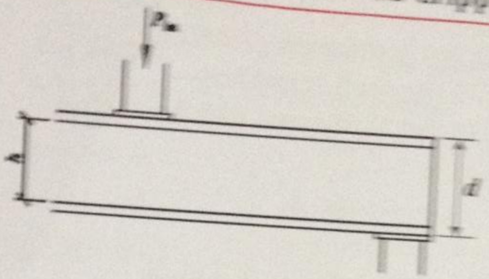
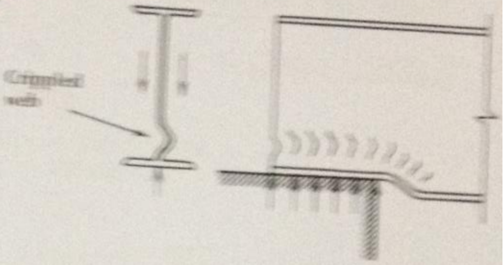
Point Load on Beams: Web Crippling





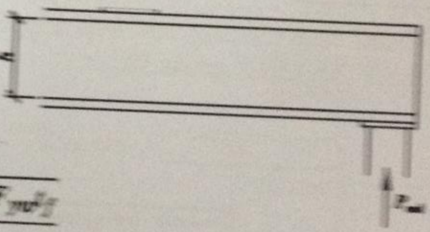
For interior loads, (i.e., point load acts at $d/2$ or more from member end)

মুখস্থ করা লাগবে না।

$$R_n = 0.80t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$= 136t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw}t_f}{t_w}}$$

Point Load on Beams: Web Crippling



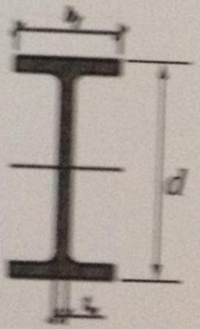
i.e. @ supports

For exterior loads, (i.e., point load acts at less than $d/2$ distance from end)

For $N/d \le 0.2$

$$R_n = 0.40t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

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



For $N/d > 0.2$

$$R_n = 0.40t_w^2\left[1+\left(\frac{4N}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$= 68t_w^2\left[1+\left(\frac{4N}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_{yw}t_f}{t_w}}$$

Point Load on Beams

LRFD: $\phi R_n \geq R_u$

$\phi \begin{cases} = 1.0 \text{ for local web yielding,} \\ = 0.75 \text{ for web crippling} \\ = 0.85 \text{ for Sidesway web buckling} \end{cases}$

মুখস্থ

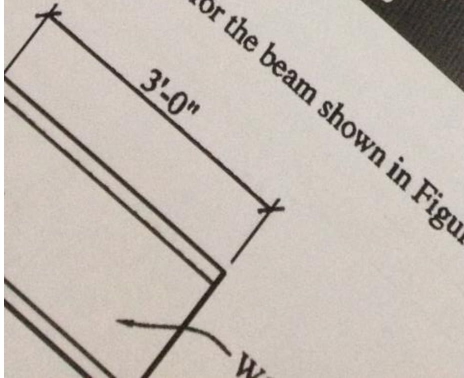
ASD: $\dfrac{R_n}{\Omega} \geq R_a$

$\Omega \begin{cases} = 1.5 \text{ for local web yielding,} \\ = 2.0 \text{ for web crippling} \\ = 1.76 \text{ for Sidesway web buckling} \end{cases}$

মুখস্থ

---

g and web crippling for the beam shown in Figure. The steel
50.



Solution:

For W18 × 50, we obtain

$k = 0.972$ in.
$t_w = 0.355$ in.
$d = 18$ in.
$t_f = 0.57$ in.

Area of web,

$\underline{\underline{A}} = t_w(N + 5k)$

$= 0.355(6 + 5 \times 0.972)$

$= 3.86 \text{ in}^2$

$\therefore \phi R_n = \phi A F_y$

1. $x = 3'\text{-}0'' > d$ and $> d/2$
2. $N = 6$ in. (given)

3a. *Web Yielding*

$$\phi_{wy}R_n = \phi_{wy}(5k + N)F_y t_w = 1.0[(5)(0.972) + 6](50)(0.355)$$
$$= \mathbf{192\ kips} > P_u = \mathbf{100\ kips\ OK}$$

3b. *Web Crippling*

As interior load,

$$\phi_{wc}R_n = \phi_{wc}0.8t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_y t_f}{t_w}}$$

$$\phi_{wc}R_n = (0.75)(0.8)(0.355)^2\left[1 + 3\left(\frac{6}{18.0}\right)\left(\frac{0.355}{0.57}\right)^{1.5}\right]\sqrt{\frac{(29{,}000)(50)(0.57)}{0.355}}$$

$$\phi_{wc}R_n = 172 \text{ kips} > P_u = 100 \text{ kips} \quad \text{OK}$$

The W18 × 50 beam is adequate for web yielding and web crippling.

Beam Bearing:

In many cases, beams are supported on concrete or masonry wall. In such a case, a steel *bearing plate* is used to transfer the beam reaction on to the support. Size of the bearing plate is based on *concrete bearing capacity* and beam *web yielding* and *web crippling* capacity.

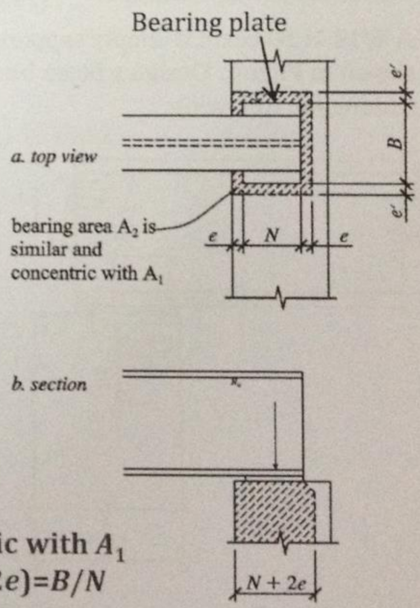
On concrete or masonry

$$R_u \le \phi R_n = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} \le 1.7 f'_c A_1$$

$\phi = 0.6$

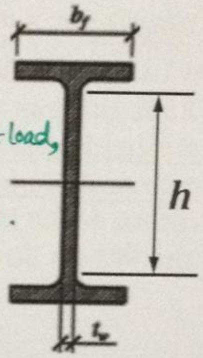
A_1 = Area of steel bearing plate= $B \times N$

A_2 = Maximum area of bearing support geometrically similar and concentric with A_1 $= (B+2e')(N+2e)$, where $(B+2e')/(N+2e) = B/N$



tension flange of a beam loaded with a concentrated load on the compression flange. It occurs when relative lateral displacement between the loaded compression flange and the tension flange is not restrained at the point of application of the load. Sidesway buckling can be prevented by providing lateral bracing to both the tension and compression flanges at the load point and preventing rotation of the compression flange. The lateral bracing at the flanges should each be designed for 1 percent of the concentrated load applied at that point. Alternatively, a pair of stiffeners may be used, designed to carry the full load, and extending from the load point to at least half the depth of the beam. Rotation of the compression flange must also be prevented.

Point Load on Beams: Sidesway web buckling



When the compression flange *is restrained* against rotation:

1. for $(h/t_w)/(L_b/b_f) \le 2.3$,

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1 + 0.4\left(\frac{h/t_w}{L_b/b_f}\right)^3\right]$$

যদি $\phi R_n <$ Point load, then we will use stiffener.

2. for $(h/t_w)/(L_b/b_f) > 2.3$,

$$R_n = \text{no limit}$$

When the compression flange is *not restrained* against rotation:

3. for $(h/t_w)/(L_b/b_f) \le 1.7$,

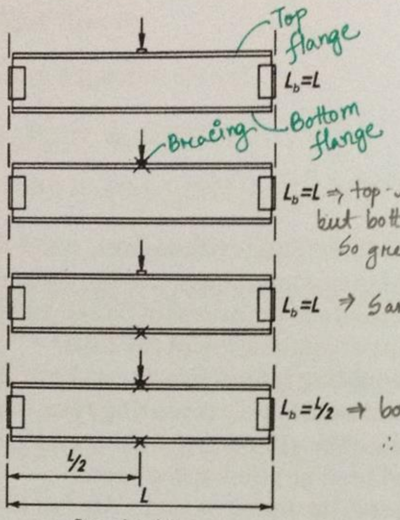
$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{L_b/b_f}\right)^3\right]$$

C_r = 960,000 ksi when $M_u < M_y$ at the location of force.
= 480,000 ksi when $M_u \ge M_y$ at the location of force.

4. for $(h/t_w)/(L_b/b_f) > 1.7$,

$$R_n = \text{no limit}$$

Point Load on Beams: Sidesway web buckling



× = Braced point

L_b = Largest laterally unbraced length along either flange at point of load.
(see AISC 360-05 Commentary Fig. C-J10.2)

$L_b = L \Rightarrow$ top - $L/2$
but bottom - $2L$
So greater is L & so $L_b = L$

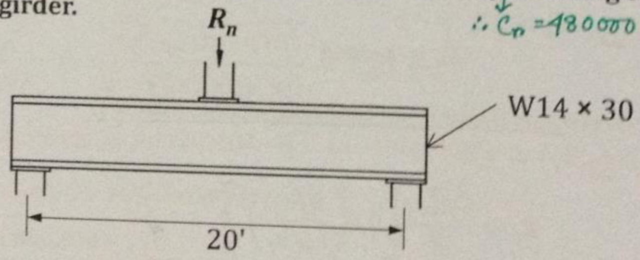
$L_b = L \Rightarrow$ Same as earlier

$L_b = L/2 \Rightarrow$ both flange unsupported length is $L/2$
$\therefore L_b = L/2$

Example: same as first example.

A W14 × 30 girder with a yield stress of 50 ksi is simply supported over a span of 20 ft and has a concentrated load applied at midspan. The top flange is restrained against rotation and is laterally braced at the ends and at the location of the load. It may be assumed that M_u and $1.5M_a$ exceed M_y at the location of the load. Determine the nominal web sidesway buckling strength of the W14 × 30 girder.

$\therefore C_r = 480000$ ksi



Solution:

From AISC manual, for W14 × 30 section, we find,
$t_f = 0.385"$, $h/t_w = 45.4$, $t_w = 0.27"$, $b_f = 6.73$

Now, $h = 45.4t_w = 45.4×0.27 = 12.26"$

$L_b = 20' = 240"$ (in accordance with AISC definition, see slide 19)

- Formula হতে comp. flange is restrained as top flange is comp. flange

Now, $(h/t_w)/(L_b/b_f) = 45.4/(240/6.73) = 1.273 < 2.3$

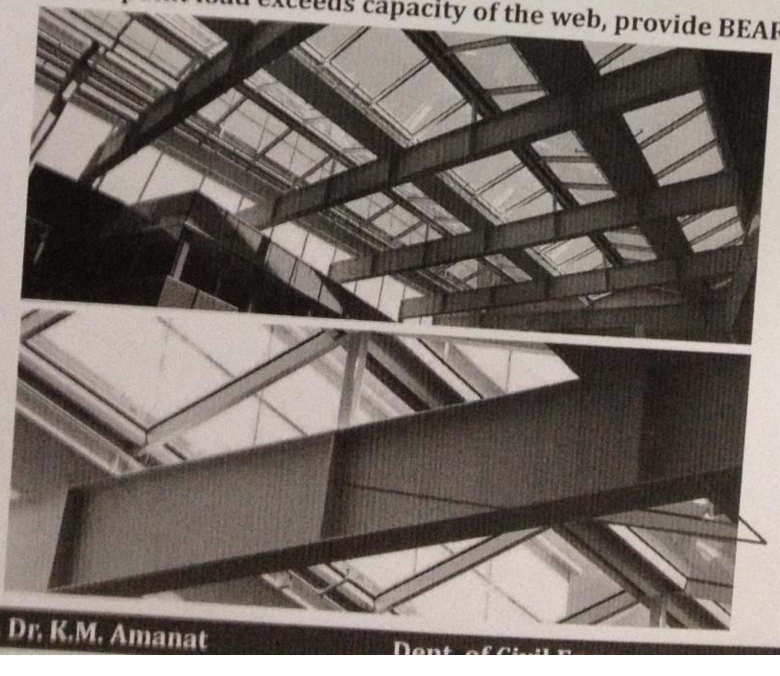
Therefore, $$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1 + 0.4\left(\frac{h/t_w}{L_b/b_f}\right)^3\right]$$

$\therefore R_n = (480000×0.27^3×0.385/12.26^2)[1+ 0.4(45.4/240/6.73)^3]$
$= 44.03$ kip

$\therefore \phi R_n = 0.85 \times 44.03^k = \underline{\underline{37.43^k}}$ ⇒ যদি design load > 37.43^k, then either
Ans.
- give stiffener or
- decrease L_b.

Point Load on Beams ✓

When point load exceeds capacity of the web, provide BEARING STIFFENER



Serviceability of Beams

- Design Limit States :-
  1. Strength Limit State
  2. Servicability " " ⟶ কতটুকু deflection হবে
  3. Other " "

☑ Serviceability, instead of strength, may and often does control the design of beams.

☑ Excessive deflection may cause damage to supported nonstructural elements such as partitions

☑ May impair the usefulness of the structure by, for instance, distorting door jambs so that doors will not open or close, or

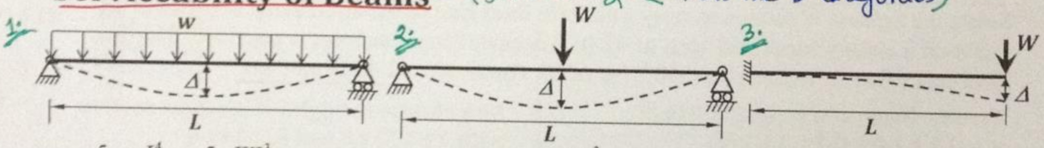
☑ May cause "bouncy" floors.

- In the above figure, ছাদে পানি জমেছে cz slab middle-এ deflect করেছে, তাই only middle- এই পানি জমে আছে।

These are serviceability problems, often unrelated to the strength of the floor system.

Serviceability of Beams

(formula মুখস্থ ⇒ all the 5 categories)



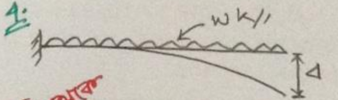
1. $\Delta = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384}\frac{WL^3}{EI}, \quad W = wL$

2. $\Delta = \frac{1}{48}\frac{WL^3}{EI}$

3. $\Delta = \frac{1}{3}\frac{WL^3}{EI}$

$$\Delta_{\max} = \beta_1 \frac{WL^3}{EI} \quad (7.6.1)$$

4.



(বই থেকে দেখবে must) Δ =

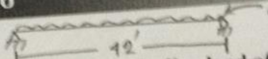
W = total *service* load on the span
L = span length
E = modulus of elasticity (29,000 ksi or 200,000 MPa for steel)
I = moment of inertia
β_1 = coefficient which depends upon the degree of fixity at supports, the variation in moment of inertia along the span, and the distribution of loading. (For a simply supported beam, $\beta_1 = 5/384$; other values are available in *AISC Manual* Tables 3–23, pp. 3–211 to 3–226)

$$\Delta_{\max} = \frac{}{48Ed}$$

For Live Load, $\Delta_{\max} \leq \frac{L}{360}$

Eq. 7.6.4, using $E = 29{,}000$ ksi becomes

$$\frac{L}{d} \leq \frac{48(29{,}000)}{10(360)f} = \frac{387}{f} \qquad (7.6.5)$$



0·5k/' DL, 1·0k/' LL

Laterally supported

Serviceability of Beams

Ex-1 Select the lightest W section to carry a uniform dead load of 0.5 kip/ft and a live load of 1.0 kip/ft on a simply supported span of 42 ft. Adequate lateral support is provided. The live load deflection is limited to L/360. Use A572 Grade 50 and ASD method.

$\Delta LL \leq \frac{L}{360}$, $F_y = 50$ ksi

Assume beam self weight 100 lb/ft.

$w = 0.5 + 1.0 + 0.1 = 1.6$ k/ft,

$M = wL^2/8 = 1.6 \times 42^2/8 = 352.8$ k-ft

Considering strength:

$M_a = \Omega M = 1.67 \times 352.8 = 589.2$ k-ft

$Z_x = M_a/F_y = 589.2 \times 12/50 = 141.4$ in^3.

Since we are using Z_x, we must select a compact section.

Considering *LL* Serviceability

$$\Delta = \frac{L}{360} = \frac{42 \times 12}{360} = 1.4 \text{ in.}$$

$$\text{Now, } \Delta = \frac{5}{384}\frac{wL^4}{EI}$$

$$\Rightarrow I = \frac{5}{384}\frac{wL^4}{E\Delta}$$

$$\Rightarrow I = \frac{5}{384}\frac{(1/12)\times(42\times12)^4}{29000\times1.4} = 1724.5 \text{ in}^4$$

Now choose the lightest *compact* section with $I \geq 1724.5$ and $Z_x \geq 141.4$

From AISC chart,

Chosen *compact* section: W21x93

(W24x68 is slender for F_y=50)

Check self-weight 93 lb/ft <100 OK.

Check shear:

$2.24\sqrt{(E/F_y)}$

$= 2.24\sqrt{(29000/50)} = 53.9$ ksi.

From chart: d=21.6, t_w = 0.58

$d/t_w = 21.6/.58 = 37.2 < 2.24\sqrt{(E/F_y)}$

$\therefore V_n = 0.6F_y d t_w$

$= 0.6\times50\times21.6\times0.58 = 375.8$ kip

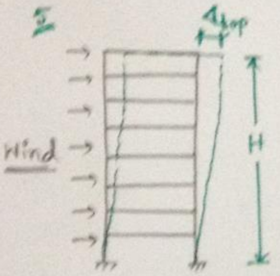
$\therefore V_n/\Omega = 375.8/1.5 = 250.53$ kip

But $V = wl/2 = 1.593\times42/2 = 33.5 < V_n/\Omega$ OK.

Final designed section: W21x93

Examples: 7.4.1, 7.6.1, 7.7.1, 7.7.2
Exercise: 7.1, 7.2, 7.3, 7.4, 7.5, 7.6, 7.7, 7.8, 7.9

Chapter 9:
Examples: 9.9.1, 9.9.2, 9.9.3, 9.9.4, 9.9.5, 9.9.6, 9.10.1, 9.10.2, 9.10.3
Exercise: 9.1 (omit cases 1, 2), 9.2, 9.3, 9.4, 9.5, 9.6, 9.7, 9.8, 9.9, 9.10, 9.12



According to Code -

$\Delta_{top} \leq H/500$

Example-1 (LRFD Method)

Assume, self weight = 50 #/'

$\therefore$ $w = 1.6 \times 1.0 + 1.2 \times (0.5 + 0.05) = 2.26$ k/'

$M_u = wL^2/8 = 2.26 \times 42^2/8 = 498.33$ k'

$M_n = \frac{M_u}{\phi} = \frac{498.33}{0.9} = 553.7$ k'

We will design compact section

$M_n = F_y Z_x \Rightarrow Z_x = \frac{553.7 \times 12}{50}$ in^3 $\Rightarrow Z_x = 132.89$ in^3

Now, $w_{LL} = 1$ k/' (As serviceability is always determined for unfactored load)

$\therefore M_{LL} = \frac{wL^2}{8} = \frac{1 \times 42^2}{8}$ k' $= 220.5$ k'

$$\therefore \Delta LL = \frac{5}{384} \cdot \frac{wL^4}{EI} = \frac{5}{384} \cdot \frac{220.5 \times 42^4}{29000 \times 144 \times I}$$

$$\Rightarrow I \geqslant 0.0831 \text{ ft}^4$$

or $I \geqslant 1724 \text{ in}^4$

From AISC Manual Charts, we choose a compact W section with

$Z_x \geqslant 132.9$ and $I_x \geqslant 1724$

We choose W 21 × 93

$I = 2070$, $Z = 221$, self wt 93 #/' > 50 #/'

Check Capacity : $w = 1.6 \times 1 + 1.2(.5 + .093)$

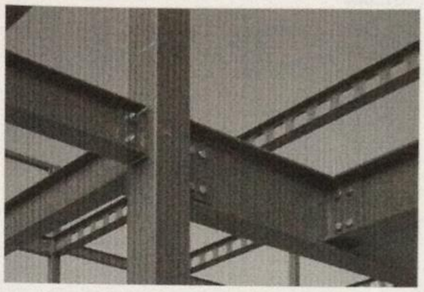
$= 2.312$ k/'

$$M_u = \frac{wL^2}{8} = \frac{2.312}{8} \times 42^2 = 509.71$$

Section capacity, $M_n = F_y Z_x = 50 \times 221 = 11050\ 5'' = 920$ k'

$\therefore$ Capacity, $M_u = \phi M_n = 0.9 \times 920 = 828$ k'

EXAMPLES



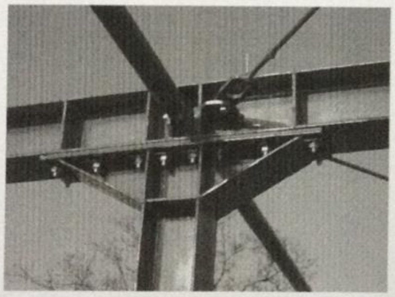
Simple framing joint

only transfers shear, no moment transmission

Top chord joint of steel truss

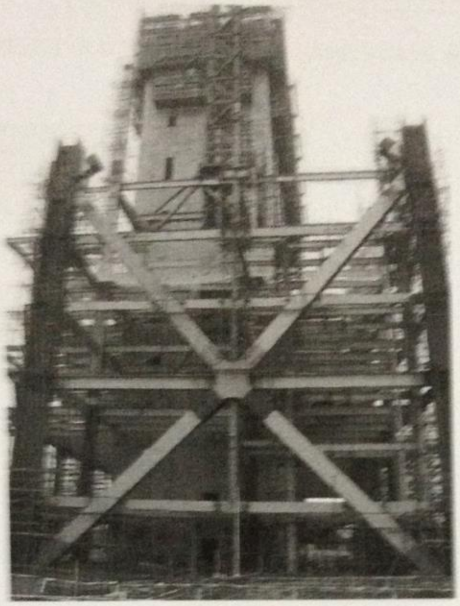
Rigid framing

EXAMPLES



Rigid framing joint

Column splice,
Simple framing



Bracing ⇒ Mega Bracing

S THE IMPORTANCE? *

critical part of steel structure design process.
east understood part of the structure.
ently the weakest part of structure.
probable location of occurrence of *stress*
ntration.
ing design procedures are generally not on the
of actual stress distribution.
nost affected part by fatigue stress.
ection failure has been regarded as the most
r cause of steel structure failure.

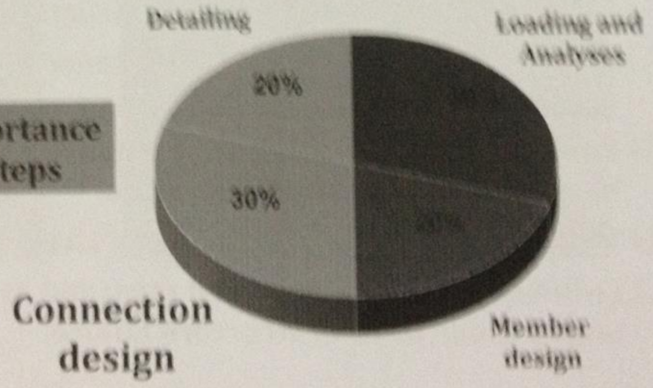
$r << a$

Connections - 1

STEPS IN DESIGN OF STEEL STRUCTURE

A. Estimation of loads, modeling and analyses.
B. Design of beams, columns and other main members.
C. Design of connections
D. Detailing.

Relative Importance Of Different Steps



STEEL SCULPTURE

University of
Alberta, Canada

A carefully designed steel structural form demonstrating various types of connections used in steel structures.

Also available at:
Minnesota State University, Mankato, USA
University of Wyoming, USA
University of Massachusetts at Amherst
and many others.

Steel Sculpture

CLASSIFICATION OF CONNECTIONS (AISC)

AISC TYPE-1. Fully Restrained (FR) Moment Connection (Rigid Framing:)
↳ Beam to column connection

- Full continuity is provided.
- Transfers both moment and shear.
- Original angles between intersecting members are held virtually constant.
- Rotational restraint of 90% or more is required.
- Used under both working and plastic design methods.

AISC TYPE-2. Simple Connections (Simple Framing:) → only shear

- Rotational restraint at member ends is negligible.
- Shear transfer only.
- Change in angle between members can be as high as 80% of the amount it would theoretically change if frictionless hinged connection could be used.
- Used under working design only.

AISC TYPE-3. Partially-rigid (PR) Connections (Semi-rigid Framing): → fully shear & partially moment transmission

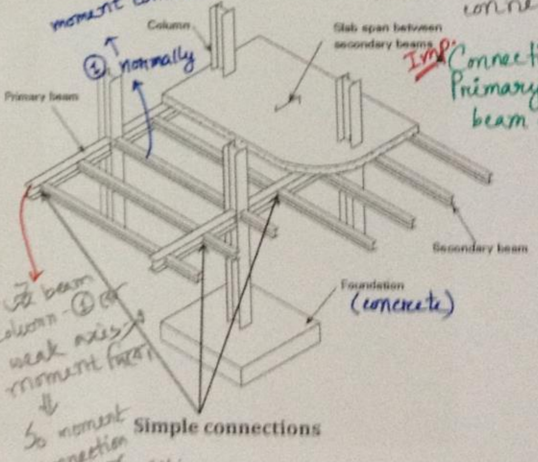
[illegible] syllabus

- Rotational restraint is between 20 to 90%
- Full shear transfer and partial moment transfer.

SIMPLE CONNECTIONS (Simple Shear Connections)

→ এই beam column কে strong axis-এ bending দিবে। So এটা Moment connection

- Simple shear connections (commonly known as framed beam connections), and now referred to as "Simple Connections" (AISC-B3.6), are used to connect beams to other beams or to column flanges or webs when simple support of the beam has been assumed.
- Design of such connections has become somewhat standardized using Part 10 of the AISC Manual, which provides the design methods and tables for Simple Shear Connections



moment connection

① normally

(concrete)

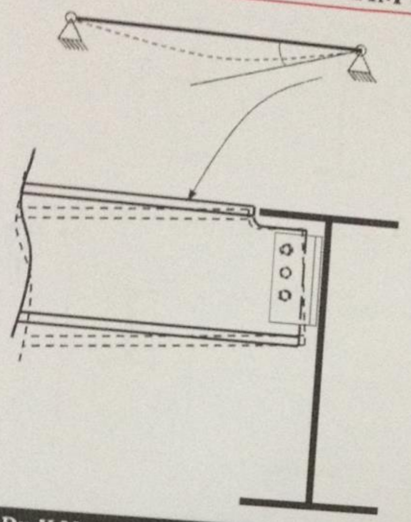
Imp. Connection with Primary & Secondary beam → is simple connection always

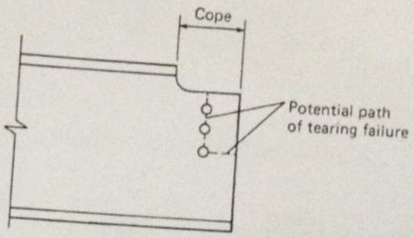
এই beam column-① কে weak axis-এ moment দিবে।
⇓
So moment connection হবে না fully.

SIMPLE FRAMED BEAM CONNECTION (only web to web connection)

Imp.

↓ for shear.





Imp. { ⊛ Shear connection always থাকবেই । i.e. web connection হবেই । Then if moment connection needed, then flange → flange connection is introduced.

SIMPLE CONNECTIONS (Simple Shear Connections)

↳ block Shear failure is critical here.

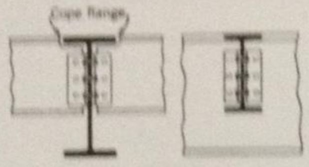
AISC TYEPS:

1. All Bolted Double-Angle Connections
2. Bolted/Welded Double-Angle Connections
3. All-Welded Double-Angle Connections
4. Bolted/Welded Shear End-Plate Connections
5. All-Bolted Unstiffened Seated Connections
6. All-Welded Unstiffened Seated Connections
7. All-Bolted Stiffened Seated Connections
8. Bolted/Welded Stiffened Seated Connections

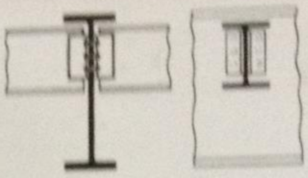
AISC Simple Connections

Simple Connection

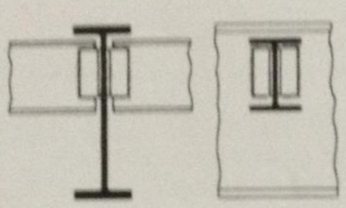
1. All Bolted Double-Angle



2. Bolted/Welded Double-Angle

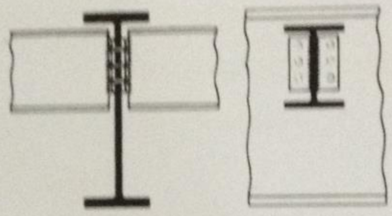


3. All-Welded Double-Angle



4. Bolted/Welded Shear End-Plate

→ angle-এর বদলে plate use করা হয়

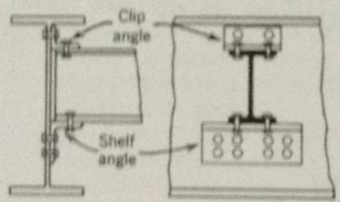


AISC Simple Connections

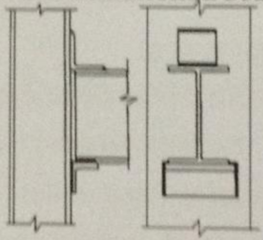
seat কে আলাদাভাবে strong করা হয় না।

Seated Connection

5. All-Bolted Unstiffened Seated

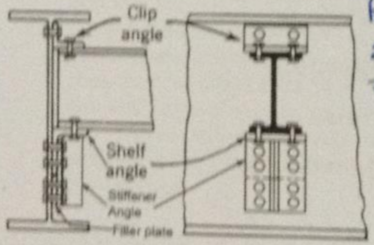


6. All-Welded Unstiffened Seated

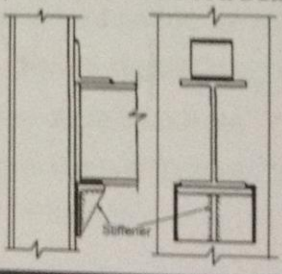


7. All-Bolted Stiffened Seated

→ seat কে different way তে stiffener দিয়ে strong করা হয়।



8. Bolted/Welded Stiffened Seated

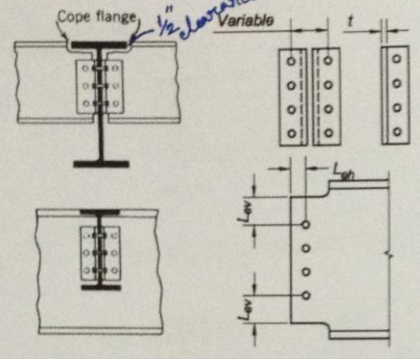


DESIGN OF SIMPLE CONNECTIONS USING AISC MANUAL

Part 10 of AISC Manual contains necessary design tables for easy design and ready reference for Simple Connections.

by AISC Manual, Part -10

All Bolted Double Angle



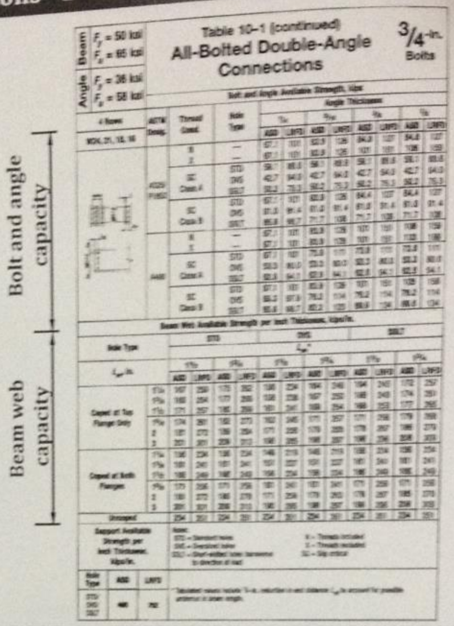


Table 10-1 (continued)
All-Bolted Double-Angle Connections
$^3/_4$-in. Bolts

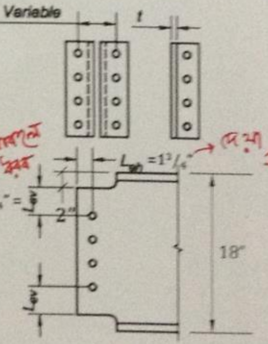
Beam F_y = 50 ksi, F_u = 65 ksi
Angle F_y = 36 ksi, F_u = 58 ksi

All Bolted Double Angle

Select an all-bolted double-angle connection between a W18×50 beam and a W21×62 girder web to support the following beam end reactions: R_D = 30 kips, R_L = 40 kips. The beam top flange is coped 2-in. deep by 4-in. long, $L_{ev} = 1^1/_4$ in., $L_{eh} = 1^3/_4$ in. Use $^3/_4$-in. diameter ASTM A325-N bolts in standard holes. Beams are A572 Grade 50 material.

→ secondary beam এর size বলে

Solution:



না দেয়া থাকলে min^m ধরব ≈$1^1/_4$"

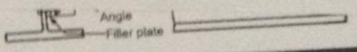
→ দেয়া না থাকলে 1.5"–2.0" এর মধ্যে assume করব

From AISC Manual Table 1-1, for W18×50, total depth d = 18", web thickness t_w = 0.355", t_f = 0.57", k = 0.97"

Girder W21×62: depth is 21". Beam is coped, i.e, beam top and girder top are at same level. Thus beam bottom shall be 21-18 = 3" above girder bottom. Thus beam bottom need not be coped.

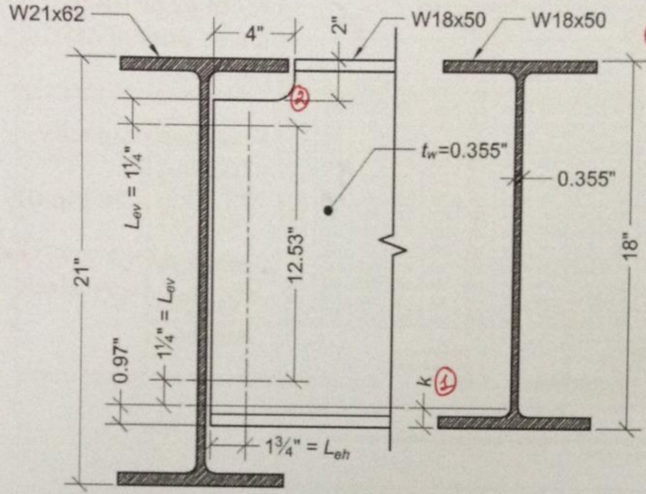
Space available for bolts, 18 -2-2(1.25)-0.97 = 12.53"

Assuming that bolt spacing = 3" (>3d = 3×$^3/_4$ = 2.25"), maximum number of bolts that may be accommodated in 12.53 inch space is 12.53/3 + 1 = 5.17 → 5 bolts.



All Bolted Double Angle

Solution (contd....):



② * Coping থাকলে, coping থেকে L_{ev} ধরব

① * Coping না থাকলে, K থেকে L_{ev} ধরব

All Bolted Double Angle

Solution (contd....):

- Normal sizes of bolts used – $\frac{5}{8}''$ to $\frac{10}{8}''$

Following ASD principle, total load to be transferred = $R_D + R_L = 30 + 40 = 70$ kip.

Allowable bolt shear capacity: কোনো dia দেয়া থাকে range এর মধ্যে যদি বলা না থাকে

A325-N bolts ⇒ Threads included in shear plane. ← Normally এটাই use করব।

$R_n = mF_vA_b/\Omega = 2.0(0.4\times120)[^{\pi}/_4(^3/_4)^2]/2=21.2$ kip

Number of bolts required: $70/21.2 = 3.3 < 5$ OK.

double shear

Provided $L_{eh} = 1.75''$, thus angle size shall be $1.75\times2 = 3.5''$. ⇒ (cz normally angle এর middle এই দেয়া হয়।)

For additional safety for beam web capacity, we consider $L_{eh} = 1.50''$

70 kip load is to be transferred by 0.355 inch thick web. Thus required web capacity is = 70/.355 = *197.2 kips per inch thickness*.

Considering web capacity, we choose page 10-20 of AISC Manual with *5-Rows of bolts* → 4 rows দিলেও হয়, but for safety we will give 5 rows।

All Bolted Double Angle

Solution (contd….):

→standard hole
→oversized hole
→slotted hole

Imp. minm 1.25
⇓
যদি satisfy না করে তাহলে Lev বাড়াতে হবে। Then bolt-3 সংখ্যা কমে যেতে পারে।
⇓
Leh-ও বাড়াতে হতে পারে যদি satisfy না করে।

lower part of the chart

Beam Web Available Strength per Inch Thickness, kips/in.													
Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1½		1¾		1½		1¾		1½		1¾	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	208	312	216	324	195	293	203	305	205	307	213	320
	1⅜	210	316	219	328	197	296	206	308	207	311	216	323
	1½	213	319	221	332	200	300	208	312	210	315	218	327
	1⅝	215	323	223	335	202	303	210	316	212	318	220	331
	2	223	334	231	346	210	314	218	327	220	329	228	342
	3	242	363	250	375	229	344	237	356	239	359	247	371
Coped at Both Flanges	1¼	197	296	197	296	185	278	185	278	197	296	197	296
	1⅜	202	303	202	303	190	285	190	285	202	303	202	303
	1½	207	311	207	311	195	293	195	293	207	311	207	311
	1⅝	212	318	212	318	200	300	200	300	212	318	212	318
	2	223	334	227	340	210	314	215	322	220	329	227	340
	3	242	363	250	375	229	344	237	356	239	359	247	371
Uncoped		293	439	293	439	293	439	293	439	293	439	293	439

Support Available Strength per Inch Thickness, kips/in.		
Hole Type	ASD	LRFD
STD/ OVS/ SSLT	585	878

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include ¼-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

From page 10-20 *lower part* of AISC Manual,

Beam web capacity
=208×0.355
=73.8 kip > 70 kip OK

- যত right-এ যাব chart-এর, তত beam-এর block shear capacity বাড়বে।

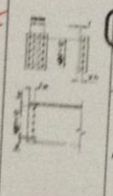
All Bolted Double Angle

Solution (contd….):

Upper Part of the chart

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections
¾-in. Bolts

5 Rows				Bolt and Angle Available Strength, kips							
W30, 27, 24, 21, 18	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness							
				¼		5/16		⅜		½	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	83.3	125	104	156	106	159	106	159
		X	—	83.3	125	104	156	125	187	133	199
		SC Class A	STD	73.8	111	73.8	111	73.8	111	73.8	111
			OVS	53.3	80.0	53.3	80.0	53.3	80.0	53.3	80.0
			SSLT	62.8	94.1	62.8	94.1	62.8	94.1	62.8	94.1
		SC Class B	STD	83.3	125	104	156	105	158	105	158
			OVS	76.2	114	76.2	114	76.2	114	76.2	114
			SSLT	82.0	123	89.6	134	89.6	134	89.6	134
	A490	N	—	83.3	125	104	156	125	187	133	199
		X	—	83.3	125	104	156	125	187	166	249
		SC Class A	STD	83.3	125	92.3	138	92.3	138	92.3	138
			OVS	66.7	100	66.7	100	66.7	100	66.7	100
			SSLT	78.4	118	78.4	118	78.4	118	78.4	118
		SC Class B	STD	83.3	125	104	156	125	187	132	198
			OVS	82.4	124	95.2	143	95.2	143	95.2	143
			SSLT	82.0	123	102	154	112	168	112	168

From page 10-20 upper part of AISC Manual,

Thickness of the angle shall be $^1/_4$"

Chosen angle

L 3.5 × 3.5 ×$^1/_4$

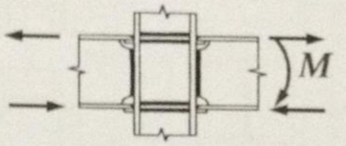
F_y = 36 ksi (A36)

N = thread is included in shear transmission
X = thread is exclude
SC = Slip Critical ⇒ (we won't use now)

Rigid Connections:

CONTINUOUS BEAM-TO-COLUMN CONNECTIONS

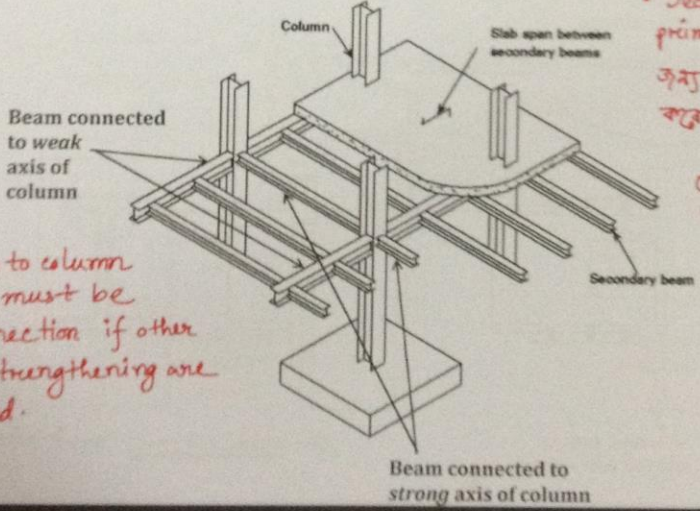
- ☐ In continuous beam-to-column connections, it is the design intent to have full transfer of moment and little or no relative rotation of members within the joint [i.e., AISC-Fully Restrained Moment Connections (FR) - rigid-frame connections].



- ☐ Since the flanges of a beam carry most of the bending moment via tension and compression flange forces acting at a moment arm approximately equal to the beam depth, it is the transfer of these essentially axial forces for which provision must be made.
- ☐ Since the shear is carried primarily by the web of a beam, full continuity requires that it be transferred directly from the web.

Rigid Connections:

CONTINUOUS BEAM-TO-COLUMN CONNECTIONS



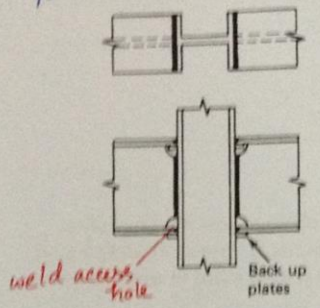
- Secondary beam primary beam-এর জন্য torsion create করে।
↓
যেন এটা না হয় তাই simply supported রাখা হয়/ shear connection

- But beam to column connection must be Moment Connection if other ways of strengthening are not adopted.

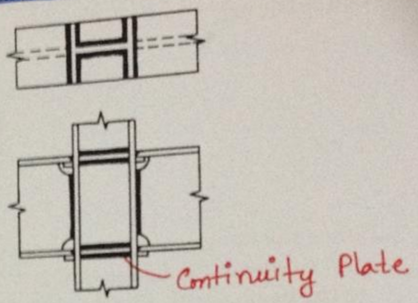
Rigid Connections:

CONTINUOUS BEAM-TO-COLUMN CONNECTIONS

- Beam directly welded to column
- Beam transfers moment to strong axis of column



(a) No column stiffeners; direct flange to flange groove welds



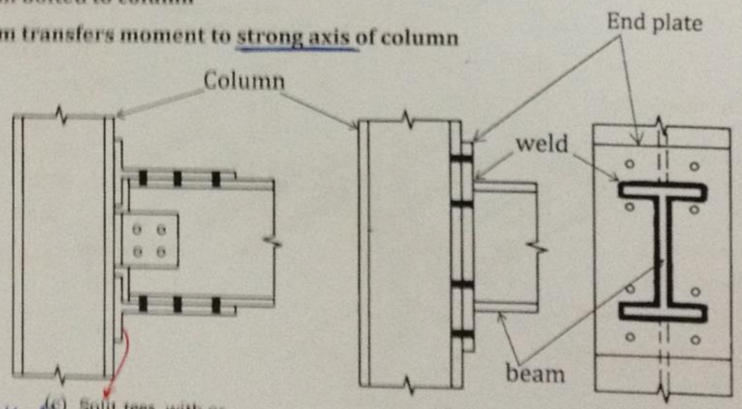
(b) Horizontal plate stiffeners

(all welded)

Rigid Connections:

CONTINUOUS BEAM-TO-COLUMN CONNECTIONS

- Beam bolted to column
- Beam transfers moment to strong axis of column



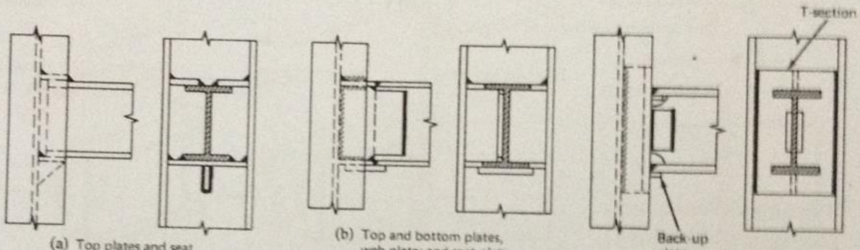
(all bolted) (c) Split tees, with or without web angles

(d) Welded end plates (most popular)

Rigid Connections:

CONTINUOUS BEAM-TO-COLUMN CONNECTIONS

- Beam welded to column
- Beam transfers moment to **Weak Axis** of column



(a) Top plates and seat bracket

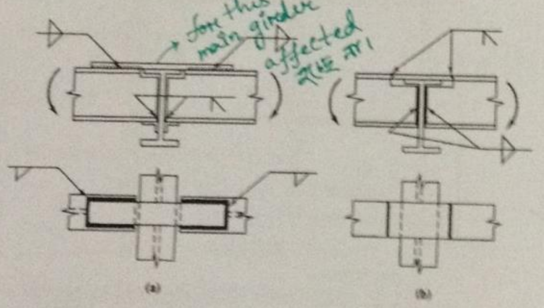
(b) Top and bottom plates, web plate; and seat plate

(c) Tee section with flange back-up plates, and small web plates

- Flange to Flange connection must ⇒ for moment transfer

Rigid Connections:

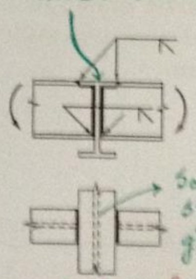
CONTINUOUS BEAM-TO-BEAM CONNECTIONS



for this main girder affected হচ্ছে না।

Tension flanges not attached to each other (a and b).

- main girder এর উপর দিয়ে moment transfer হচ্ছে



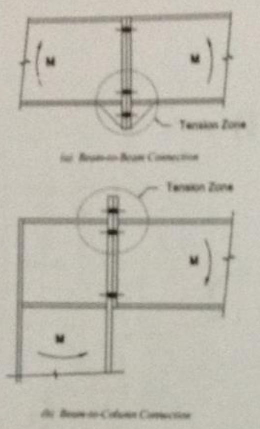
So এখানে biaxial stress create হবে girder will become weaker.

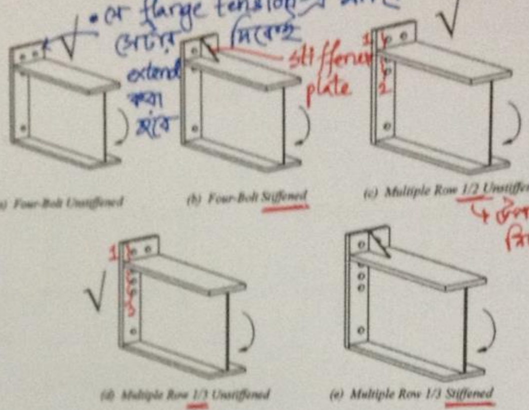
(not preferable always)

Tension flanges attached to each other.

Extended end-plate moment connection

Non-seismic application





- যে flange tension-এ থাকবে সেটার দিকেই extend করা হবে

stiffener plate

ফ্ল্যাঞ্জের ১টি row
নিচে দুইটি row

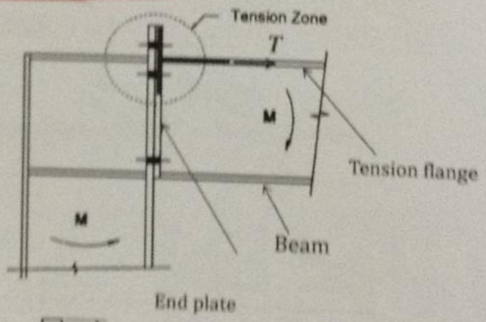
- যদি both flange-এই tension হয় (for seismic load) তাহলে both side extended হবে।

* Parameters of end plate connection →
1. Plate thickness
2. Bolts no.

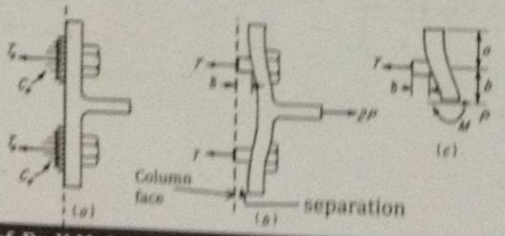
Extended end-plate moment connection

Non-seismic application

Behavior of endplate in tension zone under flexure.



Thick endplate behavior

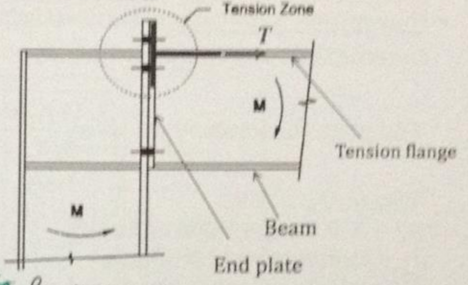


Endplate is thick and rigid enough to cause contact separation by elongation of bolt.

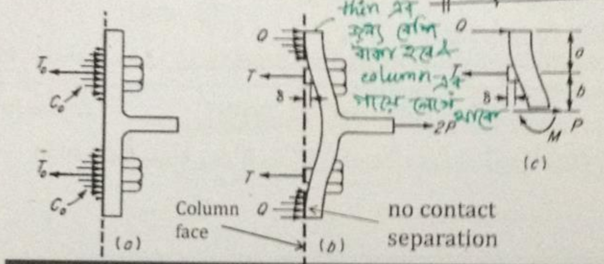
Extended end-plate moment connection

Non-seismic application

Behavior of endplate in tension zone under flexure.



Thin endplate behavior



thin এর জন্য বেশি বাঁকা হবে & column এর সাথে লেগে থাকে

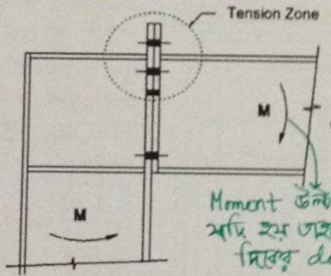
Endplate is thin and undergoes bending. This causes development of $\boxed{\text{prying force } Q.}$

cz contact separation middle এ হবে but দুই end এ হবে না।

For Prying Force, this will be indeterminate

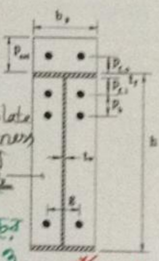
Extended end-plate moment connection

Non-seismic application: Thick endplate behavior (LRFD Design)

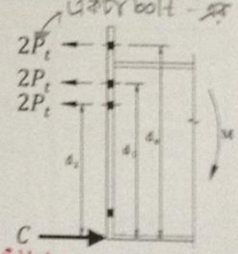


- end plate thickness

Moment উল্টোদিকে যদি হয় তাহলে নিচের দিকের design ও same way করতে হবে



একটা bolt-এর force



This figure will be given in exam (This is for 1/2 unstiffened)

Determination of bolt diameter

Moment equilibrium:

Internal Moment = $2P_td_0 + 2P_td_1 + 2P_td_2 = M_u$ external Moment

$\Rightarrow 2P_t\Sigma d_n = M_u$

$\Rightarrow 2(\phi\frac{\pi}{4}d_b{}^2F_t)\,\Sigma d_n = M_u$

for LRFD Design

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

- Assumptions →
  1. Bolt dia same
  2. Bolt force after elongation same

Extended end-plate moment connection

Non-seismic application: Thick endplate behavior

Determination of endplate thickness

$$\text{End plate thickness, } t_p = \sqrt{\frac{1.11\gamma_r \overset{0.75}{\phi} M_{np}}{\underset{0.9}{\phi_b} F_{py} Y}}$$

Where, $\phi_b = 0.9$,
$\gamma_r = 1.0$ for extended end plate,
F_{py}= Endplate yield strength,
Y = yield line mechanism parameter,
ϕM_{np} = connection strength based on bolt tension limit state.
$= \phi\,[2P_t\Sigma d_n]$, where $P_t = \frac{\pi}{4} d_b{}^2 F_t$ and $\phi = 0.75$
যে dia just বের করলাম
and F_t = 90 ksi for A325 and 113 ksi for A490 bolts.
ASTM Bolts diameters are: $^1/_2$, $^5/_8$, $^3/_4$, $^7/_8$, 1, $1^1/_8$, $1^1/_4$, $1^3/_8$, $1\,^1/_2$ inch

- যে Moment (M_u) first-এ use করে dia বের করলাম ওটাই (M_u) use করা যাবে না cz dia rounding করে একটি particular dia select করা হবে।

⇓

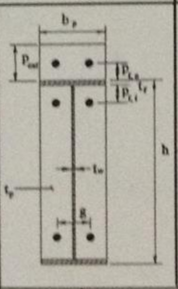
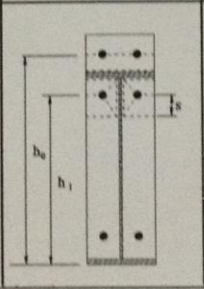
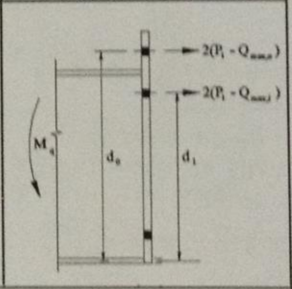
- So actual Moment (M_{np}) will be a bit different from M_u.

Extended end-plate moment connection

Non-seismic application: Thick endplate behavior

Y = yield line mechanism parameter: Four bolt unstiffened

chart will be given

	Geometry	Yield-Line Mechanism	Bolt Force Model
			
End-Plate Yield	$\phi M_n = \phi_b M_{pl} = \phi_b F_{py} t_p^2 Y$ $Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}}+\frac{1}{s}\right)+h_0\left(\frac{1}{p_{f,o}}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1\left(p_{f,i}+s\right)\right]$ Note: Use $p_{f,i} = s$, if $p_{f,i} > s$ $s = \frac{1}{2}\sqrt{b_p g}$ $\phi_b = 0.90$		

These eqns will be given

- no. of bolt-এর জন্য first-এ 4 bolt or simplest form assume করে $\frac{25}{10}$ এর Moment equilibrium এর formula-র first দুটো part দিয়ে again bolt dia বের করে check করব।

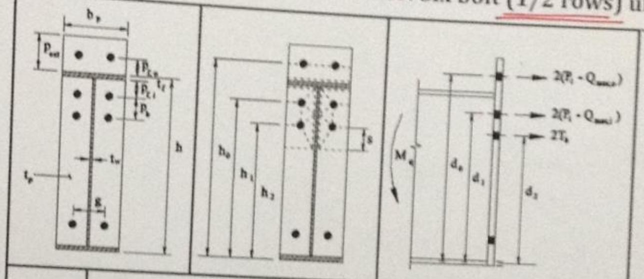
⇓

If not satisfied, we will go for 1/2 or 1/3 rows.

Extended end-plate moment connection

Non-seismic application: Thick endplate behavior

Y = yield line mechanism parameter: Six bolt (1/2 rows) unstiffened

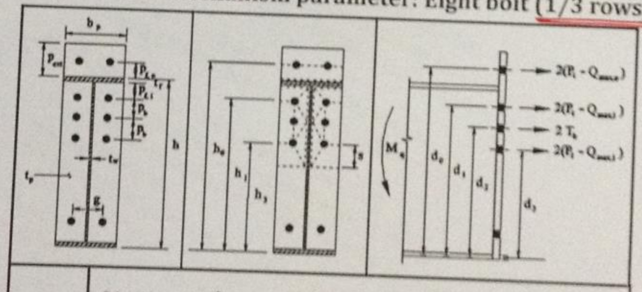


End-Plate Yield	$\phi M_n = \phi_b M_{pl} = \phi_b F_{py} t_p^2 Y$ $Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}}\right) + h_2\left(\frac{1}{s}\right) + h_0\left(\frac{1}{p_{f,o}}\right) - \frac{1}{2}\right] + \frac{2}{g}\left[h_1\left(p_{f,i} + 0.75p_b\right) + h_2\left(s + 0.25p_b\right)\right] + \frac{g}{2}$ $s = \frac{1}{2}\sqrt{b_p g}$ $\phi_b = 0.90$ Note: Use $p_{fi} = s$, if $p_{fi} > s$

Extended end-plate moment connection

Non-seismic application: Thick endplate behavior

Y = yield line mechanism parameter: Eight bolt (1/3 rows) unstiffened



End-Plate Yield	$\phi M_n = \phi_b M_{pl} = \phi_b F_{py} t_p^2 Y$ $Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}}\right) + h_3\left(\frac{1}{s}\right) + h_0\left(\frac{1}{p_{f,o}}\right) - \frac{1}{2}\right] + \frac{2}{g}\left[h_1\left(p_{f,i} + 1.5p_b\right) + h_3\left(s + 0.5p_b\right)\right] + \frac{g}{2}$ $s = \frac{1}{2}\sqrt{b_p g}$ $\phi_b = 0.90$ Note: Use $p_{fi} = s$, if $p_{fi} > s$

Lect -27

Grout → দেয়া হয় যে column থেকে load uniformly এর concrete-এ যায়। যে concrete surface কখনও 100% smooth হয় না। So steel এর সাথে সব জায়গায় সমানভাবে contact-এ থাকে না।

⇓

Grout strength $\geq$ base concrete

⇓

semisolid mix

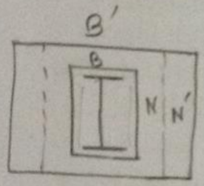
only sand + cement + water

↓

(or very fine aggregate)

- Design considerations
  - Axial tension (uplift) ⇒ specially in towers

27/9



B', N' is similar to B, N & the two areas are concentric 〉 এদের (2 conditions) match করে যত বড় area নেয়া যায়

& $\sqrt{\frac{A_2}{A_1}} \not> 2.0$ & $\not< 1.0$

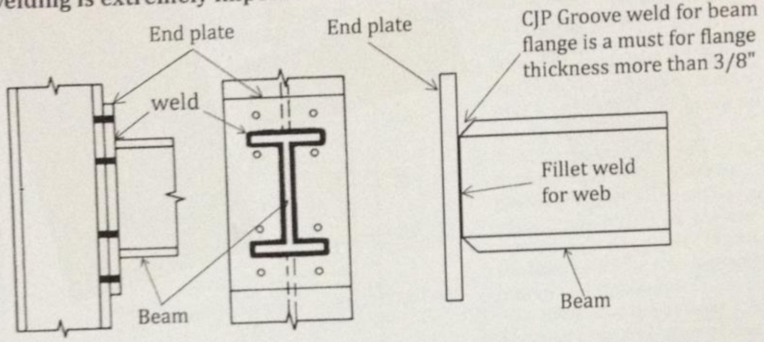
27/11 Example :

21/10 Beam Bearing : (Same as 27/11)

Extended end-plate moment connection

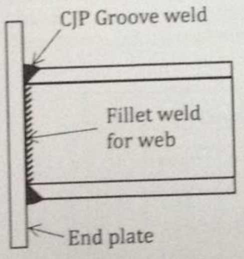
Welding of Endplate with Beam

Beam forces are transferred to the endplate through welding, which, in turn, transfers the forces to column through bolts. Therefore, proper welding is extremely important for an endplate connection.



Extended end-plate moment connection

Welding of Endplate with Beam



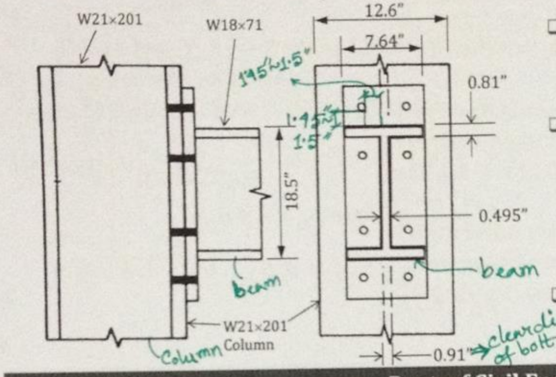
- ❑ Flange to endplate connection shall be CJP groove weld for most cases. Beam web to end plate connection can be fillet type on both sides.
- ❑ All welds shall be in accordance with American Welding Society standards, AWS D1.1
- ❑ Welding electrodes must meet AISC requirement: Charpy V-notch impact value shall not be less than 20 ft-lb at minus 20°F for the weld metal.
- ❑ Beam web shall be welded first to the endplate before welding the flanges.
- ❑ There shall not be any weld access hole on the beam web.

Extended end-plate moment connection

Non-seismic application: Extended unstiffened moment connection

Example:

A W18×71 beam (A572 Gr. 50) has to transfer 75 k-ft dead load and 140 k-ft live load moment on to a W21×201 (A572 Gr. 50)column on its strong axis through an extended end plate type connection. Suitable dimension the end plate and determine the bolt diameter and thickness of end plate (A572 Gr. 50). Use ASTM A325 bolts.



- ☐ Dimensions of W18×71 and W21×201 are obtained from AISC manual and are shown on the fig. at left.
- ☐ Initially assume two-row 4-bolt configuration. Bolts shall be placed close to the beam flange and column web in such a way that enough clear space around the bolt exists for it to be easily placed and tightened.
- ☐ Also there shall be at least $1.5d_b$ distance from free edge of end plate to bolt centre.

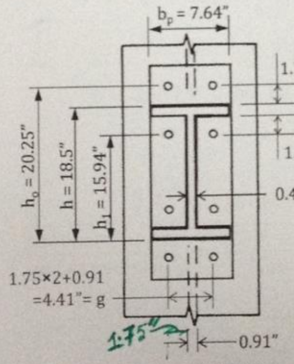
[illegible] obviously greater thickness of web (beam/column) [illegible]

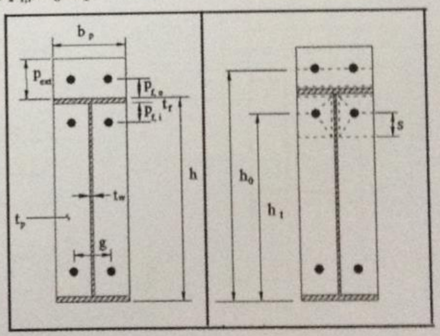
Extended end-plate moment connection

Non-seismic application: Extended unstiffened moment connection

Example contd...

Assume that bolts are about 1.75 inch away from beam flange as well as column/beam web Then we can calculate parameters like g, $p_{f,o}$, $p_{f,i}$, h_o, h_1 etc.



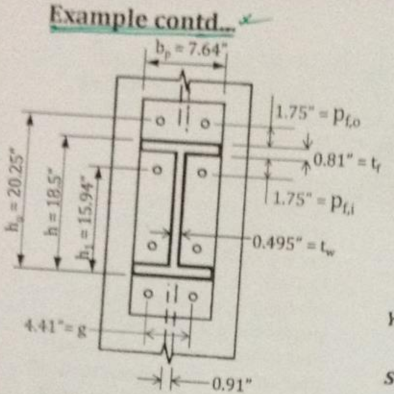


Also, $d_o = h_o - t_f/2 = 19.845"$, $d_1 = h_1 - t_f/2 = 15.535"$

Extended end-plate moment connection

Non-seismic application: Extended unstiffened moment connection

Example contd...



$d_o = 19.845"$
$d_1 = 15.535"$

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

Now, $M_u = 1.2(75) + 1.6(130) = 298$ k-ft $= 3576$ k-in

$$d_b = \sqrt{\frac{2M_u}{\pi\phi F_t \sum d_n}} = \sqrt{\frac{2\times3576}{\pi(.75)90(35.38)}} = 0.976 \approx \underline{1.0"}$$

End plate thickness, $t_p = \sqrt{\dfrac{1.11\gamma_r\phi M_{np}}{\phi_b F_{py} Y}}$

$\phi M_{np} = \phi[2P_t\Sigma d_n]$, where $P_t = \frac{\pi}{4}d_b^2F_t$ and $\phi = 0.75$

Now, $P_t = \frac{\pi}{4}d_b^2F_t = 0.7854(1)^2(90) = 70.69$ kip

$\phi M_{np} = \phi[2P_t\Sigma d_n] = 0.75[2\times70.69(35.38)] = 3751.5$ k-in.

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}}+\frac{1}{s}\right)+h_0\left(\frac{1}{p_{f,o}}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1(p_{f,i}+s)\right]$$

$s = \frac{1}{2}\sqrt{b_p g}$; use $p_{f,i} = s$ if $p_{f,i} > s$

Now, $s = \frac{1}{2}\sqrt{b_p g} = 0.5\sqrt{7.64\times4.41} = 2.9$

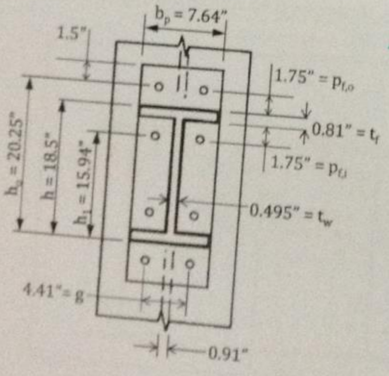
$\therefore p_{f,i} = 1.75$

এখানে যদি 2" আসত তাহলে either bolt clear distance on plate configuration বাড়াব / row of bolts বাড়াব
⇓
Later is easier.
So row বাড়াব in most cases.

Extended end-plate moment connection

Non-seismic application: Extended unstiffened moment connection

Example contd...



$$\underline{Y} = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}}+\frac{1}{s}\right)+h_0\left(\frac{1}{p_{f,o}}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1(p_{f,i}+s)\right]$$

$$= \frac{7.64}{2}\left[15.94\left(\frac{1}{1.75}+\frac{1}{2.9}\right)+20.25\left(\frac{1}{1.75}\right)-\frac{1}{2}\right] + \frac{2}{4.41}\left[15.94(1.75+2.9)\right]$$

$= 3.82\times25.67+33.61$

$= \underline{131.66}$

For extended endplate, $\gamma_r = 1.0$

$$\text{Now, } t_p = \sqrt{\frac{1.11\gamma_r\phi M_{np}}{\phi_b F_{py} Y}} = \sqrt{\frac{1.11\times1.0\times3751.5}{0.9\times50\times131.66}}$$

$$= 0.84" \approx \underline{\frac{7}{8}"}$$

↳ should be converted in this form

way
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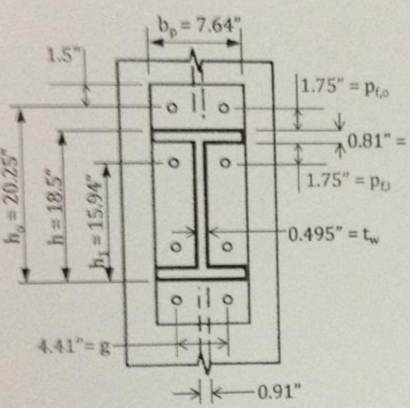
Extended end-plate moment connection

Non-seismic application: Extended unstiffened moment connection

Example contd...



Dimensional and other checks:

We choose the 1.5" ($1.5d_b$) extension of endplate beyond the outer bolts giving the total size of endplate 7.64"×25", t_p=7/8" with 1" dia. A325 bolts.

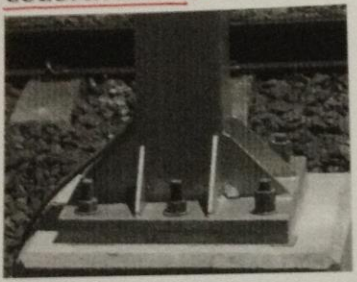
Horizontal distance of bolt center from vertical edge of endplate =(7.64-4.41)/2=1.62" > $1.5d_b$ OK

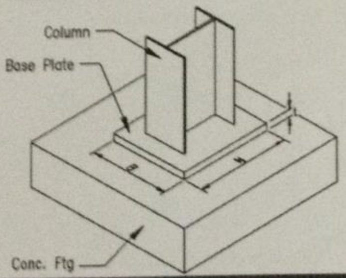
Connection design items not covered:

- ☐ Beam shear
- ☐ Column web crippling

- Grout ⇒ দেয়া হয় যে column থেকে load যেন uniformly concrete-এ যায়। concrete surface কখনও 100% smooth হয় না। S. steel এর সাথে সব জায়গায় সমানভাবে contact-এ থাকে না।

COLUMN BASE





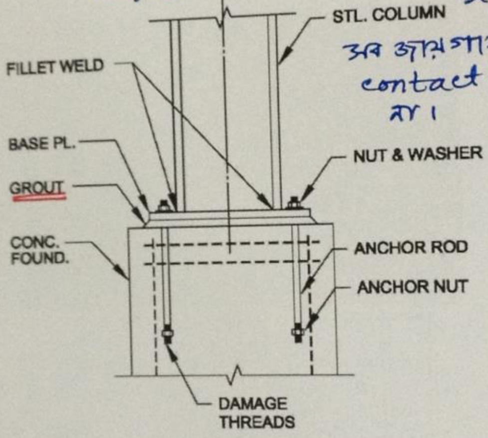
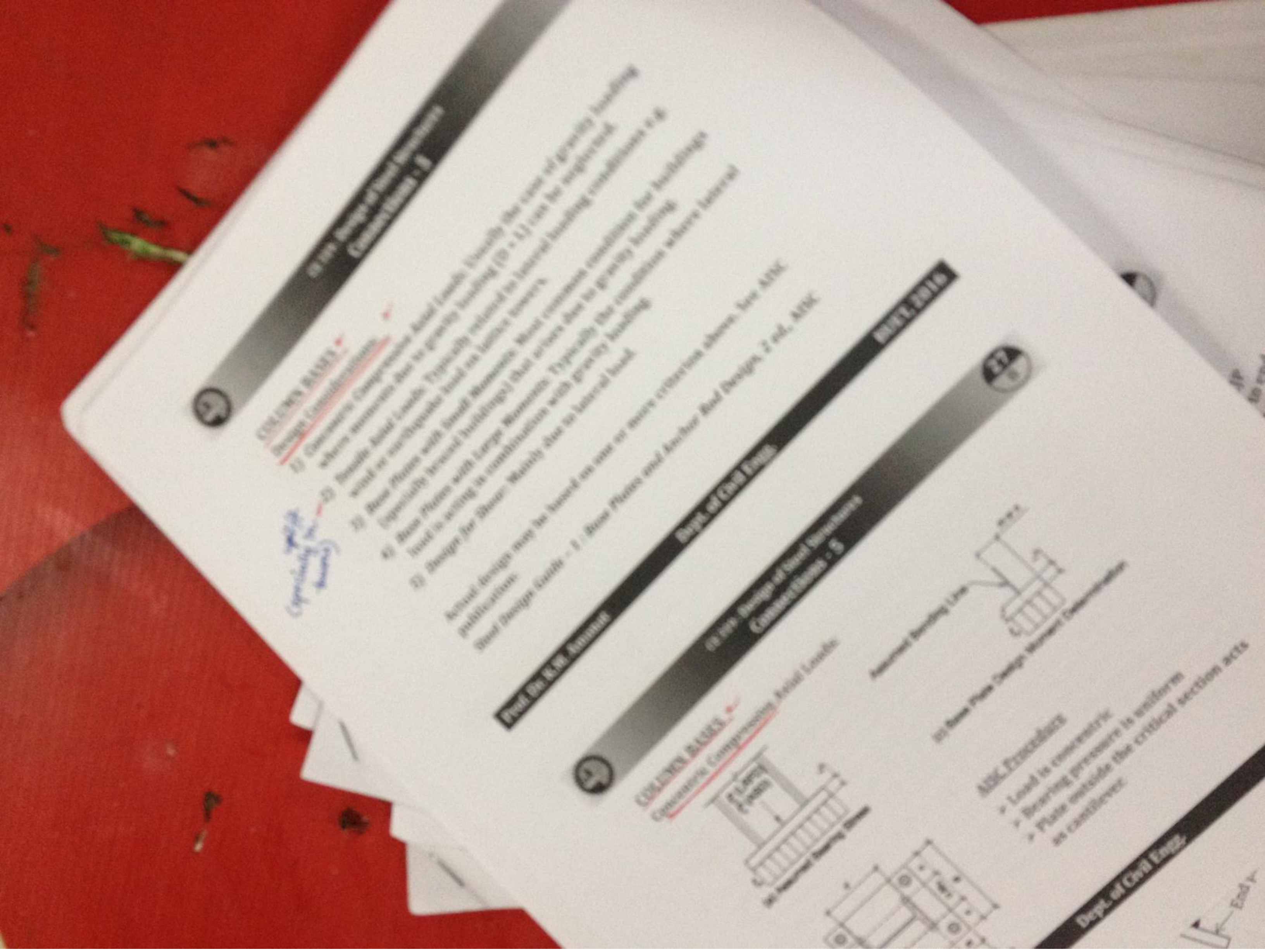


Figure 1.1. Column base connection components.

- Grout Strength ≥ base concrete
  ↓
  Semisolid mix (only sand + cement + water)
  ↓
  or very fine aggregate

COLUMN BASE

- ☐ Column base plate connections are the critical interface between the steel structure and the foundation. These connections are used in buildings to support gravity loads and function as part of lateral-load-resisting systems.
- ☐ In addition, they are used for mounting of equipment and in outdoor support structures, where they may be affected by vibration and fatigue due to wind loads.
- ☐ Base plates and anchor rods are often the last structural steel items to be designed but are the first items required on the jobsite.



CE 319: Design of Steel Structures
Connections - 8

COLUMN BASES

Design Considerations:

1) Concentric Compressive Axial Loads: Usually the case of gravity loading where moments due to gravity loading (D + L) can be neglected.
2) Tensile Axial Loads: Typically related to lateral loading conditions (e.g. wind or earthquake) load on lattice towers.
3) *Base Plates with Small Moments*: Most common condition for buildings (especially braced buildings) that arises due to gravity loading.
4) *Base Plates with Large Moments*: Typically the condition where lateral load is acting in combination with gravity loading.
5) *Design for Shear*: Mainly due to lateral load.

(specially for [illegible])

Actual design may be based on one or more criterion above. See AISC publication:
Steel Design Guide - 1: Base Plates and Anchor Rod Design, 2 ed., AISC

Prof. Dr. K.M. Amanat | Dept. of Civil Engg. | BUET, 2016

CE 319: Design of Steel Structures
Connections - 5

27

COLUMN BASES

Concentric Compressive Axial Loads:

AISC Procedure

- Load is concentric
- Bearing pressure is uniform
- Plate outside the critical section acts as cantilever

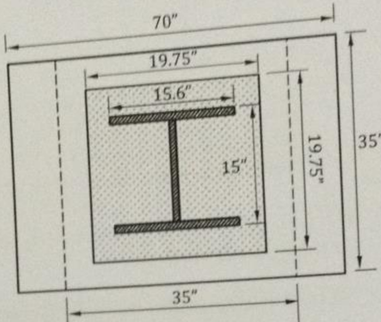
Dept. of Civil Engg.

Example:
A W14x159 column transmits an axial compressive live load of 750 kip and dead load of 300 kip on to a concrete base having a top surface area of 35-in by 70-in. Determine the size and thickness of base plate using A36 material. The concrete base has $f_c' = 4$ ksi. Follow ASD method.

Solution:
W14x159 geometry: $b_f = 15.6''$, $d = 15''$
Baseplate shall be smallest when allowable bearing stress is maximized.
Thus assume $\sqrt{(A_2/A_1)} = 2.0$ (max. value)
$\therefore f_{p(max)} = 0.85 f_c'(2.0) = 1.7 f_c' = 1.7 \times 4 = 6.8$ ksi.

$P = 750 + 300 = 1050$ kip

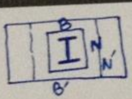
$BxN = \Omega P / f_{p(max)} = 2.5x1050/6.8 = 386.03$ in^2.
Overall plan dimension of the column is 15.6x15 which is almost square.



Initially chosen base plate

So we choose $B=N=\sqrt{(386.03)} = 19.65'' \approx 19.75''$

$\therefore$ Check $\sqrt{(A_2/A_1)} = \sqrt{(35^2/19.75^2)} = 1.77 < 2.0 \rightarrow$ Not OK



→ B', N' is similar to B, N and the two areas are concentric

এই দুই condition match করে max^m যত area নেয়া যায় & $\sqrt{\frac{A_1}{A_2}} \ngtr 2.0$ & $\nleq 1$

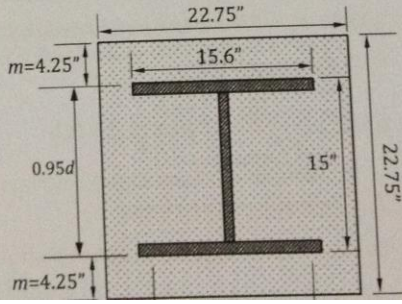
Next trial: assume $\sqrt{(A_2/A_1)} = 1.5$
$\therefore f_{p(max)} = 0.85 f_c'(1.5) = 5.1$ ksi.

$BxN = \Omega P / f_{p(max)} = 2.5x1050/5.1 = 514.7$ in^2.

We choose $B=N=\sqrt{(514.7)} = 22.68'' \approx 22.75''$

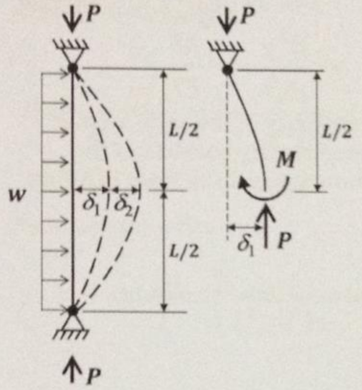
$\therefore$ Check $\sqrt{(A_2/A_1)} = \sqrt{(35^2/22.75^2)}$
$= 1.54 > 1.5 \rightarrow$ OK

From geometry (See Sheet-8),
$m = (22.75 - 0.95x15)/2 = 4.25$ in,
$n = (22.75 - 0.8x15.6)/2 = 5.135$ in,



2nd ORDER EFFECT OF AXIAL FORCE AND MOMENT AMPLIFICATION

The foregoing approach to the analysis of members subjected to both bending and axial load is satisfactory so long as the axial load is not too large. The presence of the axial load produces secondary moments, and unless the axial load is relatively small, these additional moments must be accounted for.

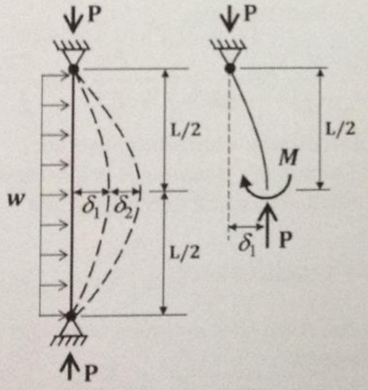


Moment at $L/2$ due to udl w, $M_o = wL^2/8$ and deflection due to w is δ_1. The M_o calculated above is called 1st order moment.

Due to δ_1, P shall now cause an additional moment $M_1 = P\delta_1$.
Thus moment shall be $M_o + M_1 = wL^2/8 + P\delta_1$

Moment M_1 is a second order moment because it can only be evaluated after the 1st order analysis and calculation of δ_1.

2nd ORDER EFFECT OF AXIAL FORCE AND MOMENT AMPLIFICATION



However the moment analysis is still incomplete.
Because, $P\delta_1$ shall now cause additional deflection δ_2 that will result additional moment $P\delta_2$

Thus, $M = wL^2/8 + P\delta_1 + P\delta_2$

Now, $P\delta_2$ shall cause additional deflection δ_3 and the process shall continue.

Therefore, in presence of axial load, the bending moment calculation becomes iterative,

$$M = wL^2/8 + P\delta_1 + P\delta_2 + P\delta_3 + \dots = M_o + M_1 + M_2 + M_3 + \dots$$

Which can be simplified as, $M = B\,M_o$

Where B is an amplification factor used to multiply first order moment M_o to take into account the second order effects.

The above phenomenon is generally termed as P-δ effect (not P-Δ).

$$B=\frac{-\frac{Pe}{EI}}{\frac{P}{EI}-\frac{\pi^2}{L^2}}=\frac{-e}{1-\frac{\pi^2 EI}{PL^2}}=\frac{e}{\frac{P_e}{P}-1}$$ Where, $P_e=\frac{\pi^2 EI}{L^2}$ = Euler load.

Now, we can write y as,

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

MOMENT AMPLIFICATION

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

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

M becomes maximum when $\sin\frac{\pi x}{L}$ is maximum. That is $\boxed{x = L/2}$

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

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

$$M_{max}=M_0\underbrace{\left[\frac{1}{1-(P/P_e)}\right]}_{\text{Moment Magnification Factor}}$$

$$M_{\max} = M_0\left[\frac{1}{1-(P/P_e)}\right] \quad \Rightarrow \quad M_{\max} = B\,M_0$$

where M_o is the un-amplified maximum moment. In this case, it results from initial crookedness, i.e. $M_o = P\,y_o$ but in general, it can be the result of transverse loads or end moments. The moment amplification factor is therefore,

$$B = \frac{1}{1-(P/P_e)}$$

Because the member deflection corresponds to a buckled shape, the axial load corresponds to a failure load—that is, a load corresponding to an LRFD formulation. Therefore, the amplification factor should be written as,

$$B = \frac{1}{1-(P_u/P_e)}$$

where P_u is the factored axial load. The form shown in last expression is appropriate for LRFD. For ASD, a different form, to be explained later, will be used. The exact form of the AISC moment amplification factor is slightly different from that shown in above expression .

V. Imp.
- Normally column check করি w.r.t weak axis. But for moment magnification, যে axis-এ bending হবে, ঐ axis-এই column capacity check করতে হবে।

Example: (contd. Previous Problem @ sheet 28)

For the last problem compute moment amplification factor and recheck the interaction.

Solution:

$P_u = 200.4$ kip,

Bending due to Q loads are in strong axis. Therefore, for moment magnification, Euler load for strong axis buckling is needed.

$KL/r_x = 1.0\times204/4.35 = 46.9 < 4.71\sqrt{(E/F_y)}$
$F_e = \pi^2E/(KL/r_x)^2 = 3.14^2\times29000/(46.9)^2 = 130$ ksi.
$P_e = F_eA_g = 130.0\times14.4 = 1872$ kip

$\therefore B = 1/[1-P_u/P_e]=1/[1-200.4/1872] = 1.12$

$\therefore$ Amplified $M_{ux} = 1.12\times1285.2 = 1439$ kip-in.

$$\frac{P_u}{\phi_c P_n}+\frac{8}{9}\frac{M_{ux}}{\phi_b M_{nx}} = 0.495+\frac{8}{9}\frac{1439}{0.9\times3020} = 0.966 < 1.0 \text{ OK}$$

If >1, then the section should be reviewed and we have to go for larger section

---

load moment for LRFD and a service load moment for ASD. … no translation"). M_{nt} will be a
$M_{\ell t}$ = maximum moment caused by sidesway (the subscript ℓt is for "lateral translation"). This moment can be caused by lateral loads or by unbalanced gravity loads. Gravity load can produce sidesway if the frame is unsymmetrical or if the gravity loads are unsymmetrically placed. $M_{\ell t}$ will be zero if the frame is actually braced. For LRFD, $M_{\ell t}$ will be a factored load moment, and for ASD, it will be a service load moment.
B_1 = amplification factor for the moments occurring in the member when it is braced against sidesway.
B_2 = amplification factor

AISC INTERACTION FORMULA

LRFD interaction equations

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left[\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right] \leq 1.0 \quad \text{For} \quad \frac{P_u}{\phi_c P_n} \geq 0.2$$

$$\frac{P_u}{2\phi_c P_n} + \left[\frac{M_{ux}}{\phi_b M_{nx}} + \underbrace{\frac{M_{uy}}{\phi_b M_{ny}}}_{}\right] \leq 1.0 \quad \text{For} \quad \frac{P_u}{\phi_c P_n} < 0.2$$

→ zero for strong axis bending of W sections.

ASD interaction equations

$$\frac{P_a}{P_n/\Omega_c} + \frac{8}{9}\left[\frac{M_{ax}}{M_{nx}/\Omega_b} + \frac{M_{ay}}{M_{ny}/\Omega_b}\right] \leq 1.0 \qquad \text{For} \quad \frac{P_a}{P_n/\Omega_c} \geq 0.2$$

$$\frac{P_a}{2P_n/\Omega_c} + \left[\frac{M_{ax}}{M_{nx}/\Omega_b} + \frac{M_{ay}}{M_{ny}/\Omega_b}\right] \leq 1.0 \qquad \text{For} \quad \frac{P_a}{P_n/\Omega_c} < 0.2$$

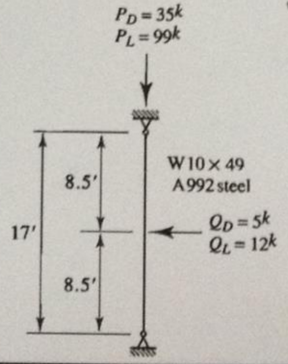
Example:

The beam–column shown in the Fig. is pinned at both ends and is subjected to the loads shown. Bending is about the strong axis. → ∴ $M_{n_y} = 0$ and $M_{uy} = 0$ Determine whether this member satisfies the appropriate AISC Specification interaction equation. Follow LRFD approach and neglect moment amplification. The column is not braced except at ⇒ laterally unsupported beam. the ends. Take $C_b = 1.32$ for flexure.

Solution:

A992 steel, $F_y = 50$ ksi.

W10x49 section:
$A_g = 14.4$ in^2, d=10 in., $S_x = 54.6$ in^3,
$r_x = 4.35$ in., $Z_x = 60.4$ in^3, $r_y = 2.54$ in.,
$r_{ts} = 2.84$ in, $h_o = 9.42$ in, $J = 1.39$ in^4, c= 1.0
$b_f = 10.0$ in, $t_f = 0.56$ in., h/t_w=23.1.
As a column: $L = 17' = 204$ in., $K = 1.0$
As a beam: $L = L_b = 17$ ft = 204 in.,
C_b =1.32



$M_{max} = M_o\left[\frac{1}{\quad}\right]$

SIGN FOR

VQ

$P_u = 1.2×35 + 1.6×99 = 200.4$ kip

$b_f/(2t_f) = 8.92 < 0.38\sqrt{(E/F_y)} \rightarrow$ compact flange.

$h/t_w = 23.1 < 3.76\sqrt{(E/F_y)} \rightarrow$ compact web.

এগুলো বের করে নিব / chart - থেকে দেখা যায় হলেও হবে।
⇓
CE chart-এ কোন section-এর পাশে কিছু লিখা থাকে না, অনুযায়ী compact section.

V. Imp.

• যদি কিছু বলা না থাকে তাহলে always Weak axis ↑ capacity বের করব।

Determine column capacity (based on weak axis):

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

$KL/r_y = 1.0×204/2.54 = 80.31 < 4.71\sqrt{(E/F_y)}$

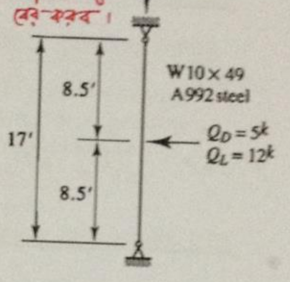
$F_e = \pi^2E/(KL/r_y)^2 = 3.14^2×29000/(80.31)^2$
$= 44.33$ ksi.

$\therefore F_{cr} = [0.658^{(50/44.33)}]50 = 31.18$ ksi

$\therefore \phi_cP_n = \phi_cA_gF_{cr} = 0.9×14.4×31.18 = 404.2$ kip

$\therefore P_u/(\phi_cP_n) = 200.4/404.2 = 0.495 \underline{> 0.2}$

$\therefore$ 1st formula applies.

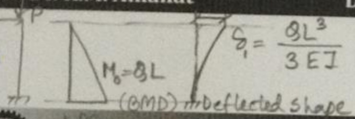


Determine capacity as a beam (strong axis):

$L_p = 1.76r_y\sqrt{(E/F_y)} = 1.76× 2.54× \sqrt{(E/F_y)} = 107.6 \text{ in.} < L_b$

$$L_r = 1.95r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_xh_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_xh_o}{Jc}\right)^2}} = 379.6 \text{ in.}$$

• Moment Magnification:



$\delta_1 = \frac{QL^3}{3EI}$ ⇒ So axial load আসলে, Bending moment gradually add হতে থাকবে।
⇓
This is known as Moment Magnification

$+P\delta_1$
$+P\delta_1$

$M = M_o + P\delta_1 + P\delta_2 + \ldots$
$= BM_o$

B = Moment Magnification Factor

$M_p = F_yZ_x = 50 × 60.4 = 3020$ kip-in

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

$M_{nx} = 1.32[3020 - (3020 - 0.7×50×54.6)[(204-107.6)/(379.6-107.6)]$
$= 3467.6 > M_p$

$\therefore M_{nx} = M_p = 3020$ kip-in

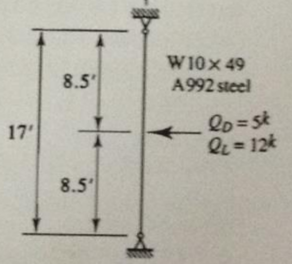
Flexure load, $Q_u = 1.2×5 + 1.6×12 = 25.2$ kip.

$\therefore M_{ux} = 25.2× 204/4 = 1285.2$ kip-in.

Only strong axis bending, $M_{uy} = 0.0$

$$\frac{P_u}{\phi_cP_n} + \frac{8}{9}\left[\frac{M_{ux}}{\phi_bM_{nx}} + \frac{M_{uy}}{\phi_bM_{ny}}\right]$$

$$= \frac{P_u}{\phi_cP_n} + \frac{8}{9}\frac{M_{ux}}{\phi_bM_{nx}} = 0.495 + \frac{8}{9}\frac{1285.2}{0.9×3020} = 0.915 < 1.0 \text{ OK}$$



Note: In the above demonstration, the 2nd order effect of axial load (moment magnification) is neglected which shall be discussed next.

Pro

factored
M_n = maximun
translatio
lanced gr
unsymme
be zero if
load mom
B_1 = amplificat
braced aga
B_2 = amplificati

resulting from sidesway.