بسم الله الرحمن الرحيم

(وأنزلنا الحديدَ فيه بأسٌ شديدٌ ومنافعُ للناس)

صدق الله العظيم   سورة (الحديد-٢٥)

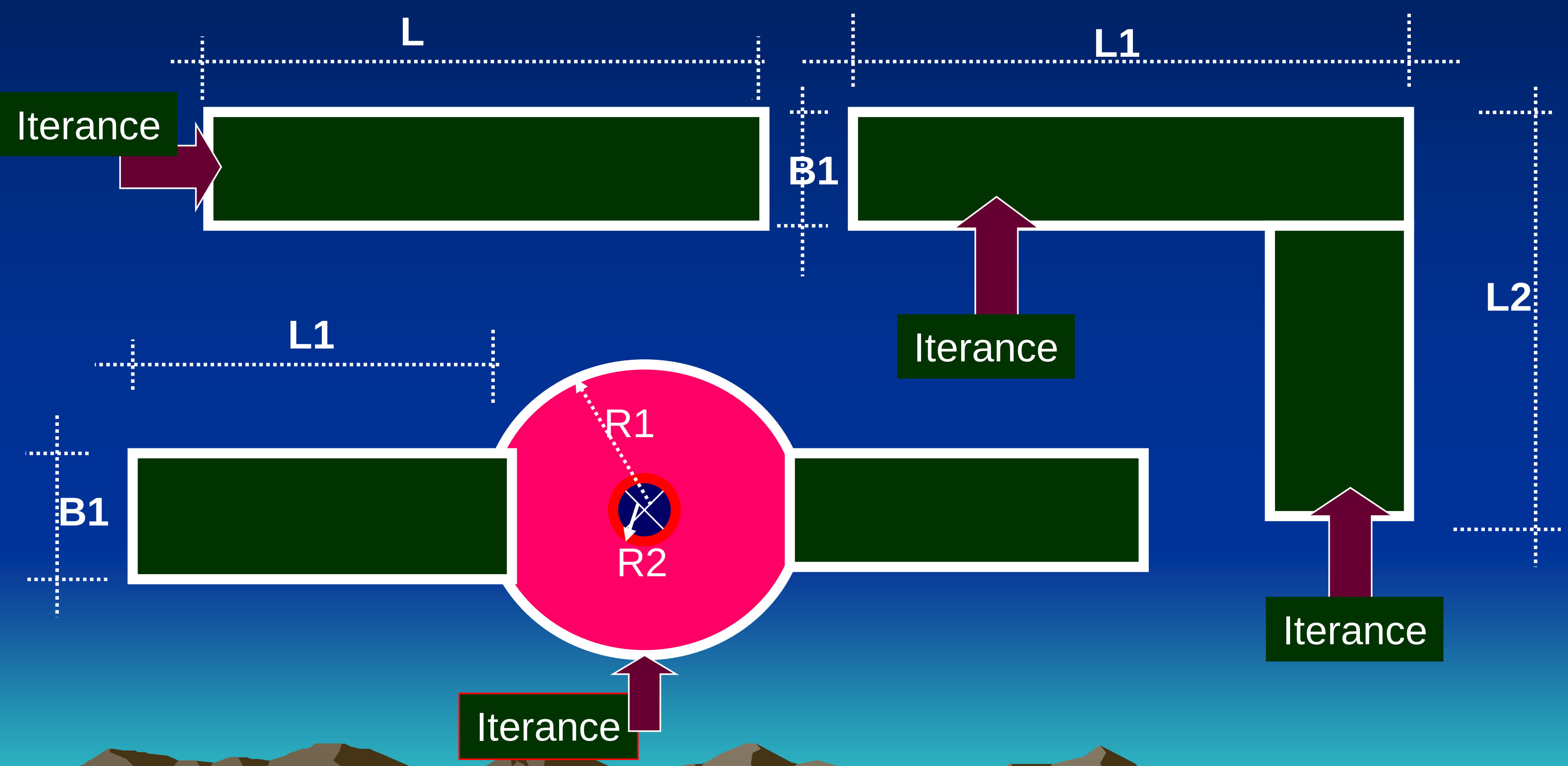
Steel Structures (III)

Part (II)

ا.د./سعد الدين مصطفى

أستاذ المنشآت والكبارى المعدنية

Proposed Areas for projects



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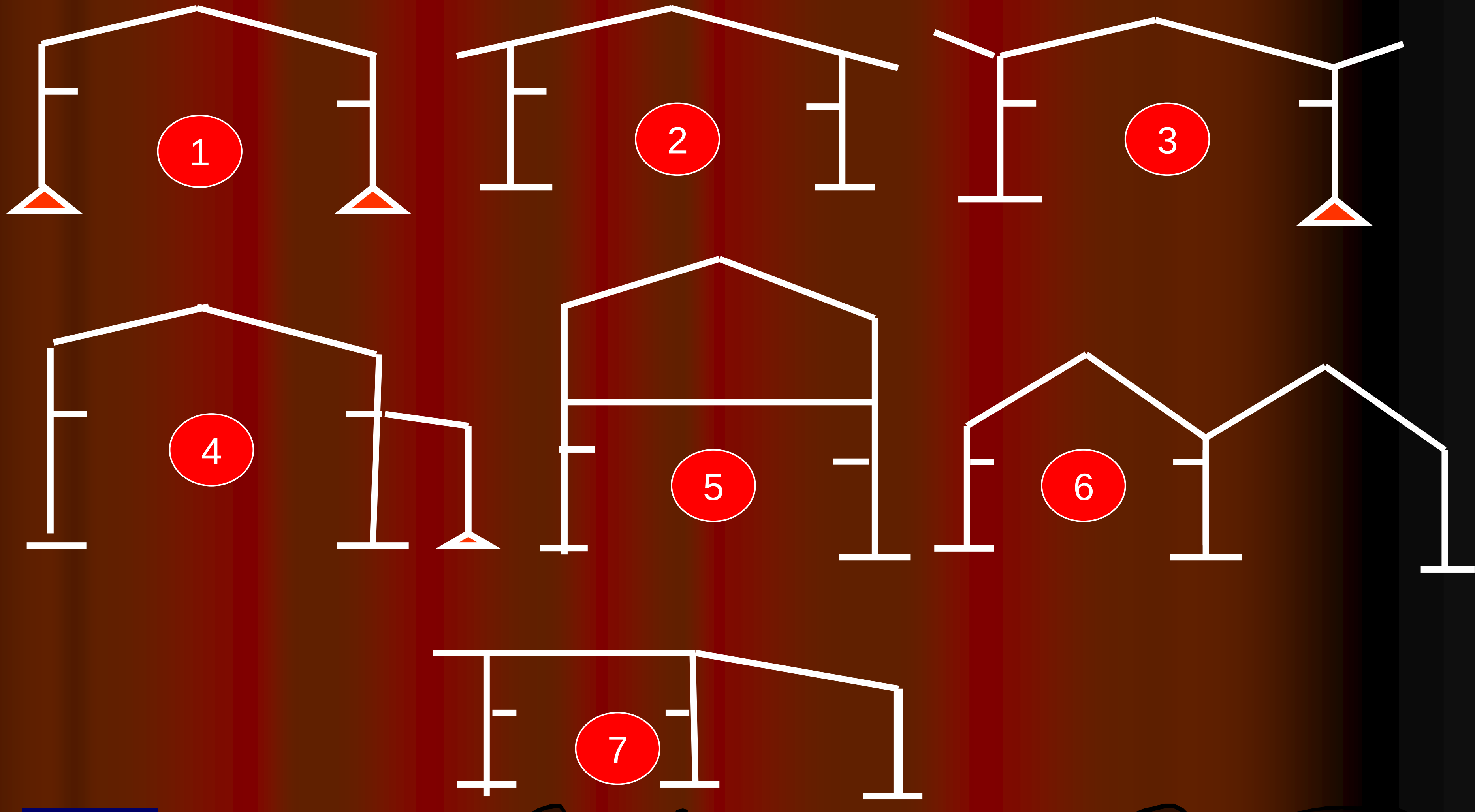
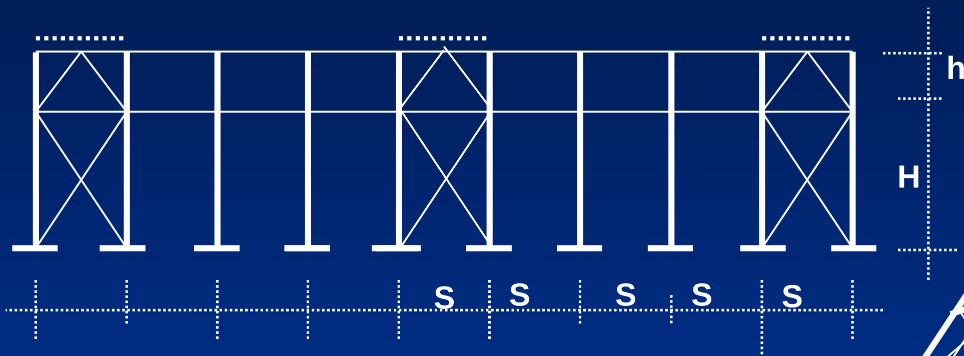
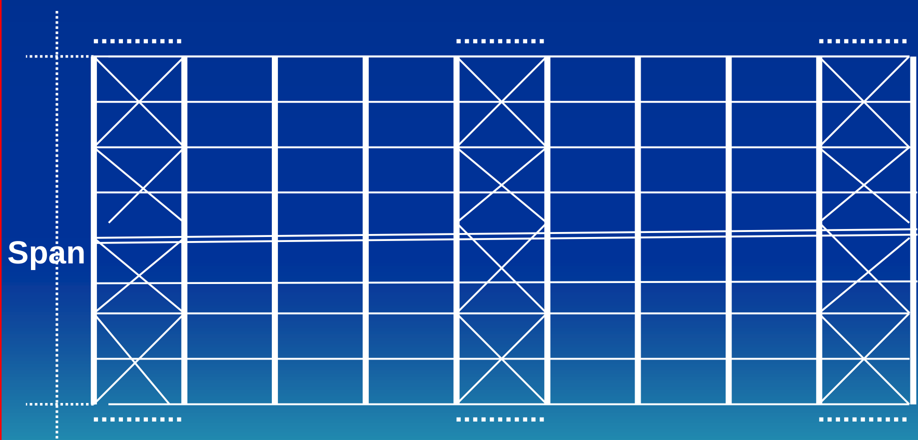
Some of Structural Systems



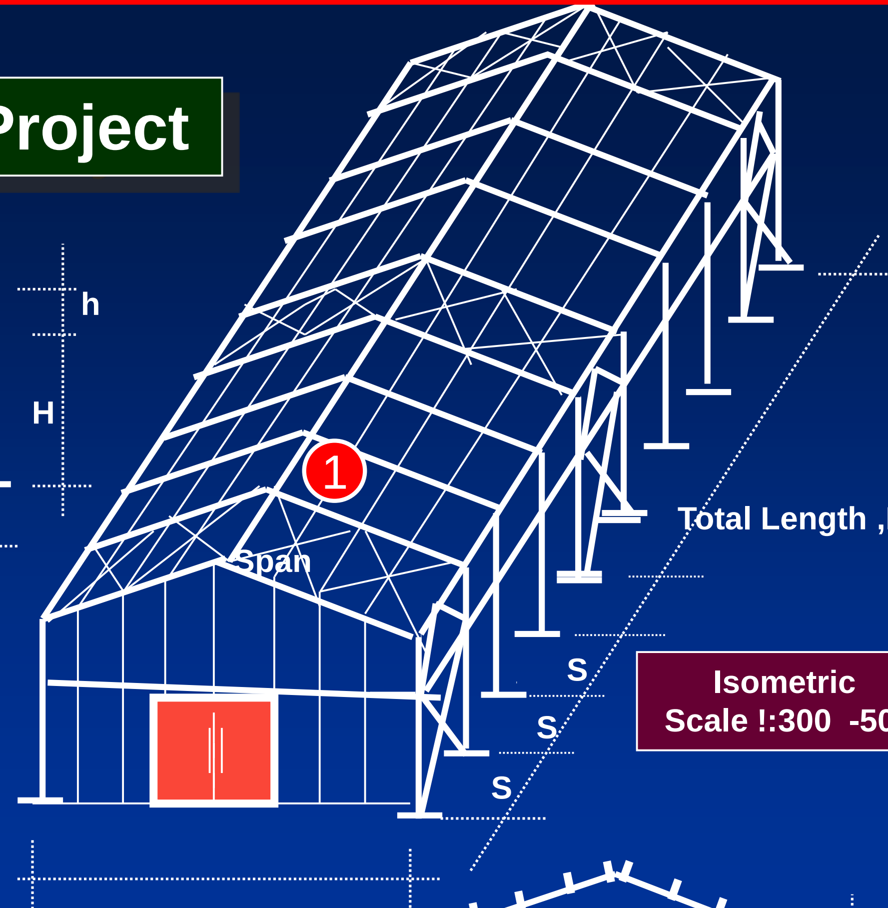
General Layout of Project



Side View- Transverse Wind Bracing
Scale !:100 = 200

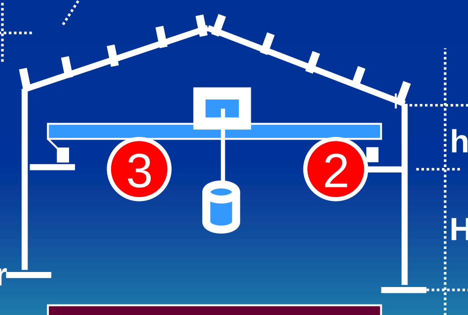


Plan - Upper Wind Bracing
Scale !:100 = 200



Isometric
Scale !:300 -500

1 Purlins
2 Crane
3 Crane Track Girder



Structural System
Scale :50

Loads Acting:

1) Dead Loads:

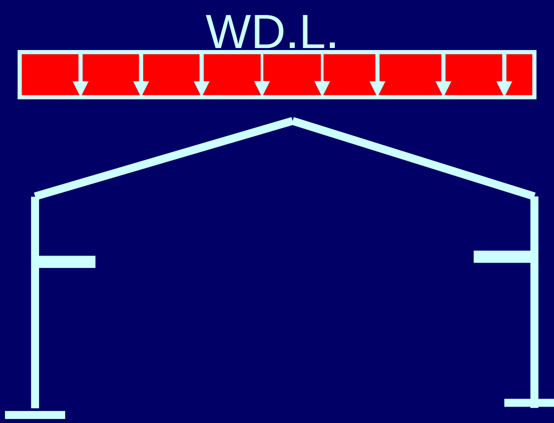
-Weight of steel Structure (Wst).

Assumed Wst [30 - 50] kg/m^2.

-Weight of covering material (Wc.m).

***Flexible Roof:**

Corrugated steel sheets. Wc= [5 - 8] kg/m^2.
Sandwich panel = [10 - 15] kg/m^2
Asbestos Wc = [15 - 25] kg/m^2
Tiels Wc = [30 - 50] kg/m^2
Glass Wc = [20 - 40] kg/m^2



***Rigid Roof:**

Concrete Wc = $\gamma . t_s$ γ = 2.5 t/m^2

Then,

WD.L. = (Ws.t + Wc.m) * S = …….. Kg/m`

2) Live Loads:

From Curve get $W_{L.L}$.according To inclined Roof . Two Cases into account:

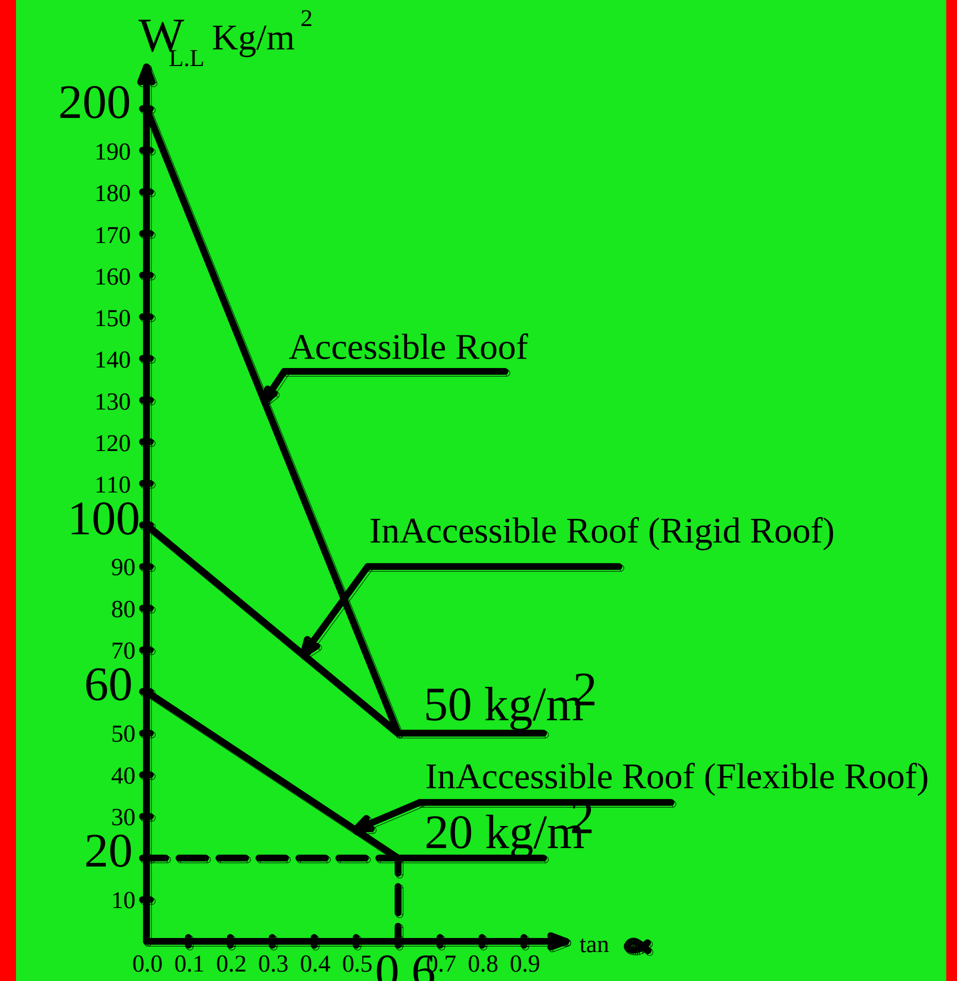
Case1 :

(a) Inaccessible Roof – Flexible)

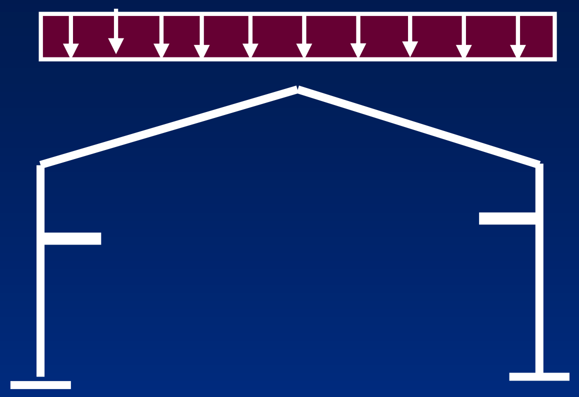
$$W'_{L.L} = 20 + (0.6 - \tan\alpha)\frac{40}{0.6}$$

(b) (Inaccessible Roof – Rigid)

$$W`_{LL} = 50 + (0.6 - \tan\alpha)\,(50/0.6)$$



Live Load on Inclined Roof



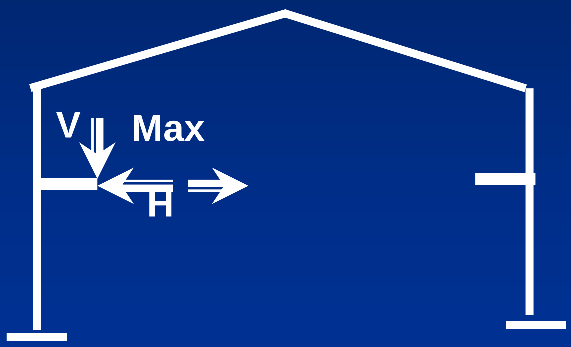
Case 2: (Accessible Roof – Rigid)

$W`_{L.L.} = 50 + (0.6 - \tan\alpha)(150/0.6)$

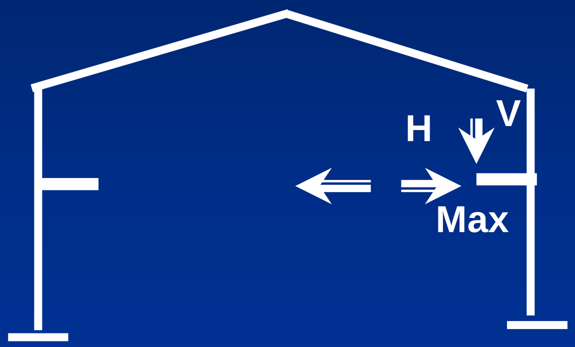
Then,

$$\boxed{W_{L.L.} = W`_{L.L} * S = ...Kg/m^2}$$

3) Crane Loads



a) Crane Left

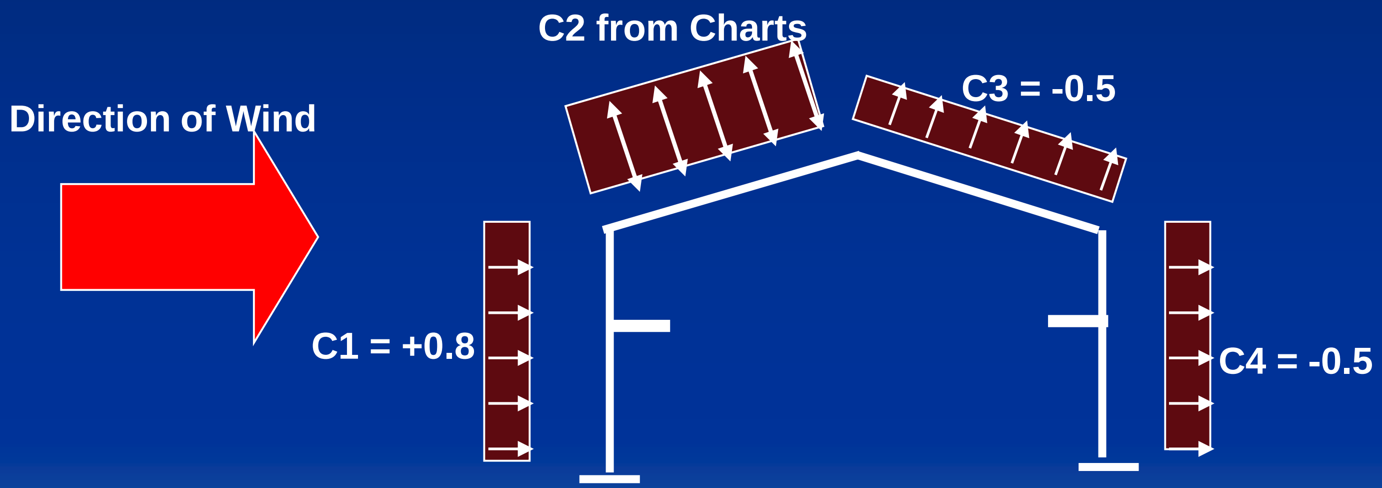


b) Crane Right

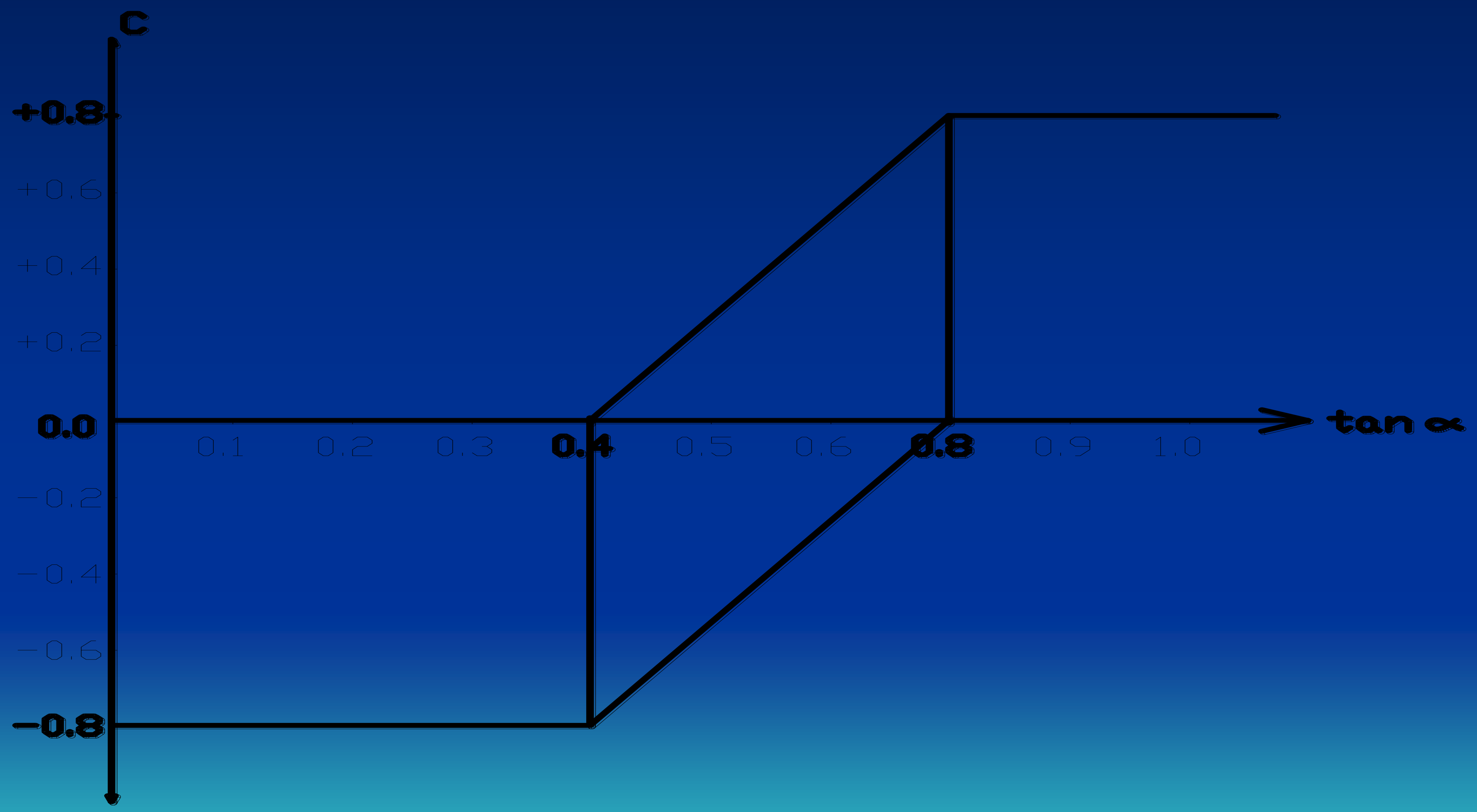
4) Wind Loads

$$W_{w.L.} = C * q * K = \ldots\ldots Kg/m^2$$

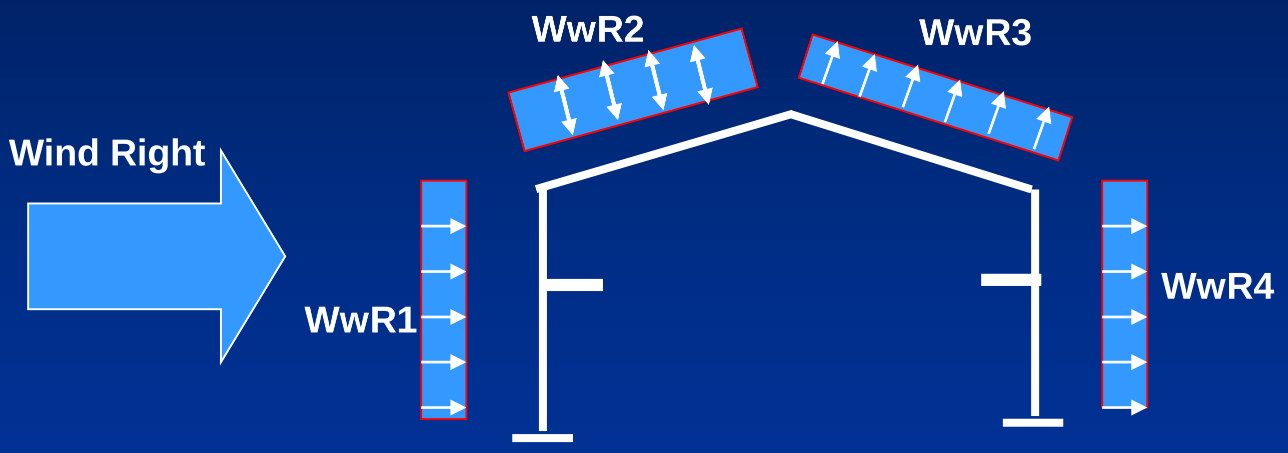
Where, Coeff. C can be determined from charts on each side of Building.



Coefficient “C”

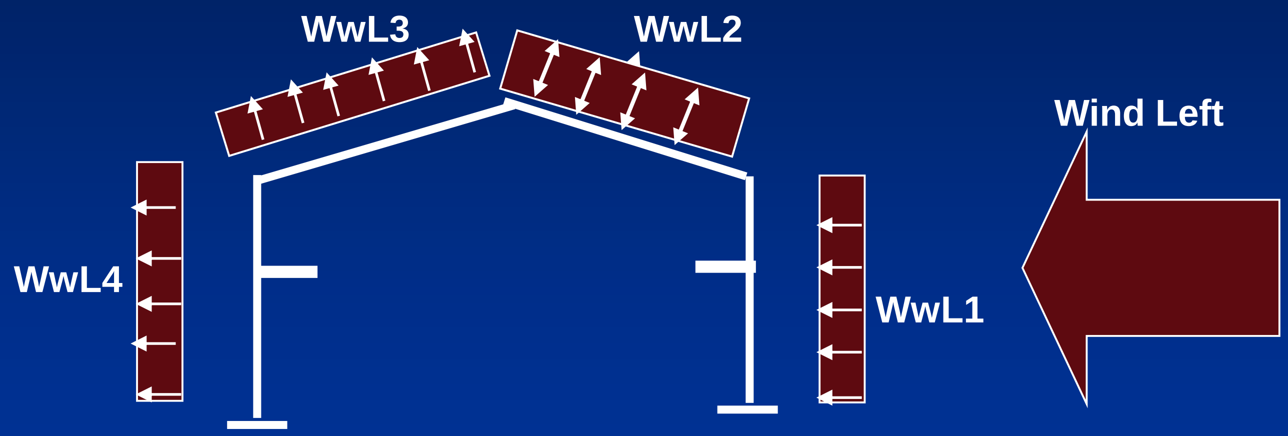


Case 1 : Wind Right



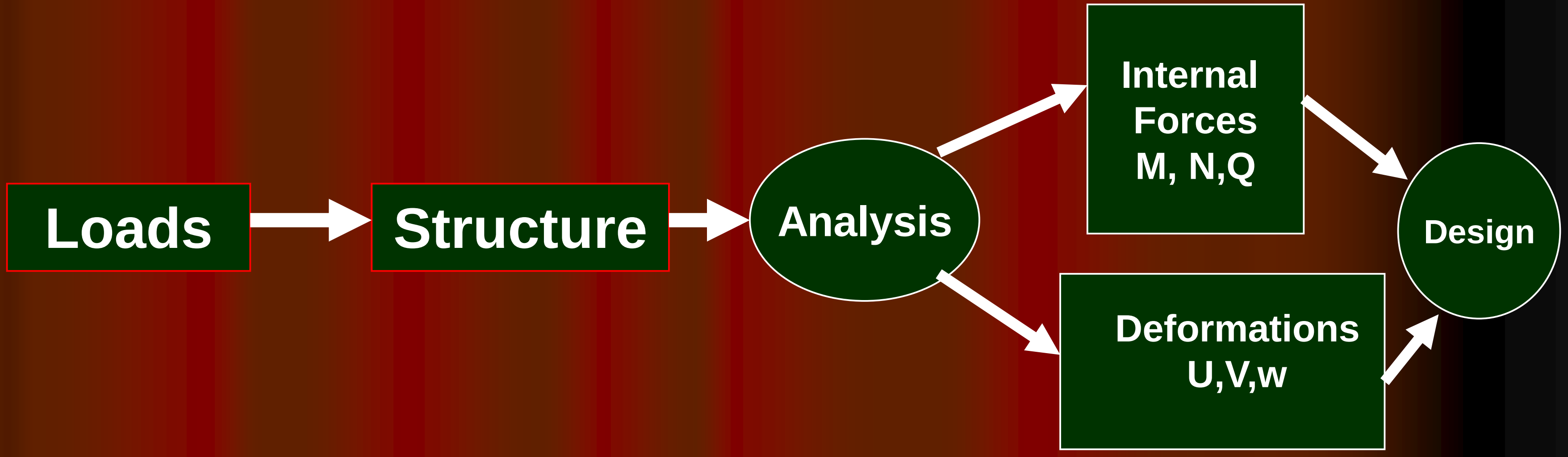
WwR1 = C1*q*K , WwR2 = C2*q*K , WwR3 = C3*q*K , WwR4 =C4*q*K

Case 2 : Wind Left



$W_{wL1} = C1*q*K$, $W_{wL2} = C2*q*K$, $W_{wL3} = C3*q*K$, $W_{wL4} = C4*q*K$

Structural Analysis



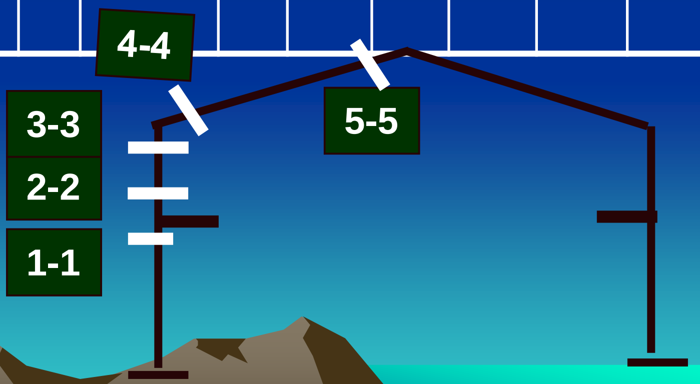
Structural Analysis of Frame :

From the applied Loads, by using Computer Program (Sap2000, Abokus, Broeken and Staad III) to determine the straining actions at different sections along the frame.

Also, the design values of the straining actions at different sections can be found.

Table (1) : Straining Actions at different sections along the Frame

Sec Number	Forces	D.L.	L.L	W.L.		Crane Left			Crane Right			Max M		Max, N	
				W.L.R	W.L.L	↓	←	→	↓	←	→	+ve	-ve	+ve	-ve
Sec. 1-1	B.M														
	S.F														
	N.F														
Sec. 2-2	B.M.														
	S.F														
	N.F														



Finish Lec100

Chapter 2
Analysis and Design of the Steel Columns

There are Two Design Methods Taking into Consideration :

(1) Allowable Stress Design (ASD)
(2) Load and Resistance Factor Design (LRFD)

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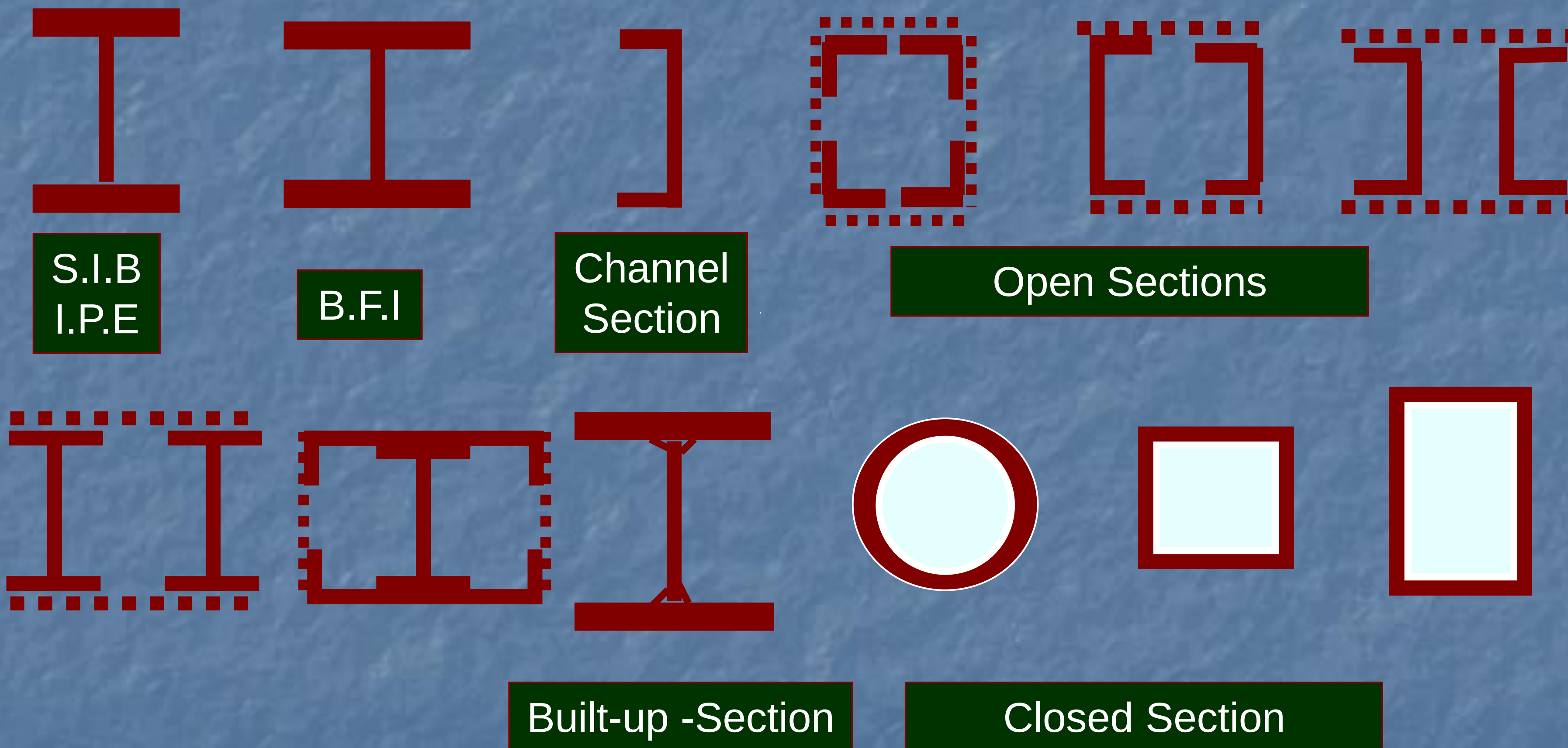
Common Columns cross Sections



Analysis of the Steel Columns

- During the design of beam-columns, the member is treated as a compression member (from buckling Phenomena) and as a beam (from lateral torsional buckling Phenomena).
- In general, the design procedure includes:

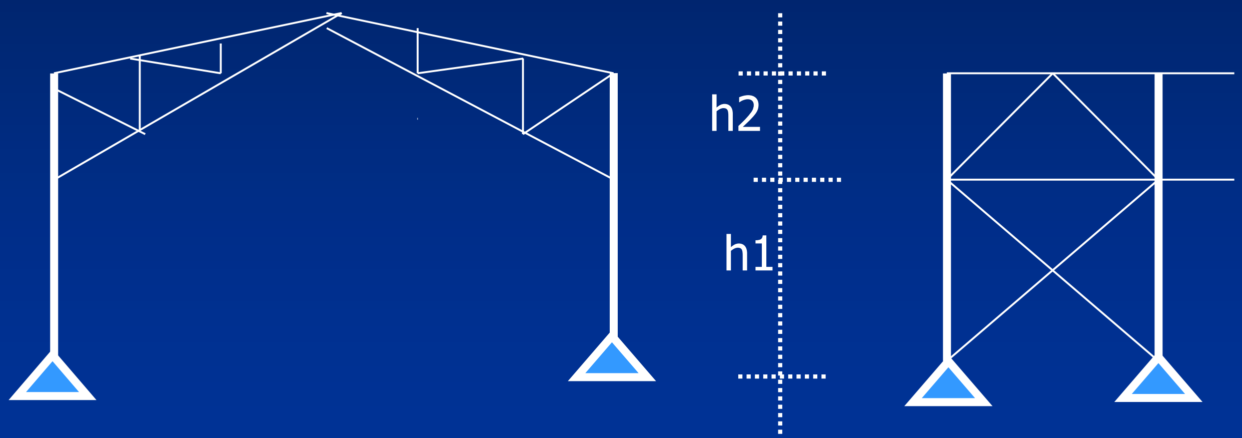
(A) Buckling Lengths ($L_{b\text{-}in}$ & $L_{b\text{-}Out}$)

(B) Design steps (Checks)

A) Buckling Lengths $L_{b\text{-}in}$ plan & $L_{b\text{-}out}$ plan

(1) Buckling Lengths $L_{b\text{-}in}$ & $L_{b\text{-}out}$ in Trussed Frame

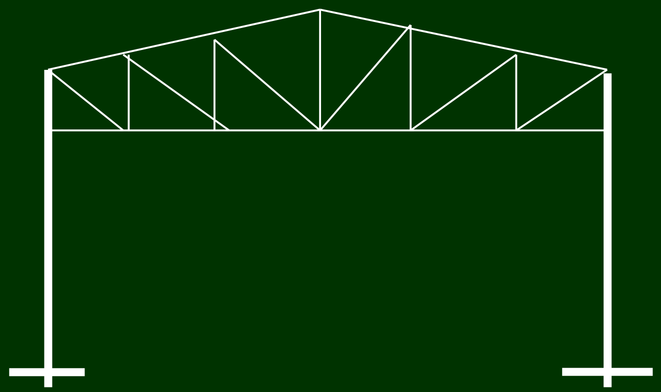
(a) Buckling Lengths of Hinged Base trussed Frames: •

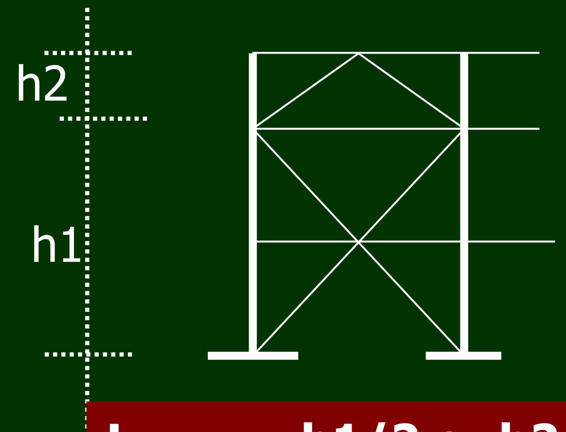


$L_{b=in} = 2.0\ (h1 + h2/2)$

$L_{b=out} = h1 \text{ or } h2 \text{ (greater)}$

- (b) Buckling Lengths of Fixed Base trussed Frames:

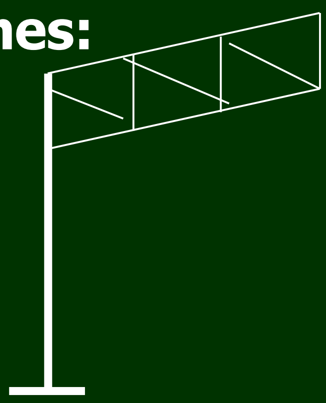


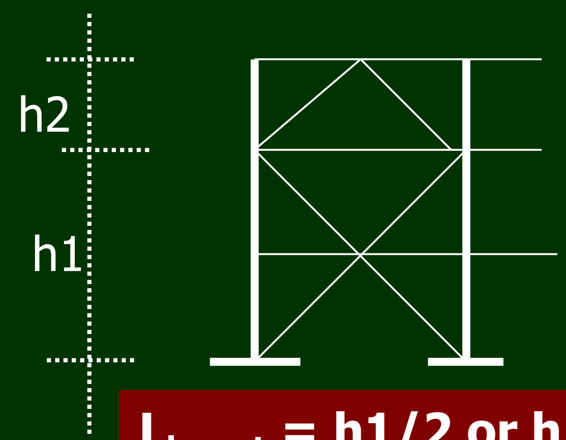


$L_{b=in}$ = 1.2 (h1 +h2/2)

$L_{b=out}$ = h1/2 or h2 (greater)

- **(c) Buckling Lengths of Columns of Cantilever trussed Frames:**

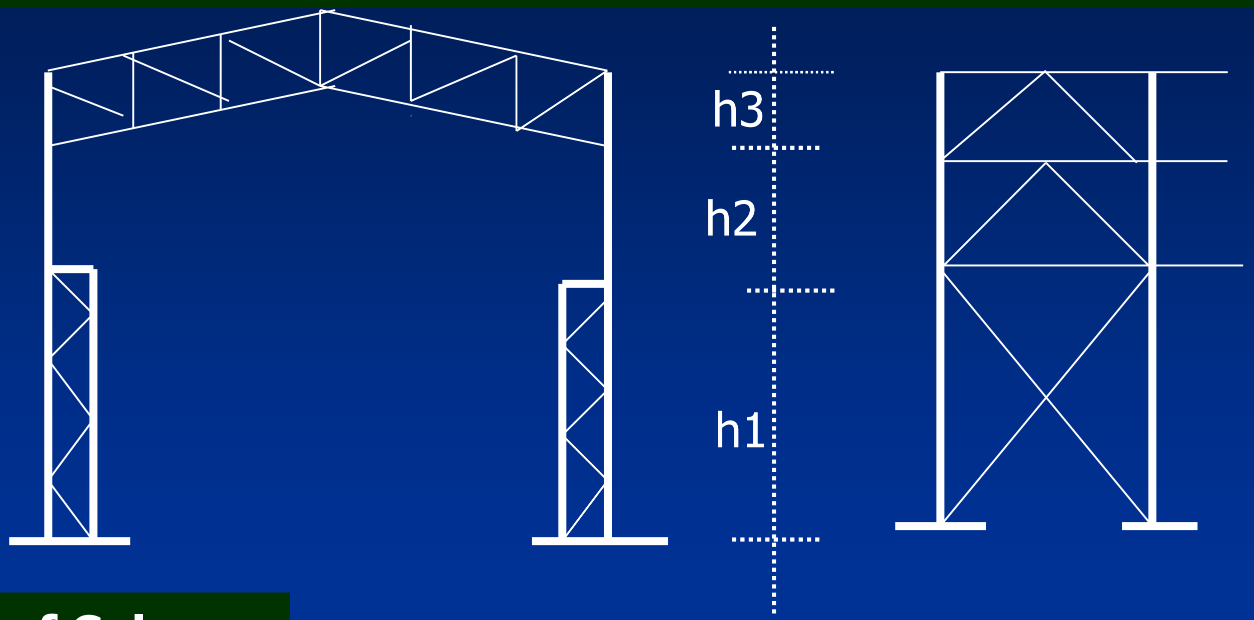




$L_{b=in}$ = 2,1(h1 +h2/2)

$L_{b=out}$ = h1/2 or h2 (greater)

(d) Buckling Lengths of Combined Columns Carrying Cranes in trussed Frames:



***** Roof Column:**

$L_{b=in}$ = 1.5 (h2 +h3/2)

$L_{b=out}$ = h2 or h3 (greater)

***** Combined Column:**

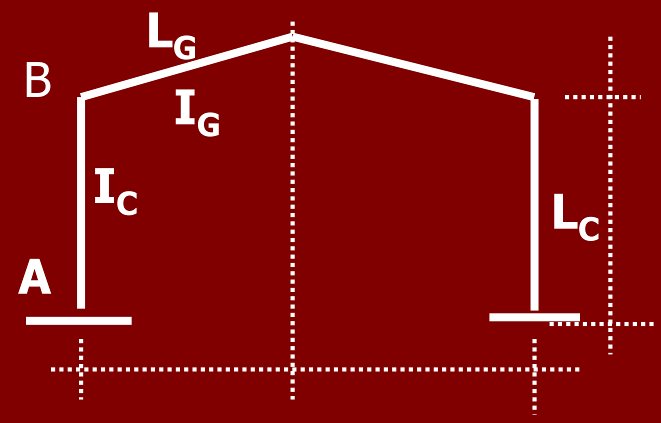
$L_{b=in}$ = 1.5 h1

$L_{b=out}$ = h1

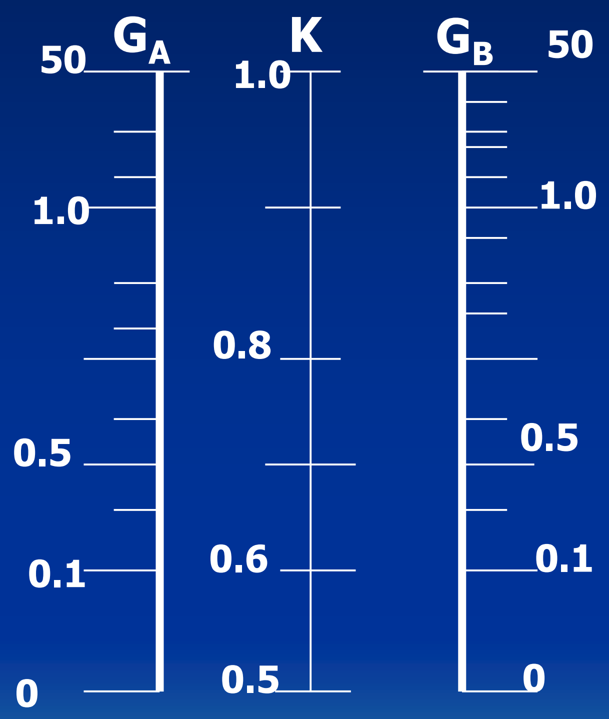
(2) Buckling Lengths Lb-in & Lb-out in Rigid Frame

- The Buckling Factor (k) for a column in a rigid frame is obtained from the alignment charts in figure (4-3 in Code page 61). The two subscripts A & B refer to the points at the two ends of the columns, while G is define by :

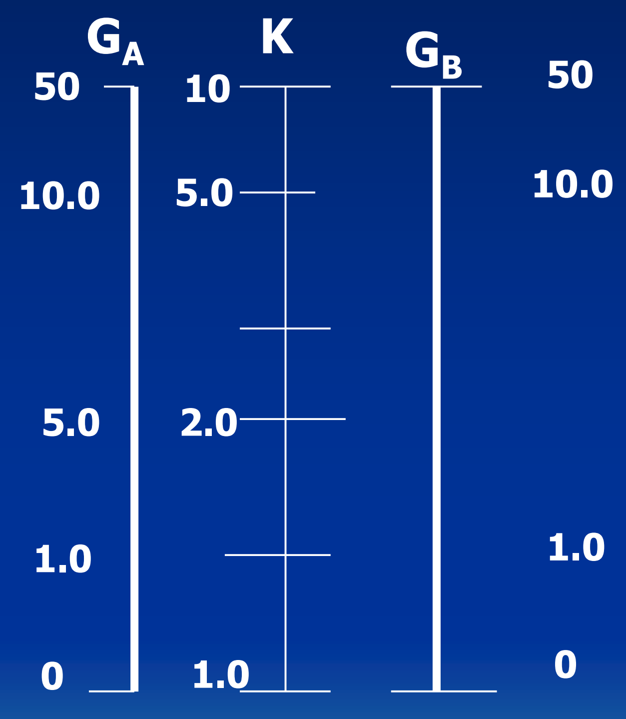
$$G = \frac{\sum (I/L)Columns}{\sum (I/L)Girders} \quad (Code\ \ pp.60)$$



Alignment Charts for Buckling Length factor (K) of Columns in Rigid Frames {Fig.4-3 pp.61 Code}

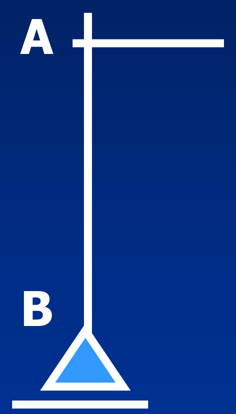


Side sway Prevented



Side sway Permitted

G - values for Columns with Special end Conditions {Code pp.60}

Column Base Condition	 	
G_B	$G_B = 10.0$	$G_B = 1.0$

Beams with Special End Conditions (Code pp.62)

Beam End Conditions		
Side sway Prevented	$(I/L)_G * 1.5$	$(I/L)_G * 2.0$
Side sway permitted	$(I/L)_G * 0.5$	$(I/L)_G *$ o.67

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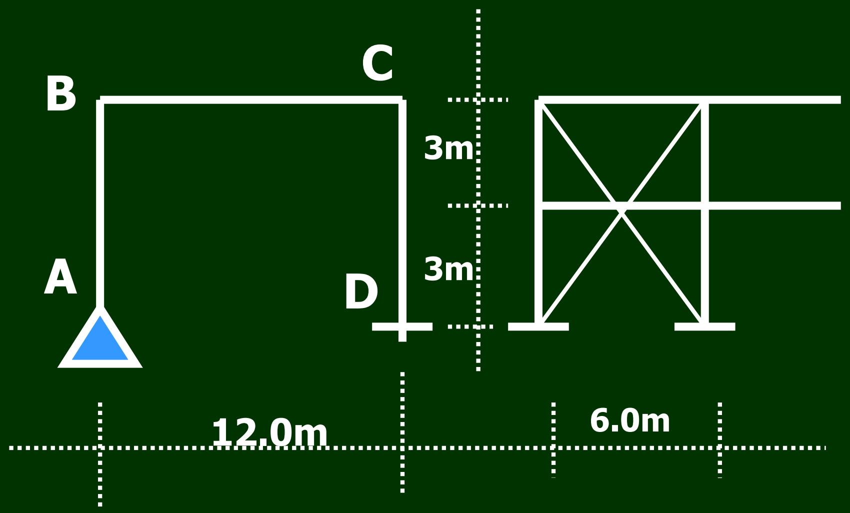
Example: Determine the buckling lengths of the Columns AB & CD in the following cases :

a) Side sway is prevented
b) Side sway is permitted



- **Column A-B**
- **Case a: Side sway is prevented:**
- **Buckling Length in plan: Lb-in =K*L**
- **GA = 10.0**
- **GB = {I/6/I/12} = 2.0**
- **From charts (p.61 Code) get K = 0.9**
- **Lb-in = 0.9*6.0 = 5.4m**
- **Lb-out = 3.0m**
- **Case b: Side sway is permitted**
- **Column A-B : Lb-in =K*L**
- **GA = 10.0**
- **GB = {I/6/I/12} = 2.0**
- **From charts (p.61 Code) get K =2.15**
- **Lb-in = 2.15*6.0 =12.9m**
- **Lb-out = 3.0m**

Column C-D

Case a: Side sway is prevented:

Buckling Length in plan: Lb-in =K*L

GA = 1.0

GB = {I/6/I/12} = 2.0

From charts (p.61 Code) get K = 0.8

Lb-in = 0.8*6.0 = 4.8m

Lb-out = 3.0m

Case b: Side sway is permitted

Column A-B : Lb-in =K*L

GA = 1.0

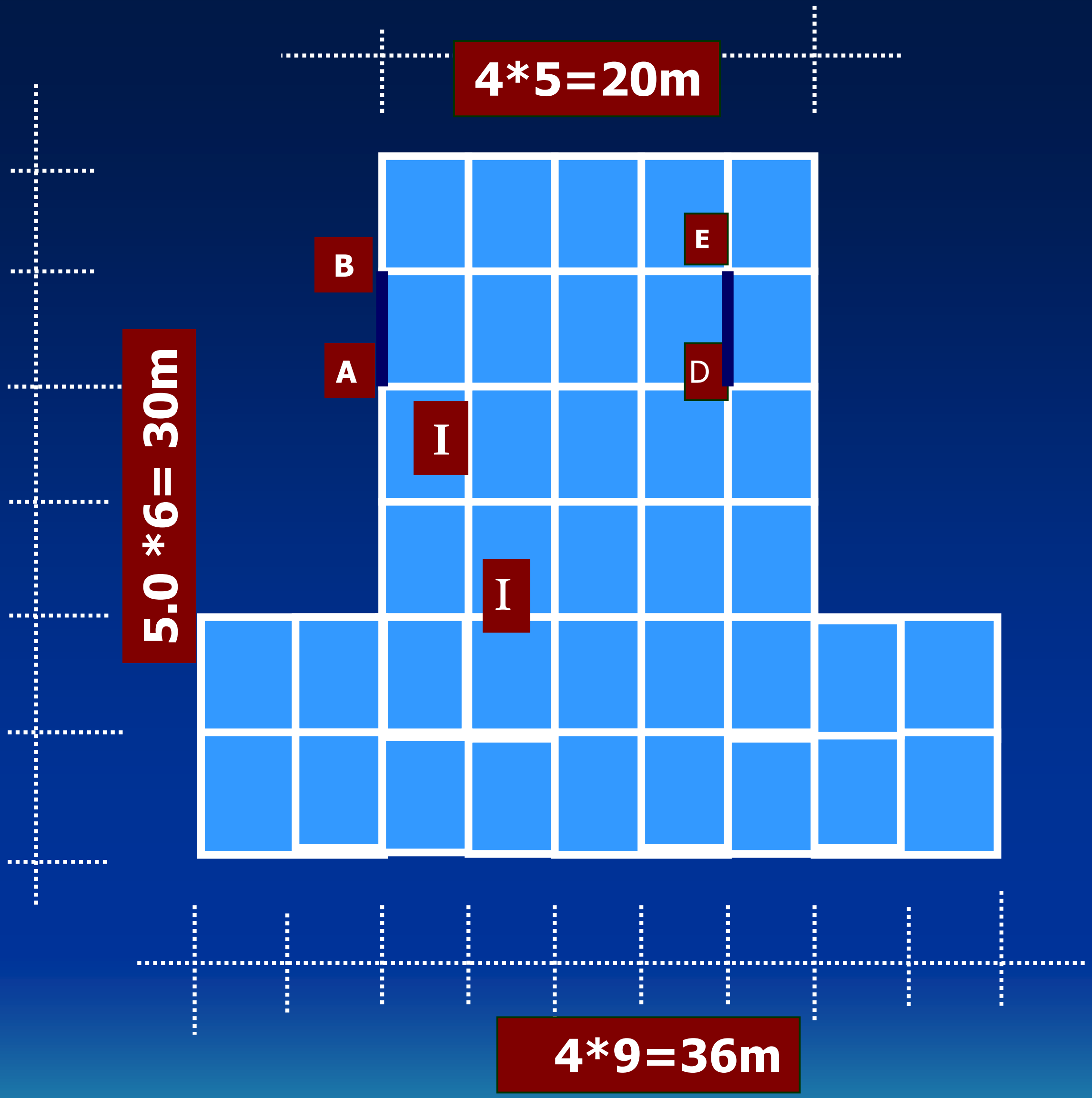
GB = {I/6/I/12} = 2.0

From charts (p.61 Code) get K =1.5

Lb-in = 1.5*6.0 = 9.0 m

Lb-out = 3.0m

Determine the Buckling Lengths of the Columns A-B and D-E, where the Side sway is prevented If the moment of inertia Of the columns is one And half moment of Inertia of the rafter



Solution

<u>Column A-B</u>

$G_A = ((1.5I/5 + 1.5I/5))/(I/4)$

$= (3I/5)/(I/4) = 2.4$

$G_B = 2.4$

From charts 4-3 PP61 get

K = o.88

Lb-in K*L = 0.88*5 =4.4m

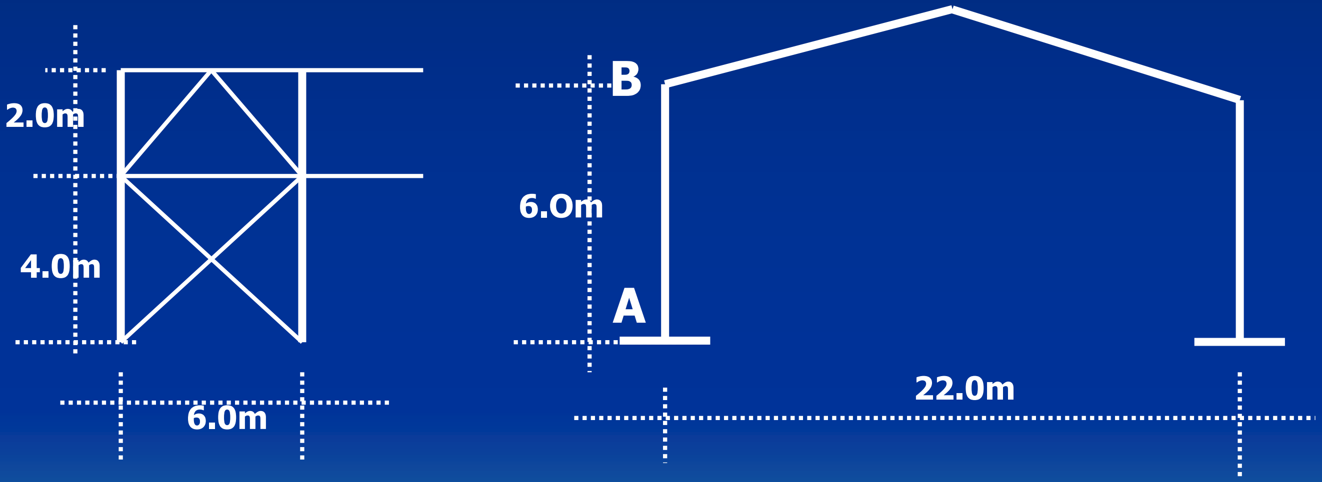
Lb-out = 5.0m

Column D-E as the same Procedure of column A-B.

Design Procedure of Columns

- (1) Data given: {MD, ND at critical section along the column, Vertical bracing system, Lb-in and Lb-out.
- (2) Estimation of the cross section
- (3) Check for Local Buckling
- (4) Check for Lateral Torsional Buckling
- (5) Check for Column Buckling
- (6) Check the interaction Equation.

- Example : Design a suitable rolled Section for the column A-B Shown the given figure if the column subjected to the following loads: MD = 20t.m and ND = 20t.



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بسم الله الرحمن الرحيم

Lec.3

Design of Steel Columns

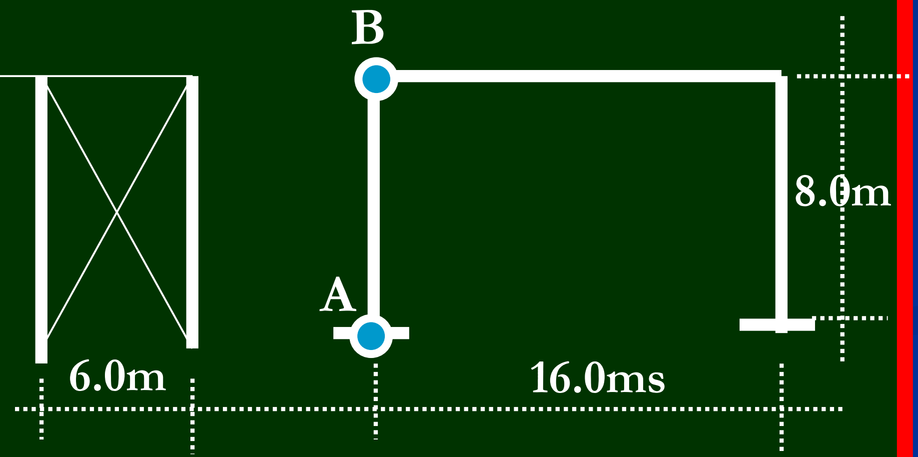
Two Cases into Consideration:

(I) Column Subjected to Axial Force Only (N.F.)

(II) Column Subjected to Axial Force and Bending Moment (N.F.& B.M.)

(I) Column Subjected to Axial Force Only

- *Example: Design a Column "AB" to carry a total load 100t . The column is hinged from both sides, the length of the column is 8.0ms (Lbx = Lby =800 cm)*



(i) Stiffness Condition:

Lbx/ix ≤ 180 ix ≥4.44cm

Lby/iy ≤ 180 iy ≥4.44 cm

(ii) Stress Condition:

Areq. = Axial Force/ $F_{assumed}$=

=100/0.7 = 142.86cm Choice B.F.I NO.360 ,ix = 15.6, iy = 7.49,

A = 181cm2

(iii) Check For Local Buckling:

Web : dw = h – 4tf = 27cm

$(dw/tw) = 27/1.25) = 21.6 \leq (58/\sqrt{Fy})$ 37.3 (Code pp.9)

O.K. Compact

Flange :

$C = (bf\text{-}2tf - tw = 12.125cm$

$(C/tf)act = (12.125/2.25)$ 5.39 $\leq (16.9/\sqrt{Fy}) = 10.9$

O.K. Compact The Section is compact

(iv) Check of Stresses:

$\lambda x = (Lbx/ix) = (800/15.6) = 51.28$

$\lambda y = (Lby/iy) = (800/7.49) = 106.8$ Take $\lambda max = 106.8$

$Fall.c = 7500/106.8^2 = o.657\ t/cm^2$

Fact = (Axial Force/Area) = (100/181) = 0.55 t/cm2 < Fc O.K. Safe

(II) Columns Subjected to Axial force and Bending Moment (N & M)

Design Procedure of Column subjected to (M &N) :

(1) Data given: {M_D, N_D at Critical Section Along the Column, Side Sway Prevented or permitted and $L_{b\text{-in plan}}$ and $L_{b\text{-out of Plan}}$.

(2) Estimation of the cross section

(3) Check for Local Buckling (Code pp. 9 to 12)

(4) Check for Lateral Torsional Buckling (Code pp. 15 to 18)

(5) Check of Stresses (Interaction Equation Code pp.25).

Lateral Torsional Buckling of the Comp. Flange

(1) Determine the Actual Unsupported Length of the Comp. Flange of the Column "L_{Uact}" and the coefficient Cb is depending on the type of load and support conditions:

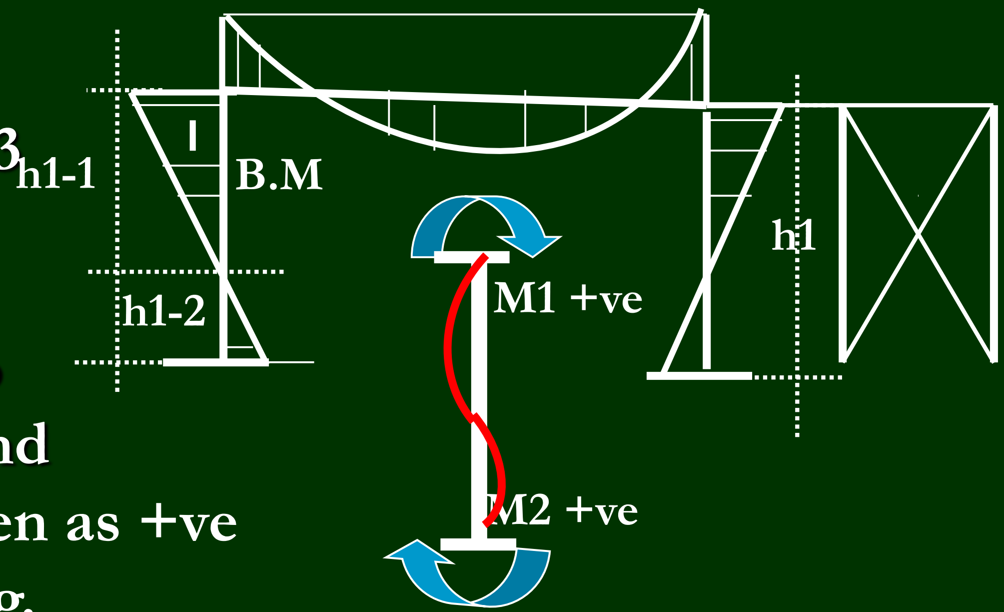
$L_{Uact} = h_{1-2}$ or h_{1-1}(greater)

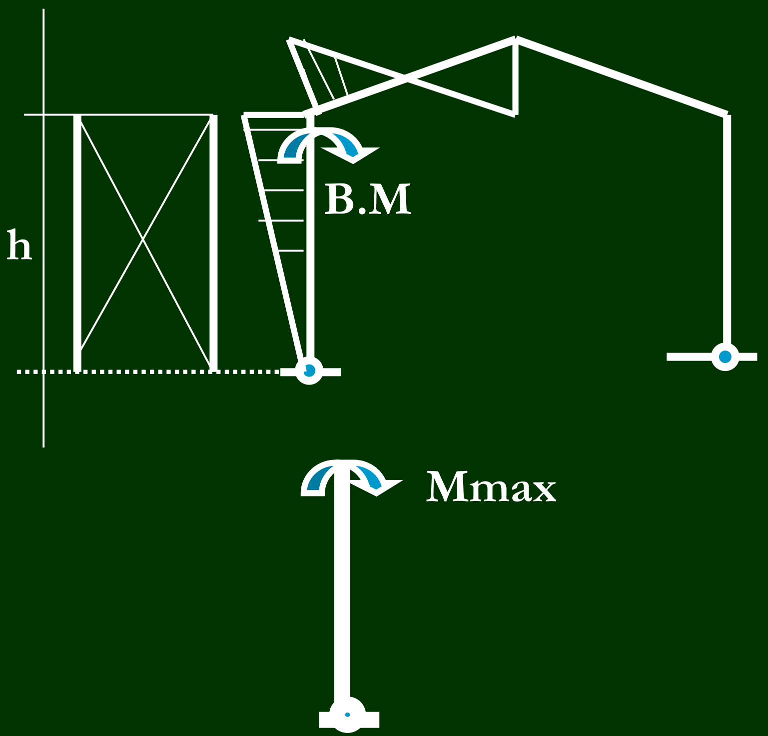
$Cb = 1.75 + 1.05\,\alpha + 0.3\,\alpha^2 \leq 2.3$

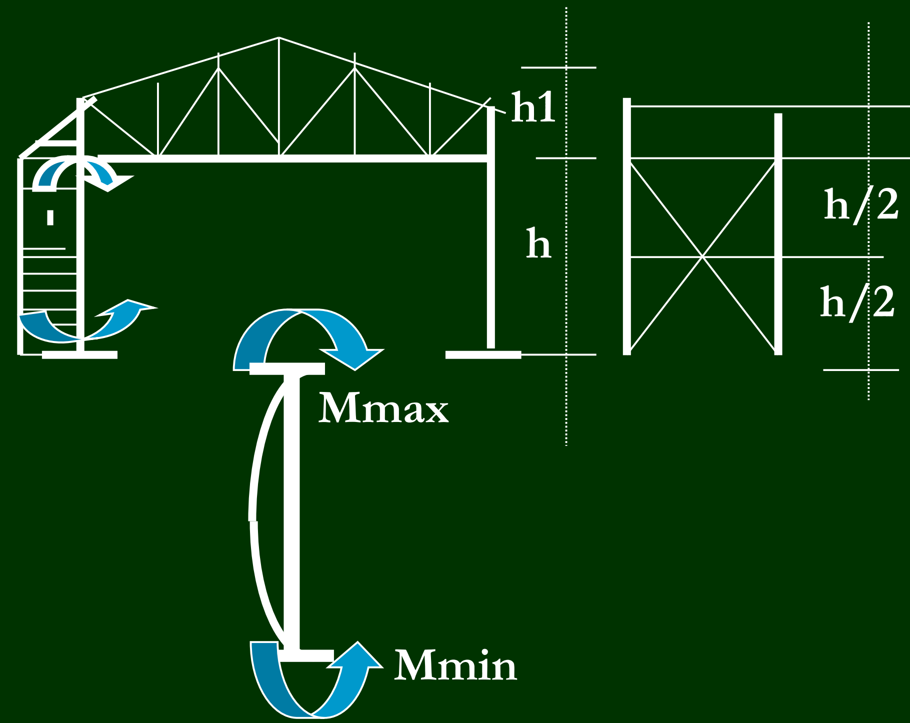
Where:

$\alpha = M_1/M_2 = +ve$

M1/M2 is the algebraic ratio of the smaller to the larger end moments of the column taken as +ve For reverse curvature bending.







$L_{Uact} = h$

$\alpha = M_{min}/M_{max} = 0.0$

$Cb = 1.75$

$L_{Uact} = h/2$

$\alpha \quad = M_{min}/M_{max} = -1.0$

$Cb = 1.75 + 1.05\,\alpha + 0.3\,\alpha^2 \leq 2.3$

(code pp.19 Eq.2.28)

(2) Determine the Actual Lateral Torsional Buckling "Fbc":

- Calculate the values if Lu-max according to the following equations (Code pp. 16):

$L_{u1max} \leq (20b_F/\sqrt{Fy})$ (Eq. 2.18 in Code)

$L_{u2max} \leq (1380.A_F/d.Fy)C_b$ (Eq. 2.18 in Code)

- If $L_{u\ act.} \leq L_{u1\ \&\ 2}\ max$ (no Lateral torsional buckling is considered)

$F_{be} = 0.64\ Fy$ if the section is compact

$F_{bc} = 0.58\ Fy$ if the section is non-compact

- If $L_{u\ act.} > L_{u1\ OR}\ L_{u2}$ max

(Then Calculate the Lateral Torsional Buckling F_{Ltb} as following (pp.18 &19 in Code):

$$FLtb = \sqrt{F_{Ltb1}^{2} + F_{Ltb2}^{2}} \leq 0.58Fy$$ (code pp.19)

<u>Firstly</u> : Find the *F_{Ltb1} as follows:*

F_{Ltb1} = {800.Af.Cb/Luact.d) ≤ 0.58Fy (Eq. 2.23 Code)

<u>Secondly</u>: Find the *F_{Ltb2} as follows :*

- When Luact < $84\sqrt{(Cb/Fy)}$

FLtb2 = 0.58Fy

- When $84\sqrt{(Cb/Fy)} \leq Luact/rT \leq 188\sqrt{Cb/Fy}$

$F_{Ltb2} = (0.64 - (Luact/rT)^2 Fy/1.176{*}.10^{5}{*}Cb)\ Fy \leq 0.58Fy$

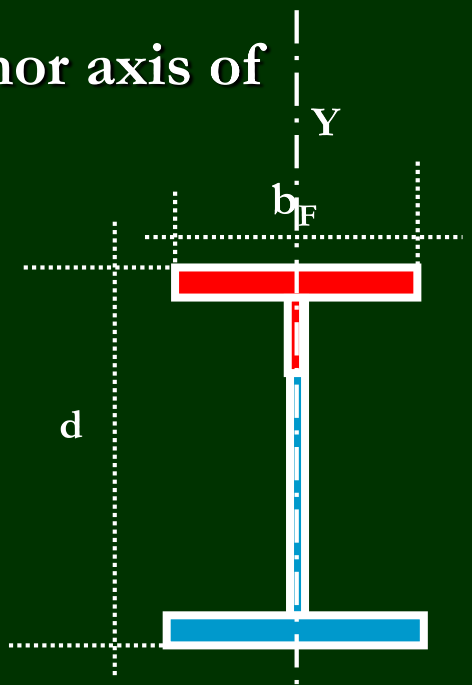
- When $Lu_{act}/r_T > 188\sqrt{Cb/Fy}$

➡ $F_{Ltb2} = 12000(Lu/r_T)Cb \le 0.58Fy$ (code pp.18)

Then,

➡ $FLtb = \sqrt{F_{Ltb1}^{2} + F_{Ltb2}^{2}} \le 0.58Fy$ (code pp.19)

Where : rT = Radius of Gyration about minor axis of section concerning the compression flange plus sixth of the compression area of web



$rT = \sqrt{Iy\text{-}y/Af} =$

$Iy\text{-}y = bf(tf)^3/12 + tw.(hw/6)^3/12$

Check of Stresses (Interaction Equation 2.35)

in Code pp.25 :

$$\frac{f_{ca}}{F_c} + \frac{f_{bx}}{F_{bcx}} A_1 + \frac{f_{by}}{F_{bcy}} A_2 \leq 1.0$$

- **Where :**

fca = Actual Compression Stresses =(N/A)

Fc = Allowable compression Stresses ($\lambda \leq 100$ or $\lambda > 100$)

fbx = Actual Bending Stresses in x-direction (Mx/Sx)

Fbcx = 0.64Fy (case of Compact or 0.58Fy in case of non-compact (Allowable bending Stresses on the compression side due to Mx

f_{by} = Actual Bending Stresses in y-direction (M_y/S_y)

F_{bcy} = Actual Bending Stresses due to

$M_y = 0.72F_y$ in Case of Compact Sec. and

$= 0.58F_y$ in Case of Non-Compact Sec.)

A1 and A2 = 1.0 for $(f_{ca}/F_c) \leq 0.15$, (Code pp.25),

Otherwise:

$A1 = (C_{mx}/(1-f_{ca}/F_{EX})$ (Code pp.25)

and $A2 = (C_{my}/(1-f_{ca}/F_{EY})$ (Code pp.25)

Where:

- Cmx and Cmy are Moment Modification Factors and to be taken according to the following:

  (i) Side sway permitted $C_m = 0.85$

  (ii) Side sway prevented *with transverse Load*

  $C_m = 1.0$ Hinge End $C_m = 0.85$ Fixed End

  without transverse Load :

  $C_m = 0.6 - 0.4(M1/M2) \leq 0.4$

- The Euler Stresses for buckling in X&Y directions:

  $F_{EX} = (7500/\lambda x^2)$ (Codepp.26 Eq.2.36)

  $F_{EY} = (7500/\lambda y^2)$ (Codepp.26 Eq.2.36)

- Example : Design a suitable rolled Section for the column A-B Shown the given figure if the column subjected to the following loads: MD = 20t.m and ND = 20t.(side sway is prevented)

(1) Data Given :

- MD = 20 t.m
- ND = 20t
- Buckling Lengths

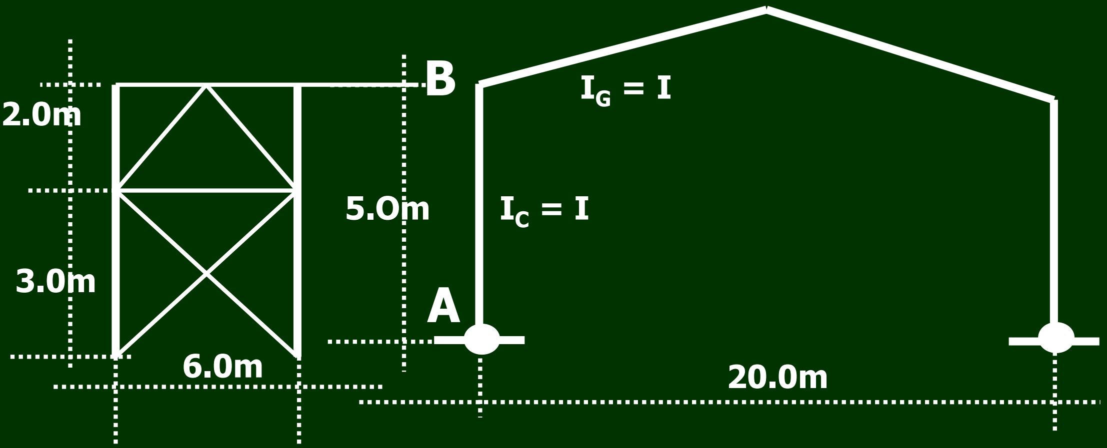
  Lb-in =K *L

  GA = 10.0

  GB = (I/5/ I/10)=2

  From Charts (4-3) (Code pp.61) K = 0.91 Then, Lb-in =K*L=4.55m

  Lb-out = Greater = 3.0m



(2) Estimation of the cross section

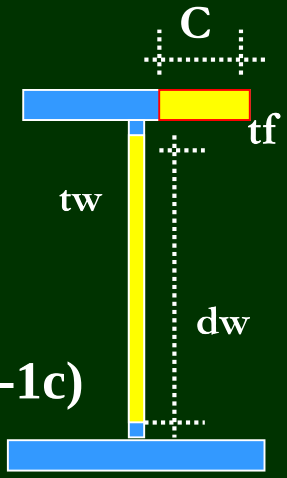
Assume F_b = 0.8 to 1.2 t/cm^2

$Zx = (M_D/F_b) = (20*100/1.0) = 2000\ cm^2$

Try B.F.I.B. No. =320

Zx = 2020cm3, Zy = 661cm3, ix = 13.7 cms, iy = 7.6cms, b = 30cms

A = 171cm2, t_F =2.2 cms, tw = 1.3cms



(3) Check for Local Buckling:

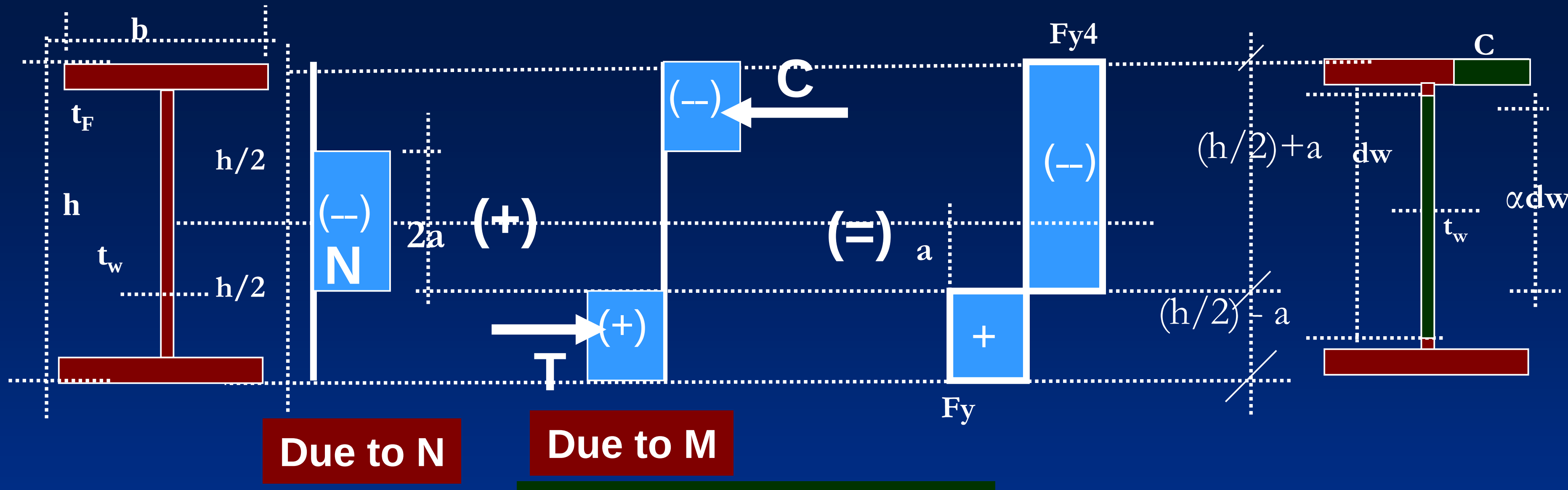
A) Flange Component (Code Page11 - table (2-1c):

$C = 0.5(b_F - t_w - 2t_F) = 12.15cm$

$C/t_F = (12.15/2.2) = 5.52 < (16.9/\sqrt{2.4}) = 10.91$ (Code table 2-1c)

Then, the flange is thus Compact.

B) Web Element (Code Page 9 table(2-1a):



Plastic Stress Distribution

From the stress Diagram distribution the value of "a" is determined as Follows:

$N = 2 \cdot a \cdot tw \cdot Fy$

$20 = 2 \cdot a \cdot 1.3 \cdot 2.4$ Then, $a = 3.2$ cm

$\alpha dw = h/2 + a - 2t_F = 21.2$ cm

$dw = h - 4t_F = 23.2$ cm

$\alpha = \alpha dw / dw = 0.914 > 0.5$

- $(dw/tw)_{Code} = \{(699\sqrt{Fy})/(13\,\alpha - 1)\} = 41.48$ (Code pp.9)
- $(dw/tw)_{Act.} = (23.2/1.3) = 17.84 < 41.48$ (O.K. Compact)

The Section is Compact.

4- Check for Lateral Torsional Buckling

- Lu act = 300cm
- Lu1 = $\{20bf/\sqrt{Fy}\} = \{20.30/\sqrt{2.4}\} = 387.3$ cm >300 Safe
- Lu2 = {1380Af/d.Fy}Cb =

  = {1380*66.0/32*2.4} 1.75 =2075.4cm > 300

  Since Lu1 &Lu2 greater than Lu act , then calculate FLtb.
- F_{Ltb1} = {800.Af.Cb/Luact.d) ≤ 0.58Fy (Eq. 2.23 Code)

  = {800*66*1.75/ 300*32) = 9.625 > 0.58 Fy

  Since FLtb1 is greater than 0.58 Fy = 1.4 t/cm2

  , no need to calculate FLtb2

(5) Check of Stresses (Interaction Equation 2.35 Code pp.25):

$$\frac{f_{ca}}{F_c}+\frac{f_{bx}}{F_{bcx}}A_1+\frac{f_{by}}{F_{bcy}}A_2\leq 1.0$$

Where:

f_{Ca} = (N/A) = 20/171 = 0.117 t/cm^2

λ_x = L_{bx}/ix = 455/13.7 =33.21

λ_y = L_{by}/iy = 300/7.6 = 39.47 Then, λmax = 39.47

F_C = 1.4- 0.000065(39.47)^2= 1.3t/cm^2

(Fca/Fc) = (0.117/1.3) = 0.09 t/cm2 < 0.15

Then, A1 and A2 = 1.0

f_{bc}= Mx/Sx = (20*100/2020) = 0.99 t/cm2

F bc = 0.64Fy = 1.536 t/cm2

$$\frac{f_{ca}}{F_c} + \frac{f_{bx}}{F_{bcx}} A_1 + \cancel{\frac{f_{by}}{F_{bcy}}} A_2 \leq 1.0$$

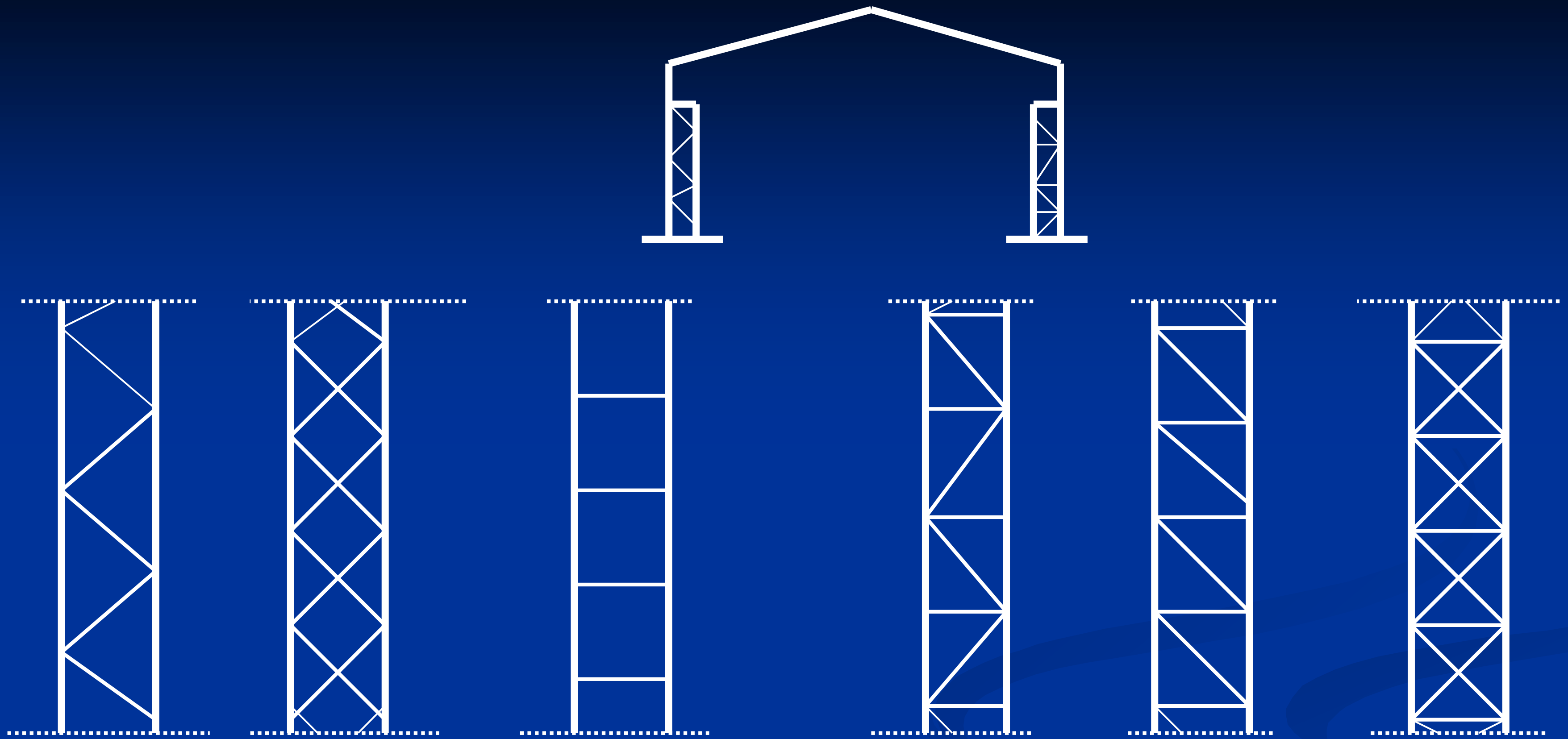
$$\frac{0.117}{1.3} + \frac{0.99}{1.536}(1.0) + \cancel{\frac{0.0}{0.0}} 1.0 = 0.734 \leq 1.0$$

O.K. Safe and Economic

بسم الله الرحمن الرحيم
Lec. 4

Design of Combined Steel Columns with Lacing and Batten Plates

4/1/20



Single Double

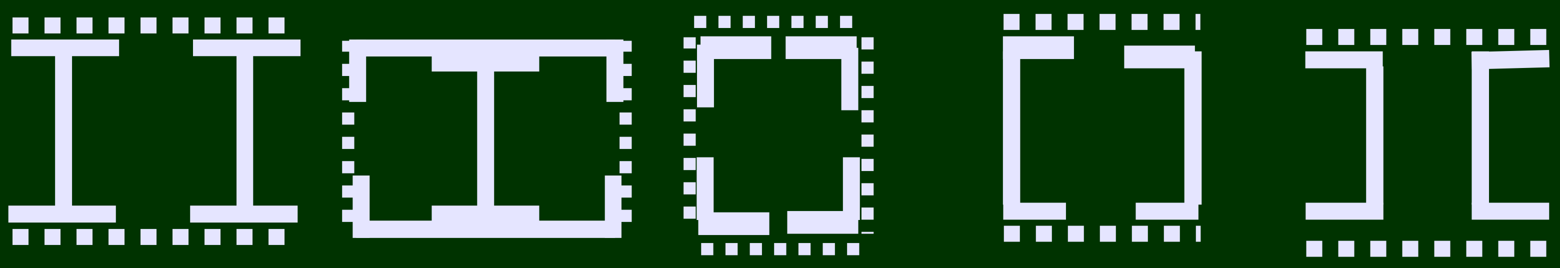
1- Lacing Systems

2- Batten System

3- Combination of Laced and Battened Systems

Systems of Laced and Battened Columns

- Generally, Columns with Combined Section are suitable in case of Large Straining actions or when rolled I-Sections are not readily available.
- The Combined Sections of the Steel Columns are usually chosen to be 2-channels, 2-IPE or 4-angles connected together by Lacing Bars or Batten Plates .



Cross Sections of Built-Up Columns

(II) Combined Columns Subjected to Axial force and Bending Moment (N & M)

Design Procedure of Combined Column subjected to (M &N) :

(1) Data given: {M_D, N_D at Critical Section Along the Column, Side Sway Prevented or permitted and $L_{b\text{-in plan}}$ and $L_{b\text{-out of Plan}}$.

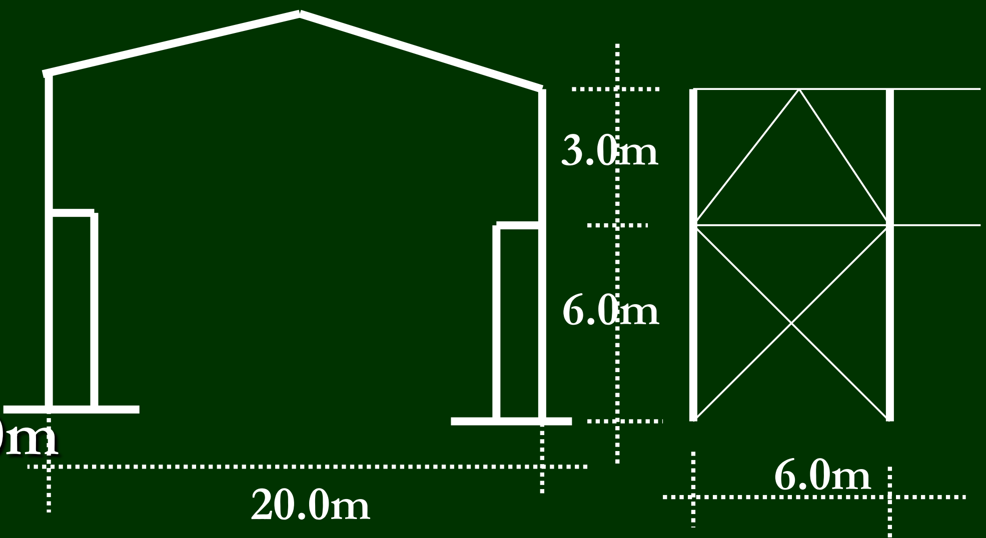
(2) Estimation of the cross section

(3) Check for Local Buckling (Code pp. 9 to 12)

(4) Check of Stresses (Interaction Equation Code pp.25).

- Example :For the pitched Roof Frame shown in Figure is required to design the "AB" as 2-channels spaced 50 cm using lacing bars, if the maximum B.M. at A=30.0t.m, the corresponding normal force, N= 30t and the maximum Shearing Force at A , Q = 4.0 t, Side sway is permitted.

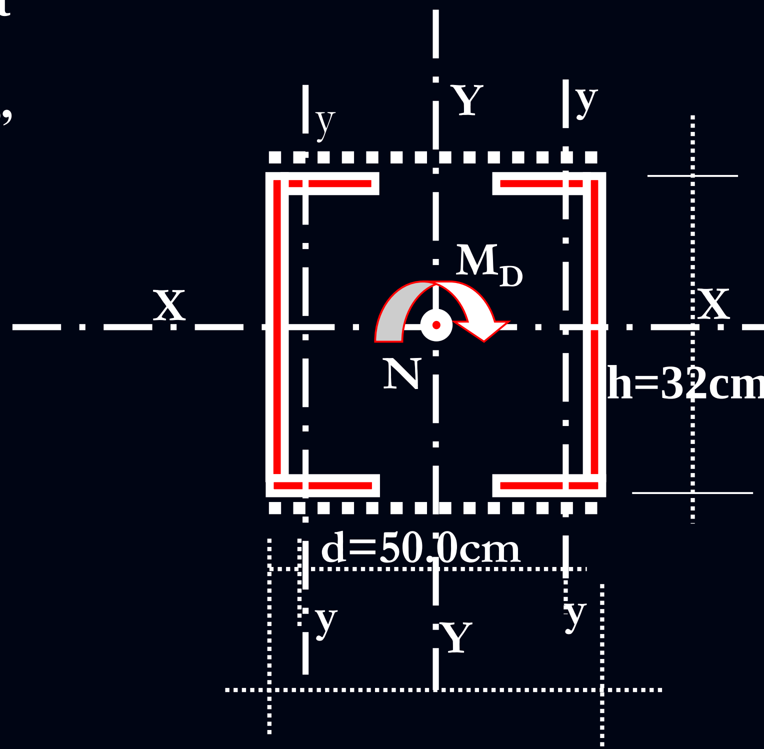
- ***(1) Data Given:***



- MD = 30 t.m
- ND = 30 t
- Q = 4.0t
- $Lb_{in\text{-}plan}$=1.5h =9.0m
- $Lb_{out\text{-}of\ plan}$ = 6.0m
- Side Sway is permitted

(2) *Estimation of the Cross Section:*

- d = Spacing between two Section Component
  = (h/10-15) = (600/10-15) =60-45cm
- Calculating the max. Compressive Force "C"
  C = -(M/d + N/2)=-(3000/50 +30/2)= -75 t
  on one Channel
- $A_{required} = (C/F_{C\ Assumed}) = (75/1.0) = 75cm^2$



Choice 2-Channels 320

d/h= (50/32) =1.56 lies between 1.5 to 2

Then Choose 2-channels 320

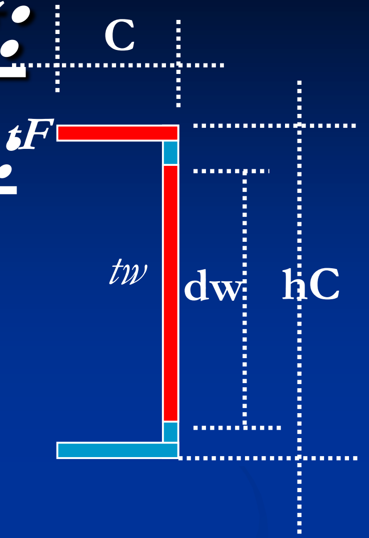
A2C = 2*75.8 = 151.6cm2

$I_{y\text{-}y\text{-}2C} = (2(597 + 75.8*25^2)) = 95944 cm^4$

$r_{X\text{-}X\text{-}2C} = r_{x1C} = 12.1cm$, $r_{y\text{-}y\text{-}2C} = \sqrt{(95944/151.6)} = 25.15cm$

$r_z = r_{y\text{-}1C} = 2.81$

(3) Check for Local Buckling:



Web Element (table2-1a Code pp. 9):

$dw = hC - 4tF = 32 - 4*1.75 = 25cm$

$dw/tw)_{act} = (25/1.4) = 17.86$

$dw/tw)_{Code} = 58/\sqrt{2.4} = 37.44$,

then, $(dw/tw)_{act} < (dw/tw)_{Code}$ O.K. Compact

Flange Element (table2-1c Code pp.11):

$C = bf$ (Code pp.11 table2.1c) $=10cm$

$C/tf)_{act} = (10/1.75) = 5.7$, $(C/tf)_{Code} = (16.9/\sqrt{2.4}) = 10.9$

$C/tf)_{act} < C/tf)_{Code}$ *O.K. Compact.*

(4) *Check of Stresses (Interaction Equation):*

Lb-in =1.5(6) =9.0m , Lb-out = 6.0m , Lz = 90cm ,

k = 1.0 (lacing bars) & K = 1.25 (Batten plate)

$$\lambda_{y-in} = \sqrt{\left(\frac{L_{b-in}}{r_{y-2C}}\right)^2 + \left(k\,\frac{L_z}{r_z}\right)^2} = 48.01 \prec 180$$

(Equivalent Slenderness ratio of Battened or Latticed Steel Columns) Code Eqs.9-1& 9-2 pp.138)

$\lambda_{x\text{-}out}$ =(Lb-out /rx) =(600/12.1) = 49.6

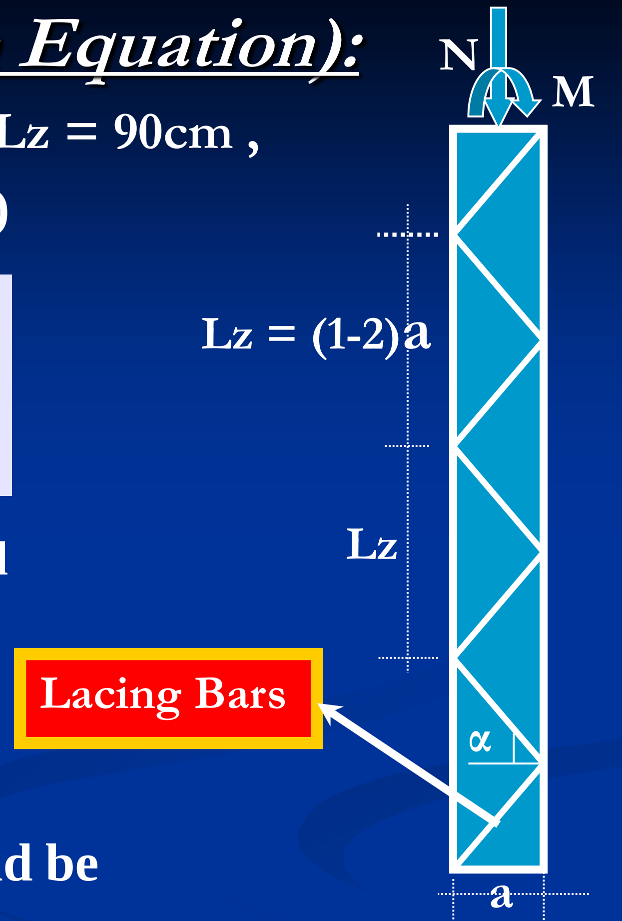
Where :

λz is the Local Slenderness ratio (Lz / rz) should be Satisfied the following Conditions:

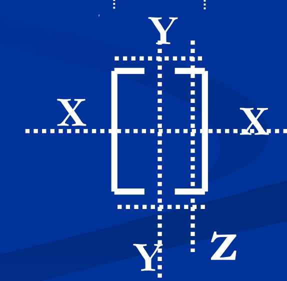
λz = (Lz /rz) < 60 and, (Code pp.136)

and < (2/3) λy-in (Code pp.136)

Then, λz = (Lz /rz) = (90/2.81) = 32.0 < 60 and < (2/3)λy-in



$rz=\sqrt{I_C/A_C}$



- F_C = $1.4 - 0.000065(49.6)^2 = 1.24$ t/cm2
- *fca* = *N/A* = *(30/151.8)* = *0.197*

Then, $Fca/F_C = (0.197/1.24) = 0.159 > 0.15$

- *A2* = $(Cmy/(1-(fca/F_{CY})))$ *(Code pp.25)*
- *Where : Cmy = 0.85 (Frame permitted side sway)* *(Code pp.26)*

$F_{EY} = 7500/\lambda y^2 = 7500/(48.01)^2 = 3.254 t/cm^2$

Then, A2 = $Cmy/(1-(fca/F_{EY})) = 0.96 < 1.0$ *take =1.0*

- *fby* = $(M/I_{y\ -2C})(d/2 + e_C)$

$= (30*100/95944*(50/2 + 2.6) = 0.86 t/cm^2$

- *Fbc* = *0.58Fy* = $1.4\ t/cm^2$

(Interaction Equation):

$$\frac{fca}{Fc}+\frac{fby}{Fbcy}A_2\le 1.0$$

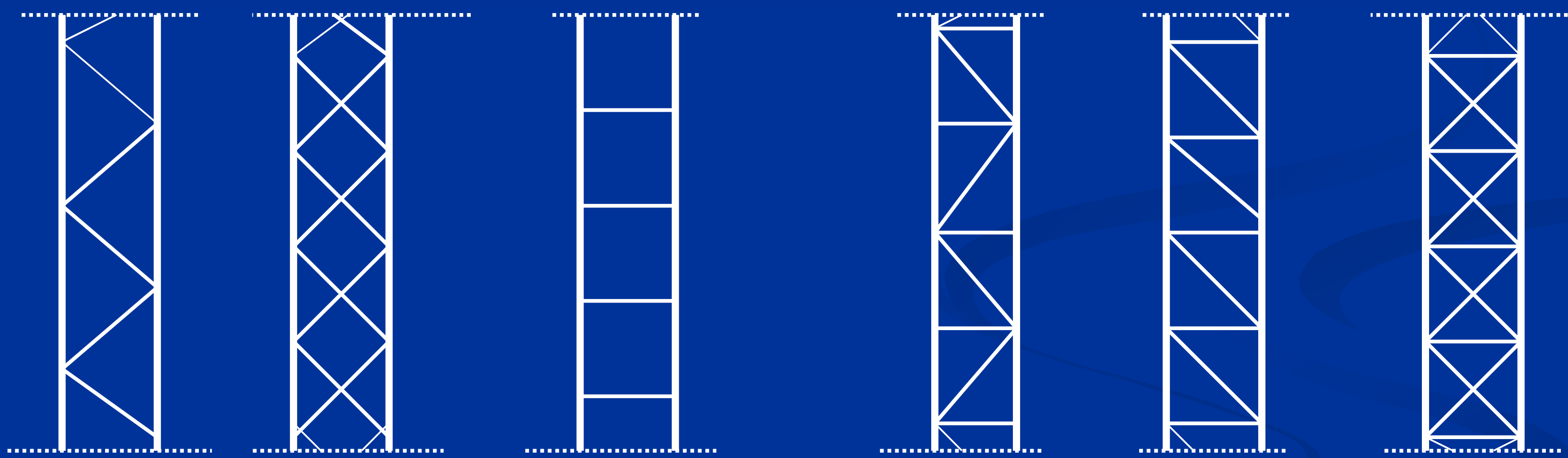
$$\frac{0.179}{1.10}+\frac{0.86}{1.4}*1.00=0.794\prec 1.0$$

O.K. Safe and Economic

Design of Lacing Bars

Introduction:

Systems of Laced and Battened Columns



Single Double

1- Lacing Systems

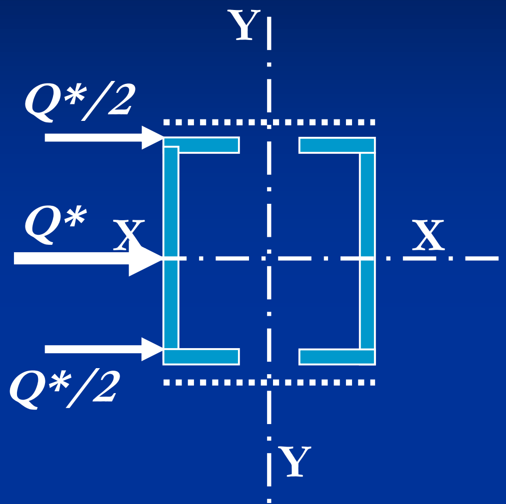
2- Batten System

3- Combination of Laced and Battened Systems

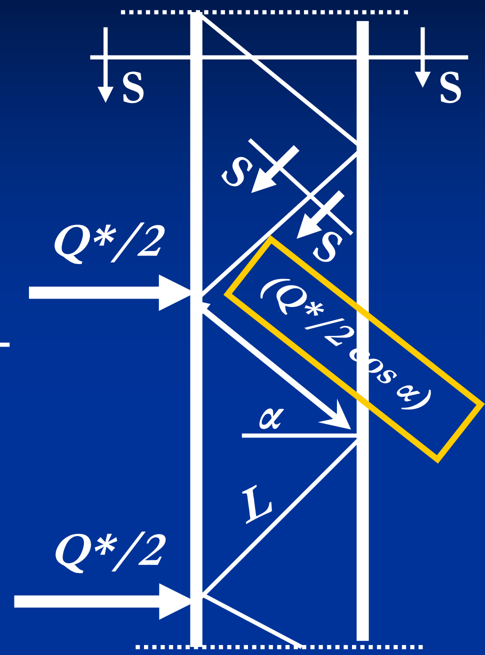
Laced and Battened Columns

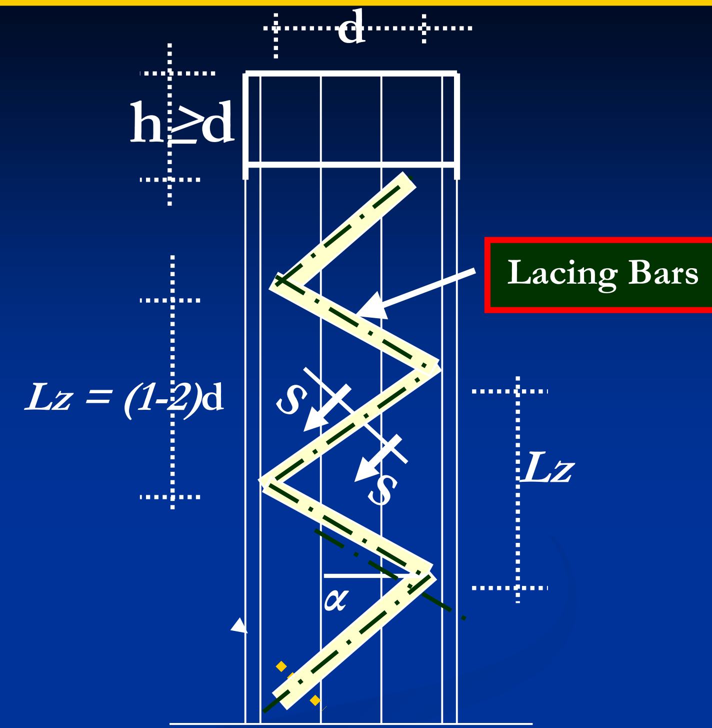
4/10/20

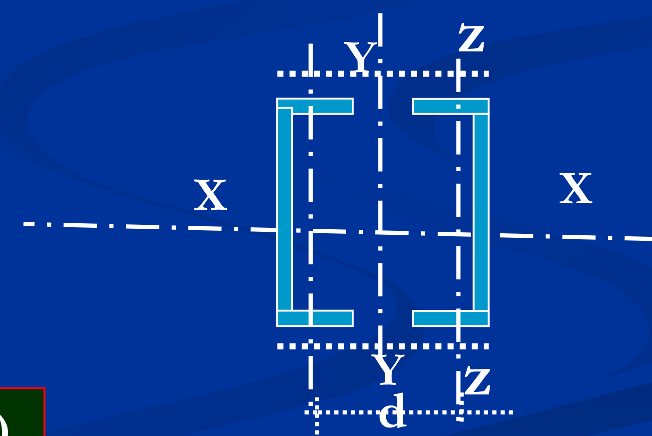
Batten plates are provided at the ends of the lacing systems.



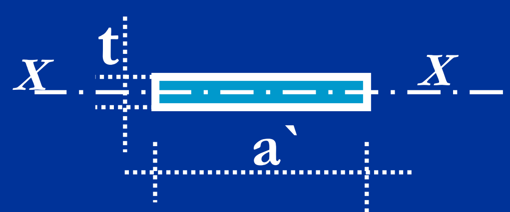
Section S-S







a =(L/10)
t =(L/40)



Section at Lacing Bars(Section S-S)

Notations on Lacing Bars and Batten Plates

(1) Lacing Bars shall be inclined at an angle of $\alpha = 50^o$ to 70^o to the axis of the member where a single interaction System is used and at an angle $\alpha = 40^o$ to 50^o where a double interaction system is used.

(2) Maximum Length between Lacing Bars ($L_z = kL$ = [(1 - 2)d] in a Single Interaction Lacing and L_z = [(0.5-1.0)d] for Double Interaction Lacing) where d is the distance between the Centroids of the main component thickness.

(3) The ratio $(L_z/r_z) \leq 140$

(4) The thickness of the plates ≤ (1/50) of the distance between the inner lines of bolts

(5) Lacing Bares are designed to resist this Force = $Q^* = (Q + 0.02\ N)$. where Q and N are the shearing and the axial forces of the column. The lacing bars are treated as diagonal members of a truss.

- Example : For the frame carrying crane shown in Figure, the crane column subjected to N=20t and M=20t.m and Q = 6.0t ,it is required to : (i) Design the welded lacing bars if the column consists of 2C 320 spaced 50cm. (ii) Design the connection of the lacing bar to the column as Revited connection.

(1) Data Given:

$Q^* = Q + 0.02N = 6.4t$

Force in Lacing Members (F_{Lacing}):

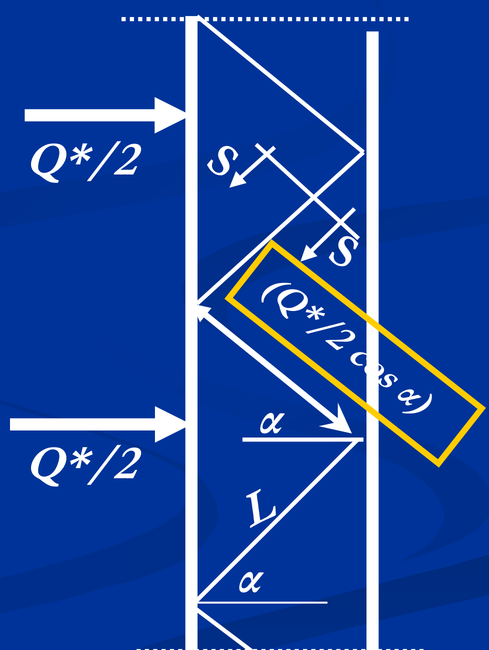
. $F_{Lacing} = Q^*/2\cos\alpha = 6.4/2\cos 45^o = \pm 4.525t$

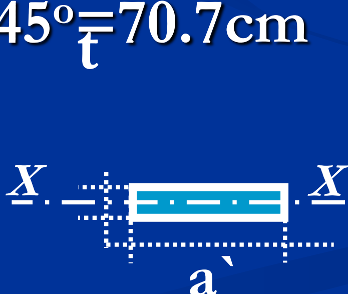
Length of Lacing $= L = 50/\cos 45^o = 70.7cm$

Assume a = L/10 = 7cm

Assume t = L/50 = 14m

Lbx = L = 70cm





Section S-S

(2) Design of the Lacing bar as a Compression Member:

- $rx = (\sqrt{Ix\text{-}x/Area}) = \sqrt{(at^3/12)/a*t} = t/\sqrt{12} = 1.4/\sqrt{(12} = 0.404$
- $\lambda x = L_{bx}/\ rx = 70/0.404 = 173.2 > 140$ Unsafe
- Use $t = 2.0$ cm
- $rx = (t/\sqrt{12}) = 2.0/\sqrt{12} = 0.577$
- $\lambda x = Lbx/rx = 70/0.577 = 121.317 > 140$
- $F_C = (7500/(121.317)^2 = 0.51 t/cm2$
- $f\,act = F_{Lacing}/Area = (4.525/(7*2) = 0.323 t/cm^2 < F_C$ O.K. Safe

(3) Design of Lacing bar as a Tension Member:

- $f_{act} = (F_{Lacing}/Area) = (4.525/(7*2) = 0.323 t/cm^2 < 1.4\ t/cm^2$
- $(L/a) = (70/7) = 10 \leq 60$ O.K. Safe

(4) *Design of the Connection between the batten plate and column as a Welded Connection:*

Assume Weld Size =s =8mm

- Force = Area of Weld *Allowable stresses in Weld

  5.525 = (L*2s)(0.2Fu) = (L*2*0.8)(0.2*3.6)

  get L = 4.8cm

- $L_{\text{weld eff.}}$ = 2s +L = (2*0.8+4.8)=6.5cm   min L = 5.0cm

(5) Design of the Connection between the batten plate and column as a Bolted Connection:

Force = 5.525 t $\leq R_{\text{Least}}$ Assume Diameter of Bolts Φ_{16}

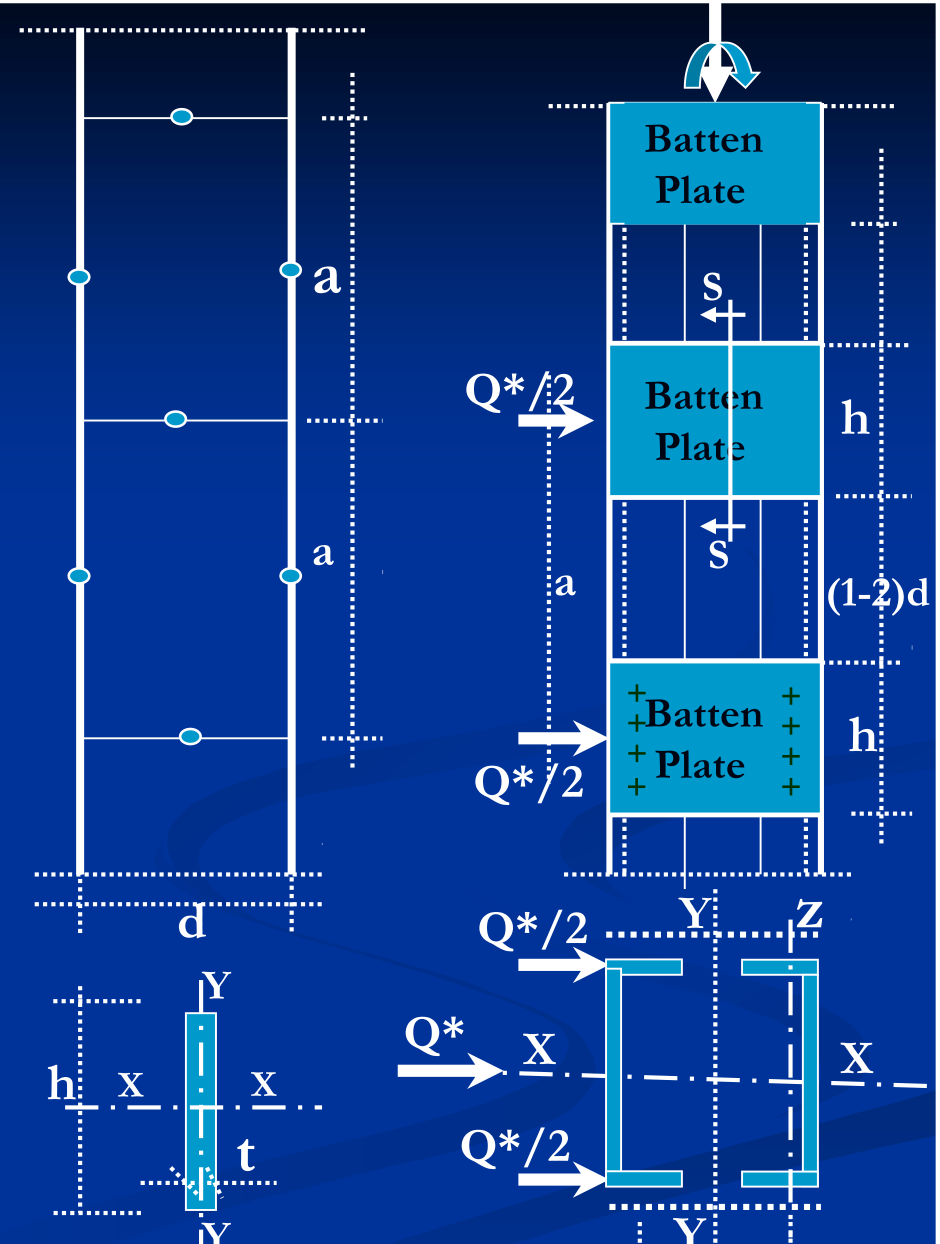
- R_{Least} is the Least of $R_{S.S.}$ and R_{bearing}.

Design of Batten Plate

- Battens shall be plates or Channels and shall be bolted or welded to the main elements.
- The system may be assumed as a virandeel girder or intermediate hinges may be assumed at mid distance to change the system into a statically determinate system.

$h=(0.75-1.25)d \approx d$

$a=h+(1-2)d$



Section in Batten Plate (S-S)

Design Steps of Batten Plate :

(1) Data Given:

$h = (0.75\text{-}1.25)d \approx d$

$\text{Thickness} = t = (h/50)$

$Q^* = Q + 0.02N$

$I_{x-x} = (t^*h^3/12)$ weld Batten

$I_{x-x} = ((t^*h^3/12) - I\Phi)$

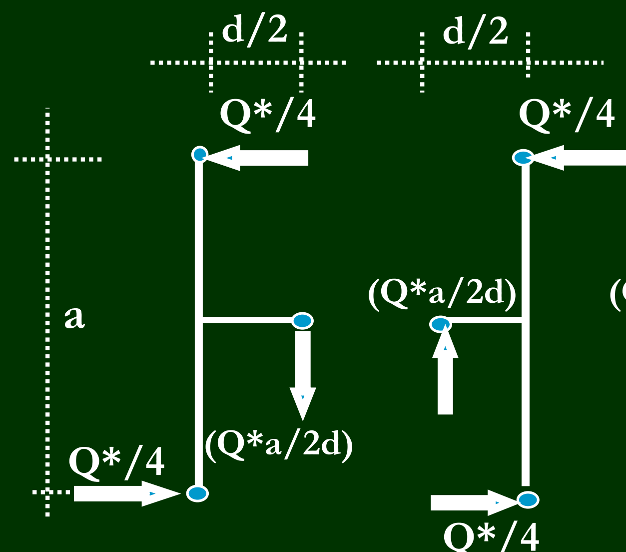
(Bolted batten)

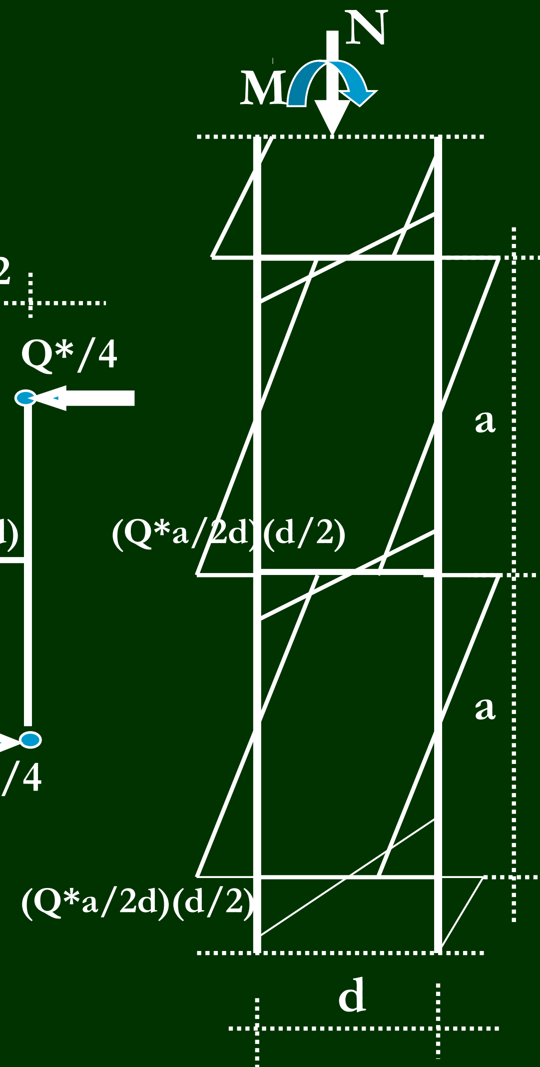
The acting straining actions on the Batten Plate are :

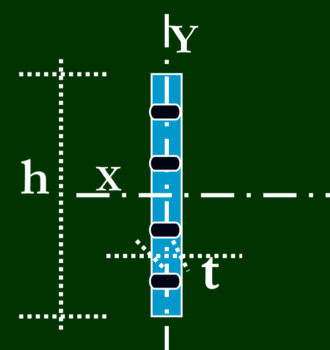
S.F. $= (Q^*.d/2a)$

B.M. $= \text{S.F.}^*(d/2)$

$= (Q.a/2d)(d/2)$







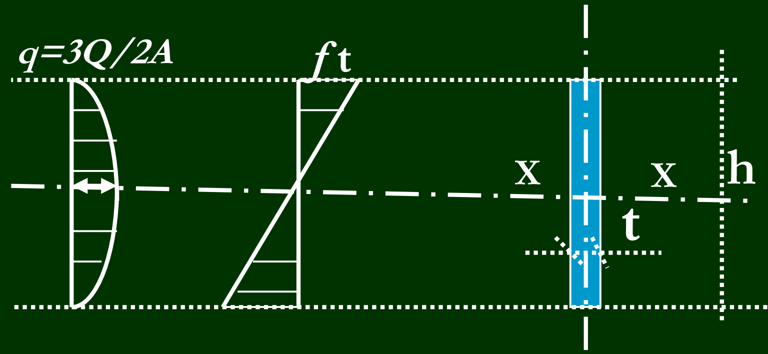
Section in Batten Plate (Bolted Conn.)

(2) Check of Flexure Stresses:

$f_b = (M_x/I_x)(h/2) = \ldots\ldots \leq 0.72F_y$

(3) Check of Shear Stresses:

$Q\ act = 1.5Q/h*t) = \ldots. \leq 0.35F_y$



(4) Design of the Connection between the batten plate and column as a Bolted Connection:

$Q \quad = \quad Q^*.a/2d$

$Mt \quad = \quad (Q^*.a/2d)\ (d/2)$

The used Bolts put either in one column or in two column (4-rows)

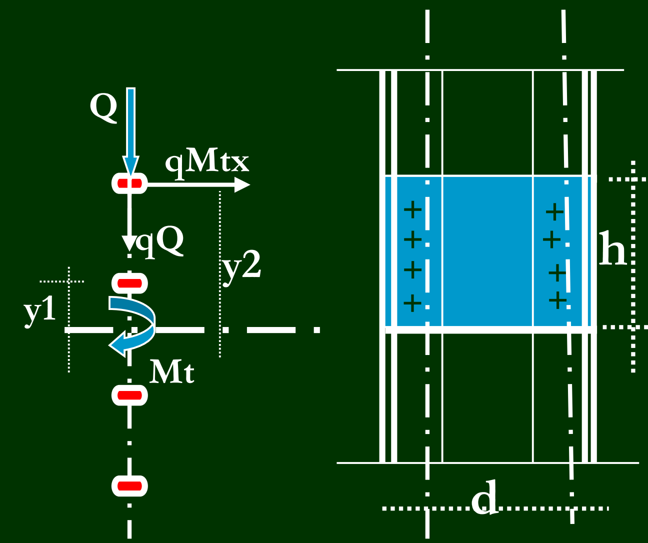
Case1 : to use one row of 4-bolts :

Check of Stresses:

$q_Q \downarrow = (Q^*/n) = \ldots..t$

$q_{Mtx} \longrightarrow = (Mt\ .\ y2)\ /\ 2(y1^2 + y2^2) = \ldots.t$

$Qtotal = \sqrt{(qQ)^2 + qMtx^2} = \ldots.t$



Section in Bolts

Rs.s.h. $= \pi\, \Phi^2 /4\ (0.25F_{U\ bolt})$ =….t

Qtotal ≤ Rs.s.h O.K. Safe

Case2 : to use two rows of 4 – bolts (n=8Φ):

Check of Stresses:

qQ ↓ = (Q*/n) = …..t

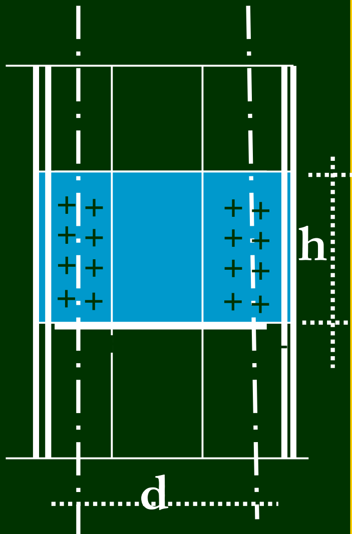
qMtx ⟶ = $(Mt\ .\ Y) / n(r1^2 + r2^2)$ = ….t

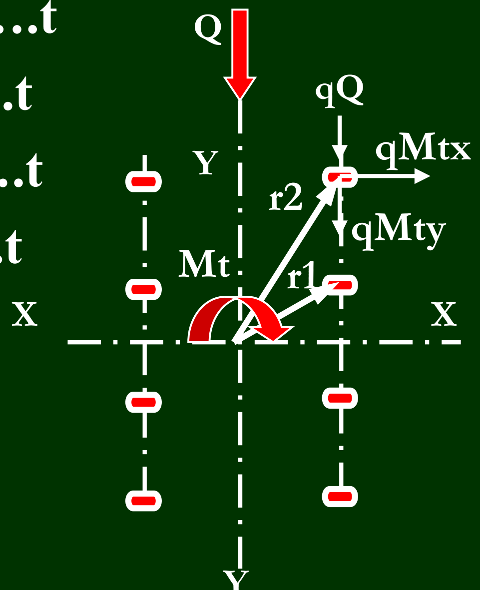
qMty ↓ = $(Mt\ .\ X) / n(r1^2 + r2^2)$ =….t

Qtotal = $\sqrt{(qQ+qMt)^2 + qMt^2}$ = ….t

Rs.s.h. = $(\pi\, \Phi 2 /4)\ (0.25F_{U\ bolt})$ =….t

Qtotal ≤ Rs.s.h O.K. Safe





Section in Bolts

4/19/20

(5) Design of the Connection between the batten plate and column as a Welded Connection:

Assume weld size = "S"

Each side is calculated to resist

A shear force = Q = (Q*.a /2d) and

torsional Moment (Q*.a/2d) (d/2)

X` = (2.b.s)(b+s/2)/(2bs +(h+2s)s)

Ix-x = ……..

Iy-y =……….

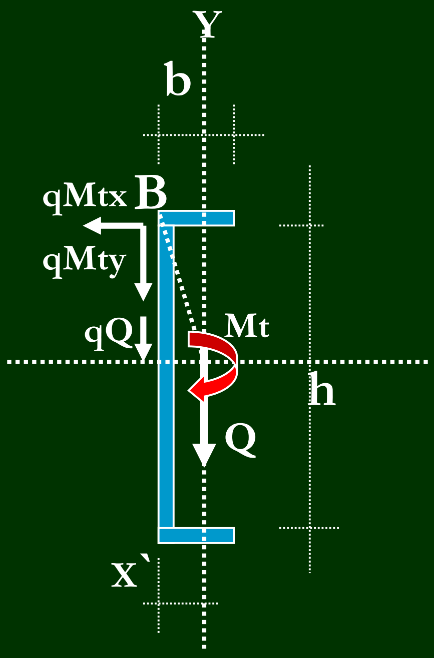
Ip = Ix-x + Iy-y

qQ B ↓ = Q/(h+2s)s =

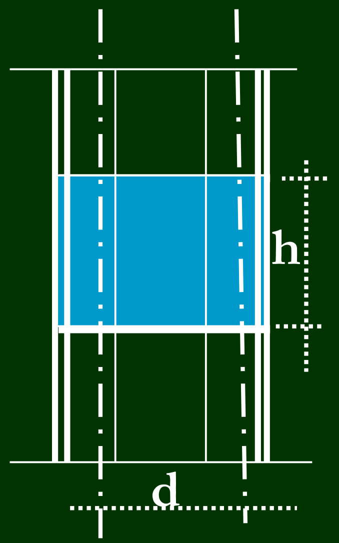
qMty B ↓ = Mt *X`/Ip =

qMtx B → = Mt*(h/2)/Ip =

qB total = $\sqrt{(qQB\downarrow + qMty\downarrow)^2 + qMtxB^2_{\rightarrow}}$ =……≤ 0.2Fu



Section of Weld



Lec. 5

Design of Steel Beams

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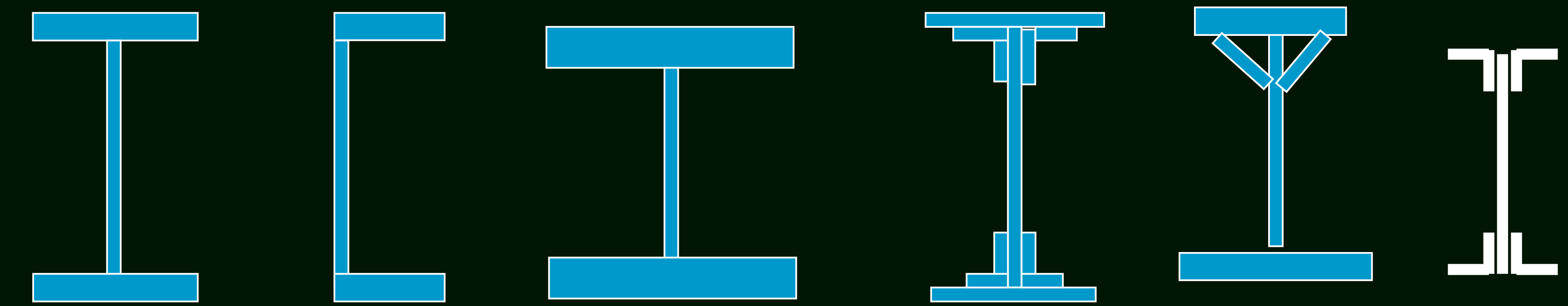
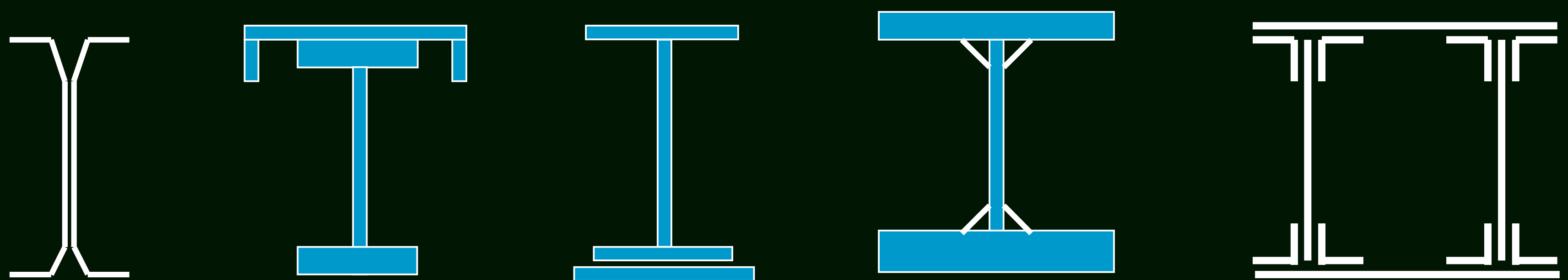
Common Beam Cross Sections



S.I.P or I.P.E　　Channel　　B.F.I.　　Built up sections



Built up sections

Performance and Behavior of Steel Beams:

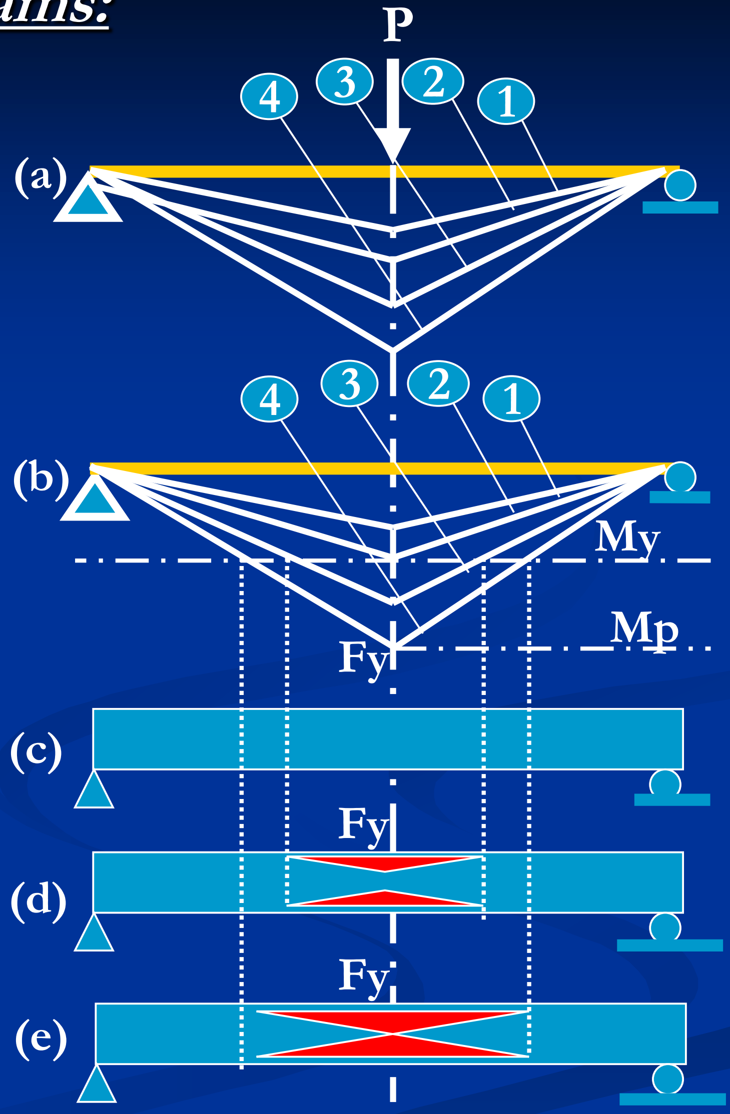
The behavior of a beam under Concentrated Load at mid-span illustrated by the following example.

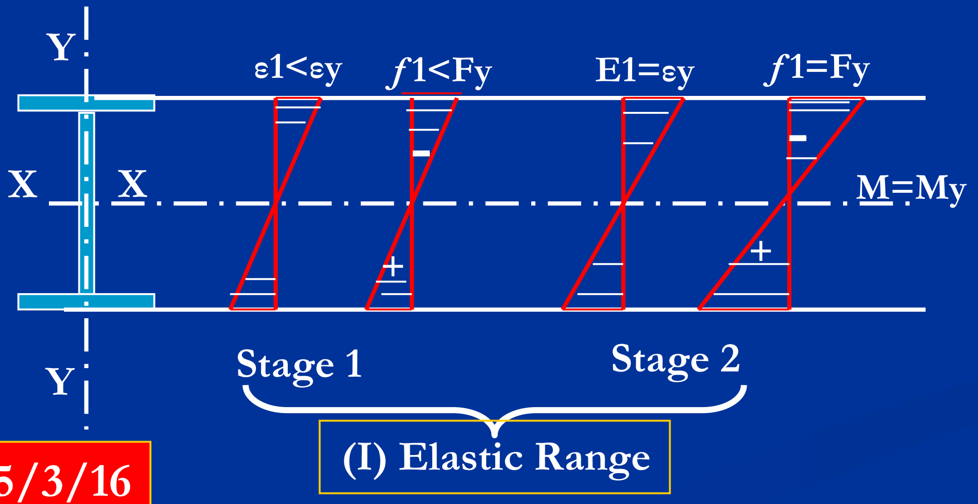
The max. moment, outer - fiber strain and stresses occur at mid-span

Successive Stages in bending are shown in This figure .

(I) Elastic Range [Stage (1) and Stage (2)]

(II) Plastic Range [Stage (3) and Stage (4)]





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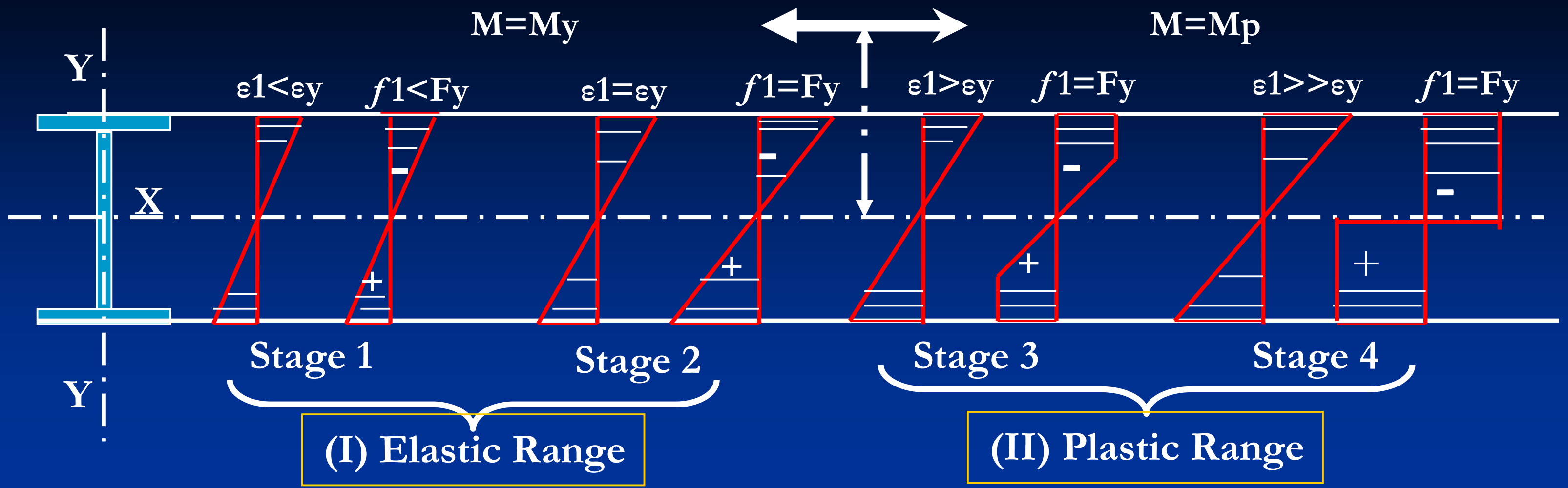
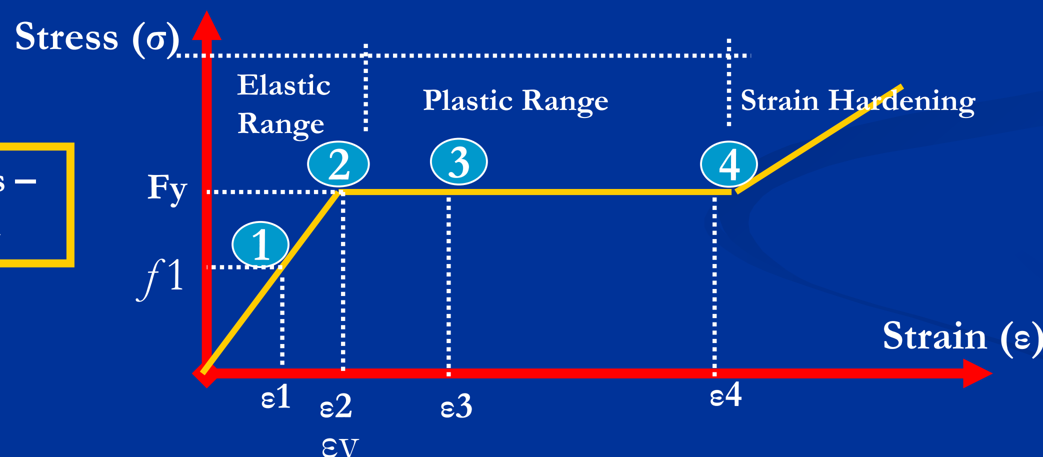


Variation in Strain and Stress Along The Depth of the Beam

Idealized Stress – Strain Diagram



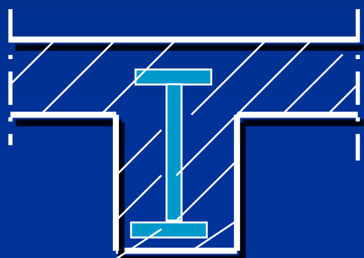
Strength in Elastic Range = My = Fy *Sx , Zx = elastic Section Modulus

Strength in Plastic Range = My = Fy *Sp , Zp = Plastic Section Modulus , and,

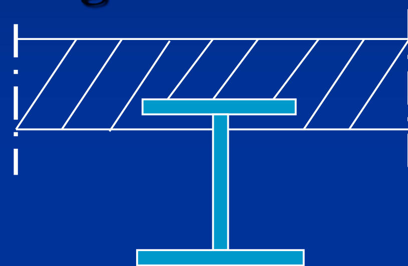
ζ =(Mp/My) = (Zx/Zp) = 1.1 – 1.18 For I- Sections and = 1.5 for Rectangular Section

Fully Laterally Supported Beams (Floor or Roof Beams)

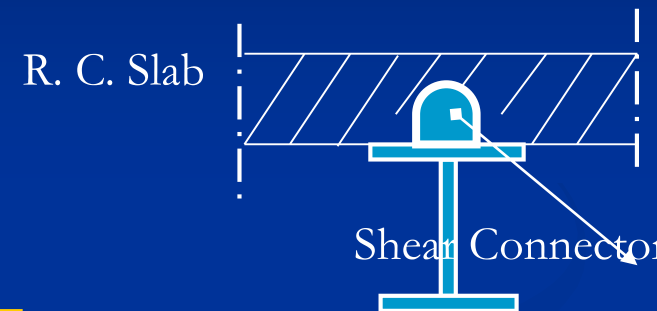
Flexural Members have the Compression Flange Restrained to Satisfy Lateral Support . This is True in Beams Supporting floors and Roofs, where the beam is in direct contact or has the top Flange embedded in the floor or Roof Slabs as the Following:



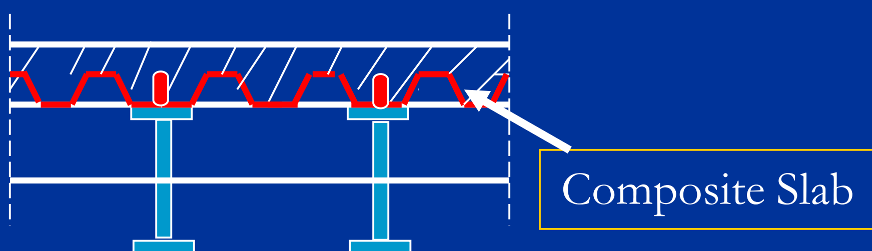
Steel Section imbedded in Concrete Section



Top Flange embedded in Concrete Slab



Top Flange Connected with R. C. Slab by Shear Connectors

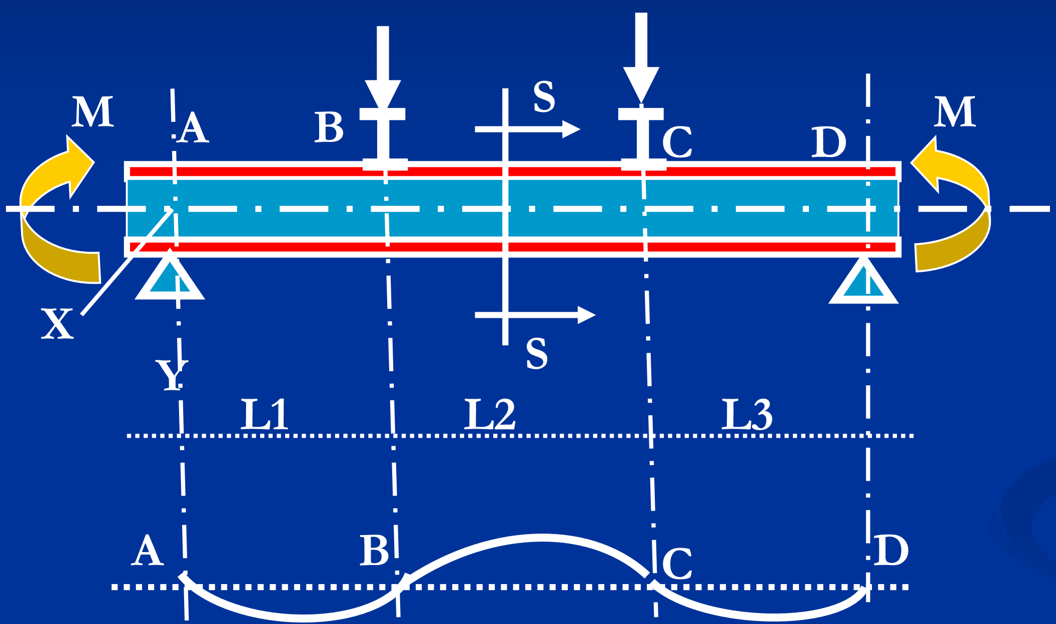


Beam is Laterally Supported by Composite Slab

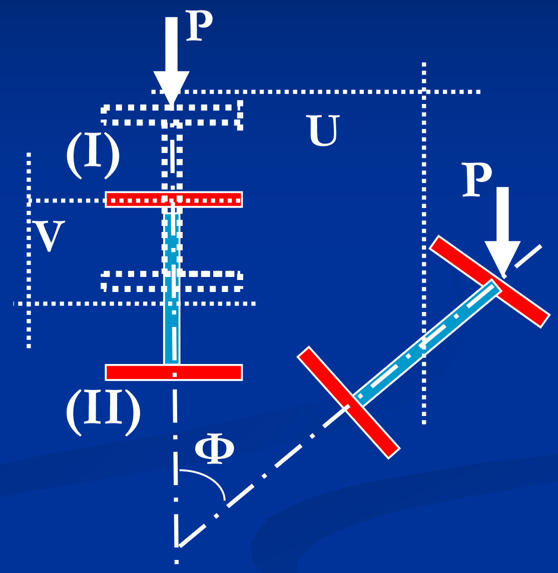
$$L_U act = 0.0$$

Laterally Unsupported Beams

Torsion will be combined with Bending when the Line of action of Load is not passing through the Shear Center of the cross section



Main Beam "A-D" provided by Lateral Supported by Lateral Beams at B & C "Buckling of Beams Provided with Lateral Secondary Beams"



Section S-S

LU act =Greater (L1, L2 & L3)

***Design Procedure of Steel Beams*:**

(1) *Data given:* {M_D, ,Q &b N_D at Critical Section Along the Beam, B.M.D, Unsupported Length "L_{Uact}" in Regions 1 & 2 .

(2) Estimation of the cross section.

Zx = ($M_D/F_{all.b}$) = …cm2 Choice S.I.B No….

(3) Check for Local Buckling (Code pp. 9 to 12)

- (C/tf)act =…. .≤ (C/tf) Code
- (dw/tw) act = …. ≤ (dw/tw) Code

(4) Check of the Safety Against Lateral Torsional Buckling:

How to Determine the Allowable Bending Stresses (Fbx)

Determine Lu act and,

$Lu1max = (20bf/\sqrt{Fy}$ and,

$Lu2max = (1380Af.Cb/d.Fy)$ (Code pp.16, Eq.2.18)

- If Lu act < Lu1 max and Lu2 max ,the section is compact and safe against Lateral Torsional Buckling > In this case Allowable Bending Stresses

$$\boxed{Fbx = 0.68Fy}$$

- If Lu act > Lu1 max or Lu2 max ,the Section is Non-Compact. In this Case Allowable Bending Stresses =0.58Fy and must be Check for Lateral Torsional Buckling as Follows.

☞ Calculate → $FLtb1 = (800.Af.Cb/d.Luact)$

If $F_{Ltb1} \geq 0.58F_y$,then the section is safe against Lateral Torsional Buckling and stop

$$\boxed{F_{bx} = F_{Ltb} = 0.58F_y}$$

If $F_{Ltb1} < 0.58F_y$,then Find the *F_{Ltb2} as follows :*

When $L_{uact} < 84\sqrt{(C_b/F_y)}$ (Code pp.18)

$\Rightarrow$ $F_{Ltb2} = 0.58F_y$

When $84\sqrt{(C_b/F_y)} \leq L_{uact}/r_T \leq 188\sqrt{C_b/F_y}$

$\Rightarrow$ $F_{Ltb2} = (0.64 - (L_{uact}/r_T)^2 F_y / 1.176*.10^{5}*C_b)F_y \leq 0.58\ F_y$

When $L_{u\,act}/r_T > 188\sqrt{C_b/F_y}$

$\Rightarrow$ $F_{Ltb2} = 12000(L_u/r_T)C_b \leq 0.58F_y$ (code pp.18)

Then,

$$F_{Ltb} = \sqrt{F_{Ltb1}^2 + F_{Ltb2}^2} \leq 0.58F_y \quad \text{(code pp.19)}$$

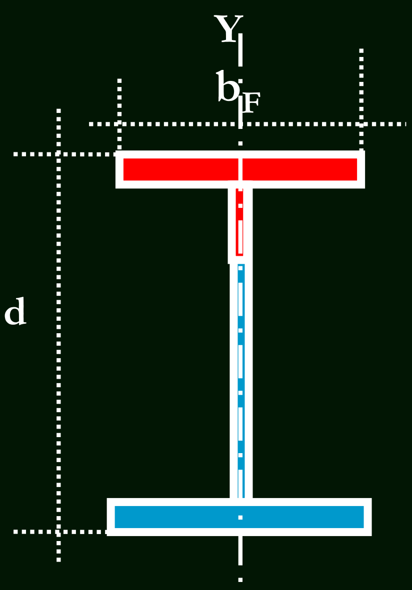
$\Rightarrow$ $\boxed{F_{bx} = F_{Ltb}}$

5/9/16

Where : rT = Radius of Gyration about minor axis of section concerning the compression flange plus sixth of the compression area of web

$rT = \sqrt{Iy\text{-}y / Af} =$

$Iy\text{-}y = tf.(bf)^3/12 + (hw/6)\,(tw)^3/12$



Summary of Allowable Bending Stresses:

Section	Fbx	Fby
Compact - Supported	0.64Fy	0.72Fy
Compact - Unsupported	F_{Ltb}	0.72Fy
Non-Compact - Supported	0.58Fy	0.58Fy
Non-Compact - Unsupported	F_{Ltb}	0.58Fy

(5) Check for Deflection :

The Calculated Deflection due to WL.L only

$$\Delta = \left(\frac{PL^3}{48EI}\right) \quad Concentrated \quad Load \quad Simple \quad Span$$

$$\Delta = \left(\frac{5WL^4}{384EI}\right) Uniform \;\; Distributed \quad Loads \quad Simple \quad Span$$

$$\Delta = \left(\frac{5L^2}{48EI}\right)\left(M_S - 0.1\left(\frac{M1}{M2}\right)\right) Uniform \;\; Distributed \quad Loads$$

$$Continous \quad Span$$

Where : Ms is the Absolute value of the mid-span moment

and M1 and M2 unequal end moments

Δact = Δ = Mid-span Deflection

ΔCode = See Code (pp.132 table 9-1)

(6) Check of Stresses :

Bending Stresses = $f bc = M_D * 100 / Zx$ = .. Fb = …t/cm^2

Shear Stresses = q act = Q/A_W =..qall. Shear = 0.35Fy

Equivalent Stresses = $\sqrt{f bc^2 + 3q^2 act}$ =…$t/cm^2 \leq 1.1 * F_{cb}$

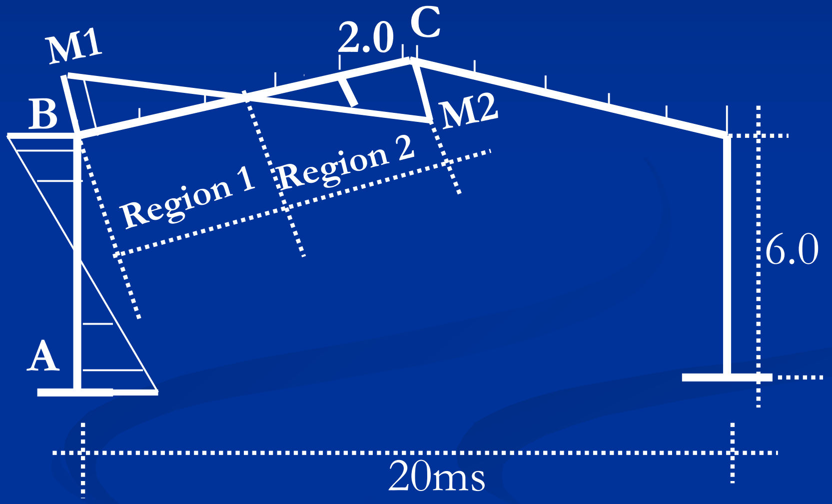
Where:

Fbc =(0.64Fy in Compact Sec. or 0.58Fy in Non-compact Sec.)

Example: Design the girder B-C shown in the following Frame.The design values of the straining action are (MD = 20.0t.m, Q =4t and N= -5t , M1 = M2 , Spacing between Purlins = 2.1 m, $W_{L.L}$ = 0.3t/m.

(1) Data Given :

M_D=20.0t.m, Q =4t and N= -5t



(2) Estimate of the cross Section

$Zx = (MD/f_{\text{Assume}}) = 1667.0\text{cm}^3$

From table Choice B.F.I No.300

tw=1.1cm, tf=1.9cm., ix=13cm

iy=7.57, I_X = 30820cm^4, Iy=9240cm^4

A = 149cm^2, Ix = 25170cm^4, Iy=8560cm^4, Zx=1680cm^3

(3)Check for Local Buckling :

Web Element:

$Dw = (h - 4tf) = 30 - 4*1.9 = 22.4cm$

$(dw/tw)_{act} = (22.4/1.1) = 20.36$,

$(dw/tw)_{Code} = (127/\sqrt{Fy}) = 82$ (Table 2-1a Code p.9)

$(dw/tw)_{act} \leq (dw/tw)_{Code}$ Compact

Flange :

$C = 0.5(bf-tw-2tf) = 12.55cm$

$(C/tf)_{act} = (12.55/1.9) = 6.6$, $(C/tf)_{Code} = (16.9/\sqrt{Fy}) = 10.9$ (table 2-1c)

$(C/tf)_{act} < (C/tf)_{Code}$ Compact , The section is Compact.

(4) Check for Lateral Torsional Buckling:

Region 1 :

L_U act $= (LG/2) = 10.1/2 = 505.0cm$

L_{U1} max $= 20bf/\sqrt{Fy} = 387.3$ cm , $\alpha = 0.0$

$Cb = 1.75 + 1.05\alpha + 0.03\alpha^2 = 1.75$

L_{U2} max $= 1380.Af.Cb/(h.Fy) = 1911.87$ cm

- $L_U$act > Lu1max and < LU2max ,then

The Section is Unsupported

The section will be treated as a Non-Compact and must be checked for Lateral Torsional Buckling as the Following:

$FLtb1 = (800.Af.Cb./Lu.d) = (800*30*1.9*1.75)/505*30) = 5.27 t/cm^2$

$FLtb1 > 0.58Fy = 1.4 t/cm^2$, then Stop

$FLtb = FLtb1$, $Fbx = FLtb = 1.4\ t/cm^2$

The Section is Safe against Lateral Torsional Buckling.

Region 2 :

$L_U act$ = Spacing between purlins = 210 cm

$L_{U1}\ max$ = $20bf/\sqrt{Fy}$ = 387.3 cm

α = (0.6M2/M2) = 0.6

Cb = $1.75 + 1.05\alpha + 0.03\alpha^2$ = 2.4 >2.3 take Cb = 2.3

$L_{U2}\ max$ = 1380.Af.Cb/(h.Fy) = 2512.75 cm

- $L_U act$ < Lu1max and LU2max , then

  The section is Compact and,

  The section is supported safe against Lateral Torsional Buckling. In this Case Allowable Bending Stresses

  Fb = 0.68Fy

(5) Check for Deflection:

$$\Delta_{Act} = \left(\frac{5W_{L.L.}L^4}{384EI} \right) Uniform\ \ Distributed\ \ \ Loads\ \ \ Simple\ \ \ Span$$

$$\Delta_{Act} = \left(\frac{5*0.3*(1000)^4}{384(2100)(30820)100} \right) = 1.2cm$$

$$\Delta_{Code} = \left(\frac{Span}{200}\right) = \frac{1000}{200} = 5cm$$

$$\Delta_{Act} < \Delta_{Code} O.K.Safe$$

(6) Check of Stresses

Bending Stresses = $f$bc = M_D *100/Zx =2000/1680 = 1.19 t/cm^2

Shear Stresses = q act = Q/A_W =(4.0/26.2*1.1)= 0.139 < 0.35Fy = 0.84 t/cm2

Equivalent Stresses = $\sqrt{f bc^2 + 3q^2 act} = \sqrt{1.4741}$ =1.21 t/cm^2 ≤ 1.1* Fbx

End of Lec 5

بسم الله الرحمن الرحيم

Lec.600

Analysis and Design of Steel Connections

Different Types of Steel Connections:

(1) Concentric Shear Connections

(i) Truss member Connections
(ii) Splices of Truss Members
(iii) Splices of Column Flanges
(iv) Continuity of Beams
(v) Splices of Girder Flanges

(2) *Eccentric – Shear Connections*

(i) Crane Bracket Connections
(ii) Flexible Beam to Column Connections
(III) Flexible Secondary–to-Beam Connections
(iv) Splices of Column` Web
(v) Splices of Beam`s Web

(3) Pure Tension Connections

(i) Hanger Connections

(ii) Truss-to-Column Connections

(4) Moment Connections

(i) Beam to Column T Connections

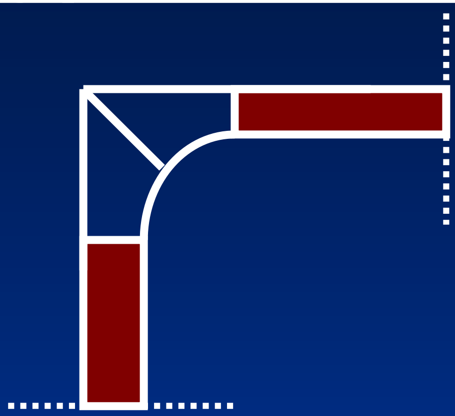
(ii) Bracket Connections

(iii) Beam Column Rigid Connections

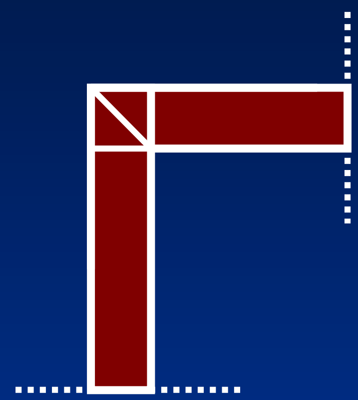
(iv) Seat Angle Connections

(v) Column Truss Connections

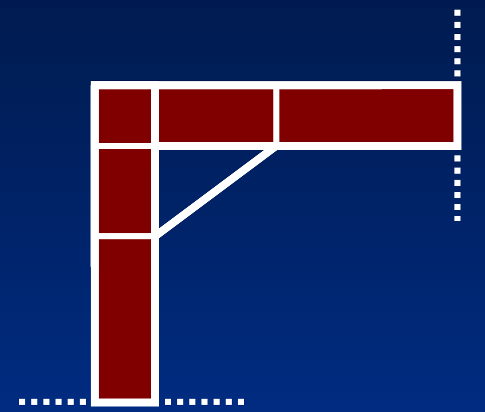
Types of Corner Connections:



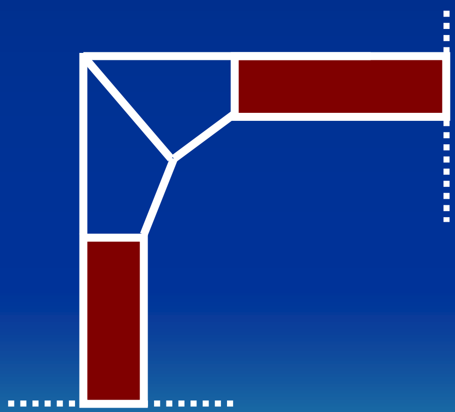
(i) Curved Corner



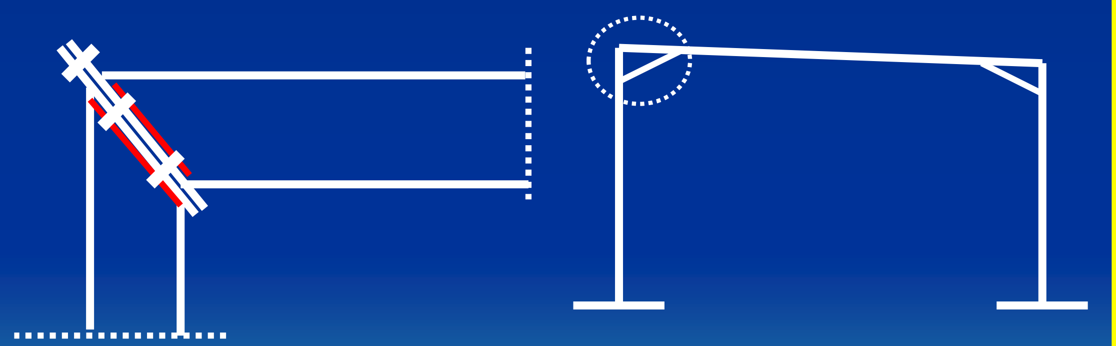
(ii) Square Corner



(iii) Corner with Haunch



(iv) Corner with tapered Haunch

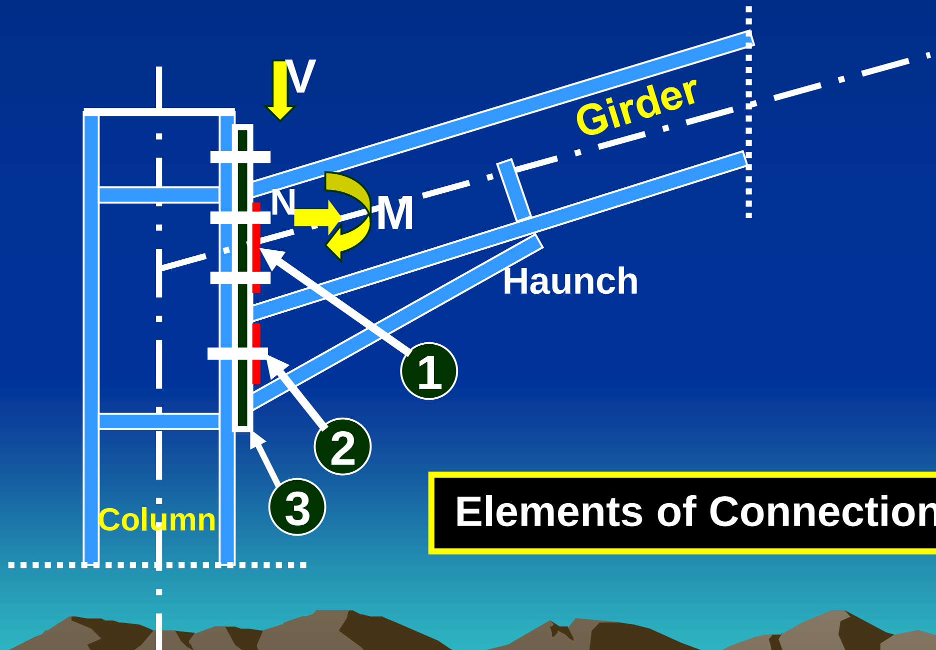


(v) Corner with Diagonal End Plate

Design of Pitched Roof Frame Connections
Rigid Beam – to - Column Connections.

Corner Connections (Rigid Connections):

Rigid Connections are those transmitting the B.M.,S.F. and N.F from one Structure Element to the Other.



Elements of Connection { 1 **Weld** 2 **Bolts** 3 **End Plate** }

Design Steps of Corner Connections:

***(I)Data Given*: B.M., S.F., N.F., Bolts, Column and Girder Cross Sections,**

(2) Design of Weld:

(i) Check For weld thickness

(Use Haunch or not)

$$N` = N \cos\alpha - Q \sin\alpha$$

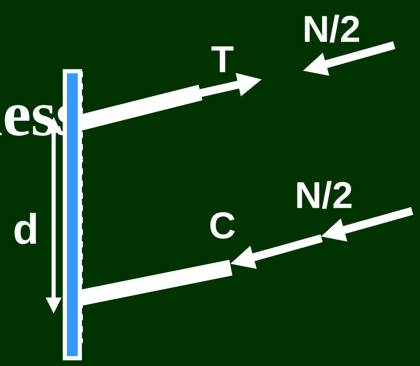
$$Q` = N \sin\alpha + Q \cos\alpha$$

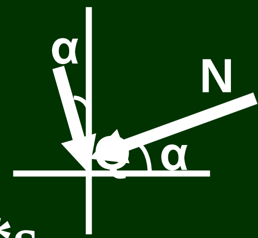
$$T = C = (M / d)$$

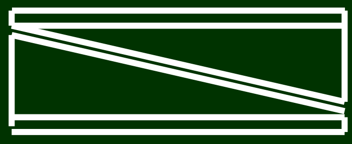
$$A = (T – N`/2) / (0.2F_U)$$

$$Area = [(bf + (bf – tw -2tf)]*s$$

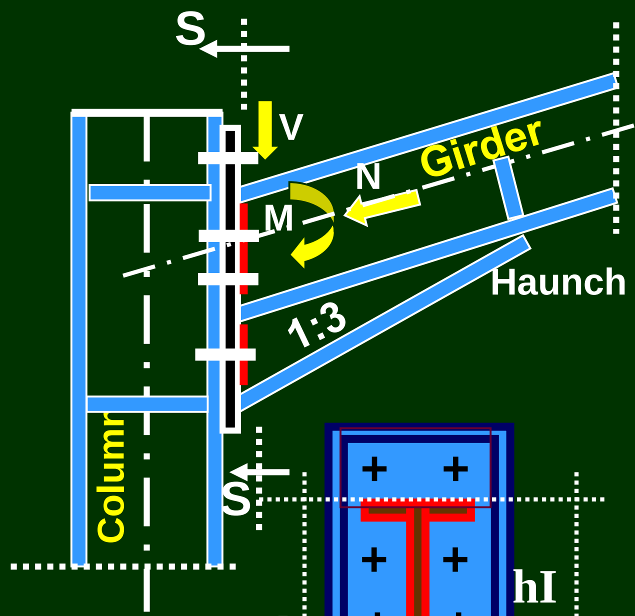
get "s"

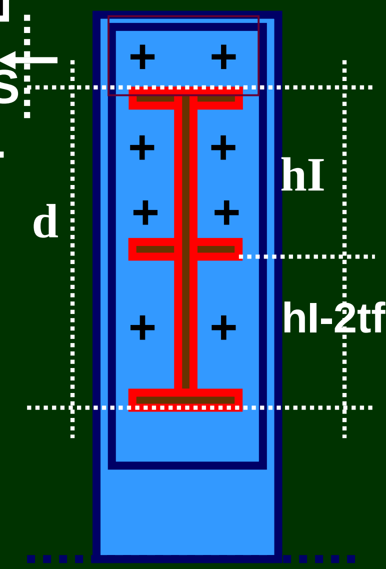






Haunch





Section S –S

If "s" greater than thickness of the flange then, use Haunch

(ii) Flange Weld:

$T = C = (M/d)$ $\quad d = h_I + (h_I - 2tf)$

$Area = (T - (N`/2)/0.2F_U = (bf*sf + (bf - tw - 2tf)sf$

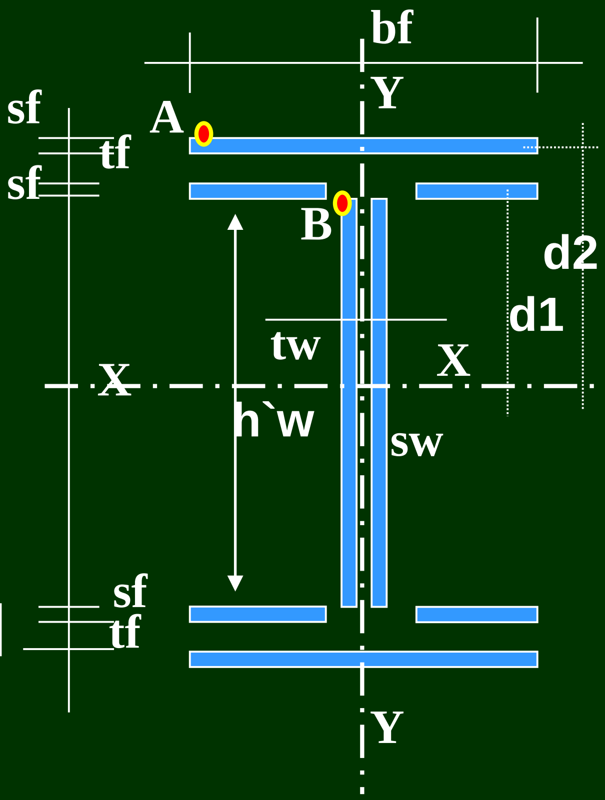
get sf $\quad Sf < tf$

(iii) Web Weld :

$Area = (Q`/0.2Fu)$

$Area = [(d - 4tf)sw]*2$

sw = …. Not less than 4.0mm



(iv) Check of Stresses:

$Ix = 2(sw*h`w^3)/12 + 4[b`f(sf)^3/12 +$

$(b`f*sf)*d1^2] + 2[bf*sf^3/12 + (bf*sf)(d2)2]$

$F_A = (Mx/Ix)*y_A + (N`/A) = \ldots.. < 0.2F_U$

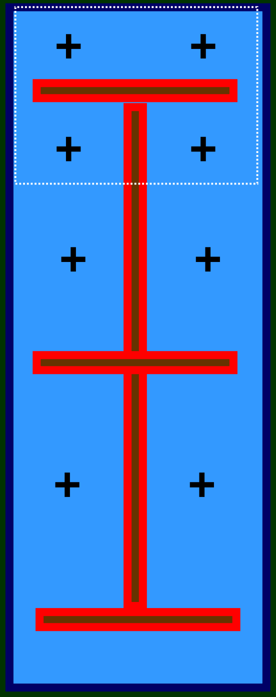
$F_B = (Mx/Ix)y_B + (N`/A) = \ldots.. < 0.2F_U$

$q_{yB} = (Q`/A_{weld}) = ...t/cm^2 < 0.2F_U$

$q_{Eq} = \sqrt{Fb^2 + 3qv_A^2} = ...< 0.2F_U *1.1$

(3) Design of High Strength Bolts:

(i) Approximate Method:



$T = (M/d\cos\alpha)$

Assume to use 2 rows of 2–bolts each round the tension flange ($n\Phi = 4$)

Tension force / Bolt = $(T/n\Phi)$ =…t

Try M(20) (10.9)

From table (6-3) (Code pp106)

Get, Tall = 15.43 *0.8 = 12.4

, Ps = 4.93t

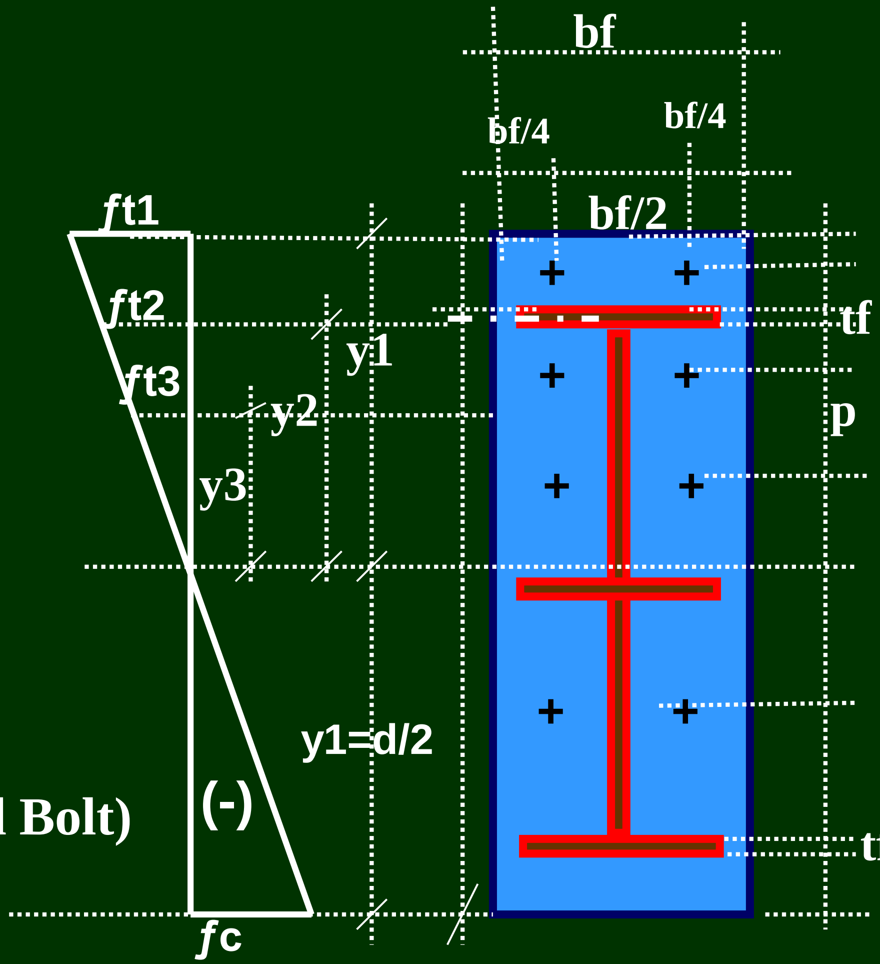
(ii) Exact Analysis:

$Ix = bf\,(d)^3/12 = \ldots.cm^4$

$ft1 = (M * y1 / Ix) = \ldots..\ t/cm^2$

$ft1 = (M * y2 / Ix) = \ldots..t/cm^2$

$ft1 = (M * y3 / Ix) = \ldots\ldots t/cm^2$



Check for External Force in Bolts due to Moment

$T_{ext.b.M}$

= (Actual Stress*Stressed Area Around Bolt)

$T1_{Ext.b.M} = [((ft1+ft2)/2 * (bf*(y1-y2)] / 2 \text{ or } 4 = \ldots.t \leq Tall = 15.43*0.8t$

$T2_{Ext.b.M} = [((ft2+ft3)/2 * (bf*(y2-y3)] / 2 \text{ or } 4 = \ldots t \leq Tall = 15.43*0.8t$

$T3_{Ext.b.M}$ = [(*f*t3)/2 * (bf*y3)] / 2 or 4 = …t ≤ Tall = 15.43*0.8t Safe

$T_{Ext.b.}$ = N`/Total nΦ resisting the external tension force = …t

T Total = $T_{ext.b.M.}$ (greater) ± $T_{Ext.b.}$ +*P*=….≤ 0.8Tall (Case of HSB. 10.9)

=….≤ 0.8Tall * 0.7 (Case of HSB. 8.8)

→ (Code pp109 -Eq.6.16)

(4) Check of Shear on Bolts:

Q/Bolt = Q/ nΦ =..t ≤ Ps = (μT/γ) (4.93t) Connection subjected to Shear only.

≤ ((μT-T ext,b)/γ) Connection subject ed to Shear and Tension.

(5) Check of Prying Force on Bolts (Eq.6-24 Code pp115) :

The Prying Force can be determined using the following Equation:

$$P = \left[\frac{\frac{1}{2} - \frac{w t^4{}_p}{30 a b^2 A_s}}{\left(\frac{3a}{4b}\right)\left(\frac{a}{4b}+1\right) + \frac{w t^4{}_p}{30 a b^2 A_s}} \right] . T_{ext.b.M.} OR\ T_{ext.b.}$$

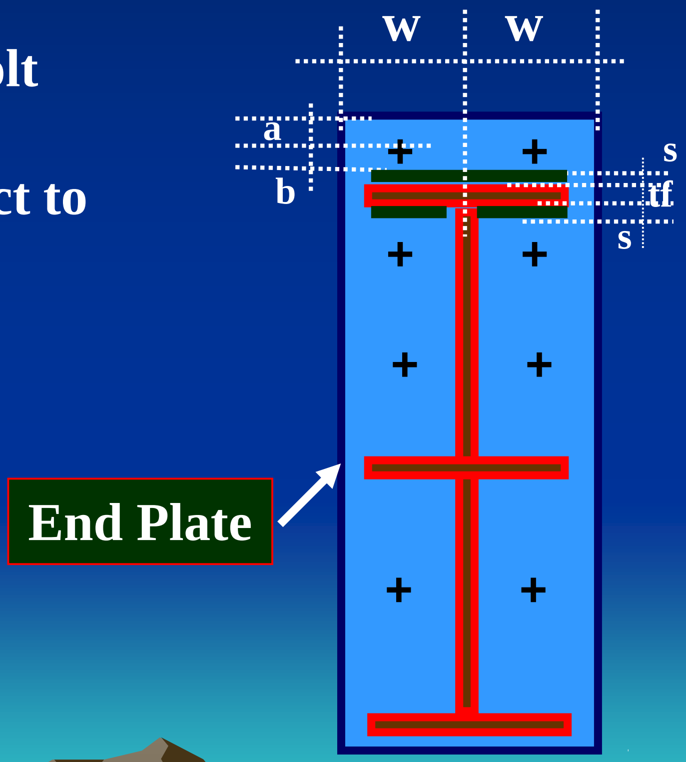
t_P = Thickness of End Plate

As = Bolt Stress Area (table 6-3 Code pp106)

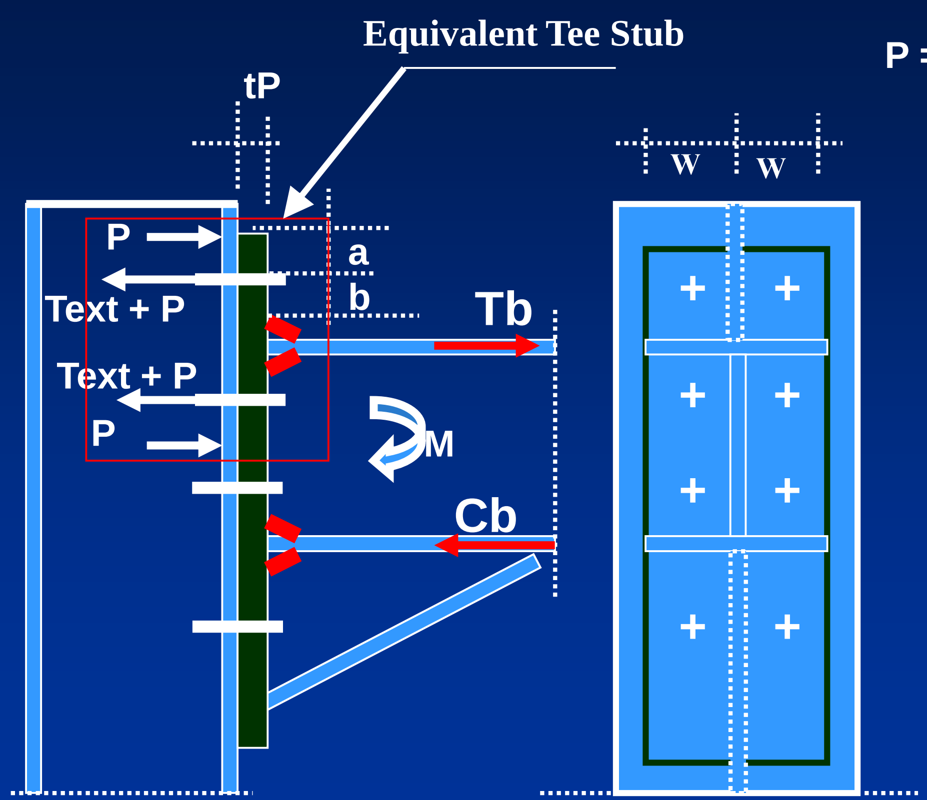
$T_{Ext.\ b}$, $T_{Ext.\ b.\ M.}$ = Applied External Tension Force on one bolt column due to either external tension force (Text.b) or due to the replacement of the applied moment (M) by two equal external and opposite forces (Tb=Cb=(M/d).

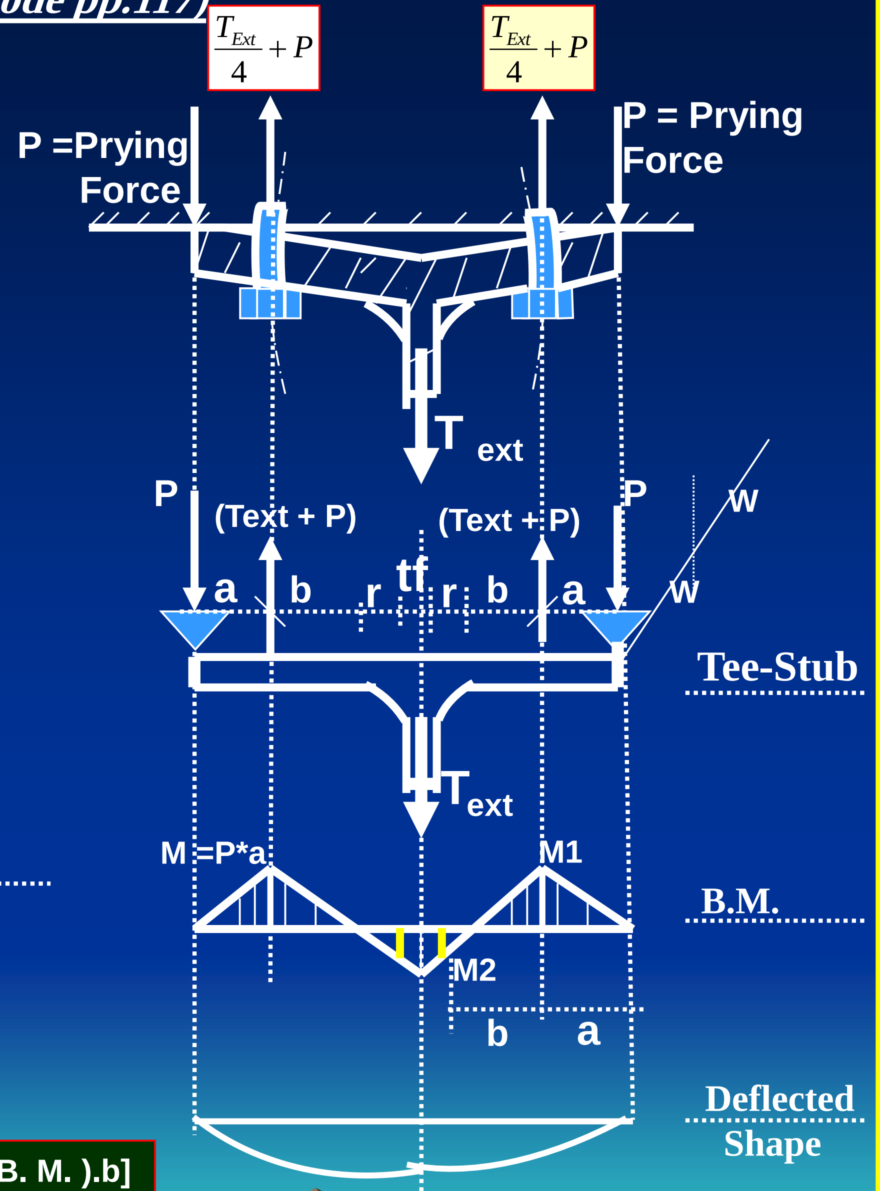
a , b = Bolt outer overhanging and inner dolt dimensions in cm

W = End Plate flange breadth with respect to one column of bolt



Determination of the Prying Force "P" (Code pp.117)





End Plate Moment Connection

M1 =P*a

M2 =P*a – [(T ext.b. OR Text. B. M.).b]

Equivalent Tee Stub Connection

(6) Design of End Plate :

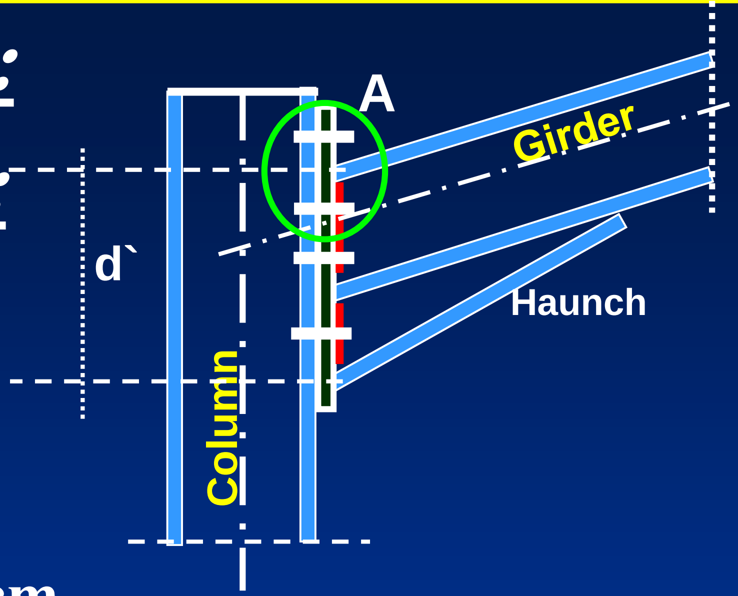
Approximate Method:

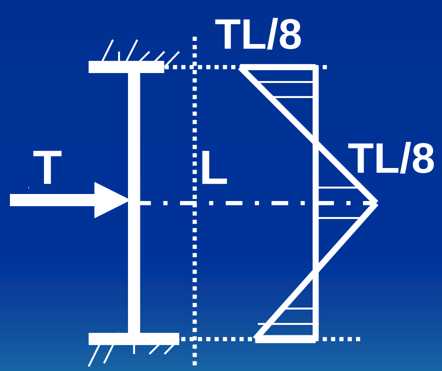
L = 3Φ + 2s + t`f = …cm

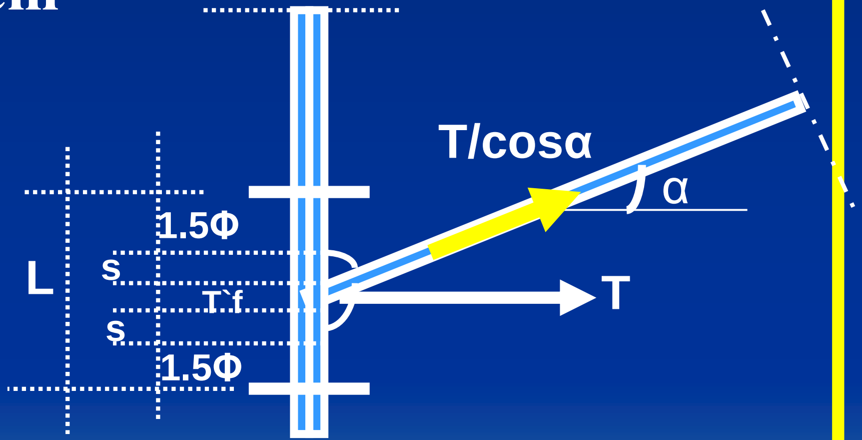
T = M/d` =….t

M = (TL/8) = …t.cm

t plate = √6M/bf*Fb = …cm





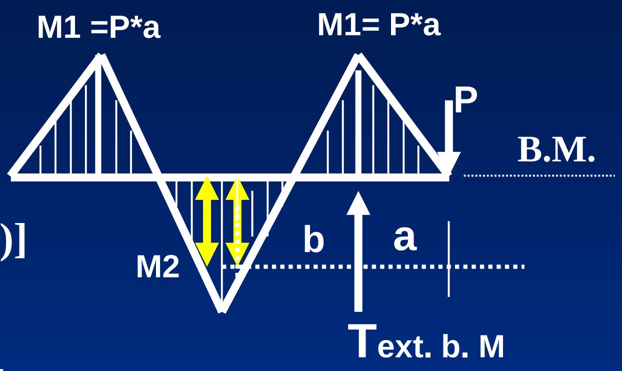


Detail A

Exact Method get After Calculating Prying Force.

(6) *Determination of the End Plate thickness*

<u>Exact Method</u> **:**



(i) Compute the exact bending moments:

M1 =P*a **Eq.6-28 Code pp.118)**

M2 = P*a – Text, b, m.*b [Eq.6-28 Code pp.118)]

(ii) Compute the exact required end plate thickness:

$t_P = \sqrt{}$ 6(Greater of M1 or M2)/2w*Fb = (Eq.6-27 Code pp.118)

Where : Fb = 0.72 Fy

if t_P is of Large , it can be reduced by using a Stiffener

(7) *Check for Safety Requirements for Beam to Column Connections (Code 119 Clause 6-9-4):*

(i) Column Web at the Vicinity (بالقرب- جوار)Compression beam Flange "Crippling of the Column Web" :

$t_{wc} \geq [b_b . t_b / (t_b + 2t_p + 5k)]$ (Eq.6.29 Code pp.119)

If Eq. (6.29) is not satisfied, use a pair of Horizontal stiffeners to achieved the following Equation:

$2b_{st} . t_{st} \geq [b_b . t_b - (t_b + 2t_p + 5k) . t_{wc}]$ (Eq.6.30 Code pp.119)

In order to prevent the Local Buckling in this stiffeners :

$(b_{st} / t_{st}) \leq (25/\sqrt{F_y})$ (Eq.6=31 Code pp. 119)

(ii) Column Flange at the Location of the Tension Beam Flange

Bending of the column Flange is prevented if:

$t_{fc} \geq (0.4 \sqrt{b_b . t_b})$ (Eq.6-32 Code pp.119)

If Eq. (6.32) is not satisfied, use a pair of Horizontal stiffeners to achieved the condition of equation (6-30).

(iii) Distorsion of the Web at Beam to Column Connection .

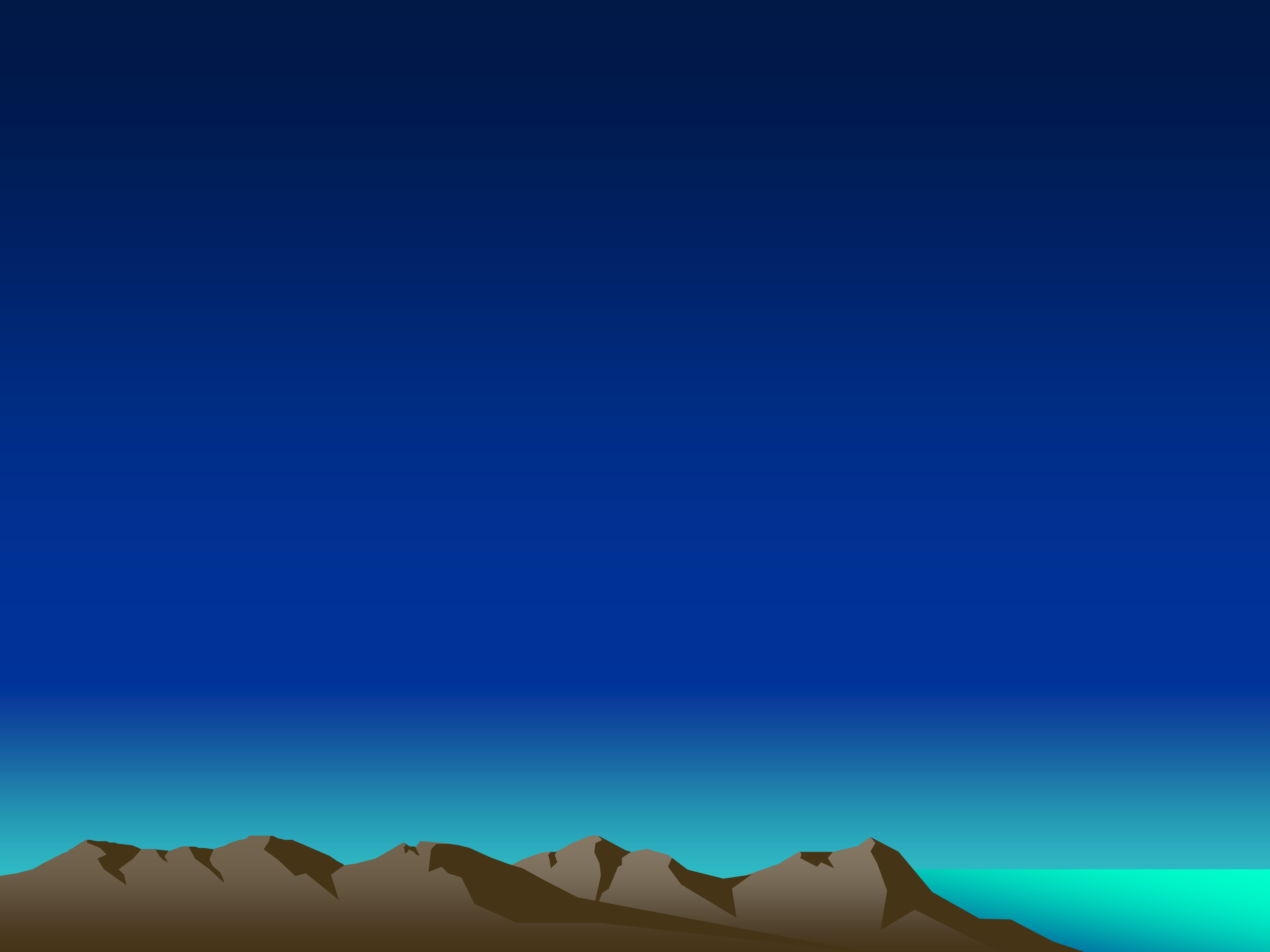
Distorsion of the Column Web is Prevented if :

$t_{wc} \geq (M/d_b)/[(0.35F_y)h_c]$ (Eq.6.33 Code pp.120)

If Eq.(6.33) is not satisfied use either a or b:

a- a double plate to lap over the web to obtain the total required thickness.

b- a pair of diagonal stiffeners in the direction of the diagonal compression as given in Eq.(6.34 Code pp.120)



بسم الله الرحمن الرحيم

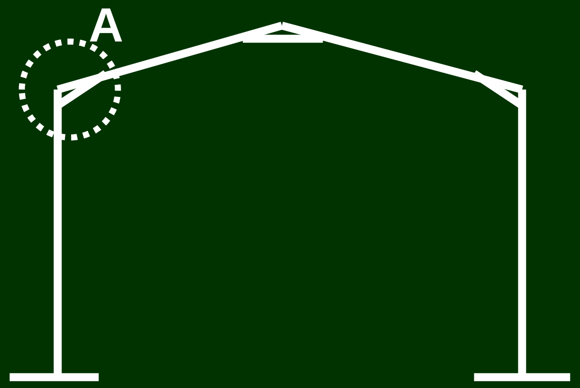
Lec.700

Analysis and Design of Steel Connections

7/1/19

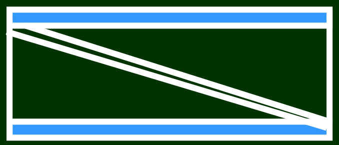
Example: Design a rigid frame Connection at "A" in the shown Figure , the Connection is subjected to M = 20 t. m, Q = 10.0t & N = 5.0t, The beam section is a SIB No. 360 and

the Column section is a BFI No. 300.



(1) Data Given :

- Md = 20 t.m , Q = 10.0t & N.F. = 5.t ,
- Bolts Grade (8.8) ,
- Column Section is a BFI. No. 300 and
- Girder Section is a SIB No. 360,
- Haunch will be a part of a beam girder with a height equals to (36-2 tf) =32.1cm and cuts to two parts .



Haunch

θ =26.565°

α =11.365°

- (Column Section) hIc = 30, twc = 1.1 , tfc = 1.90 , bfc = 30
- (Girder Section) hIb= 36, twb = 1.3 , tfb = 1.95 , bfb= 14.3

(2) Design of Weld:

h1 = hI / cosα = 36.73cm , h2 = (hI-2tf) / cos θ = 36.06cm,

tf1 = tf1 / cosα = 2.0cm , tf2 = (tf1/ cos θ) = 2.2cm and,

d = (h1 +h2) = 72.8 cm

(i) Check For weld thickness

(Use Haunch or not)

N` = N cosα - Q sinα = -8.82t

Q` = N sinα + Q cosα = 6.87t

T = C = (M / h1) = 54.45t

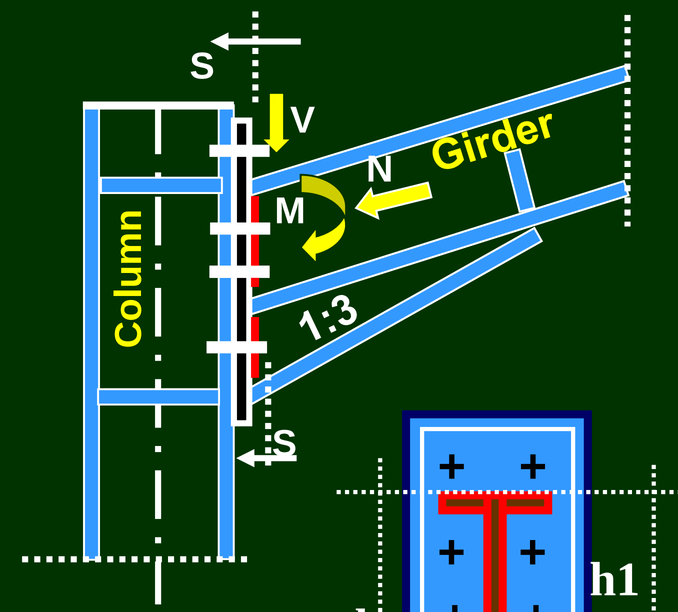
A = (T – N`/2) / (0.2FU) = 69.5cm^2

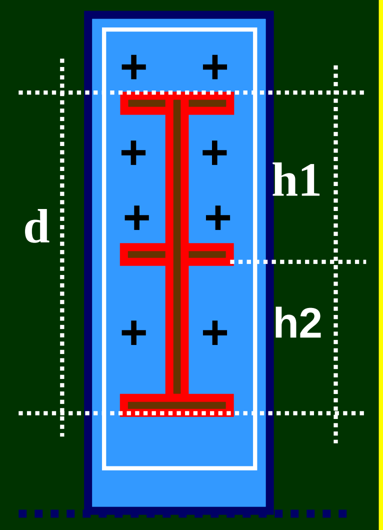
Area of Weld Around Flange =

A weld = [(bf + (bf – tw -2tf)]*s

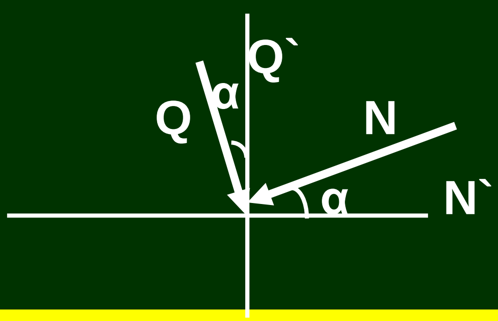
get “s” s = 2.97cm > tfb

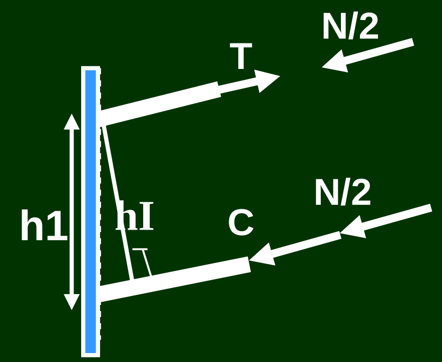
then, use Haunch





Section S –S





(ii) Flange Weld:

$T = C = (M/d) = (20*100/\ 72.8) = 27.47\ t$

$Area = (T - (N`/2)) / 0.2FU = (bf*sf + (bf - tw - 2tf)sf = [(27.47 - 4.41)/0.72]$

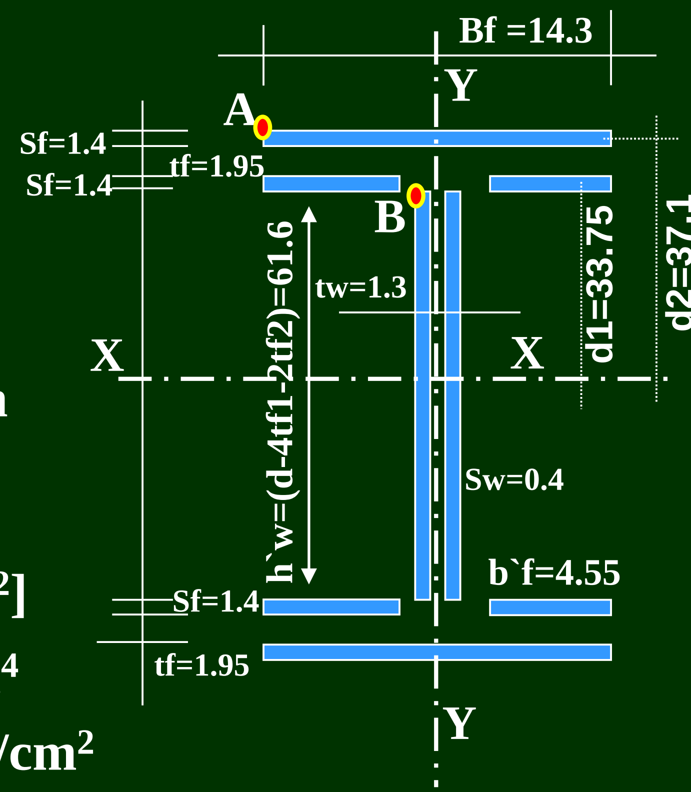
get $sf = 1.36$ cm   take $sf = 1.4cm < tf$ O.K.

(iii) Web Weld :

$Area = (Q`/0.2Fu) = (6.87/0.2*3.6) = 9.54cm^2$

$Area = [(d - 4tf1 - 2tf2)\ sw]*\ 2$

$sw = 0.08cm = 0.8mm$ take $sw = smin = 4.0mm$



(iv) Check of Stresses:

$$Ix = 2(sw*h`w^3)/12 + 4[b`f(sf)^3/12 + (b`f*sf)*d1^2]$$
$$+ 2[bf*sf^3/12 + (bf*sf)(d2)2] = 128756.0cm^4$$

$$F_A = (Mx/Ix)*y_A - (N`/A) = 0.511 < 0.2F_U = 0.72\ t/cm^2$$

$$F_B = (Mx/Ix)*y_B - (N`/A) = 0.460 < 0.2F_U = 0.72t/cm^2$$

$q_{yB} = (Q`/A_{weld\ Web}) = (6.87/123.2) = 0.06 t/cm^2 < 0.2F_U = 0.72 t/cm^2$

$q_{EqB} = \sqrt{F_B^2 + 3q_{yB}^2} = 0.47 t/cm^2 < 0.2F_U *1.1 = 0.79 t/cm^2$

(3) Design of High Strength Bolts:

(i) Approximate Method:

T = (M/d) = (2000 / 72.8) = 27.47t

Assume to use 2 rows of 2–bolts each round the tension flange (nΦ= 4)

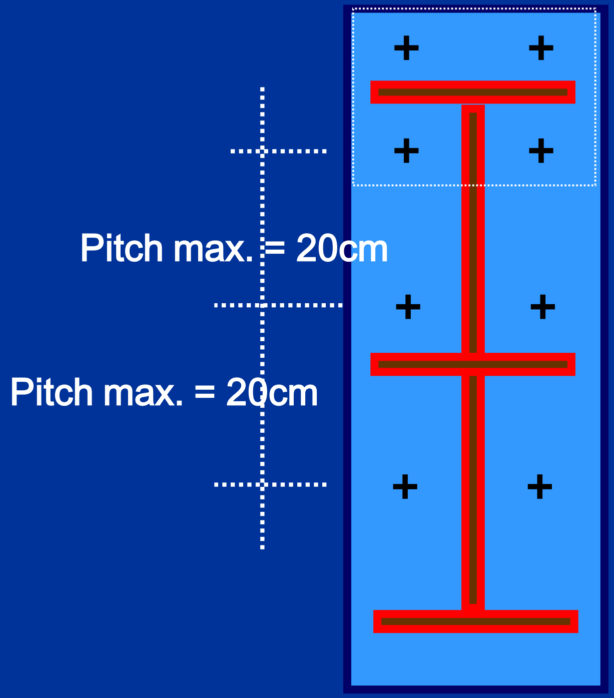
Tension force / Bolt = (T /nΦ) = 6.87t

Try M(20) (8.8)

From table (6-3) (Code pp106)

Get, Tall = 15.43 *0.8 *0.7 = 6.91t

, Ps = 4.93*0.7 = 3.45 t



Add two rows in the Compression zone at maximum pitch 20cm

(ii) Exact Analysis:

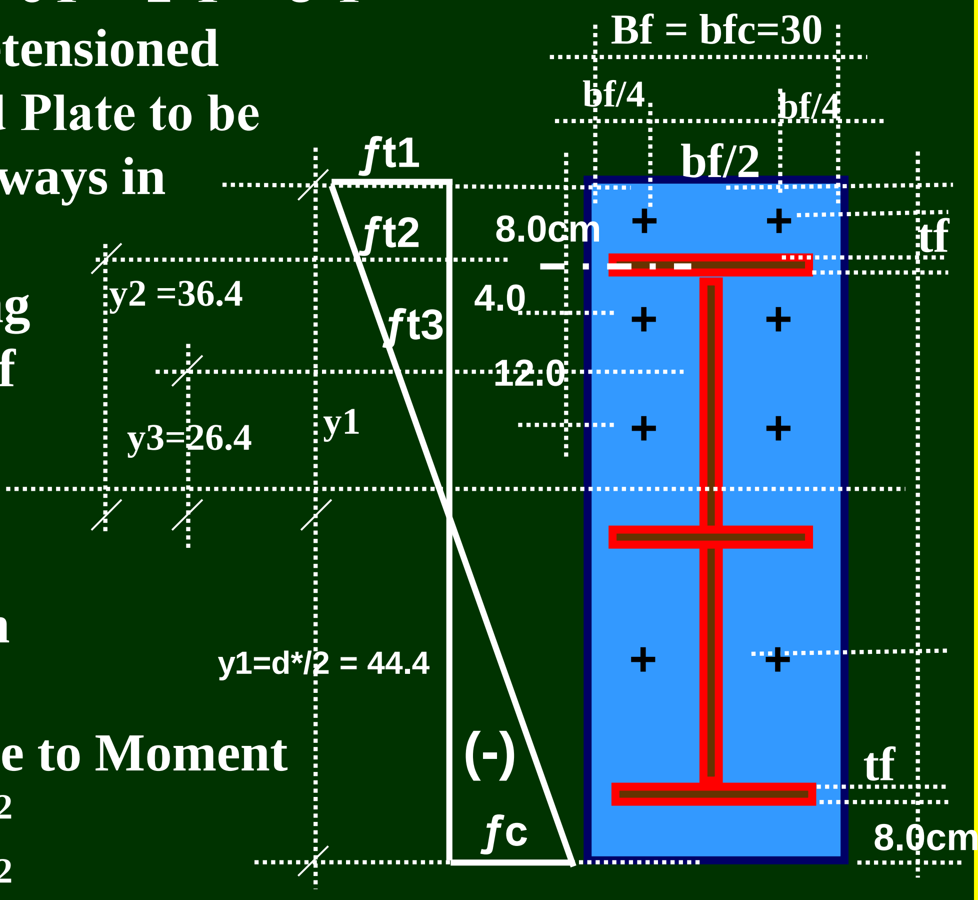
- Assume the required number of bolts (H.S.B.) in the Tension side from the following Equation $N\Phi t = (M/(0.8d^*0.6T) = 3.8\Phi \approx 6\Phi$
- Add to the calculated ($N\Phi t$) a row of bolts in the compression zone to get the total number of bolts ($N\Phi$) $= 6\Phi + 2\,\Phi = 8\,\Phi$
- Since the Pretension Force in Pretensioned Bolts (HSB) "T" causes the Head Plate to be always in Compression, and is always in Contact with Column.

  The Section resisting the straining actions is the total contact area of the end plate and its inertia is calculated as follows:

  $bf = bfc = 30cm$

  $d^* = d + 2(8.0) = 72.8 + 16 = 88.8cm$

  $Ix = bf\,(d^*)^3/12 = 1750567.7\ cm^4$



Stress Distribution Due to B.M.

Check for External Force in Bolts due to Moment

$ft1 = (M * y1 / Ix) = 0.051\ t/cm^2$

$ft2 = (M * y2 / Ix) = 0.042\ t/cm^2$

$ft3 = (M * y3 / Ix) = 0.03\ t/cm^2$

$T_{Ext.\ b.\ M}$ = (Actual Stress * Stressed Area Around Bolt)

$T1_{Ext.b.M} = [((f t1+f t2)/2 * (bf*(y1-y2))] / 2 = 5.6t \leq Tall = 15.43*0.8*0.7 = 8.64\ t$

$T2_{Ext.b.M} = [((f t2+f t3)/2 * (bf*(y2-y3))] / 2 = 5.4t \leq Tall = 8.64t$

$T3_{Ext.b.M} = [((f t3)/2 * (bf*(y3))] / 2 = 6.00 \leq Tall = 8.64t$

$T_{Ext.\ b.N}$ = N`/Total nΦ resisting the external tension force = (8.82/8) = -1.1t

T Total/Bolt $= T_{Ext.\ b.\ M.}$ (greater) $\pm T_{Ext.b.N} + P = \ldots \leq 0.8Tall*0.7 = 8.64t$

= [6.0 -1.1 + 0.50] = 5.40 < 8.64 O.K. Safe

(Case of HSB. 8.8) (Code pp109 - Eq.6.16)

(4) Check of Shear on Bolts:

Q/Bolt = Q`/ nΦ = (6.87/8) = 0.86t $\leq$ $((\mu T - T_{ext,b.N})/\gamma) = (Ps - (T_{ext.b.N}/\gamma))$

= (4.93 – (1.1/1.05)) = 3.88 *0.7 (Bolt Grade 8.8) = 2.72t Safe

(Connection subjected to Shear and Tension).

γ = 1.25 (Case I) and = 1.05 (Case II) [Code pp.105]

(5) Check of Prying Force on Bolts: (Eq. 6-24 Code pp.115)

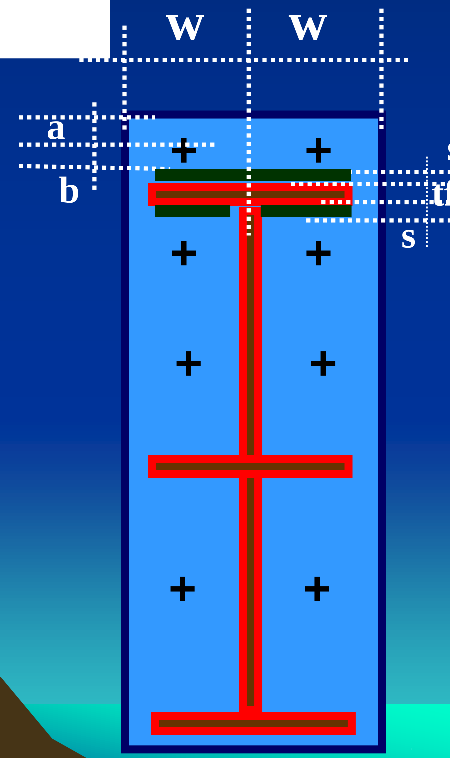
The Prying Force is determined using the following Equation:

$$P = \left[\frac{\dfrac{1}{2} - \dfrac{w t^4{}_p}{30ab^2 A_s}}{\left(\dfrac{3a}{4b}\right)\left(\dfrac{a}{4b}+1\right) + \dfrac{w t^4{}_p}{30ab^2 A_s}} \right] . T_{ext.b.M} . OR\ T_{ext.b.}$$

a = 4cm , b = 1.9 cm , W = 15cm

As = 2.45cm², $T_{Ext.\ b.\ M} = 6.0t$

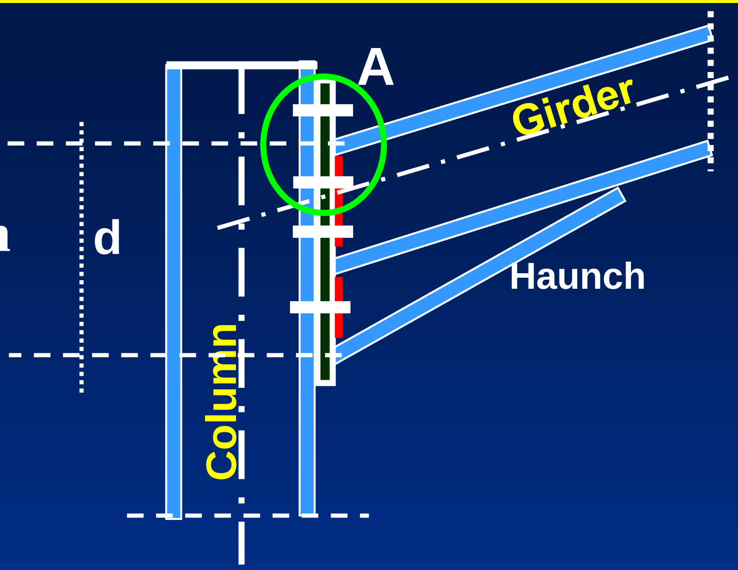
$T_{Ext.\ b.N}$ *= -1.1t , tp = 2.1 cm*



p = +0.5 t

(6) Design of End Plate :

Approximate Method:



Φ = 20mm , sf = 14mm , tf1= 20mm

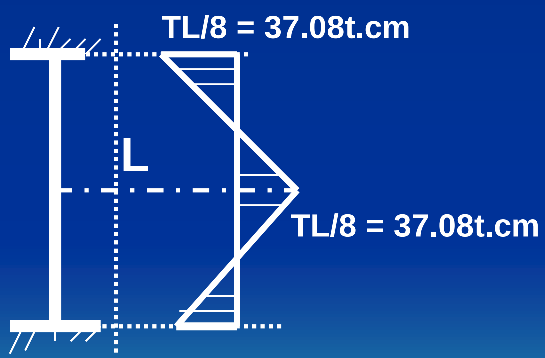
L = 3Φ + 2s + t\`f = 10.8 cm

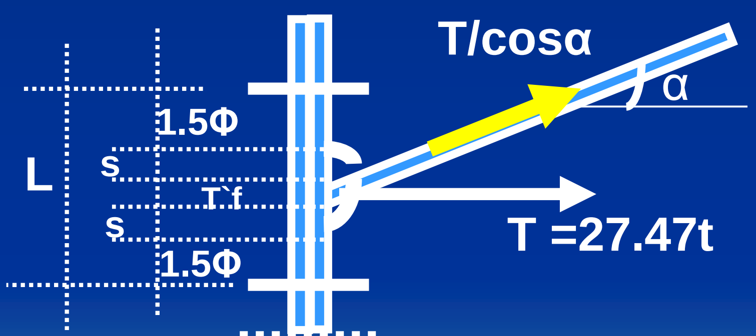
T = M/d = 27.47t

M = (TL/8) = 37.084 t.cm

t plate = tp = $\sqrt{6M/bf*Fb}$ = 2.07cm

Fb = 0.72Fy =1.728t/cm2





Detail A

Exact Method :

(i) Compute the exact bending moments:

M1 =P*a =0.50 *4 = 2.0 t. cm (Eq.6-28 Code pp.118)

M2 = P*a – $T_{Ext, b, m.}$*b = 2.0 – (-1.1)*1.9 = 4.1 t.cm

[Eq.6-28 Codepp.118)]

(ii) Compute the exact required end plate thickness:

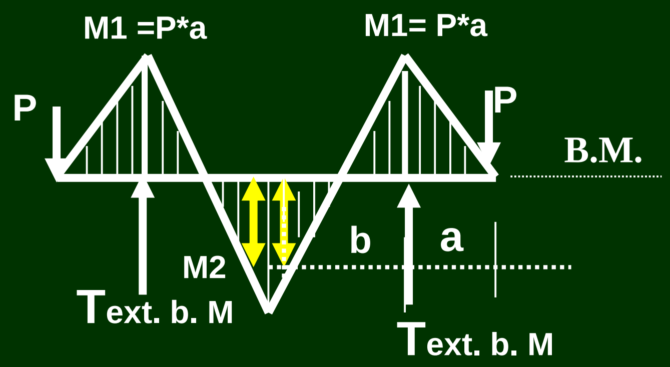
t_p =√ 6(Greater of M1 or M2)/2w*Fb (Eq.6-27 Code pp.118)

Where : Fb = 0.72 Fy

tp = √6 * 4.1/2*15*0.72*2.4 = 0.70 cm

take tp = 10.0mm O.K.

tp is reasonable and no large,



(7) *Check for Safety Requirements for Beam to Column Connections (Code 119 Clause 6-9-4):*

(i) Column Web at the Vicinity (بالقرب- جوار)Compression beam Flange "Crippling of the Column Web" :

K = 2tfc =2*1.9= 3.8 cm

L = (5k + 2tp + tb/cosθ) = 25.4 (Code 121)

Resisting Area = (L * twc) = 27.94 cm2

Applied Area = Area of Beam Flange

= 14.3*1.95= 27.889 cm2

Crippling of Web is prevented if

Resisting Area ≥ Applied Area

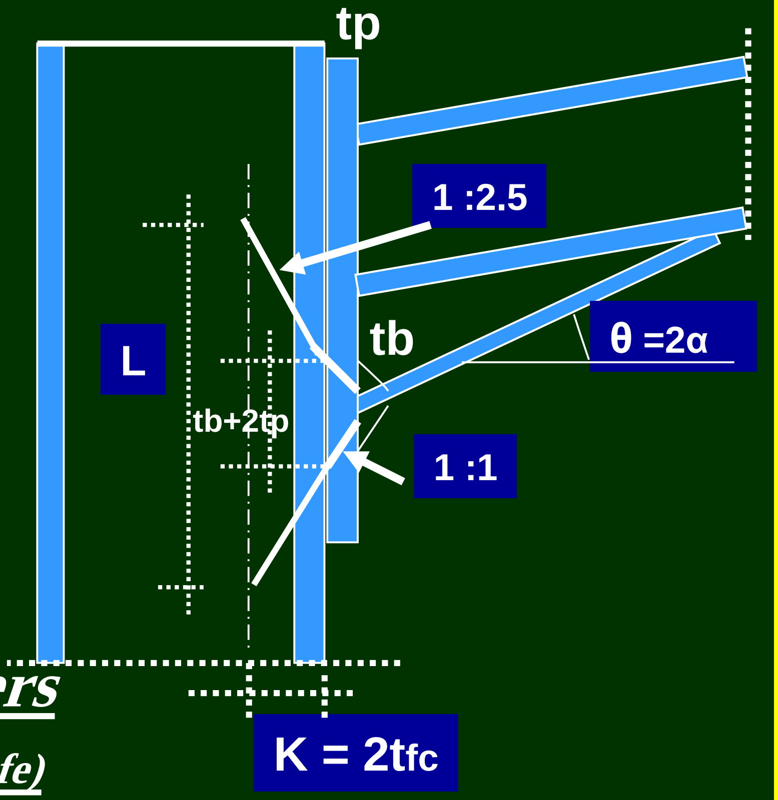
i.e. L*Twc ≥ (bb*tb) (Eq.6.29 Code pp.119)

Twc ≥ (bb*tb / L) =1.10 = Twc act. Safe

No Use a pair of Horizontal Stiffeners

If Resisting Area ≤ Applied Area (Unsafe)

Use a pair of Horizontal Stiffeners



Required Stiffeners Areas (Ast) $\geq$ (Applied Area – Resisting Area)

$$Ast = 2bst * tst \geq (bb*tb - L*twc) \qquad \text{(Eq.6-30 Code pp.119)}$$

Where : $bst = (h_{IC} - 2\,t_{FC})$,

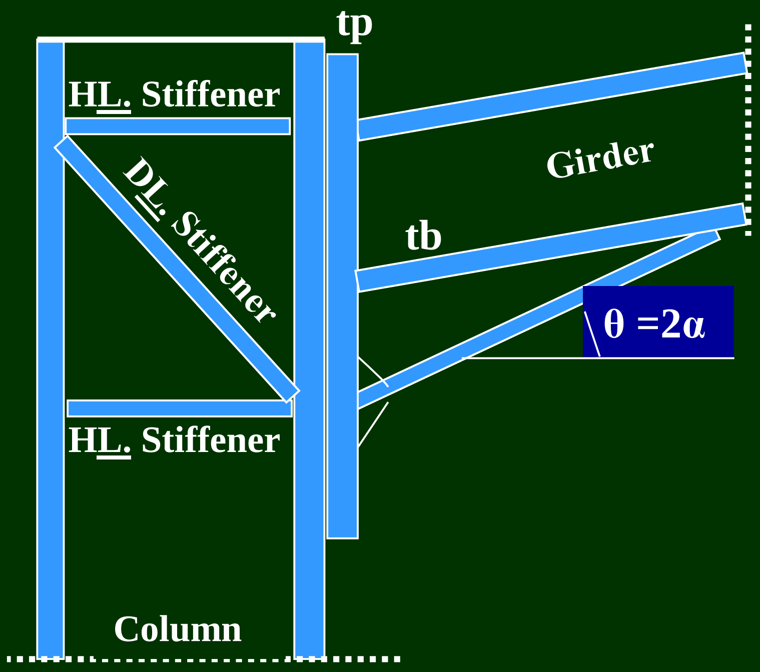
$L = (5K + 2tp + tb/\cos\alpha)$ Get $tst \geq \ldots cm$ (1)

In order to prevent the Local Buckling in this stiffeners:

$(bst / tst) \leq (25/\sqrt{Fy})$ (Eq.6.31 Code pp. 119)

Then, $tst \geq (bst/16.1)$ *(2)*

From (1) and (2) take "tst" greater



(ii) (*Check Bending of Tension Column Flange*):

Bending of the Column flange is Prevented if :

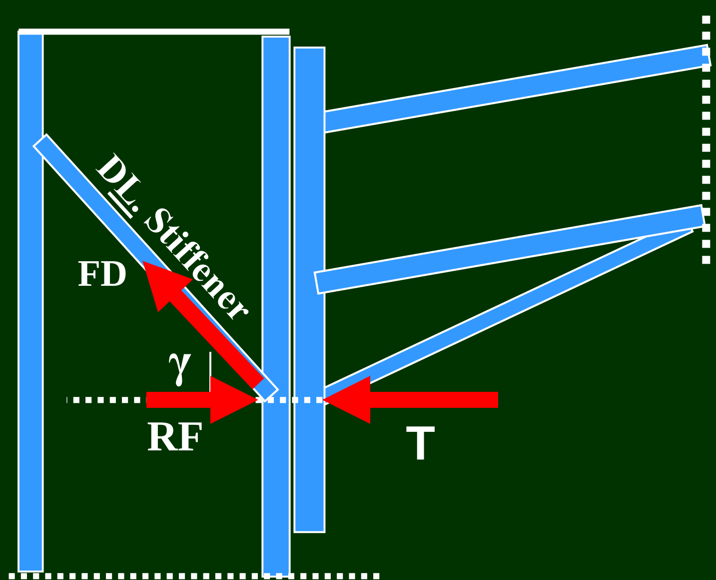
tfc ≥ 0.4√bb*tb **(Eq.6-32 Code pp119)**

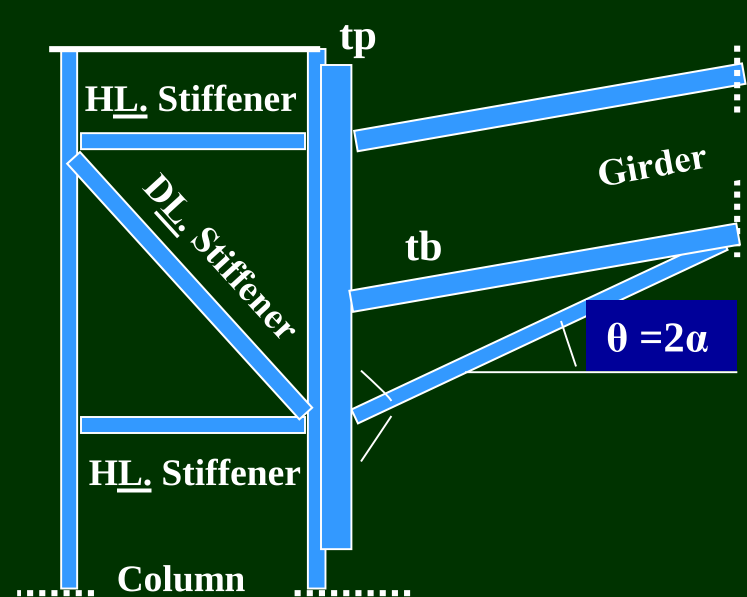
0.4√14.3*1.95 =2.11cm > tfc = 1.9 Unsafe

If Equation 6.32 is not satisfied, use a pair of horizontal Stiffeners fulfulling the condition of Equation (6.30).

tst ≥ [(bb*tb – (tb + 2tp + 5k)*twc) / 2*bst] **[Eq. (6.30 Code p.119]**

tst ≥ [(14.3*2.0) – (25.4)*1.1 / 14.45] = 0.03 cm take tst =tmin. = 4mm





(iii) (Stability Check of the Corner Web)

Distorsion of the Column Web is Prevented if :

Applied Force T = (M / (d) = 27.47t

Resistance Force = RF = (hIc *twc*q all.sh.)= (hIc*twc*0.35Fy) = 27.72t

For Safe Against Distorsion of the Column Web [RF ≥ T]

Then, twc ≥ (M/d)/[(0.35Fy)hc]=1.09 = 1.1 cm = twc act

(Eq.6.33 Code pp.120)

- **If Eq.(6.33) is not satisfied use either (a) or (b):**

(a) - A Pair of Diagonal Stiffeners in the Direction of the Diagonal Compression as Given in Eq.(6.34 Code pp.120)

Remaining Force "Rem.F" =Applied Force "T"– Resistance Force "RF*"

Rem.F = (M/db) – (0.35Fy)*hc* twc]

Force in Diagonal Stiffener = FD = Rem.F / cos γ

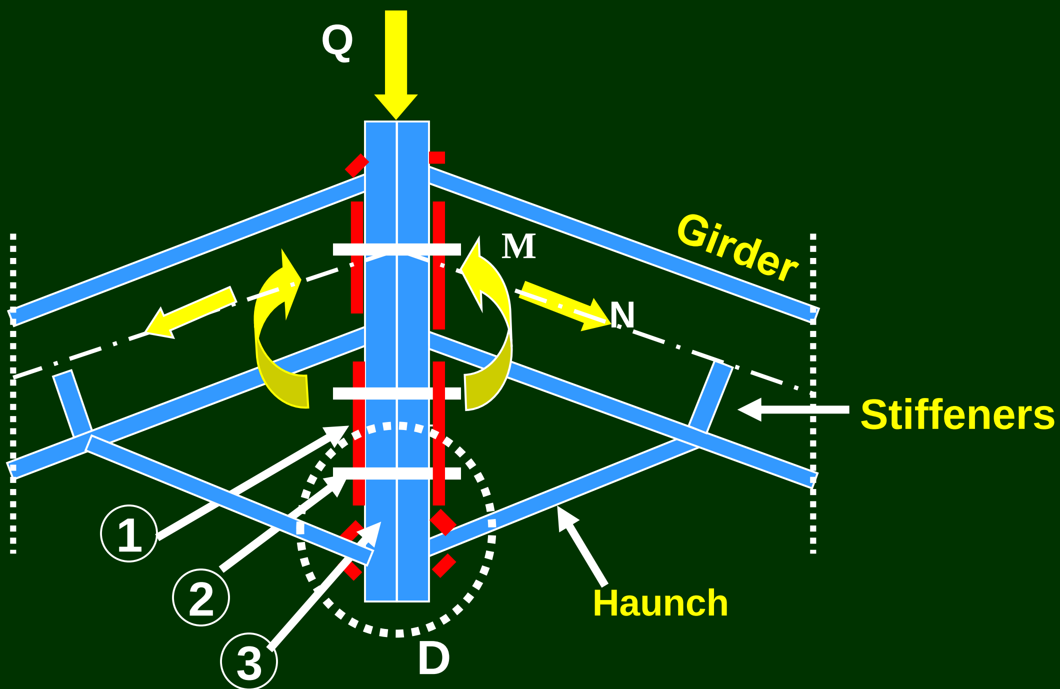
FD = Area of Stiffeners * Allowable Stresses = 2*(bst *tst) * Fall.t

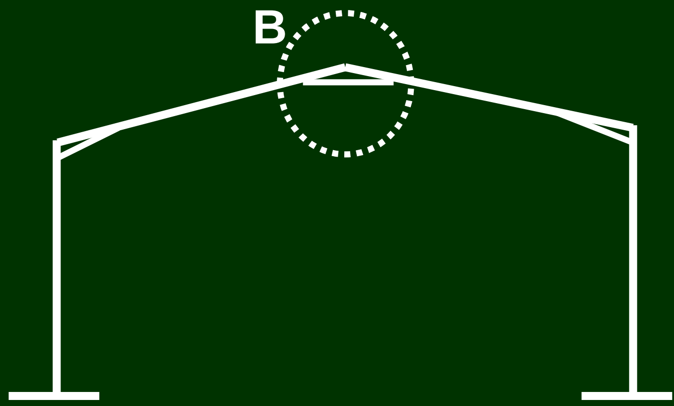
tst = (FD/2bst * Fall.t)

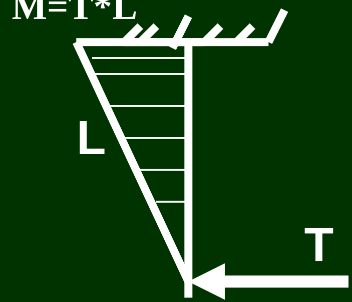
(b) – A double Plate to Lap Over the Web to Obtain the Total Required Thickness.

Apex Connections (Rigid Connections):

Apex Rigid Connections are Those Transmitting the B.M.,S.F. and N.F from One Structure Element to the Other.

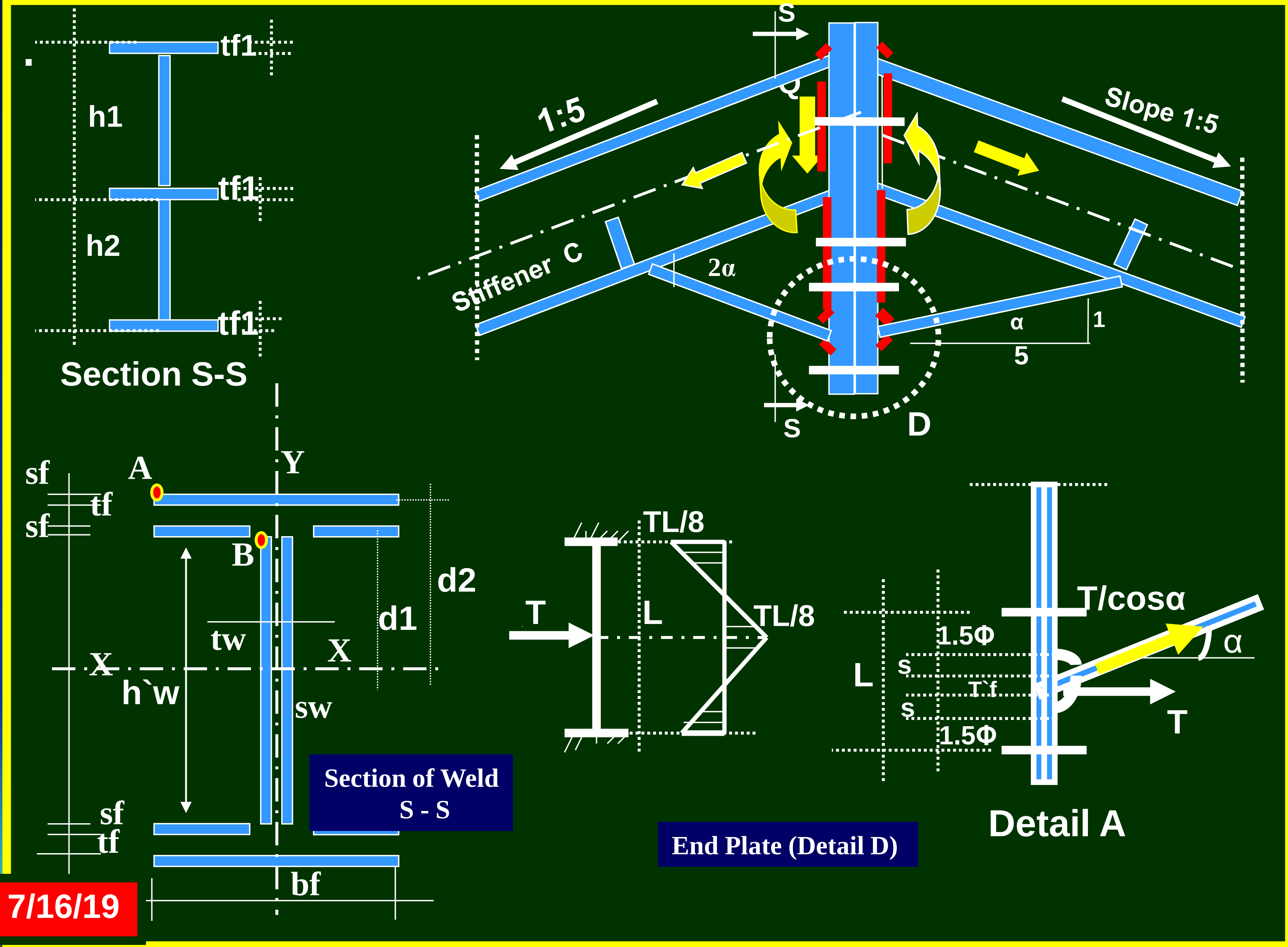






End Plate (Detail D)

Elements of Connection { 1 **Weld** 2 **Bolts** 3 **End Plate** }


Section S-S
h1
h2
tf1
tf1
tf1
S
S
Q
1:5
Slope 1:5
Stiffener C
2α
α
1
5
D
A
B
Y
X
X
sf
tf
sf
sf
tf
tw
h`w
sw
d1
d2
bf
Section of Weld S - S
T
L
TL/8
TL/8
End Plate (Detail D)
T/cosα
α
T
L
1.5Φ
s
T`f
s
1.5Φ
Detail A

Design Steps of Apex Connections:

***(1) Data Given*:**

B.M., S.F., N.F., Bolts, Column and Girder Cross Sections.

(2) *Design of Weld:*

(i) Check For weld thickness (Use Haunch or not).

(ii) Flange Weld.

(iii) Web Weld .

(iv) Check of Stresses.

(3) *Design of High Strength Bolts:*

(i) Approximate Method.

(ii) Exact Analysis.

(4) *Check of Shear on Bolts:*

(5) *Check of Prying Force on Bolts:*

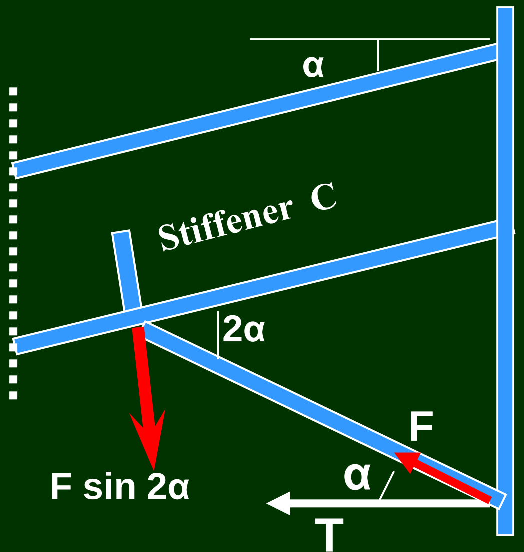
(6) *Design of End Plate :* **(i)** ***Approximate Method (ii) Exact Method.***

Design of the Stiffener "C":

- Determine the HL. Force "T".
- Find The Force "F" = T / cosα
- Force in Stiffener "C" = FC = F sin 2α
- FC =0.5* [(bst*tst)*Fall.t] = 0.5[(bst*tst)*1.4]

get tst

Where : bst = 0.5*(bb - twb)



Force in Stiffener

7/18/19

End Lec.700

7/19/19