বলা না থাকলে আমরা force per unit length বের করি।

1) c.g. of weld

$$L\bar{x} = L_1x_1 + L_2x_2 + L_3x_3$$

$$\Rightarrow (8+2\times6)\bar{x} = 6\times3 + 8\times0 + 6\times3$$

$$\Rightarrow \bar{x} = 1.8''$$

$$P = 15^k$$

$$M = Pe = 15(8+6-1.8)$$

$$= 183\,k''$$

2) I_p about c.g.

$$I_p = \underbrace{\left[\frac{6\times1^3}{12} + 6\times1\times4^2\right]\times2 + \frac{1\times8^3}{12}}_{I_x}$$

$$+ \left[\frac{1\times6^3}{12} + 1\times6\times1.2^2\right]\times2 +$$

$$\underbrace{\left[\frac{8\times1^3}{12} + 8\times1\times1.8^2\right]}_{I_y}$$

$= 23.5.6 + 79.87$

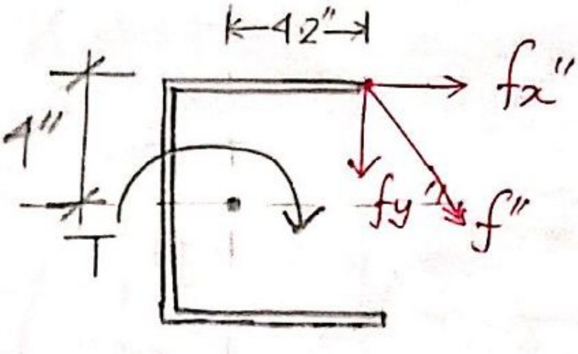
$= 315.5$

3) Stress

Direct stress $= \frac{P}{L} = \frac{15}{6\times2+8} = 0.75 k/'' \downarrow$

Due to moment

Maximum stress shall occur at a point furthest from c.g. i.e. points A or B.



$$f_x'' = \frac{T y}{I_p} = \frac{183 \times 4}{315.5} = 2.32 \rightarrow$$

$$f_y'' = \frac{T x}{I_p} = \frac{183 \times 4.2}{315.5} = 2.44 \downarrow$$

$$\text{Resultant} = \left\{ (2.44 + 0.75)^2 + 2.32^2 \right\}^{1/2} \text{ k/}''$$

$$= 3.96 \text{ k/}''$$

Steel Structure - 13   1 November 2015

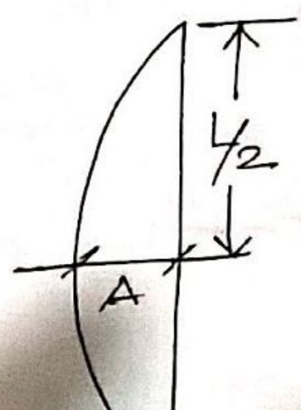
Compression Member - 1

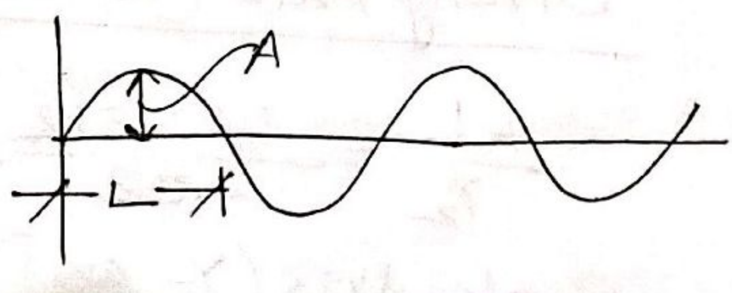
Column: (primarily compression member, assisted by small bending)

* Euler's Buckling Theory is applicable to elastic buckling only.

General Eq^n of curvature: $$\frac{\dfrac{d^2y}{dx^2}}{\left[\sqrt{1+\underbrace{\left(\dfrac{dy}{dx}\right)^2}_{\text{for small deflection zero.}}}\right]}$$

$$y = A \sin kz$$





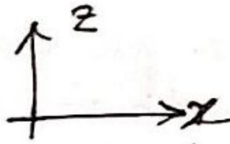
sine curve

'A' can be figured out using non-linear higher-order equations.

Steel Structure-14   2 November 2015

Angle and wide flange এর জন্য connection design easier.

Weak Direction



normal to the plane → Y axis

General tendency is to buckle along the weak axis.

সেই axis-এ, moment of inertia কম,

AISC 2005/BNBC 2015 - অনুসারী

ksi, inch, k
for stress, for length, for force

remember { Concrete এর formula psi তে
steel এর formula ksi তে

Strong Axis (x-axis)

$$\frac{KL}{r_x} = \frac{1.0 \times 312}{4.32} = 72.2$$

Weak Axis (Y-axis)

$k = 0.5$

$$\frac{KL}{r_y} = \frac{0.5 \times 312}{2.01} = 77.61$$ (Governs, to stay on the safe side)

$\frac{KL}{r}$ যত বেশী columnএর capacity তত কম।

$$4.71\sqrt{\frac{E}{F_y}}$$

$$= 4.71\sqrt{\frac{29000}{50}}$$

$$= 113.4 > \frac{KL}{r_y}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \times 29000}{(77.61)^2} = 47.52$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$$

$$= 32.17 \text{ ksi}$$

$$P_{cr} = A_g F_{cr} = 13.3 \times 32.2 = 428.11^k$$

$$\text{ASD} \Rightarrow \frac{P_{cr}}{\Omega} = \frac{428.11}{1.67} = 256.4^k$$

Design এর ক্ষেত্রে steel section নেয়া হবে lightest therefore more economic.

* আমরা সবসময় elasto-plastic buckling prefer করি, এবং $\frac{KL}{r} < 113$ নেয়া হয়।

W section-এ r_y is minimum.

* use three trials in exam.

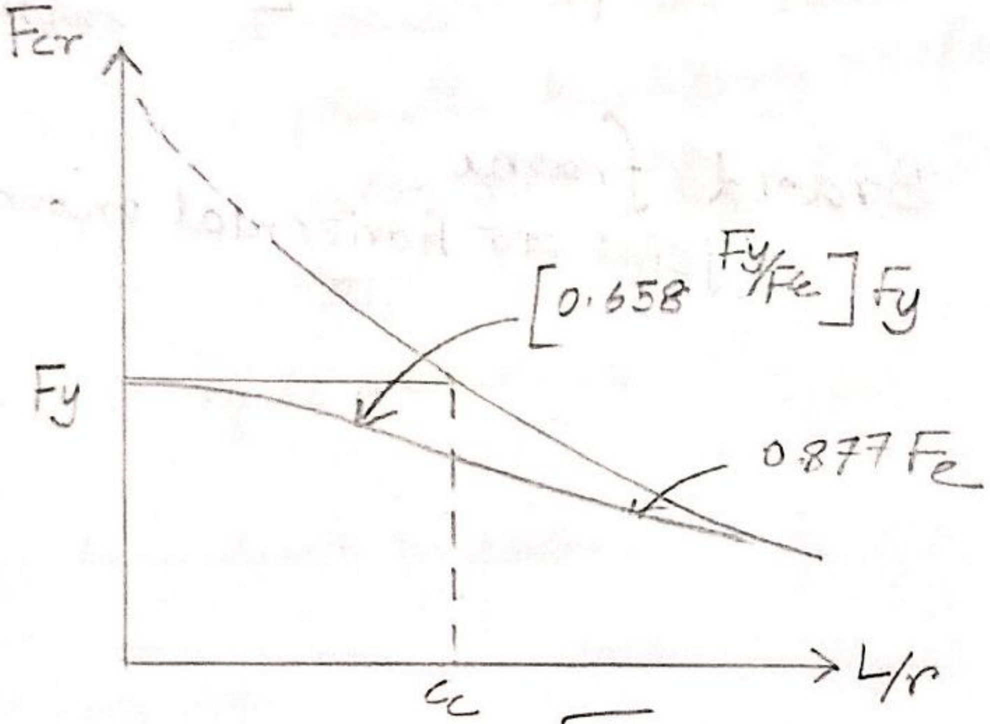
Steel Structure-15 — 4 November 2015

* Bracings prevents sway.

$$\text{Stiffness} = \frac{EI}{L}$$ → যেই axis এ bending তার about এ moment of inertia

Braced frame
(joint এর horizontal movement হবেনা)

Effect of Residual Stresses in Compression Members



$= 4.71\sqrt{\frac{E}{F_y}}$

Steel Structures-17 9 November 2015

Flexural Members

Beam connected to column → girder

spandrel

খুব বড়
দুইটা column এর মাঝখানের very short beam.

depth and length is very same

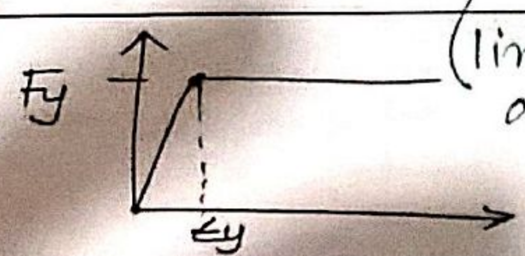
Truss acts as a beam. এর top chord এ compression, bottom chord এ tension থাকে,

$$\sigma = \frac{Mc}{I} = \frac{M}{I/c} = \frac{M}{S}$$

$$\Rightarrow M = \underset{\downarrow F_y}{\sigma} \underset{\hookrightarrow \text{section modulus}}{S}$$

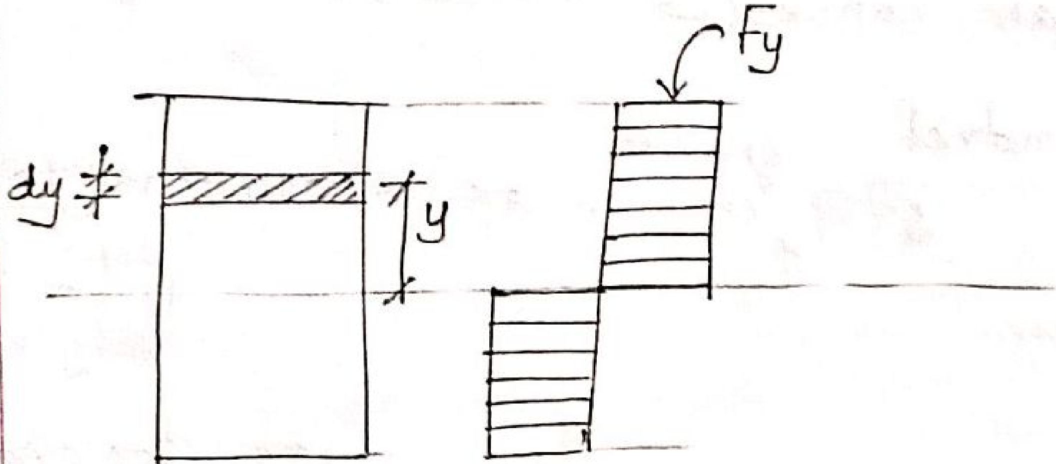
Maximum moment capacity, $M_n = M_y = S_x F_y$

Basic Stress- Strain Relationship



(linear at first, then almost horizontal)

সবগুলো fiber yield করে গেলে plastic moment পাওয়া যাবে।



$$dM = F_y \, dA \, y$$

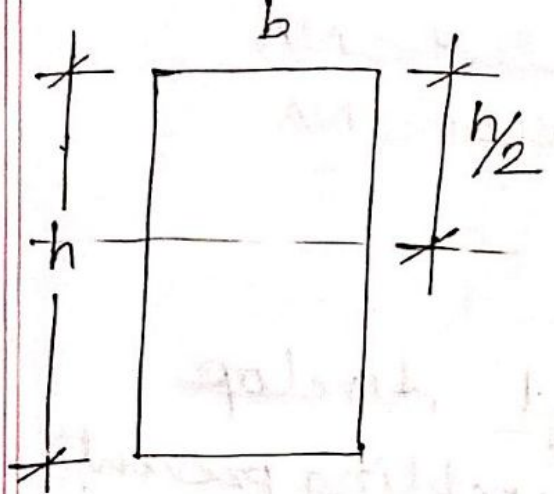
$$M_p = \int F_y \, dA \, y$$

$$= F_y \underbrace{\int y \, dA}_{\text{moment of area about the Neutral axis}}$$

Plastic Section Modulus: $\boxed{Z = \int y \, dA}$

(Chi) কাই $\boxed{\xi = \frac{M_p}{M_y} = \frac{Z}{S}}$ shape factor

Shape Factor, ξ



Plastic condition এ neutral axis বের করার নিয়ম হলো- cross sectional area কে দুইভাগে ভাগ করা, যেহেতু, tension area = compression area

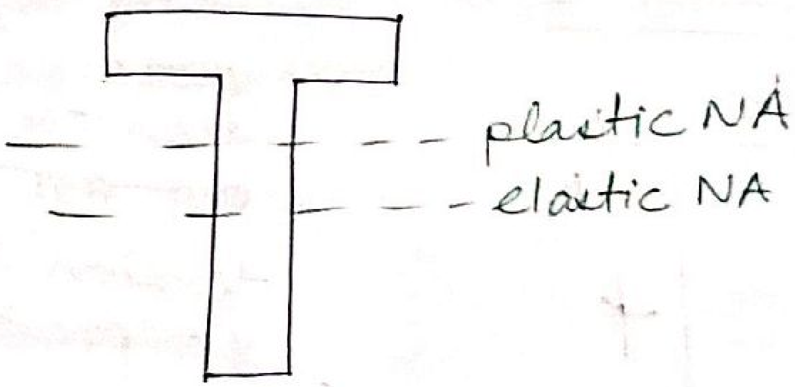
$$S_x = \frac{I}{c} = \frac{bh^3}{12} \Big/ \frac{h}{2} = \frac{bh^2}{6}$$

$$Z = \int y\,dA = \left(b \times \frac{h}{2}\right) \times \frac{h}{4} \times 2$$

$$= \frac{bh^2}{4}$$

$$\xi = \frac{Z}{S} = \frac{6}{4} = 1.5$$

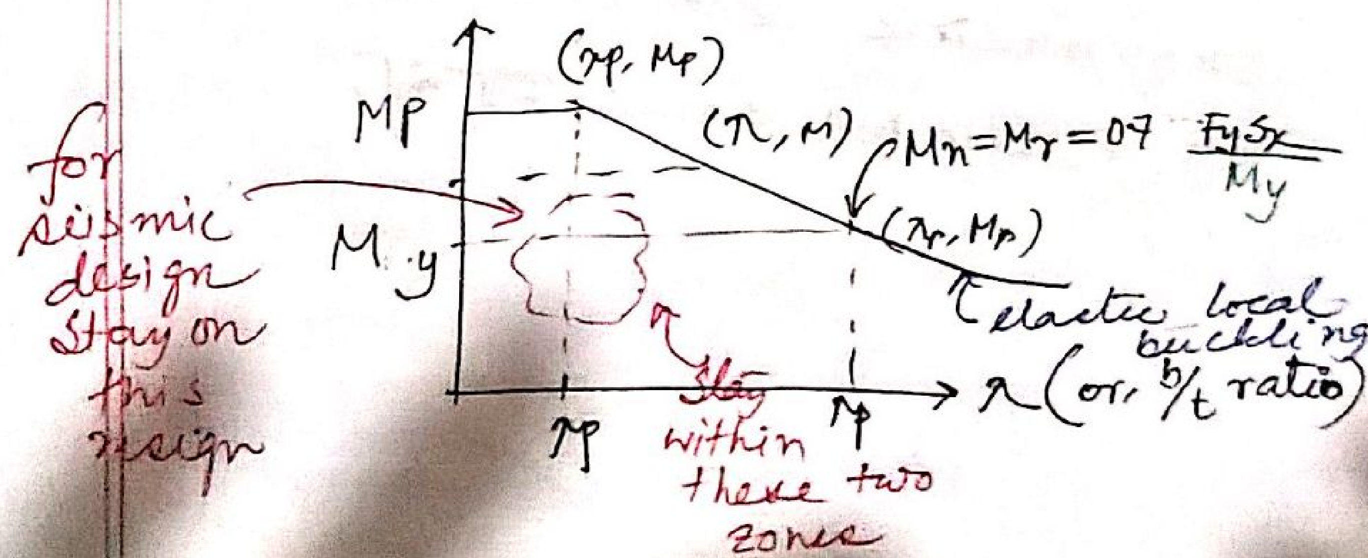
unsymmetric section এ elastic NA এবং plastic NA same হয় না।



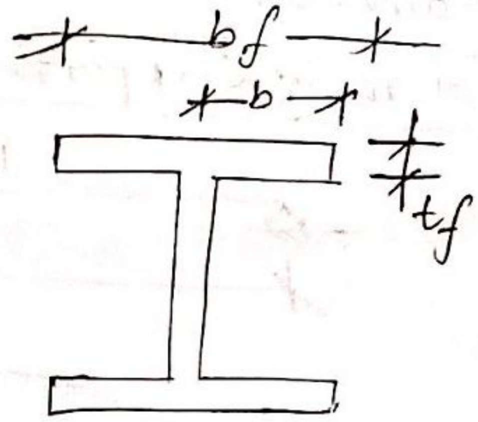
* plastic moment develop করতে গেলে flange গুলোর local buckling prevent করতে হবে

আমাদের limit state হচ্ছে plastic moment capacity

$$M_n = M_p = F_y Z_x$$



Design capacity

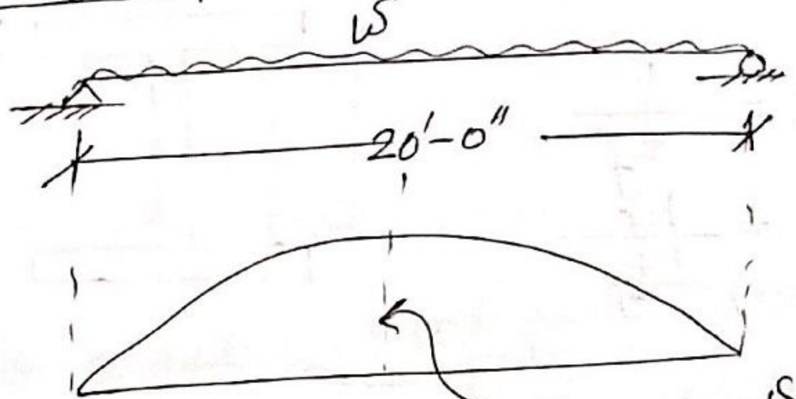


$$\lambda = \frac{b}{t_f}$$

$$= \frac{b_f}{2t_f}$$

Nominal Moment Capacity of a Beam Section

Laterally Supported Beams: ASD Design



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

$DL = 0.2\ k/'$

$LL = 0.8\ k/'$

$w = 0.2 + 0.8 = 1.0\ k/' = \frac{1}{12}\ k/''$

$L = 20' = 240''$

Distance এর unit ব্যবহার inch এ হবে

$M = \frac{wL^2}{8} = \frac{(1/12) \times 240^2}{8} = 600\ k''$

Required ASD capacity

$\Omega = 1.67$

Required Nominal capacity

$= 600 \times 1.67 = 1002\ k''$

For compact section, $M_n = M_p = F_y Z_x$

$$\Rightarrow Z_x = \frac{M_P}{F_y} = \frac{1002}{36} = 27.83 \text{ in}^3$$

Choose a section where $Z_x \geq 27.83 \text{in}^3$

A36, $F_y = 36$ ksi

From AISC manual table, we choose W8X31

$Z_x = 30.4$

check, $M_n = M_p = F_y Z_x = 36 \times 30.4$
$= 1094 K''$

$$M = \frac{M_u}{\Omega} = \frac{1094}{1.67} = 655.3 K''$$

$$\omega = \frac{8M}{L^2} = 0.091 \ K/'' = 1.092 \ K/' \ (-DL)$$

$$\frac{1}{0.092^k} = 92^\# > 31^\# \quad \underline{ok}$$

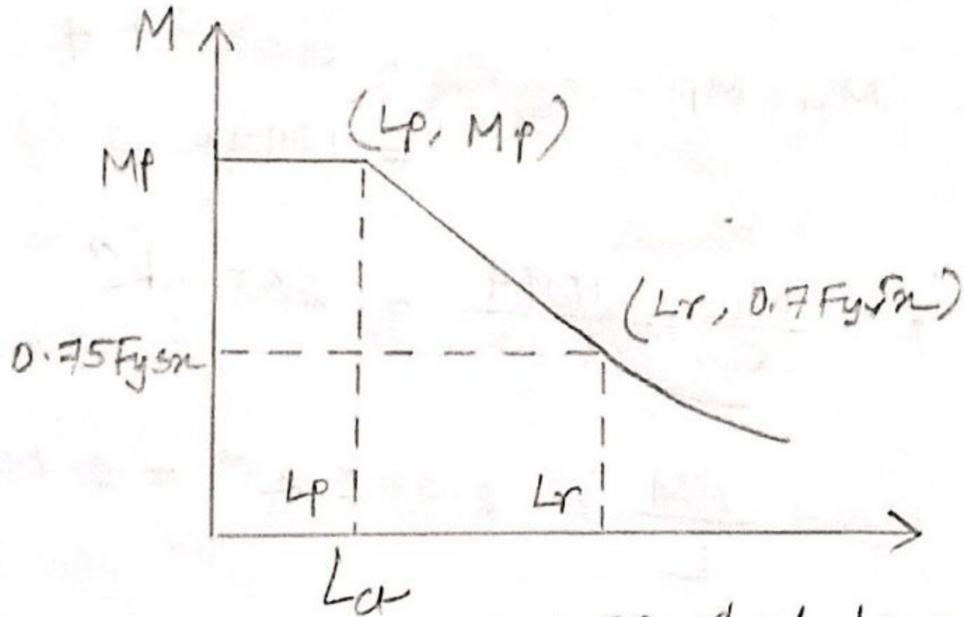
- Lateral torsional buckling depends on the unsupported length.

(Derivation → Not important, দরকার)

$\underbrace{M_{cr} > M_{PM}}$ plastic moment এ fail করবে,

হলে ভালো, ok

* Formula মুখস্থ করতে হবে না।



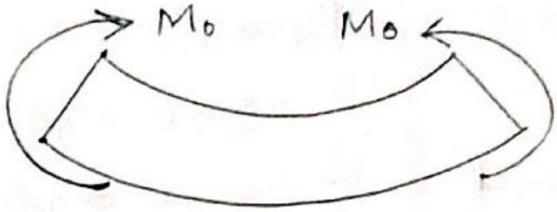
no need to memorize

$$\frac{L_p}{r_y} = 1.76\sqrt{\frac{E}{F_y}} = \frac{300}{\sqrt{F_y}} \rightarrow \text{ksi}$$

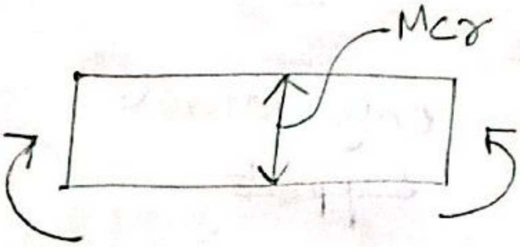
$$L_r = 1.95\, r_{ts} \frac{E}{0.7F_y}\sqrt{\frac{J_c}{S_x h_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_x h_o}{J_c}\right)^2}}$$

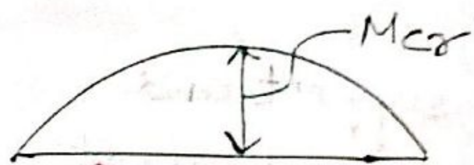
Symmetric shape, $c = 1$

C_b = modification factor.



যেহেতু দুইটা moment একই, তাই constant moment, but real life-এ moment change হয়ে যাবে। তাই, C_b factor দিব।

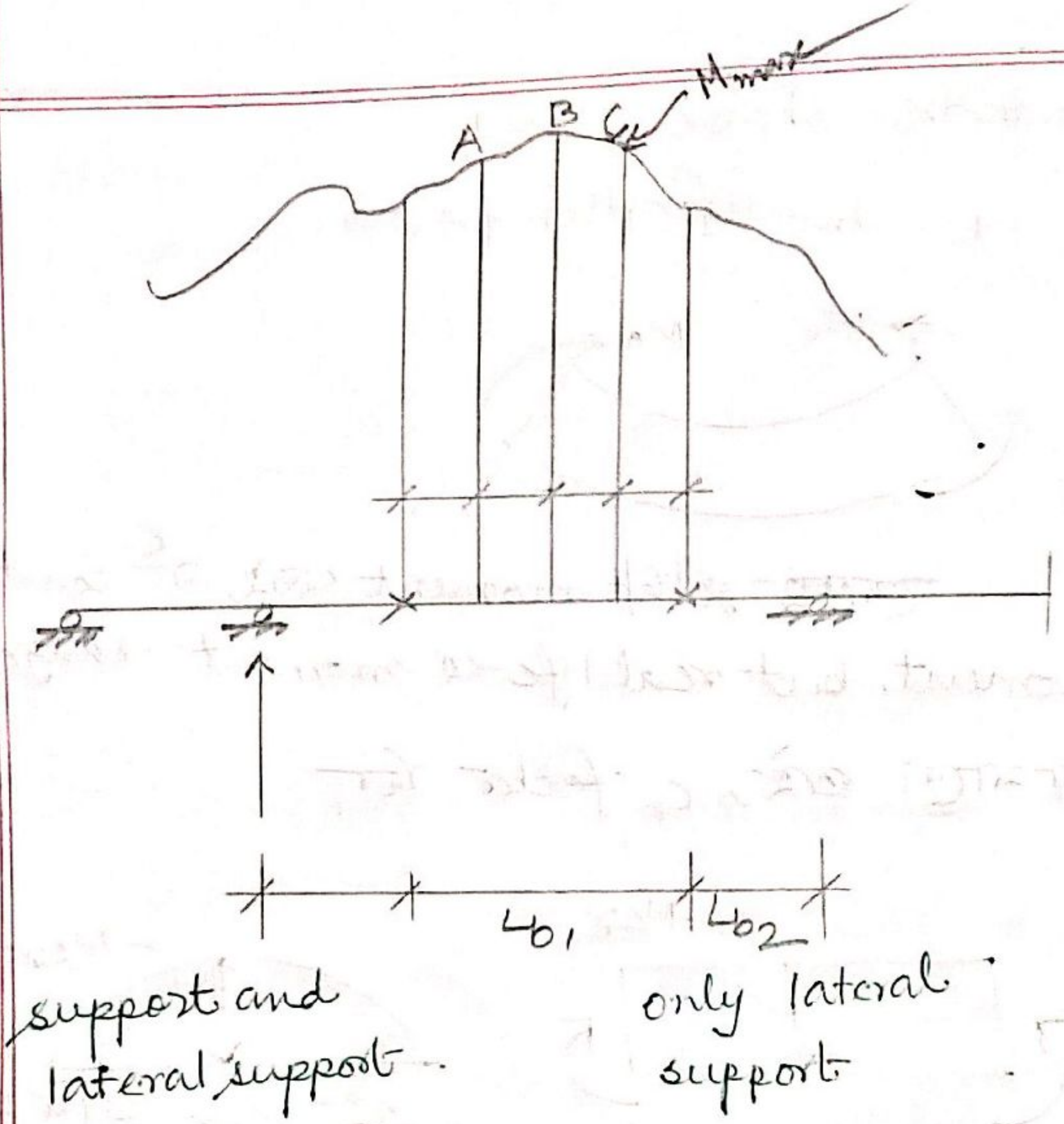




এই M_{cr} বেশি হয়।

তাই, $C_b > 1$,

$C_b \sim 1-3$



A
B
C
M_max
L_{b1}
L_{b2}
support and lateral support
only lateral support

W14×74 $F_y = 50\,ksi$

From, AISC chart → compact section

$A = 21.8\,in^2$	$S_x = 112$	$r_{ts} = 2.82$
$b_f = 10.1''$	$r_y = 2.48$	$h_o = 13.4$
$t_f = 0.785''$	$Z_x = 126$	$J = 3.87$
		$C_w = 5990$

1) Continuous lateral support → No lateral Bracing

Compact section

$$M_n = M_p = F_y Z_x = 50 \times 126 = 6300\,k''$$

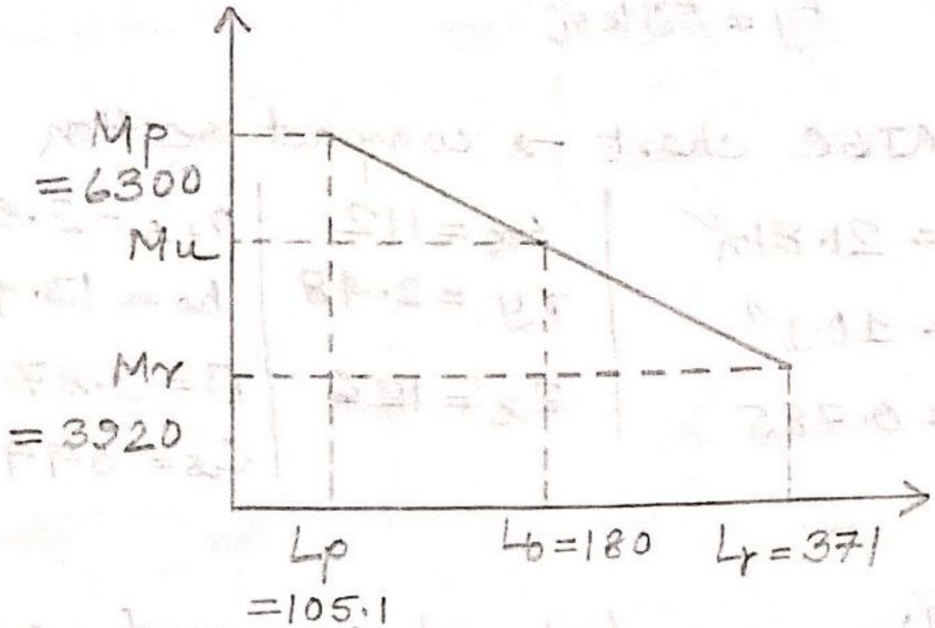
LRFD: $\phi M_n = 0.9 \times 6300 = 5670\,k''$

2) Unbraced Length = 15′, $C_b = 1$

Find L_p:

$$\frac{L_p}{r_y} = 1.76\sqrt{\frac{E}{F_y}} \Rightarrow L_p = 1.76\sqrt{\frac{29000}{50}} \times 2.48$$

$$= 105.1''$$



$$L_r = 1.95 \times 2.82 \times \frac{29000}{0.7 \times 50} \sqrt{\frac{3.87 \times 1}{112 \times 13.4}}$$

$$\times \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7 \times 50}{29000} \times \frac{112 \times 13.4}{3.87 \times 1} \right)^2}}$$

$$= 371.3''$$

$$L_r = 371''$$

$$M_r = 0.7 \times F_y \times S_x$$

$$= 0.7 \times 50 \times 112$$

$$= 3920 \, k''$$

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

$= 1 \times 5630$

$= 5630$ k"

* সবকিছু inch-এ convert করবো,

<u>LRFD:</u> $M_n = 0.9 \times 5630$

$= 5067$ k"

<u>W10×12</u> $F_y = 50$ ksi

From AISC chart → compact section

$A =$	$b_f = 3.96''$	$t_f = 0.21''$
$S_x = 10.9$	$r_y = 0.785$	$Z_x = 12.6$
$r_{ts} = 0.983$	$h_o = 9.66$	$J = 0.0547$

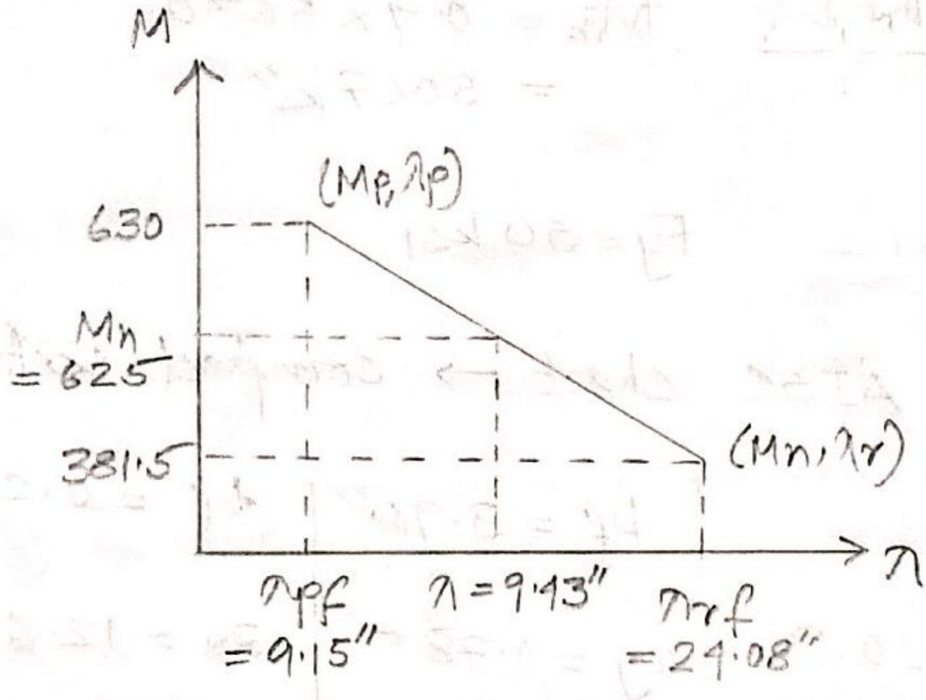
continuous lateral support → no lateral bracing

$$\lambda = \frac{b_f}{2t_f} = \frac{3.96}{2\times 0.21} = 9.43$$

For compact section, $\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$

$$= 9.15'' < \lambda$$

$$M_p = F_y Z_x = 50\times 12.6 = 630K''$$



$$M_n = 0.7 F_y S_x = 0.7\times 50\times 10.9$$

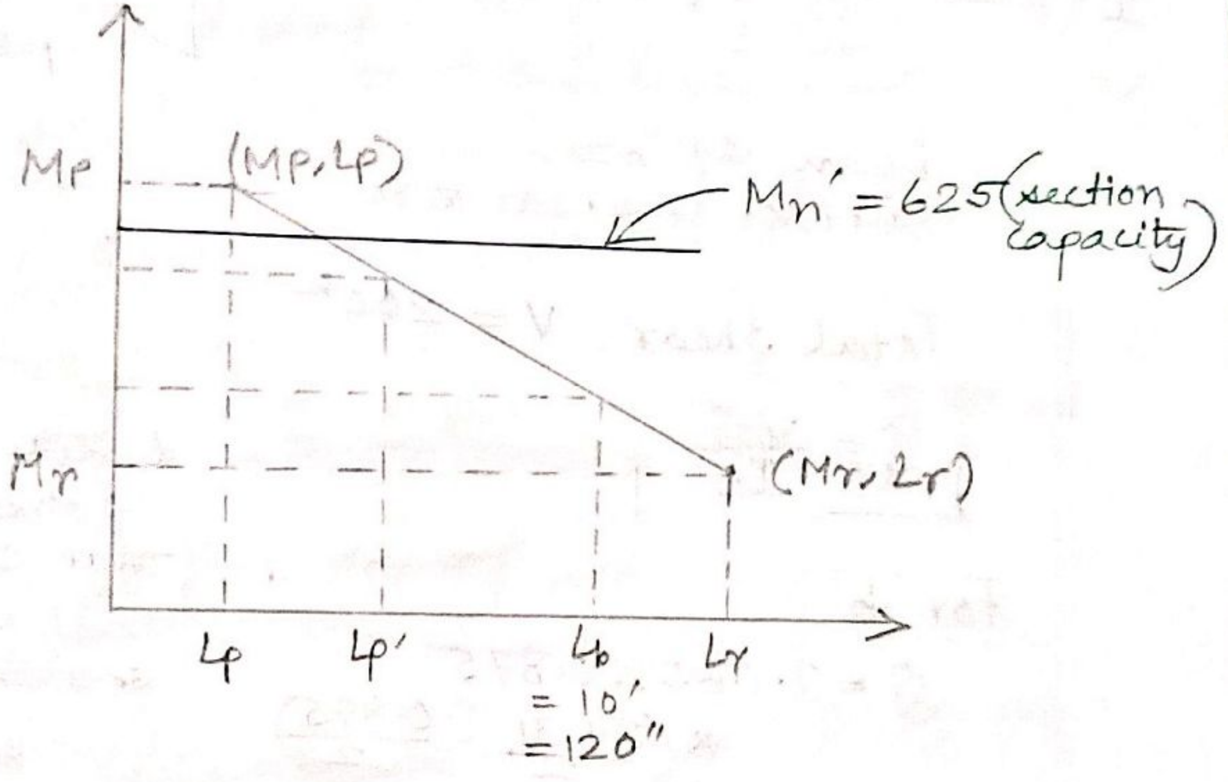
$$= 381.5$$

$$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}} = 24.08$$

Cross-section capacity,

$$M_n' = M_p - (M_p - 0.7 F_y S_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)$$

$$= 625\ k''$$



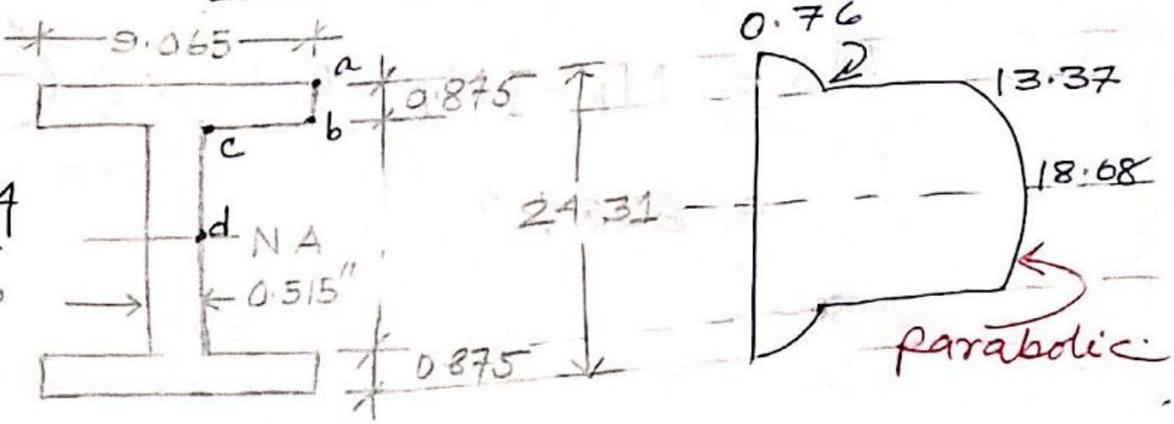
Theoretically possible হলেও practically Mp এর কাছে যেতে পারবে না, তাই Mn'-কেই maximum limit ধরতে হবে,

Shear in Beam

W24X94

$I = 2700$

$V =$



Beam এর open surface এ shear zero

Total shear, $V = 200^k$

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

t হচ্ছে যেই point এ stress বের করা হচ্ছে ঐ level-এর cross sectional area

For 'b'

$$Q = 9.065 \times 0.875 \times \left(\frac{24.31}{2} - \frac{0.875}{2}\right)$$

$$= 92.94$$

$$\tau_b = \frac{200 \times 92.94}{2700 \times 9.065}$$

$$= 0.76$$

At c

Q same as before $= 92.94$

$t = 0.515$

Q হচ্ছে যেখানে stress বের করতে চাই ওই টুকরের area-moment.

$$\tau_c = \frac{200 \times 92.94}{2700 \times 0.515} = 13.37$$

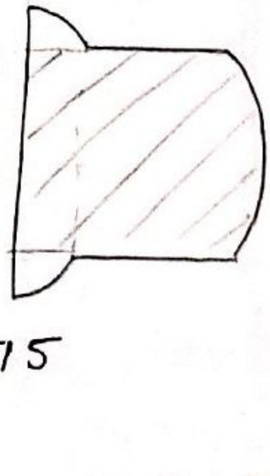
At d

$$Q = 92.94 + 0.515 \times 11.28 \times \frac{11.28}{2} = 125.7$$

$$\tau_d = \frac{VQ}{It} = \frac{200 \times 125.7}{2700 \times 0.515} = 18.08 \text{ ksi}$$

Shear contribution of web

$$V_{web} = \text{Area of stress diagram in web} \times t_w$$



$$= \frac{13.37 + 4 \times 18.08 + 13.37}{6} \times (11.28 \times 2) \times 0.515$$

$$= 191.8^k$$

$$\% \text{ contribution} = \frac{191.8}{200} \times 100 = 96\%$$

total shear এর 96% web এ আছে।

** for 'I' cross section a basic design consideration is: web takes the shear but the flexural stress goes to the flange.

$$\text{Average stress} = \frac{V}{A_w}$$ area of web

(flange area has been ignored due to its limited contribution to shear)

$$= \frac{200}{11.28 \times 2 \times 0.515}$$

$$= 17.21$$

$$\frac{\text{Avg}}{\text{max}} = \frac{17.21}{18.08} = 95\%$$

(Avg. stress is 95% of max^m stress, so detailed calculation is avoided and avg. stress is considered in design)

Point Load on Beams

→ target is to avoid web crippling, web buckling

* point load is a shear transfer through the web. → this transfer has to be handled precisely

if concentrated load surpasses the capacity the beam has to be strengthed at that point → using stiffeners (at the supports of I-sections)

stiffener → bearing stiffener (যেখানে point load সেখানে দরকার হয়।)

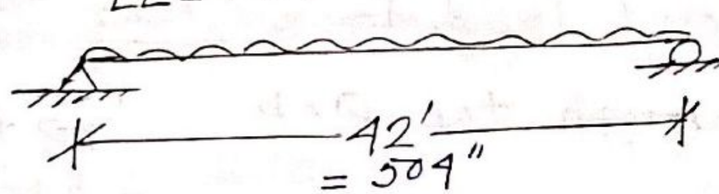
stiffener → intermediate stiffener (web এর shear capacity বাড়ানো হয়।)

Concrete বা যেকোনো structure এর জন্য deflection এর limit $\frac{L}{240}$, floor এ sensitive fitting থাকলে limit $\frac{L}{480}$

* steel structure -এ আমরা $\frac{L}{360}$ follow করব (যদি না দেওয়া থাকে)

21-11

$LL = 1.0\ k/'$ $\rightarrow 0.0833\ k/''$, $DL = 0.5\ k/'$ $\rightarrow 0.04167\ k/''$



$\Delta_{LL} \le \frac{L}{360}$

$F_y = 50\ ksi$, ASD

strength design এবং serviceability design দুটোই করতে হবে।

* 100 lb/ft unit weight এমন heavy section beam এ use করা হয়। 50 lb/ft – 70 lb/ft typically use করা হয়।

* convert all lengths to inch

$$\text{Assumed self wt} = 100\ \#/' = 8.33\ \#/'' = 0.0083\ k/''$$

$$w = (0.0833 + 0.04167 + 0.0083)\ k/'' = 0.133\ k/''$$

$$M = \frac{\omega L^2}{8} = \frac{0.133 \times 50 ft^2}{8} = 4232.5 k''$$

$$M_n = \Omega M = 1.67 \times 4232.5 = 7068 k''$$

Assuming compact section.

$$M_n = F_y Z_x$$

$$\Rightarrow Z_x = \frac{M_n}{F_y} = \frac{7068}{50}$$

$$= 141.4 \text{ in}^3$$

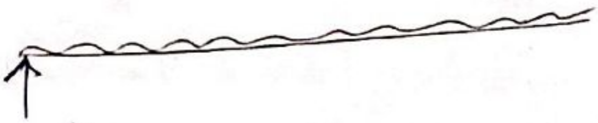
Max^m deflection $= L/360 = 1.4''$

$$\frac{5}{384} \frac{\omega L^4}{EI} = 1.4 \text{ (for LL)}$$

$$\Rightarrow \frac{5}{384} \times \frac{0.083 \times 50 ft^4}{29000 I} = 1.4$$

$$\therefore I = 1723 \text{ in}^4$$

deflection is a real phenomena, so it is always figured out using service/unfactored load.



33.5^k

Selected W21X93

Check for shear Shear $= 33.5^k$

Steel Structure -22 29 November 2015

Connections (inherently steel structure এর weakest part)

Concrete structure → cast in site
steel structure → assembled in site

steel sections maximum 20' length এর হয়।

(Beam এর flange কে column এর flange এর সাথে connect করতে হবে।) → Moment transfer হয় flange দিয়ে।
Shear transfer হয় web দিয়ে।

Steel Structure - 23 — 30 November 2015

All Bolted Double Angles

Design parameters
- Angle এর size, thickness
- Bolt এর dia, number of bolts

thread condition
- 'N' → threads included in the shear plane
- 'X' → threads excluded in the shear plane
- 'SC' → slip critical condition

* Girder is never coped

Continuous Beam-to-Column connection

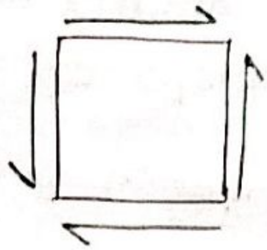
(শুধু web দিয়ে connection দিলে কাজ হবে না, flange to flange connection লাগবে।)

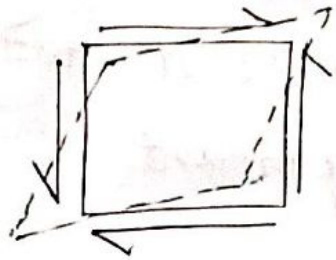
(c.g. is a fictitious point.)

* column এ axial force transmit হয় flange দিয়ে।
since it's 90% of the x-sec area.

Steel Structure - 24

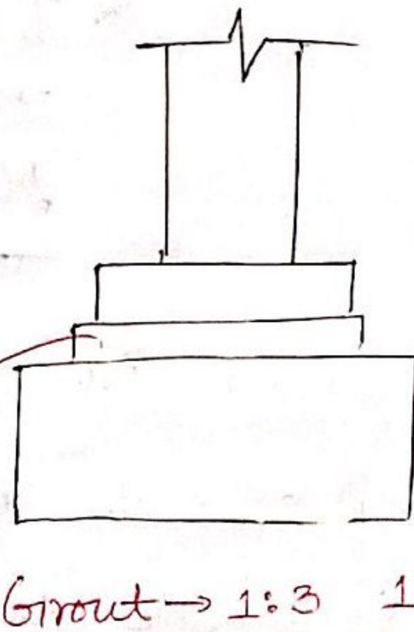
6 December 2015





Pure shear state

Column Base:



Grout → 1:3 1:4 হয়

Load uniformly distribute করার জন্য.

Moment আসলেই Anchor Rod এর design consideration - এ আনতে হবে।

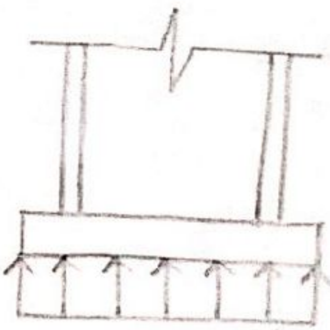
Theoretically compression এ anchor rod এর দরকার হয় না।

column base. ঠিক না থাকলে beam কে connect করতে পারব না, তাই column base অনেক important.

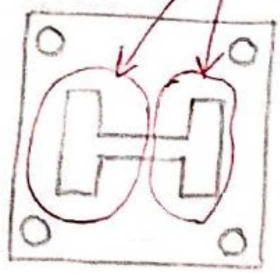
* Telecommunication Tower - এ যেই চারটা leg থাকে, সেগুলোর দুটোয় compression, দুটোয় tension আসে।

Column Base এর Design Consideration:

1) Concentric Compressive Axial Loads
2) Tensile Axial Loads
3) Base Plates with small Moments
4) Base Plates with Large Moments
5) Design for shear



দুটো portion এর যেকোনোটাই cantilever হতে পারে.



24-11

Example

Beam - Column

Column $\phi_c P_n > P_u$

$$\frac{P_u}{\phi_c P_n} \leq 1.0 \quad \boxed{\text{LRFD}}$$

$$\frac{P_a}{P_n/\Omega} \leq 1.0 \quad \boxed{\text{ASD}}$$

In general, $\dfrac{\text{Required strength}}{\text{Available strength}} \leq 1.0$

$$\frac{P_r}{P_c} \leq 1.0$$

Including flexural Reinforcement

$$\frac{P_r}{P_c} + \frac{M_r}{M_c} \leq 1.6$$

$M_r \rightarrow M_u$ (LRFD) or M_a (ASD)

$M_c \rightarrow \phi M_n$ (LRFD) or $\dfrac{M_n}{\Omega}$ (ASD)

Interaction formula considering two axes bending,

$$\frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1.0$$

AISC Design Equations

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \quad \text{when } \frac{P_r}{P_c} > 0.2$$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \quad \text{when } \frac{P_r}{P_c} \leq 0.2$$

LRFD Interaction Equations

$$\frac{P_u}{\phi_c' P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

when, $\frac{P_u}{\phi_c P_n} > 0.2$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

when $\frac{P_u}{\phi_c P_n} \leq 0.2$

ASD Interaction Equations:

$$\frac{P_a}{P_n/\Omega_c} + \frac{8}{9}\left(\frac{M_{ax}}{M_{nx}/\Omega_b} + \frac{M_{ay}}{M_{ny}/\Omega_b}\right) \le 1.0$$

when, $\frac{P_a}{P_n/\Omega} > 0.2$

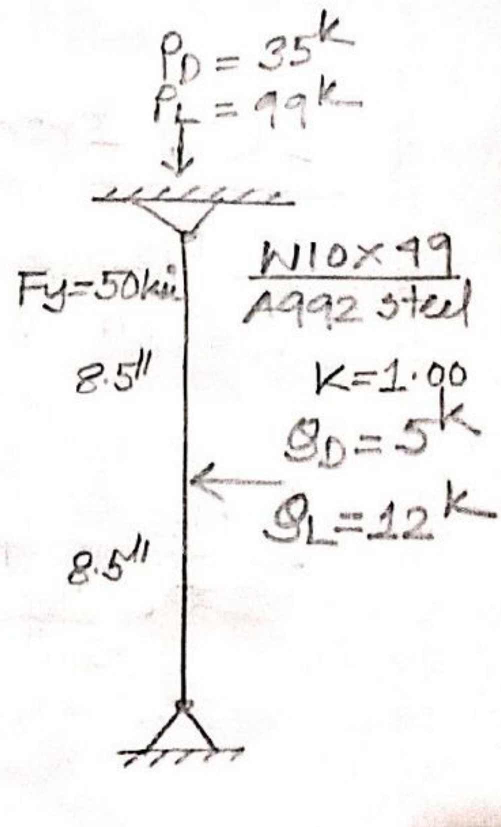
$$\frac{P_a}{2P_n/\Omega_c} + \left[\frac{M_{ax}}{M_{nx}/\Omega_b} + \frac{M_{ay}}{M_{ny}/\Omega_b}\right] \le 1.0$$

when, $\frac{P_a}{P_n/\Omega} \le 0.2$

Example (LRFD)

Strong axis bending due to lateral load, Q.

(Neglect moment amplification)



SolN:

Properties of W10×49,

$A_g = 14.4$	$d = 10''$	$S_x = 54.6$
$r_x = 4.35$	$Z_x = 60.4$	$r_y = 2.54$
$r_{ts} = 2.84$	$h_o = 9.42$	$J = 1.39, \quad c = 1.0$

Given, $\boxed{C_b = 1.32}$

Calculation করে দেখাতে হতে পারে

$b_f = 10$

$t_f = 0.56$

$h/t_w = 23.1$

$C_b = 1.32$

As a column, $L = 17' = 204''$

" " beam, $L_b = 17' = 204''$

$$P_u = 1.2 \times 35 \times 1.6 \times 99 = 200.4^k$$

Compact section

Determine col$^{\underline{m}}$ capacity,

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

$$\frac{KL}{r_y} = \frac{1 \times 204}{2.54} = 80.31 < 4.71\sqrt{E/F_y}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r_y}\right)^2} = 44.33 \text{ ksi}$$

$$F_{cr} = \left[0.658^{F_y/F_e}\right] F_y = 31.18 \text{ ksi}$$

$$\phi_c P_n = 0.9 A_g F_{cr}$$

$$= 0.9 \times 14.4 \times 3.181$$

$$= 404.2^k$$

$$\frac{P_a}{\phi_c P_n} = \frac{200.4}{404.2} = 0.495 > 0.2$$

only strong axis bending,

$M_{uy} = 0$

Flexural capacity as a beam considering

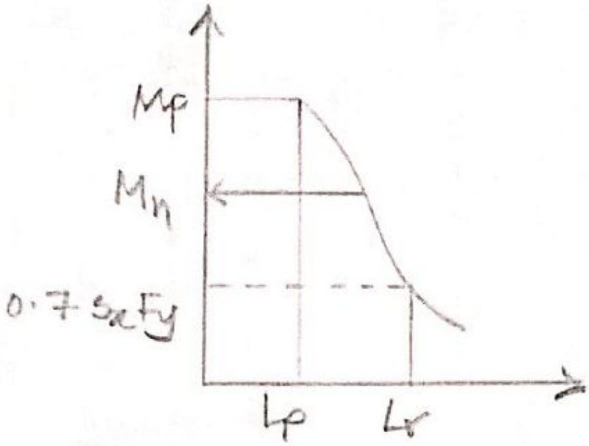
LTB

span, $L_b = 204''$

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

$$= 107.6'' < L_b$$

$$L_r = 1.95\, r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{J_c}{S_x h_o}} \sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_x h_o}{J_c}\right)^2}}$$



$$= 379.6''$$

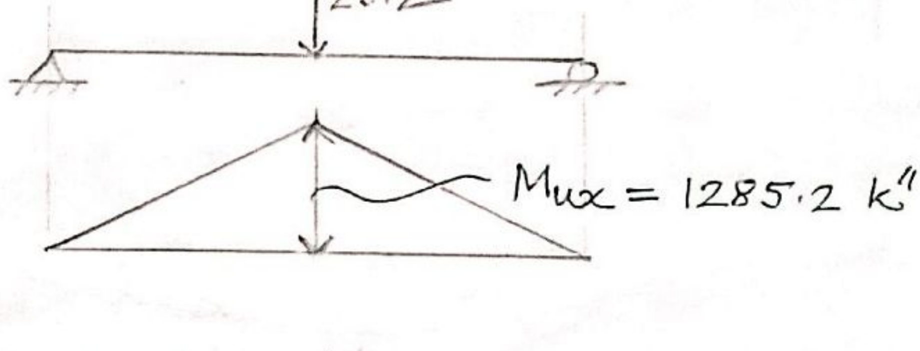
$$M_p = F_y Z_x = 3020\ K''$$

$$M_n = C_b\left[M_p - (M_p - 0.75\, S_x F_y)\frac{L_b - L_p}{L_r - L_p}\right] \le M_p$$

$$= 3467.6\ K'' > M_p$$

$$M_n = M_{nx} = M_p = 3020$$

$$P_u = 1.2 \times 5 + 1.6 \times 12 = 25.2^K$$

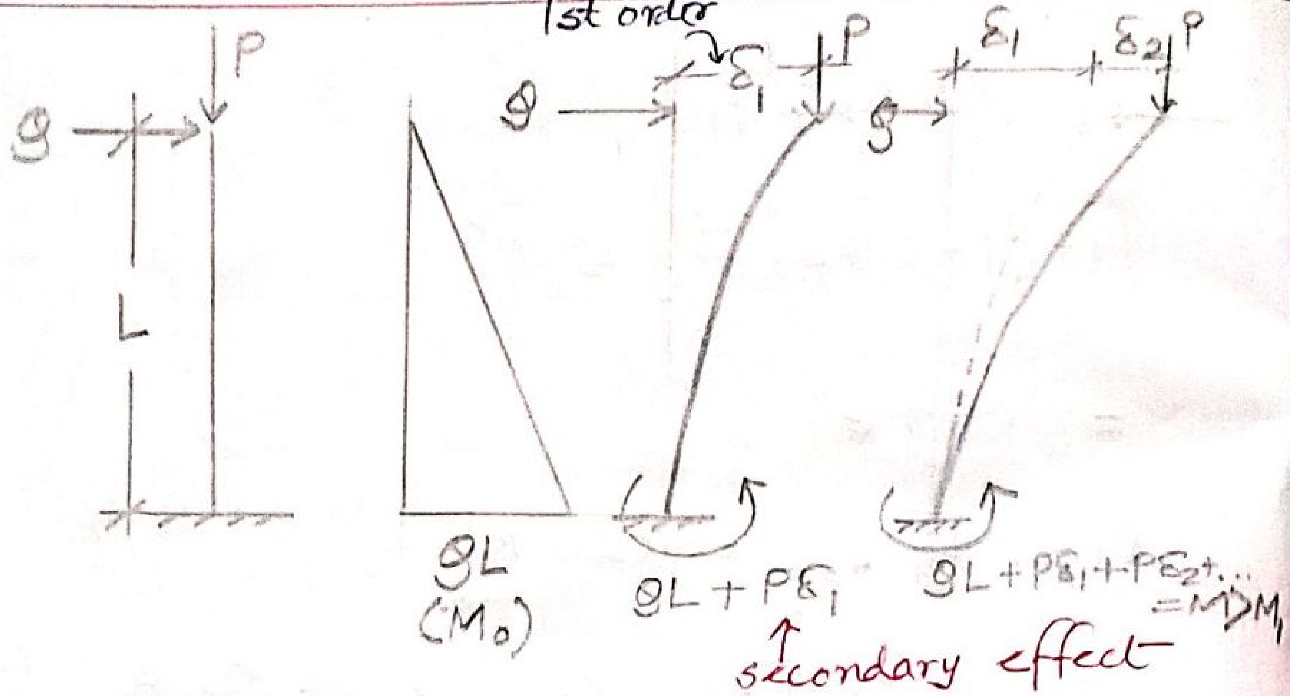


Interaction Equation

$$0.495 + \frac{8}{9} \times \frac{1285.2}{0.9 \times 3020} = 0.915 < 1.0$$

ok

Steel Structure - 26 10 December 2015



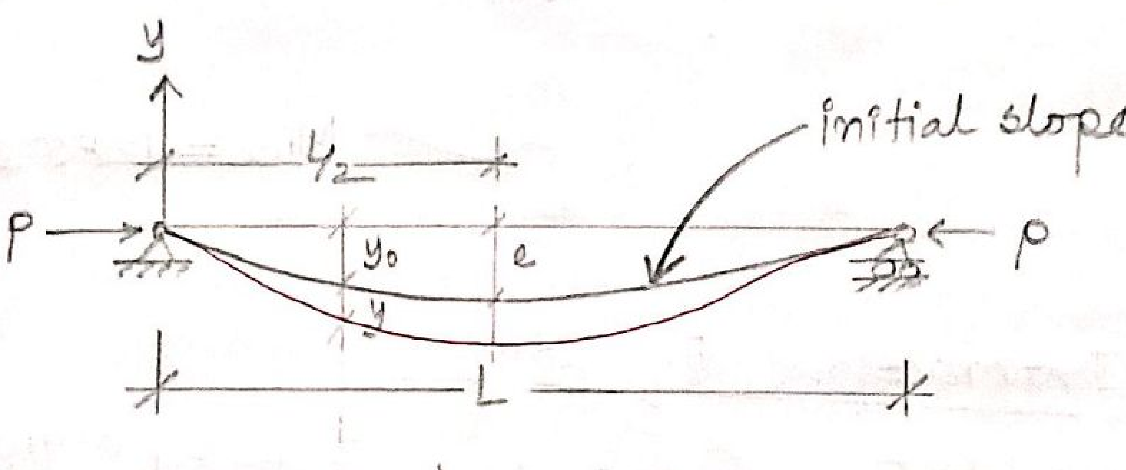
Load দিলে stress develop করে এবং সাথে সাথে একটা deflection ও হবে।

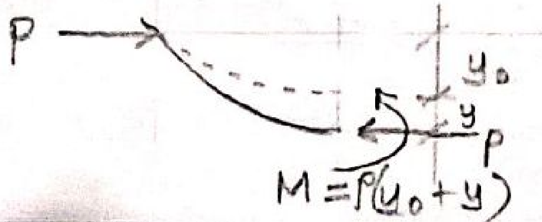
Moment including secondary effect,

$$M = B M_o$$

$\downarrow$

?





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

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

$$\Rightarrow \frac{d^2y}{dx^2} = -\frac{P}{EI}(y + y_0)$$

$$\Rightarrow \frac{d^2y}{dx^2} + \frac{Py}{EI} = -\frac{Py_0}{EI}$$

Boundary condition

A function that satisfies both BC ($y = 0, x = 0$ and $y = 0, x = L$) and the differential equation.

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

Putting, y in the differential equation.

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

$$\Rightarrow B = \frac{-\frac{Pe}{EI}}{\frac{P}{EI} - \frac{\pi^2}{L^2}} = \frac{-e}{1 - \frac{\pi^2 EI}{PL^2}} = \frac{e}{\frac{P_e}{P} - 1}$$

where, $P_e = \frac{\pi^2 EI}{L^2}$

= Euler Load

Now, $y = B \sin \frac{\pi x}{L} = \frac{e}{\frac{P_e}{P} - 1} \sin \frac{\pi x}{L}$

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

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

M is maximum when, $x = L/2$.

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

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

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

$$= B M_0$$

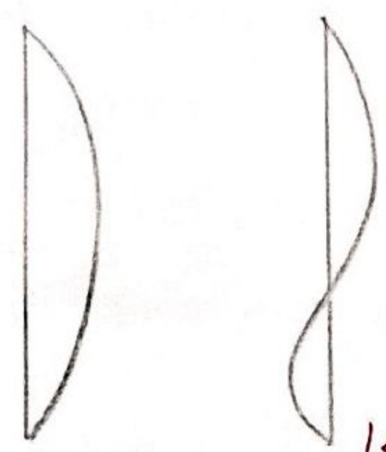
Moment amplification or, magnification factor, $B = \frac{1}{1 - P/P_e}$

Buckling represents a limit state → LRFD

$$B = \frac{1}{1 - \frac{P_a}{P_b}}$$

Moment magnification হবে-
axis-এর bending হচ্ছে তার
r নিয়ে করতে হবে।

Braced frame → P-small delta effect ($P-\delta$)
unbraced frame → P-capital delta effect ($P-\Delta$)



worst condition
(moment আর load
এর deflection একই দিকে)

less severe condition