CE - 388

CE - 388

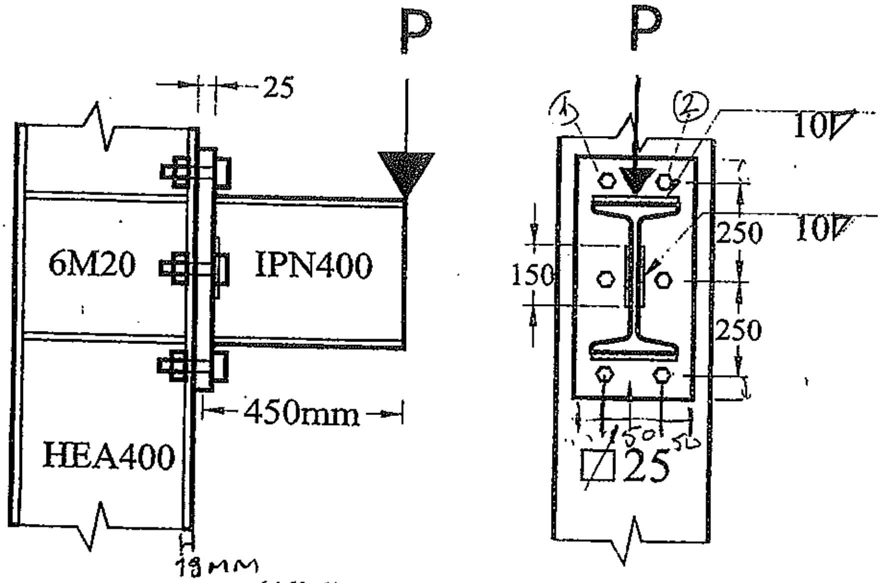
MİDTERM

$$\left(\frac{\tau}{\tau_{all}}\right)^2 + \left(\frac{\sigma}{\sigma_{all}}\right)^2 \leq 1.0$$

388

ID No:	Name:	Section:

4. *(30 pts)*. The IPN400 bracket is first welded (leg length=10mm, throat length=7mm) to a 25mm thick end plate and then connected to the flange of a HEA400 steel column by 6M20 ISO 8.8 grade high strength bolts, as shown. Calculate the allowable value of P that can be carried safely by the connection based on TS648 requirements. Assume EY loading and Fe.37 steel. Take $\sigma_{çem}$= 4.0 t/cm^2, and τ_{em}=2.4 t/cm^2 for the bolts.

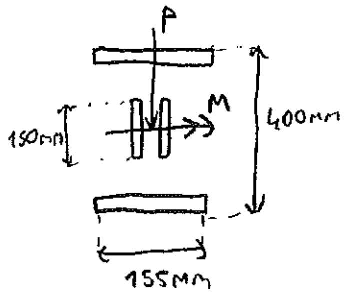


IPN 400
$b_f = 155$ mm

HEA 400
$t_f = 19$ mm

(All dimensions are in mm)

Welded Connection $t_e = 7$MM



$A_w = (2\times15.5 + 2\times15.0)(0.7) = 42.7$ cm^2

$I_w = 8073.75$ cm^4

$f_x' = 0 \quad f_y' = \frac{P}{42.7} = 0.0234\,P \quad f_z' = 0$

$f_x'' = 0 \quad f_y'' = 0 \quad f_z'' = \frac{(45P)(20)}{8073.75} = 0.0892\,P$

$M = 45P$ t.cm

$$f_r = \sqrt{(0.0234P)^2 + (0.0892P)^2} = 0.1\,P$$

τ_{all} for weld is 1.1 t/cm^2 $\quad P = \frac{1.1}{0.1} = \underline{11\ t}$

Bolts

$\Sigma y_i^2 = 2\times25^2 + 2\times50^2 = 6250$ cm^2

$A_b = 3.14$ cm^2

Normal force in bolts 1 and 2

$N_1, N_2 = \frac{(45P)(50)}{6250} = 0.36\,P$

shear in each bolt

$V_{iy} = \frac{P}{6} = 0.167\,P$

For each bolt $V_{em} = 2.4\times\pi = 7.54$ t

$N_{em} = 4.0\times\pi = 12.57$ t

$$\left(\frac{0.167P}{7.54}\right)^2 + \left(\frac{0.36P}{12.57}\right)^2 \leq 1.0$$

$0.0362P \leq 1.0 \rightarrow \underline{P = 27.6\ t}$

∴ Welding governs $\boxed{P_{max} = 11\ t}$

Check bearing around bolt holes

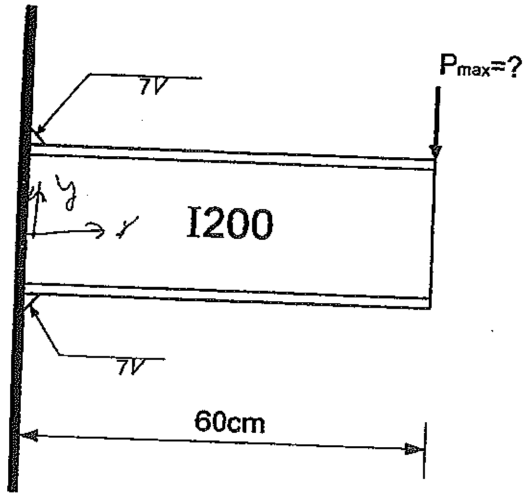
Shear in each bolt $= 0.167 \times 11 = 1.84$ t

$$\sigma_{bearing} = \frac{1.84\ t}{(1.9)(2.0)} = 0.48\ t/cm^2 \quad OK$$

If there is eccentricity there will be also torque on bolts and weldings

Name:..Student No:....................Section:...........

(20 Pts) 4. Calculate P_{max} which can be transferred to a column safely if a 60-cm long I200 bracket is connected to the column face



a) By welding along its flanges as shown in Fig. A,
b) By an end plate and 4M20 turned bolts as shown in Fig B.
Use St 37.

a)

$P/2$

$60P/20 = 3P$

$3P$

$P/2$

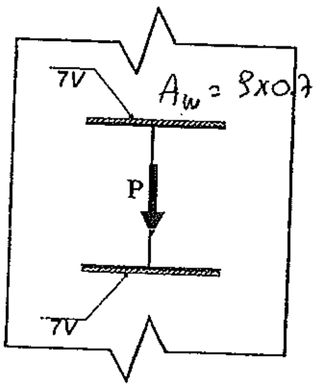
$$f_x = \frac{3P}{6.3} = 0.48P\ t/c^2$$

$$f_y = \frac{0.5P}{6.3} = 0.08P\ t/c^2$$

$$f = \sqrt{f_x^2 + f_y^2} = 0.48P\ t/c^2$$

$$0.48P \le \sigma_{all} = 1.1\ t/c^2$$

$$\therefore\ P_{max} = 2.27^{t}$$



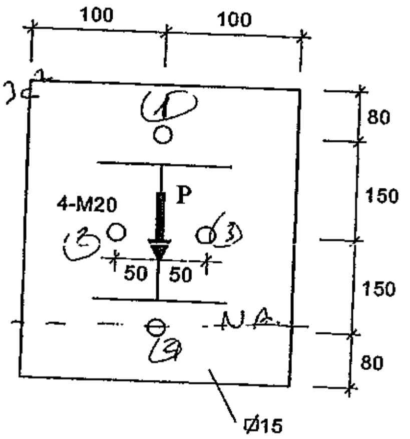


Figure A

Figure B

b)

$$\Sigma y^2 = 2\times 15^2 + 1\times 30^2 = 1350$$

Critical bolt → (1) $\quad P_1 = \frac{(60P_{max})(30)}{1350} = 1.33P_{max}$

Normal stress, $\sigma_b' = \frac{1.33P_{max}}{3.1415} = 0.42P_{max}\ t/c^2$

Shear stress, $\tau' = \frac{(P_{max}/4)}{3.1415} = 0.08P_{max}\ t/c^2$

$$\left(\frac{0.08P_{max}}{1.4}\right)^2 + \left(\frac{0.42P_{max}}{1.12}\right)^2 \le 1.0 \Rightarrow P_{max} = 2.6^{t}$$

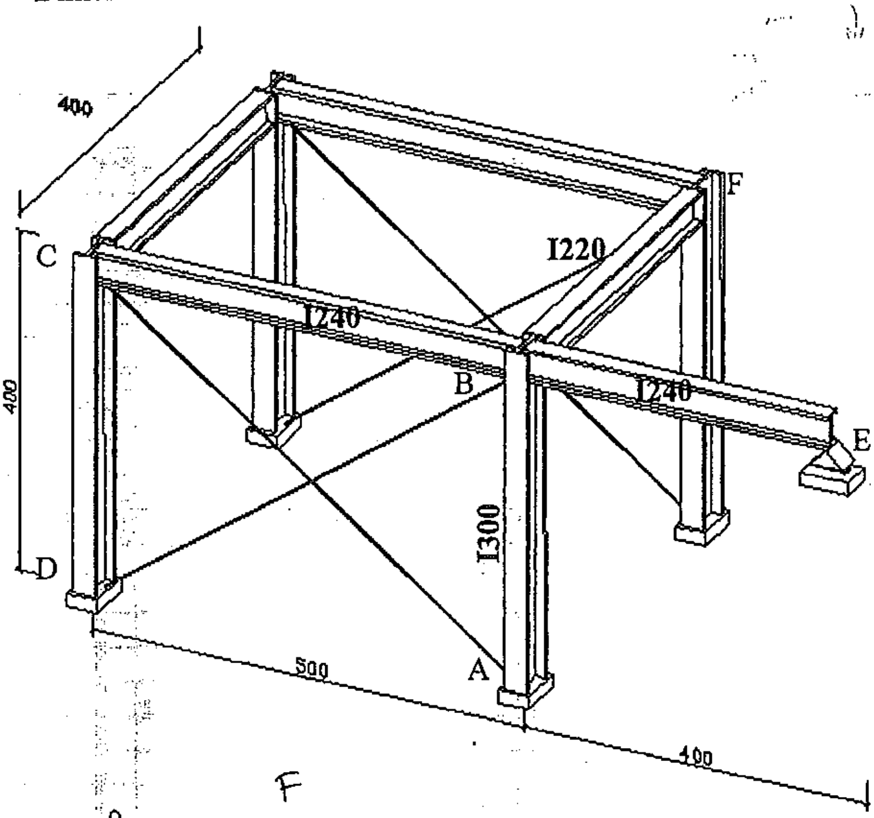
Bearing: $\tau_b = \frac{2.6}{4} = 0.65^{t}$ shear of bolt

$$\sigma_{bearing} = \frac{0.65}{1.5\times 2.40} = 0.22\ t/c^2 \quad O.K.$$

Dr. Uğur Polat	**Middle East Technical University** **Department of Civil Engineering** **CE388 – Fundamentals of Steel Design** **Tutorial #1** Duration: 60 min. March.07, 2011		
		Total	

ID No:	**Name:**	**Section:**

1. Calculate the compressive load capacity of column AB based on TS648 requirements and S235 (St37) steel. Column bases are fixed in all directions and E is a roller support. Dimensions are in cm.



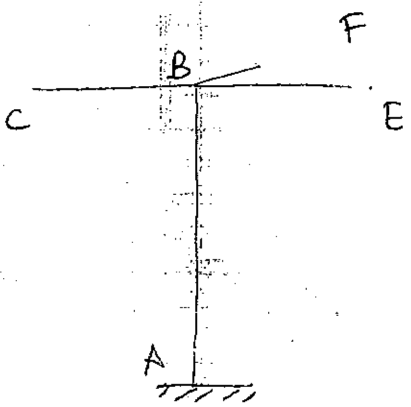
$I240 \rightarrow I_x = 4250\ cm^4$
$I_y = 221\ cm^4$

$I220 \rightarrow I_x = 3060\ cm^4$
$I_y = 162\ cm^4$

$I300 \rightarrow I_x = 9800\ cm^4$
$I_y = 451\ cm^4$

sidesway permitted

$\rightarrow$ sidesway prevented



$G_A = 1$ (fixed end)

$$G_B = \frac{\Sigma I_c / L_c}{\Sigma I_g / L_g}$$

$$G_B = \frac{(451/400)}{(3060/400) + (4250/500) + (4250/400)}$$

$= 0.04$

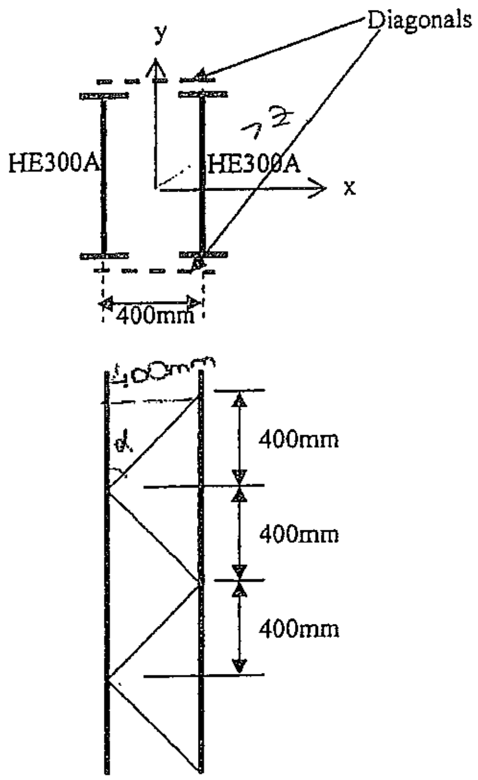
$K = 0.64$ (sidesway prevented

$K = 1.23$ (sidesway permitted)

12 TOPKARA

$L = 3.5m$

2. The latticed built-up column shown has an unsupported length of 3.5m and made of S235 (St37) steel. The HE300A wide flange struts are tied together using diagonal bars made of St.52 steel. The effective length factors for buckling in YZ and XZ planes are $k_x = 0.85$ and $k_y = 0.90$, respectively. Based on TS648 requirements;



a) Choose a proper diameter for diagonals,

b) Calculate the allowable compressive load capacity of the column.

Wide Flange $I_x = 59200$

$I_y = 19400$

a) $\alpha = 45°$.

$$M_d = \frac{Q}{2\sin\alpha}$$

<table>
<tr><td rowspan="8">Dr. Oğuzhan Hasançebi

Dr. Uğur Polat

Dr. Ahmet Türer</td><td rowspan="8">Middle East Technical University
Department of Civil Engineering

CE388 – Fundamentals of Steel Design

TUTORIAL TEST 1

Duration: 100 min. 18/19 March 2010</td><td>Q1</td><td></td></tr>
<tr><td>Q2</td><td></td></tr>
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<tr><td>Total</td><td>/100</td></tr>
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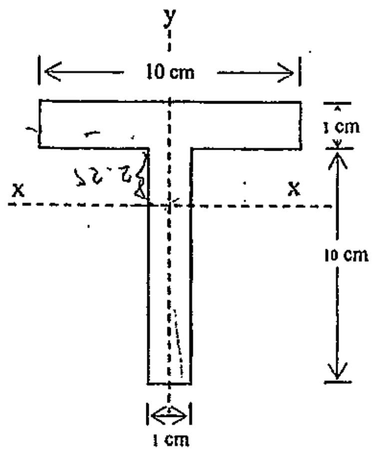
ID No:	**Name:**	**Section:**

1. (30 *pts*) Determine the following section properties for the T-section shown.



Cross-sectional area (A) ✓

Moment of inertia about x-axis (I_{xx})

Moment of inertia about y-axis (I_{yy})

Radius of gyration about x-axis (i_x)

Radius of gyration about y-axis (i_y)

Elastic section modulus (W)

Plastic section modulus (Z_p)

Shape factor (s)

$I_x = \frac{1}{12} \times 1 \times 10^3 + 1 \times 10 \times (2{,}75)^2 + \frac{1}{12} \times 1^3 \times 10 + (2{,}75)^2$

$I_x = 235{,}4\ cm^4$

$I_y = 84{,}17\ cm^4$

$i_x = \sqrt{\frac{I_x}{A}}$ $\quad i_y = \sqrt{\frac{I_y}{A}}$

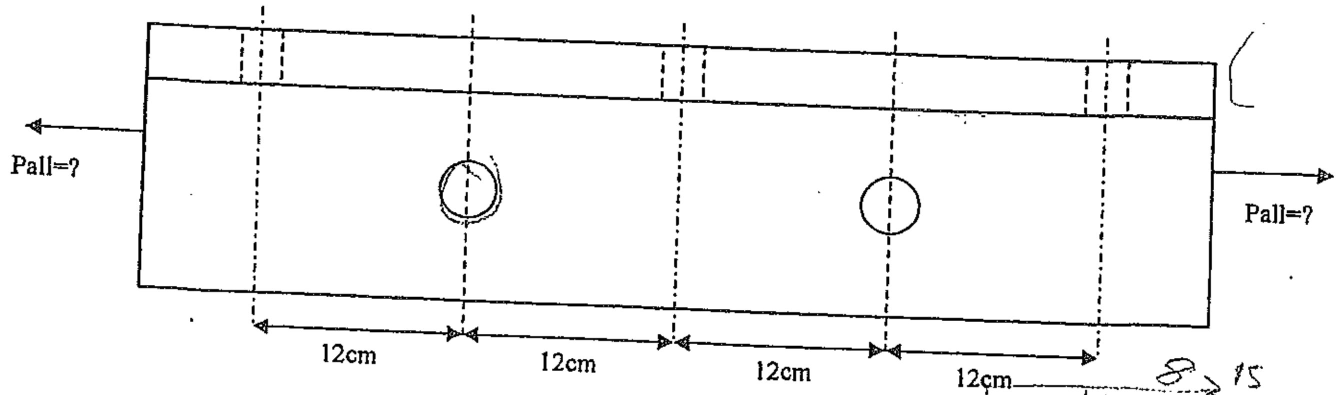
$W = \frac{I}{c} = \frac{235{,}4}{7{,}75} = 30{,}32$

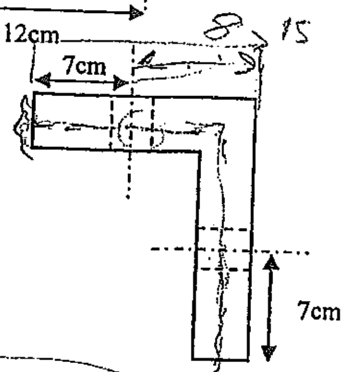
$Z = \frac{M_p}{\sigma} = \frac{(10 \times 5 + 5 \times 1)\sigma}{\sigma} = 55$

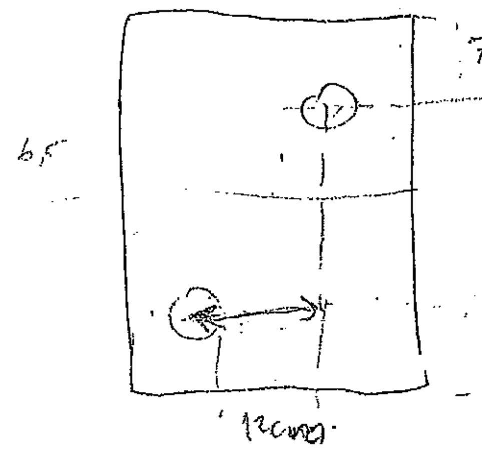
$s = \frac{Z}{W} = \frac{55}{30{,}32} = 1{,}81$

ϕ

2. (35 pts) Calculate the allowable axial load capacity of the L150x15 shape (St37) that has ϕ20mm holes as shown in the figure below.

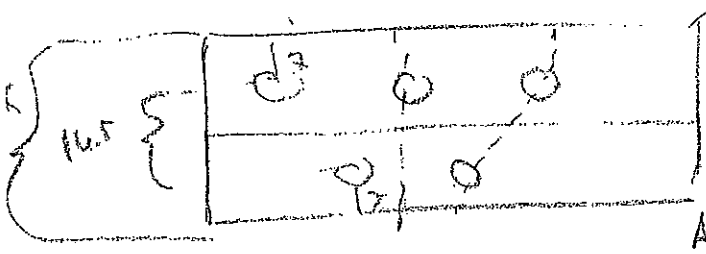






$8-\left(\frac{1,5}{2}\right) = 7,25$

$A_g = 28.5 \times 1.5$



1 hole

$A_{net} = 28.5 \times 1.5 - 1.5 \times 2$

2 hole

$28.5 \times 1.5 - 2 \times 2 \times 1.5 + \frac{12^2}{4 \times 14.4}$

take min A_{net}

$\boxed{A_{net} \leq A_g \times 0.85}$

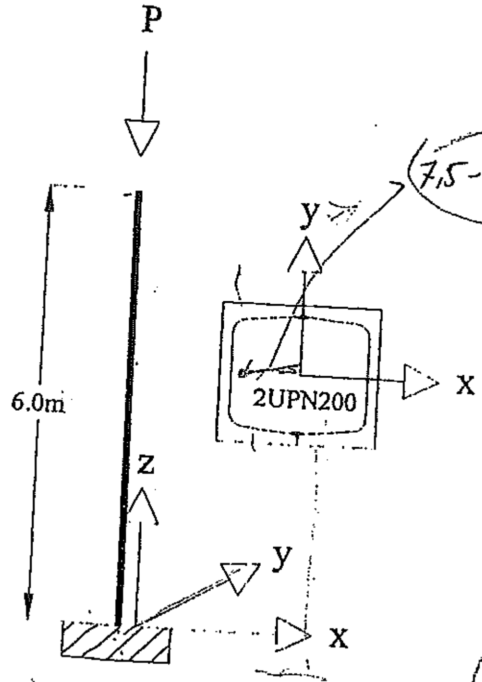
36.37

$\sigma_{all} = \sigma_y \times 0.6$ 2.4 1.44

$P_{all} = \sigma_{all} \cdot A_{net}$ 36.37

C 200

3. (*35 pts*) The cantilever column shown has a box section of 2UPN200 welded face-to-face. Calculate the Euler elastic buckling load $\mathbf{P_{cr}}$ if the steel material is S235 (St.37).



$7{,}5-(2{,}01)=5{,}49$

I_{x1}

$I_x = 2 I_{x1}$

$I_y = 2 I_{y1} + 2 A (5.49)^2$

$\lambda = \sqrt{\frac{2 \times 2 E}{\sigma}}$ $\qquad \lambda = \frac{\pi^2 \times 2100}{\sigma}$

λ_p

min

$A = 32.2 \times 2 = 64.4$

$k = 2$

$P_{cr} = \frac{\pi^2}{}$

$I_x = 2 I_x = 1910$

Euler formula,

$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2_{max} \to \lambda_y} = (3.14)^2 (2.1) \times 10^6$

$I_y = \left(I_y' + A \times (7.5-2.01)^2\right) 2$

$\boxed{I_y < I_x}$

$\sigma_{cr} < \sigma_p = 0.8 \sigma_y = 192$

$i_x = \sqrt{\frac{I_{xx}}{A}}$ $\qquad i_y = \sqrt{\frac{I_y}{A}}$

$\lambda_x = \frac{kL}{i_x}$ $\qquad \lambda_y = \frac{kL}{i_y}$

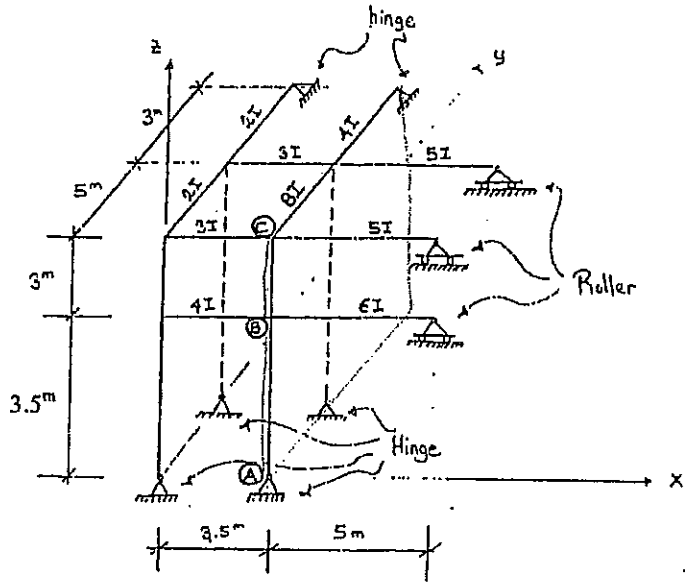
600 cm.

$P_{cr} = \sigma_{cr} \times A =$

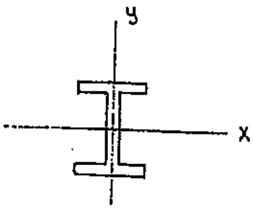
Dr. Oğuzhan Hasançebi Dr. Uğur Polat Dr. Ahmet Türer	**Middle East Technical University** **Department of Civil Engineering** **CE388 – Fundamentals of Steel Design** **TUTORIAL TEST 2** Duration: 100 min. 06/07 April 2010	Q1	
		Q2	
		Q3	
		Total	/100

ID No: **Name:** **Section:** 3

1. (40 *pts*) A three dimensional frame is shown in the figure. The column ABC shall be designed as a standard I profile. Assume St37 steel.



for columns AB and BC



$I_x = 20I$

$I_y = I$

a) Compute effective buckling length for columns AB, BC for buckling in x-z plane ($L_y = K_y.L$) and for column AC for buckling in y-z plane ($L_x = K_x.L$) by using alignment charts in TS648.

I_y buckling about y axis

buckling about x axis I_x

b) Assuming P = 15 tons, L_y = 6.5 m and L_x = 5.5, select an appropriate standard I-profile for column ABC.

a)

$G_A = 10$

$$G_B = \frac{\Sigma(I/L)_{columns}}{\Sigma(I/L)_{beams}} = \frac{(I/3.5)+(I/3)}{(4I/3.5)+(6I/5)\times 0.5} = 0.355$$

$K_{AB} = 1.75$ (with G_A and G_B)

$L_{AB} = 1.75 \times 3.5$

$L_{AB} = \underline{6.125m}$

$$G_C = \frac{(I/3)}{(3I/3.5)+(5I/5)\cdot 0.5} = 0.246$$

$K_{BC} = 1.1$ (with G_B and G_C)

$L_{BC} = 1.1 \times 3$

$L_{BC} = \underline{3.3m}$

Column AB, BC sidesway permitted

$G_A = 10$

$$G_C = \frac{(20I/6.5)}{(8I/5)} = 1.923$$

$K_{AC} = 0.9$ with (G_A and G_C)

$L_{AC} = 0.9 \times 6.5$

$L_{AC} = \underline{5.85 m}$

Column AC is sidesway prevented

b) Choose I section, calculate λ, find σ_{all} from Table
find F from ($\sigma_{all} \times A_{I\,section}$), check with $P = 15$ tons

Choose IPN 300* $\rightarrow$ $\lambda = \frac{K \cdot L}{i} = \frac{6.5}{2.56 \times 10^{-2}} = 254$

$$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2} \rightarrow \lambda = \frac{KL}{i}$$ higher λ, smaller i] more critical

Choose IPN 600* $\rightarrow$ $\lambda = \frac{6.5}{4.3 \times 10^{-2}} = 151.2 \rightarrow \sigma_{all} = 363.6$ kgf/cm^2

$$P = \sigma_{all} \times \underbrace{A}_{\text{from table}} = 363.6 \text{ kgf/cm}^2 \times 254 \text{ cm}^2 = 92.3 \text{ ton}$$

Choose IPN 400* $\rightarrow$ $\lambda = \frac{K \cdot L}{i} = \frac{6.5}{3.13 \times 10^{-2}} = 207$

$P = \sigma_{all} \times A = 193.5$ kgf/cm^2 $\times$ 118 cm^2 $= \underline{22.83}$ ton > 15 ton $= P$

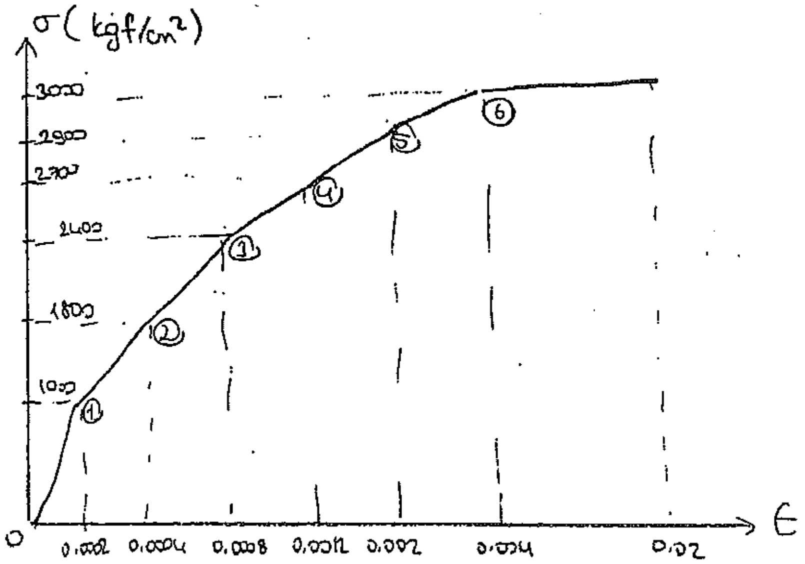
We can use $\underline{\text{IPN 400}}$

2. (30 points) The stress strain data for a special steel is given as follows:

Strain	Stress (kgf/cm^2)
0	0
0.0002	1000
0.0004	1800
0.0008	2400
0.0012	2700
0.0020	2900
0.0040	3000
0.0200	3000

a) Plot the stress-strain curve of the steel by connecting consecutive data points by straight line segments.

b) Using tangent modulus approach and a factor of safety equal to 2.0, determine the allowable compressive stresses for two columns with $\lambda = 80$ and 150.



Column with $\lambda = 80$

$0 < \sigma_{cr} < 1 \text{ t/cm}^2$

$$E = \frac{\sigma_1 - \sigma_0}{\epsilon_1 - \epsilon_0} = \frac{1-0}{(0.0002-0)} = 5000 \text{ t/cm}^2$$

$$\sigma_{CR} = \frac{\pi^2 \times 5000}{80^2} = 7.703 \text{ t/cm}^2$$

$1 < \sigma_{cr} \le 1.8 \text{ t/cm}^2$

$$E = \frac{\sigma_2 - \sigma_1}{\epsilon_2 - \epsilon_1} = \frac{1.8 \text{ t/cm}^2 - 1.0 \text{ t/cm}^2}{0.0004 - 0.0002} = 4000 \text{ t/cm}^2$$

$$\sigma_{cr} = \frac{\pi^2 \times 4000}{80^2} = 6.16 \text{ t/cm}^2$$

$1.8 \text{ t/cm}^2 < \sigma_{cr} < 2.4 \text{ t/cm}^2$

$$E = \frac{\sigma_3 - \sigma_2}{\epsilon_3 - \epsilon_2} = \frac{2.4 \text{ t/cm}^2 - 1.8 \text{ t/cm}^2}{0.0008 - 0.0004} = 1500 \text{ t/cm}^2$$

$$\sigma_{cr} = \frac{\pi^2 \times 1500}{80^2} = 2.31 \text{ t/cm}^2 \quad (\text{between } 1.8 \text{ t/cm}^2 - 2.4 \text{ t/cm}^2)$$

$$\sigma_{all} = \sigma_{cr}/2 = 1.155 \text{ t/cm}^2$$

column with $\lambda = 150$

$0 < \sigma_{cr} < 1 \ t/cm^2$

$$E = \frac{\sigma_1 - \sigma_0}{\epsilon_1 - \epsilon_0} = 5000 \ t/cm^2 \qquad \sigma_{cr} = \frac{\pi^2 \times 5000}{150^2} = 2.19 \ t/cm^2 \ \times$$

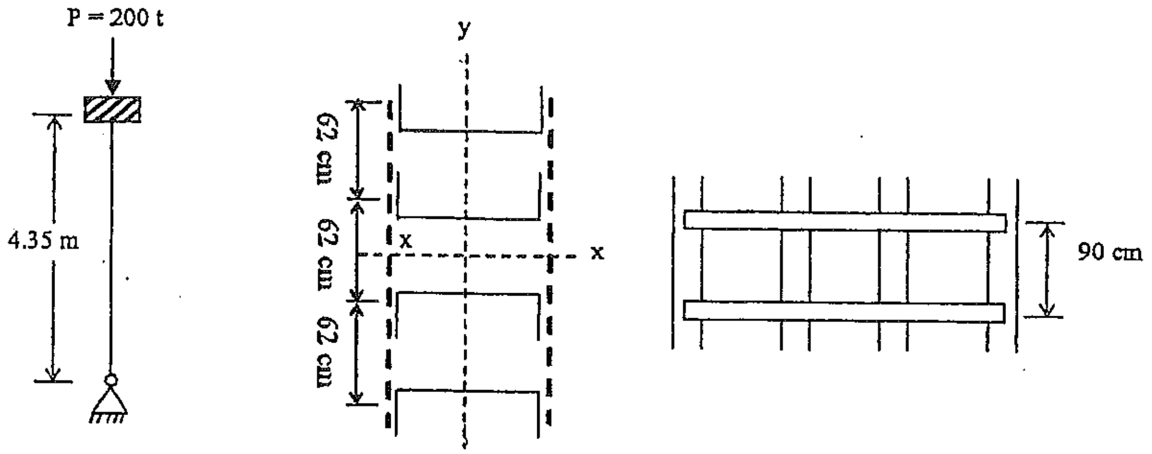
$1 < \sigma_{cr} < 1.8 \ t/cm^2$

$$E = \frac{\sigma_2 - \sigma_1}{\epsilon_2 - \epsilon_1} = 4000 \ t/cm^2 \qquad \sigma_{cr} = \frac{\pi^2 \times 4000}{150^2} = 1.752 \ t/cm^2 \ \checkmark$$

$(1 < 1.752 < 1.8)$

$$\sigma_{all} = \frac{\sigma_{cr}}{FS} = \frac{\sigma_{cr}}{2} = \frac{1.752}{2} = 0.876 \ t/cm^2$$

3. (30 points) A built-up column shown in the figure is made up of four channel [300 sections and carries a load of P = 200 tons. Check whether the built-up column is safely designed or not according to TS648. Assume St37 steel.



buckling about y axis → built-up column behaves as a solid column, No ne
No need to increase λ_y.

buckling about x axis → built-up column behaves as a open column.
It has battens in that direction.
We need to increase λ_x

4 member linked, $m' = 4$

$$\lambda_{xi} = \sqrt{\lambda_x^2 + \frac{4}{2} \cdot \lambda_{1x}^2} \qquad \lambda_{yi} = \lambda_y \qquad \lambda_1 \leq \frac{1}{2} \lambda_y \left(4 - 3 \frac{w_{xi} \cdot S}{F \cdot \sigma_{cem}}\right)$$

For UPN 300; $A' = 58.8 \text{ cm}^2$

→ It is made up four sections $A = 4A' = 4 \times 58.8 = 235.2 \text{ cm}^2$

For UPN 300; $I_y' = 8030 \text{ cm}^4$ $\quad I_y = 4 I_y' = 4 \times 8030 = 32120 \text{ cm}^2$

$I_x' = 495 \text{ cm}^4$

$$I_x = \left[(495 + 58.8 \times (62+31)^2) + (495 + 58.8 \times (31)^2) + (495 + 58.8 \times 31^2) + (495 + 58.8 (62+31)^2\right] = \underline{1132.116 \text{ cm}^4}$$

$$i_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{1132116}{235.2}} = 69.38 \text{ cm}, \quad i_y = \sqrt{\frac{I_y}{A}} = 11.69 \text{ cm}$$

Buckling about x axis

$\lambda_{xi} = \sqrt{\lambda_x^2 + \frac{m'}{2} \lambda_{1x}^2}$, $\lambda_x = \frac{k_x \cdot L}{i_x} = \frac{(2.0)(435cm)}{69.38\ cm} = 12.54$

$\lambda_{1x} = \frac{S_1}{i_1} = \frac{90}{2.9} = 31.03$

$\lambda_{xi} = \sqrt{12.54^2 + \left(\frac{4}{2}\right)(31.04)^2} = 45.65$, $\omega_{x_i} = \underline{\underline{1.215}}$ from table

Buckling about y axis

$\lambda_y = \frac{k_y \cdot L}{i_y} = \frac{(2.0)(435cm)}{(11.69cm)} = 74.42$

$\omega_{yi} = 1.535$

→ Buckling about y axis is more critical,

$\omega_{cr} = \omega_y = 1.535$

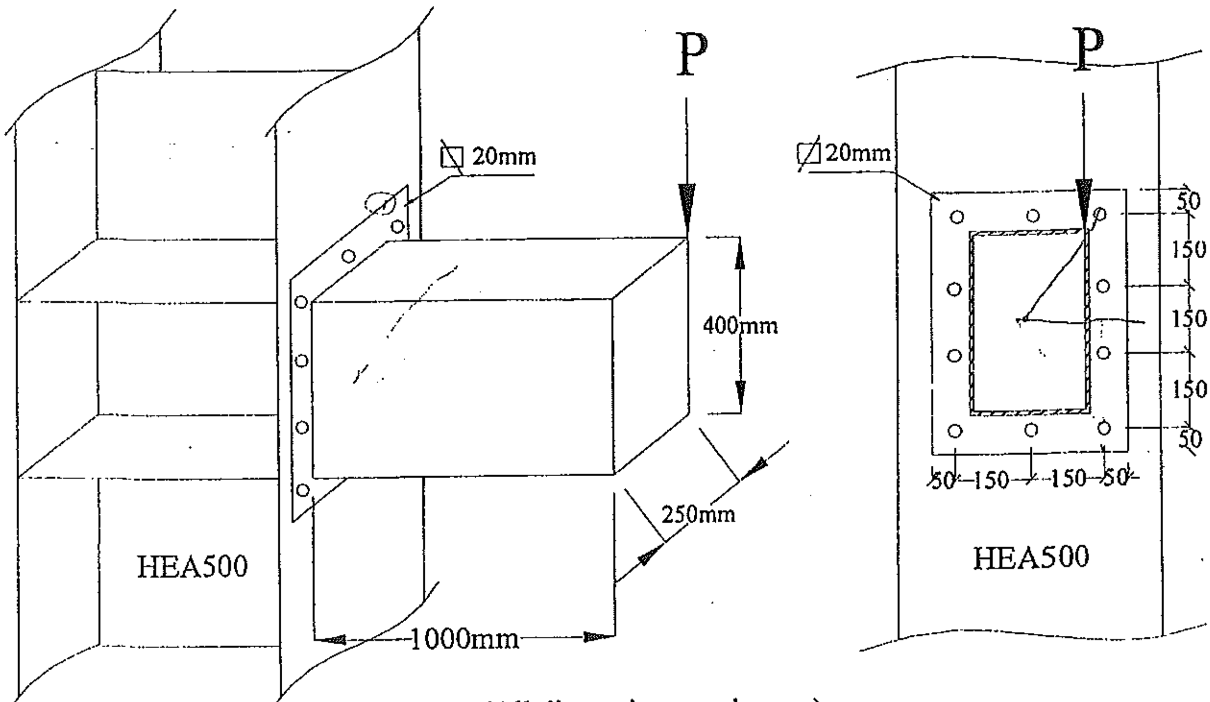
$\sigma_{beam} = \frac{1.44}{1.535} = 0.94\ t/cm^2$

$P_{all} = (0.94)(235.2) = 221.1$ ton > 200 tons

not safely designed.

Dr. Uğur Polat	**Middle East Technical University** **Department of Civil Engineering** **CE388 – Fundamentals of Steel Design** **Tutorial #3** Duration: 60 min. May 26, 2011	
		Total

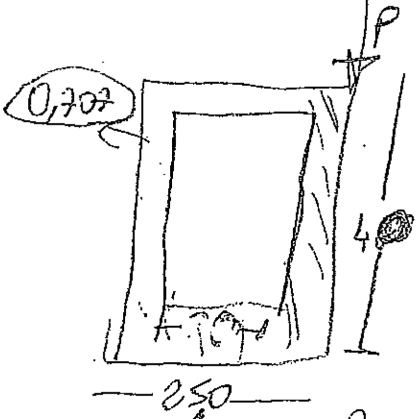
A steel bracket with a box section is first attached to a 20mm thick end plate by 10mm welding all around its x-section and then connected to the flange of a HEA500 steel column by 10M16 unfinished black bolts, as shown. Calculate the allowable value of P that can be carried safely by the connection based on TS648 requirements. Assume EY loading, **Fe37** steel and $\tau_{all} = 1.1$ t/cm^2 for welding.



(All dimensions are in mm)

$M = 100P$

$$j = [(22,5)^2 + (15)^2] \times 4 + [(7,5)^2 + (15)^2] \times 4 + [(22,5)^2] \times 2$$



$V = \frac{\pi d^2}{4} \times \tau$ 22,8

$\boxed{0,707}$

$t = 0,7$

$$I_x = \frac{1}{12}(25)(40)^3 - \frac{1}{12}(25 - 2\times0,707)(40-2\times$$

$$I_y = \frac{1}{12} 40(25)^3 - \frac{1}{12}(40 - 2\times0,707)(25 - 2\times0,70$$

34

$P_y = 0,707P$

$M = 0,707P(12,5)$

$\cancel{M = 0{,}707P}$ $M_z = 12{,}5Pt$

$A_w = 2(25+40) = 91$ cm²

$$\sigma_{xx}'' = \frac{M_z \cdot y}{I_z} = \frac{(12{,}5Pt)(20)}{I_z}$$

$$\sigma_{yy}'' = \frac{M_z \cdot x}{I_z} = \frac{(12{,}5Pt)(12{,}5)}{I_z}$$

$$\boxed{\sigma_z = \frac{100P\,(20)}{I_x}}$$

$$\sigma_y' = \frac{P}{91}$$

$I_x = 22$

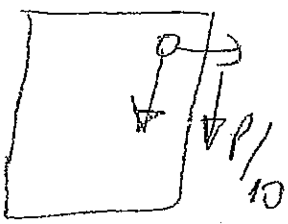
$\sigma_r = 0{,}09P \leq 1{,}1$

$$\sqrt{(\sigma_{xx}'')^2 + (\sigma_{yy}'' + \sigma_y')^2 + (\sigma_z)^2} = \left[\frac{\pi (0{,}8)^2}{4} \times 1{,}1\right]$$

b) $J = \left[(22{,}5)^2 + (15)^2\right] \times 4 + \left[(7{,}5)^2 + (15)^2\right] \times 4 + \left[(22{,}5)^2\right] \times 2$

$M = \boxed{100P}$

$\sigma_{xx}'' \; \frac{\overset{15P}{\cancel{100P}}(22{,}5)}{J}$ $\qquad \sigma_y' = \frac{P}{10}$



$$\sigma_{yy} = \frac{\overset{15P}{\cancel{100P}}(15)}{J}$$

$$\sqrt{(\sigma_{xx}'')^2 + \sigma_y^2 + (\sigma_y')^2} =$$

(Normal stress bulunmaz / bükülmeden gelen)

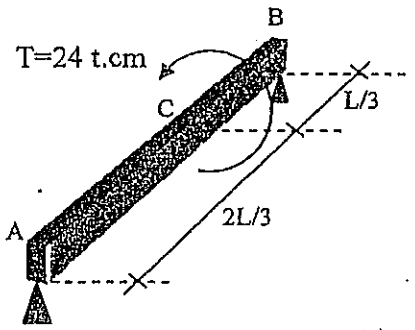
$$\frac{(100P \times 20)}{I_x} = \sigma_z$$

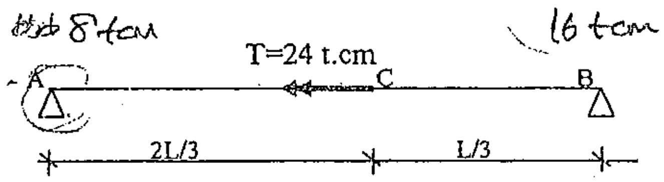
Dr. Çetin YILMAZ Dr. Uğur POLAT Dr. Ahmet TÜRER	**Middle East Technical University** **Department of Civil Engineering** **CE388 – Fundamentals of Steel Design** **Tutorial #3** Duration: 75 min	Q1	/ 25
		Q2	/ 25
		Q3	/ 50
		Total	

ID No:	**Name:**	**Section:**

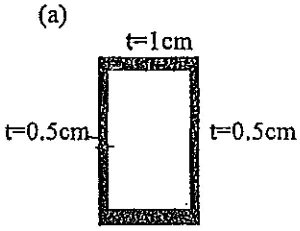
1. (*25 pts*) Calculate the maximum shear stress due to torsion if the beam has
   - **a)** a closed box section
   - **b)** an open section

   as shown in the figure below. Indicate the direction and distribution of shear stresses over the section. The ends of the beam are restrained against twisting at joints A and B but free to warp.



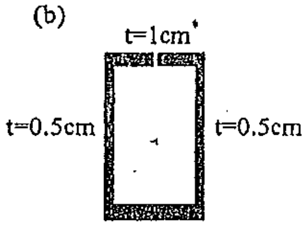


(a)



Outer dimensions = 20cm × 40cm.

(b)

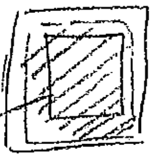


Outer dimensions = 20cm × 40cm.
(*the cut at the top flange is negligibly small in width but continuous along the length of the beam.)

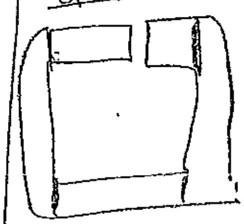
For thin walled tubular, noncircular closed sections

$$\tau_u = \frac{Tt}{2At}$$

smaller A thickness



open

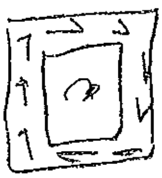


$$J = \frac{1}{3}\sum b_i t_i^3$$

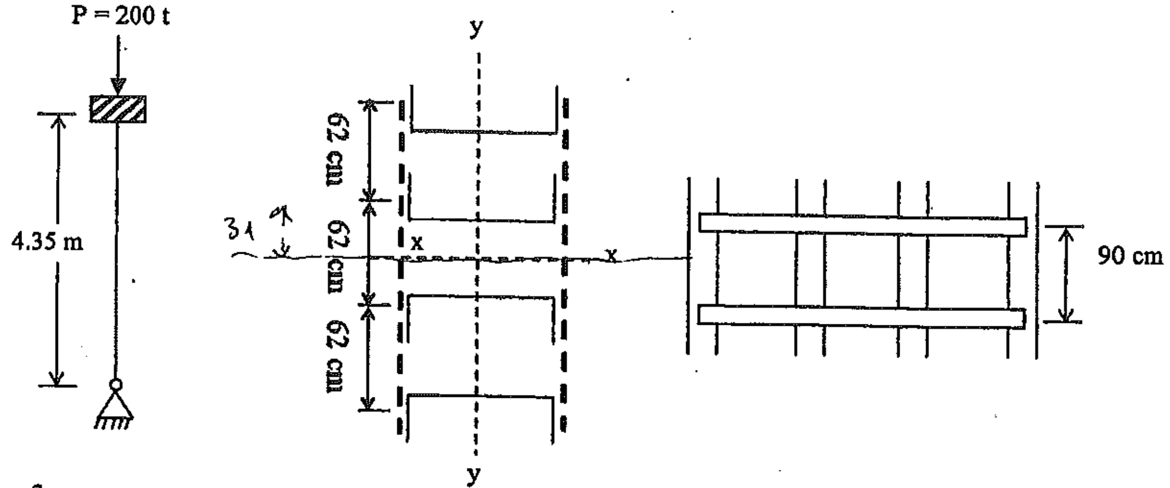
$$\tau = \frac{T t}{J}$$

maximum shearing stress in a solid rectagular section $\tau_{max} = \frac{Tt}{\alpha b t^2}$

$$\boxed{\tau = \frac{16}{2 \cdot A \cdot 0{,}5} =}$$



3. (30 points) A built-up column shown in the figure is made up of four channel [300 sections and carries a load of P = 200 tons. Check whether the built-up column is safely designed or not according to TS648. Assume St37 steel.



$A_1 = 58.8\ cm^2$

$I_{x1} = 4.95\ cm^4$

$I_{y1} = 8030\ cm^4$

$i_{x1} = 2.90\ cm$

$i_{y1} = 11.70$

$A = 4(A_1) = 4.(58.8) = 235.2\ cm^2$

$I_x = 2[I_{x1} + A(31)^2] + 2[I_{x1} + A.(93)^2] = 1132116\ cm^4$

$i_x = \sqrt{\frac{I_x}{A}} \quad i_y = \sqrt{\frac{I_y}{A}}$

Bucked colounm design

$\lambda_{x1} = \sqrt{\lambda_x^2 + \frac{m'}{2}\lambda_1^2}$

$\lambda_x = \frac{K_x . L}{i_1} = \frac{(2.0)\ (435)}{69.38} = 12.54$

$\lambda_1 = \frac{S_1}{i_1} = \frac{90}{2.90} = 31.03$

$\lambda_{x1} = \sqrt{12.54^2 + \frac{4}{2}(31.03)^2}$

$\lambda_{x1} = 45.6$

$w_{x1} = 1.22$ (from table)

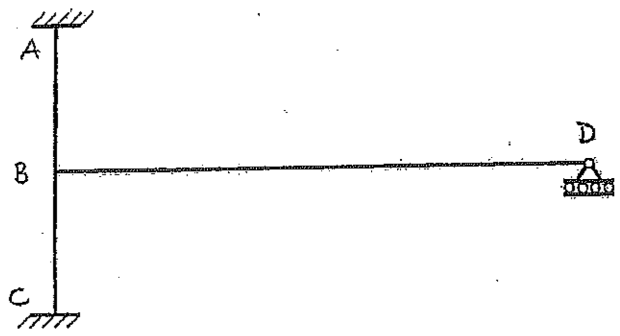
Backing about y-axis

$\lambda_y = \frac{K_y . L}{i_y} = \frac{(2.0)(435)}{11.69} = 74.4$, $w_{y1} = 1.53$ (from table)

POP-QUIZ-1 (Duration:10 minutes)

NAME OF STUDENT:......SOLUTION............SIGNATURE:..........................DATE:.............

QUESTION: Determine both the "*statical indeterminacy*" and the "*kinematical indeterminacy*" of the given frame structure.



Statical Determinacy :

$D_T = 3n + r - e - x$

$D_T = 0 + 7 - 3 - 0 = \underline{\underline{4}}$

Indet 4°

Kinematical Indeterminacy (including axial deformations)

3 @ B , 2 @ D $\rightarrow$ 5 Indet 5°

$\Rightarrow$ 1 Lateral drift, 2 rotations (@B & @D) $\longrightarrow$ 3 Indet 3°

(ignoring axial deformations)

	Middle East Technical University Department of Civil Engineering CE485 – Fundamentals of Steel Design Tutorial #1		
Dr. Çetin YILMAZ Dr. Uğur POLAT Dr. Ahmet TÜRER	Duration: 110 min. Oct.27, 2009		

Q1	/03
Q2	/04
Q3	/08
Q4	/06
Q5	/04
Q6	/20
Q7	/25
Q8	/30
Total	

ID No: **net**   **Section:**

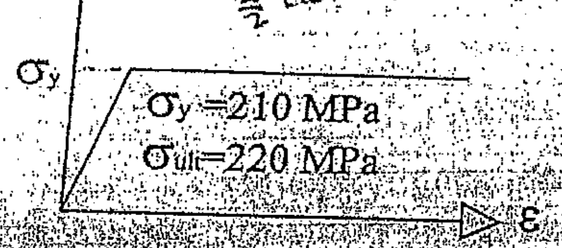
1. (*3 pts*) What is ductility?

Ductility is defined as extensive load carrying capacity without failure.

2. (*4 pts*) What is the allowable tensile stress σ_{cem} given by TS648 for the following steel materials?

$\frac{3}{5}\,210 = 126$ MPa ✓

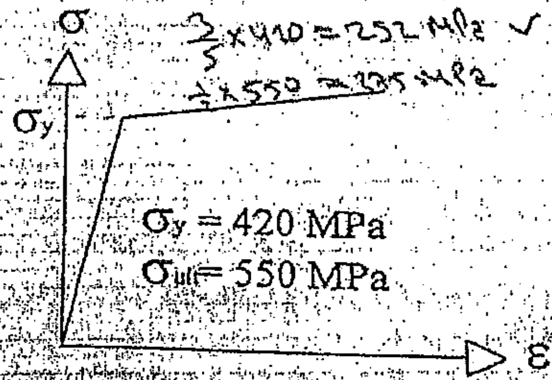
$\frac{1}{2}\,220 \rightarrow 110$ MPa



σ_{cem} = 220 MPa

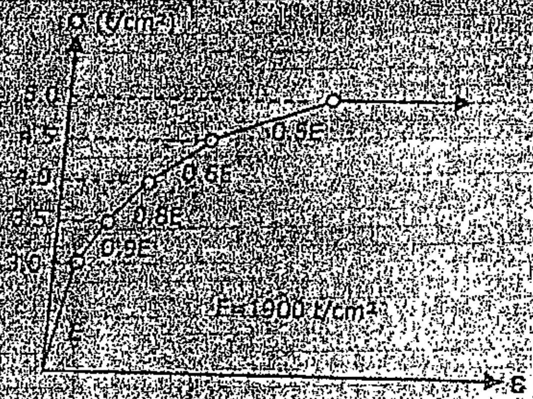
$\frac{3}{5} \times 420 = 252$ MPa ✓

$\frac{1}{2} \times 550 = 275$ MPa



σ_{cem} = 252 MPa

3. (*8 points*) Given the stress-strain diagram shown, obtain the allowable stress for a slenderness ratio of λ=65 using a Factor of safety of 2.0



$\sigma_{cr} = \frac{\pi^2 E_t}{\lambda^2}$

1) $0 < \sigma_{cr} < 3$

$\sigma_{cr} = \frac{\pi^2 \times 1900}{65^2} = 4{,}438 > 3$ ✗

2) $3 < \sigma_{cr} < 3{,}5$

$\sigma_{cr} = \frac{\pi^2 \times 1900 \times 0{,}9}{65^2} = 3{,}995 > 3{,}5$ ✗

3) $3{,}5 < \sigma_{cr} < 4$

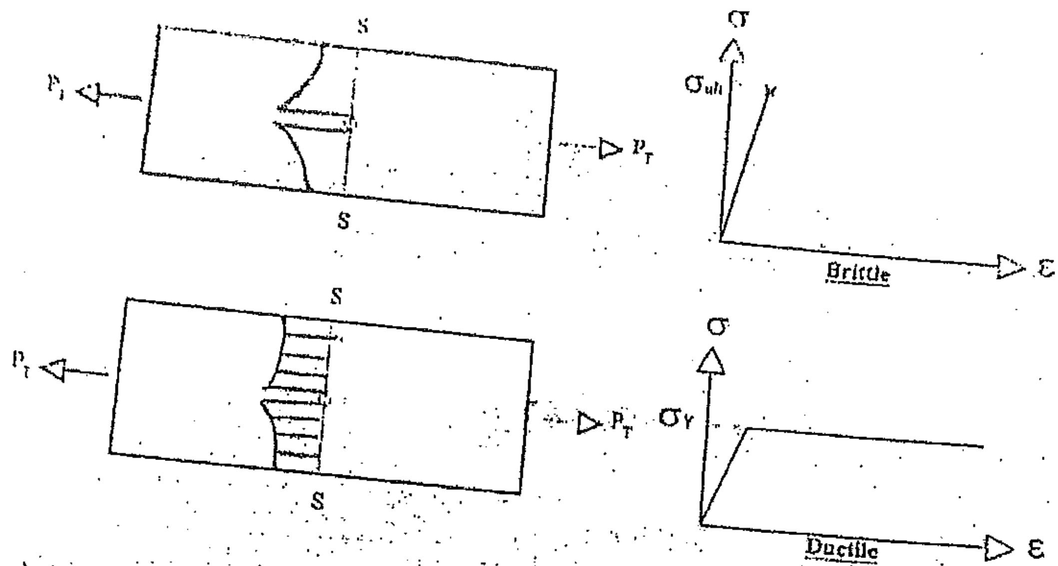
$\sigma_{cr} = \frac{\pi^2 \times 1900 \times 0{,}8}{65^2} = 3{,}55$ MPa < 4 ✓

$\gamma = 2 \Rightarrow \sigma_{all} = \frac{3{,}55}{2}$

$\sigma_{all} = 1{,}775$ MPa

4. (*6 pts*) Plot the axial stress distribution along section S-S just before failure if the plate is made of a
   a) Brittle material
   b) Ductile material

What is the major difference between the resulting stress distributions?



5. (*4 pts*) A structural element is obtained by connecting two identical steel plates, as shown. Assuming that the element length is very short and buckling is not an issue, select by underlining the statement which correctly describes the relationship between the tensile load capacity P_T and the compressive load capacity P_C and briefly explain why.

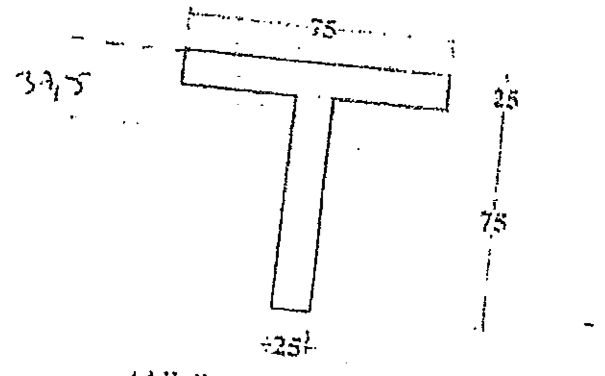
$\boxed{P_T < P_C}$ $\qquad$ $P_T = P_C$ $\qquad$ $P_T > P_C$

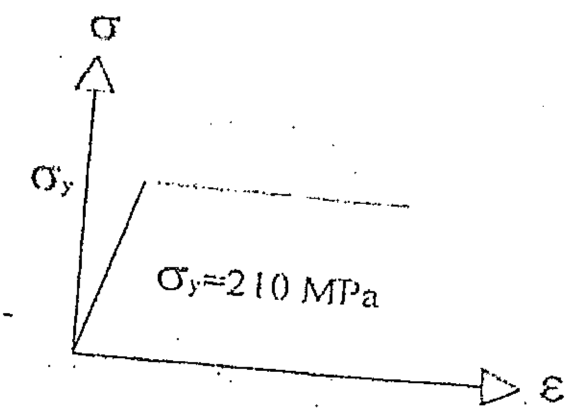
Justification:

tension, we use net area
compression, we use gross area } So A tension < A compression

so $P_T < P_C$

6. *(20 pts)* For the section and the material behavior shown, calculate the yield moment capacity M_Y and the fully plastic moment capacity M_P.





(All dimensions are in mm)

a)

$$\bar{y} = \frac{25\times75\times\frac{25}{2} + 25\times75\times\left(\frac{75}{2}+25\right)}{25\times75\times2} = 37,5 \text{ mm}$$

$$I = \frac{1}{12}\times25\times75^3 + \frac{1}{12}\times75\times25^3 + 25\times75\times50^2 = 3320313 \text{ mm}^4$$
$$= 3,32\cdot10^{-6}$$

$$\sigma_y = \frac{M_y}{I}c \qquad 210000 = \frac{M_y}{3,32\cdot10^{-6}}\times 62,5\times10^{-3} \Rightarrow M_y = 11,156 \text{ kN.m}$$

b) Top flange area and the bottom web area are equal. Since plastic theory, equal area so NA is in the between flange section and the web section.

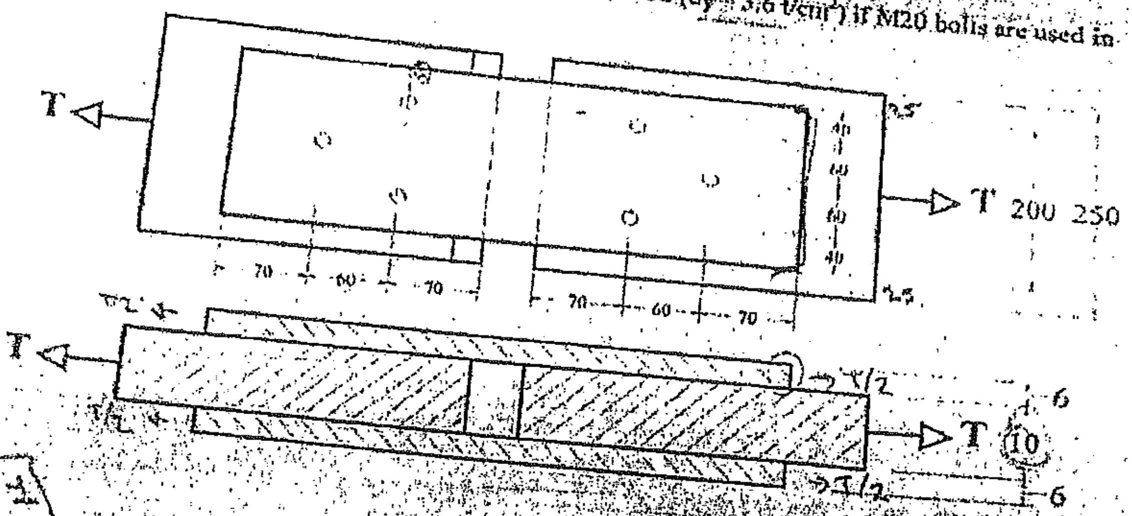
$\bar{y} = 75$ mm $\qquad A_w = 25\times75 = A_f = 75\times25 = 1,875\cdot10^{-3}$ m²

$$\sigma_{y_1} A_w = \sigma_{y_2} A_f$$
$$\sigma_{y_1} = \sigma_{y_2}$$

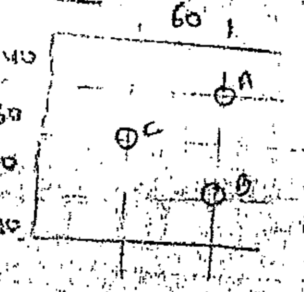
$$M_p = 1,875\cdot10^{-3}\times210.000\,(12,5+37,5)\times10^{-3}$$

$$M_p = 19,6875 \text{ kN.m}$$

7. *(25 pts)* Calculate T_{max} allowed by TS648 for St.52 ($\sigma_y = 3.6\ t/cm^2$) if M20 bolts are used in the connections.



Member 1



$\phi = 20+2 = 22$ mm

Path AB

$\boxed{12(200 - 2\times22) = 1872\ mm^2 \rightarrow \text{Critical}}$

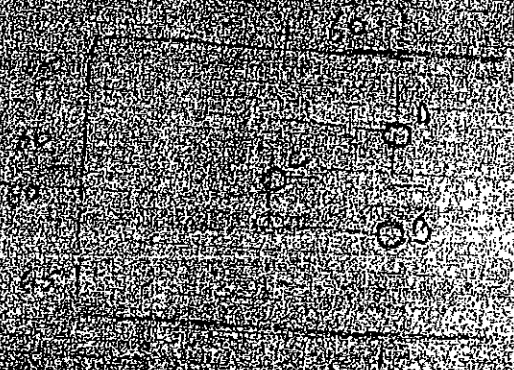
Path ACB

$12\left(200 - 3\times22 + \frac{60^2}{4\times60}\times2\right) = 1968\ mm^2$

$t = 6+6 = 12$ mm.

$0.85\ A_g = 12\times200\times0.85 = 2040\ mm^2$

Member 2



Path AB

$10(250 - 2\times22) = 2060\ mm^2$

Path ACB

$10\left(250 - 3\times22 + \frac{60^2}{4\times60}\times2\right) = 2140\ mm^2$

$0.85\ A_g = 0.85\times10\times250 = 2125\ mm^2$

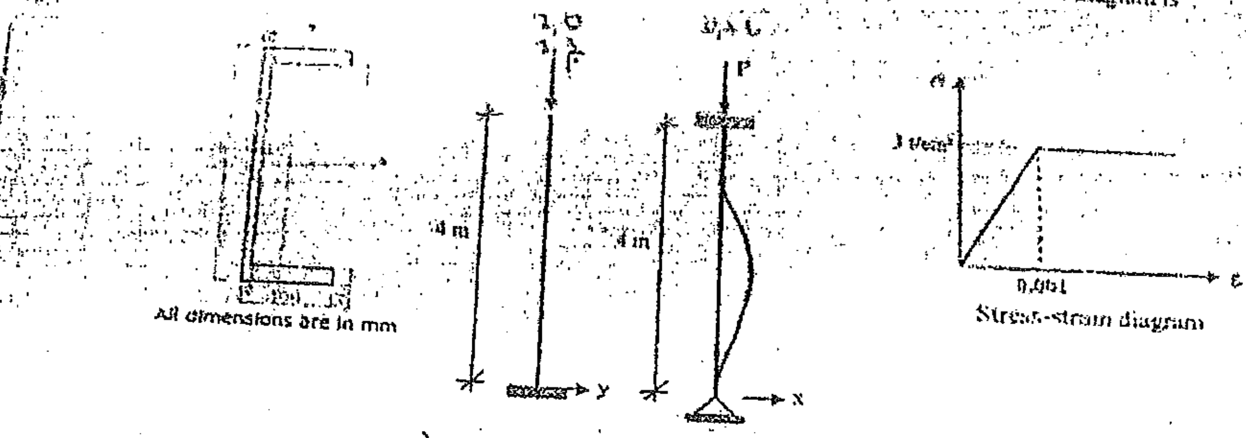
[illegible]

The factor of safety w.r.t. yield stress is used

$\times\ 360\ MPa = 216\ MPa$ $\qquad T = 216\ MPa \times 1872\ mm^2 = 404352\ N$

$(T_{max})_{all} = 404,352$ kN

8. *(30 pts)* Find the maximum load P_{cr} that the column can carry if the stress-strain diagram is as shown.



$L_e = 2 \times 4 = 8\,m.$ $\qquad$ $L_e = 4 \times 0{,}7 = 2{,}8\,m$

$$\bar{x} = \frac{10 \times 300 \times 5 + 90 \times 20 \times 55 \times 2}{90 \times 20 \times 2 + 10 \times 300} = 32{,}3\ mm$$

$\bar{y} = 150\ mm$

$$I_x = \frac{1}{12} \times 20 \times 300^3 + \frac{1}{12} \times 90 \times 20^3 \times 2 + 2 \times 90 \times 20 \times 140^2$$

$I_x = 93180000\ mm^4 = 9{,}318 \cdot 10^{-5}\ m^4$

$$I_y = \frac{1}{12} \times 10^3 \times 300 + 10 \times 300 \times 27{,}3^2 + 2 \times \frac{1}{12} \times 90^3 \times 20 + 2 \times 90 \times 20 \times 27{,}3^2$$

$I_y = 6481144\ mm^4 = 6{,}48 \cdot 10^{-6}\ m^4$

$A = 300 \times 20 + 2 \times 90 \times 10 = 6600\ mm^2 = 6{,}6 \cdot 10^{-3}\ m^2$

$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{9{,}318 \cdot 10^{-5}}{6{,}6 \cdot 10^{-3}}} = 0{,}1188\ m \qquad \lambda_x = \frac{8\ m}{0{,}1188\ m} = 67{,}34$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{6{,}48 \cdot 10^{-6}}{6{,}6 \cdot 10^{-3}}} = 0{,}031\ m \qquad \boxed{\lambda_y = \frac{2{,}8}{0{,}031} = 90{,}32}$$

$$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2\,300}{90{,}32^2} = 3{,}248\ t/cm^2 > 3\ t/cm^2$$

$P_{cr} = 3 \times 66 = 198\ ton$

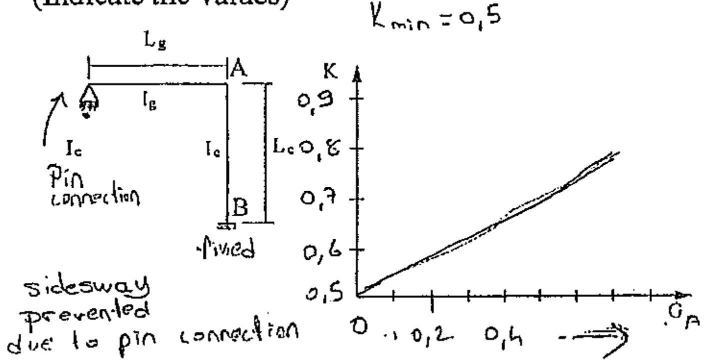
Dr. Çetin Yılmaz

CE485
SUPPLEMENTARY EXAM
PART 1 (Closed Book)
(Duration: 30 minutes; 30 points)

Name:..... ....:.Student No: .....Section:...........

1. (3 Points) Give a sketch for effective length factor vs. G for the column AB, if $G = \dfrac{I_c/L_c}{I_g/L_g}$.

(Indicate the values)



$G_B = 1$ (fixed end)

$$G_A = \frac{I_c/L_c}{I_g/L_g \cdot 1,5}$$

$G_{AB} \Rightarrow G_A$ & G_B indicates since G_B is exact, only G_A effects the effective Length.

2. (4 Points) Explain the critical slenderness ratio λ_p as stated in TS648. What will be its value for a steel with yield stress σ_y=3.5t/cm^2?

λ_p states the elastic ⇔ inelastic buckling limit.

$$\sigma_y = \frac{\pi^2 E}{\lambda_p^2} \Rightarrow \lambda_p = \sqrt{\frac{\pi^2 E}{\sigma_y}} \Rightarrow \lambda_p = 76,91$$

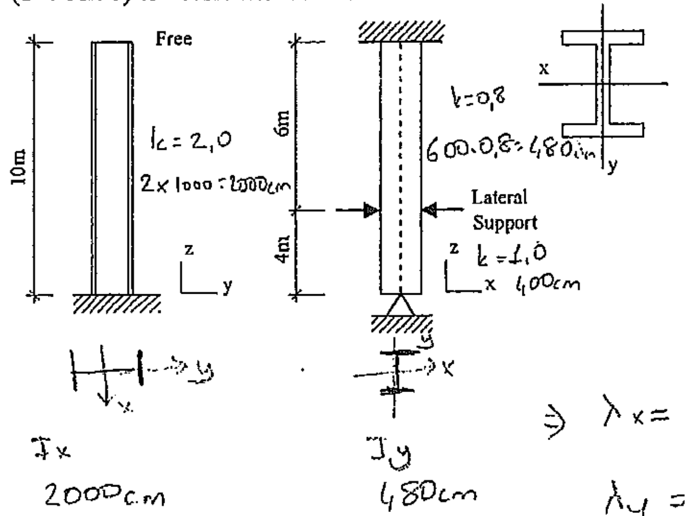
3. (3 points) Indicate the following values:

a. Poisson's ratio for St52:

b. Ultimate strain for St37: $\dfrac{2400}{2100 \cdot 10^3} = 1,142 \cdot 10^{-3}$

c. Ultimate strength for St60:

4. (3 Points) Determine the maximum slenderness ratio of the column shown.



I300
F=69.0cm^2
I_x=9800cm^4
I_y=451cm^4

$$i_x = \sqrt{\frac{I_x}{F}} = 11,92$$

$$i_y = \sqrt{\frac{I_y}{F}} = 2,55$$

$$\Rightarrow \lambda_x = \frac{2000}{11,92} = 167,78$$

$$\lambda_y = \frac{480}{2,55} = 188,23 \rightarrow \text{maximum slenderness.}$$

Dr. Çetin Yılmaz
Dr. Ahmet Türer

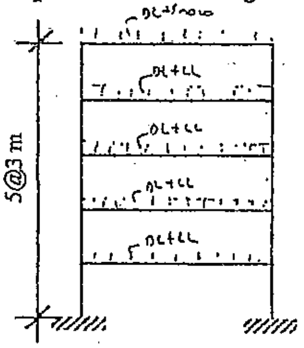
Nov. 7, 2004

Q1(18)	
Q2(18)	
Q3(10)	
Q4(18)	

CE485
TERM TEST 1
PART 2 (Open Book)
(Time allowed: 90 minutes for 64 points)

Name:..Student No:....................Section:...........

1. (18 Points) Shown residential building is located in Ankara. Calculate the equivalent **earthquake forces** acting on each story.



Given; $I = 1.0$
DL = 250 tons/floor
Snow = 75 kgf/m² = 0,075 t/m²
Floor Area = 400 m²
LL = 200 kgf/m² = 0,2 t/m²
EQ1 EQ Zone 3 → $A_0 = 0.2$
Z1 Soil Type Z2
Concentrically braced steel frame type structure
(Earthquake loads are carried only by steel cross-bracings)
(Deprem yüklerinin tamamının merkezi çaprazlı perdelerle taşındığı çelik yapı tipi)

$W_1 = W_2 = W_3 = W_4 = 250 + 0.2 \times 400 \times 0.3 = 274$ t

$W_5 = 250 + 0.075 \times 400 \times 0.3 = 259$ t

$\Sigma W_T = 4 \times 274 + 259 = 1355$ t

$T_1 = C_t \cdot H_N^{3/4} = 0.08 \times 15^{3/4} = 0.61$ sec

For soil type Z2 → $T_A = 0.15$, $T_B = 0.40$

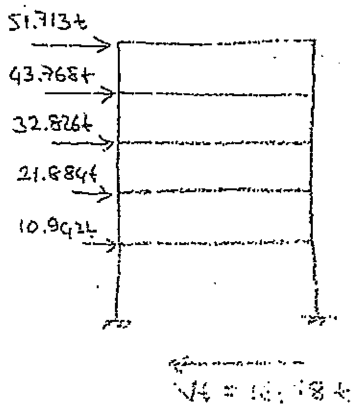
Since $T_1 > T_B$ → $S(T_1) = 2.5\left(\frac{T_B}{T_1}\right)^{0.8} = 2.5\left(\frac{0.40}{0.61}\right)^{0.8} = 1.784$

$A(T_1) = A_0 \cdot I \cdot S(T_1) = 0.2 \times 1.0 \times 1.784 = 0.3568g$

$R = 3$

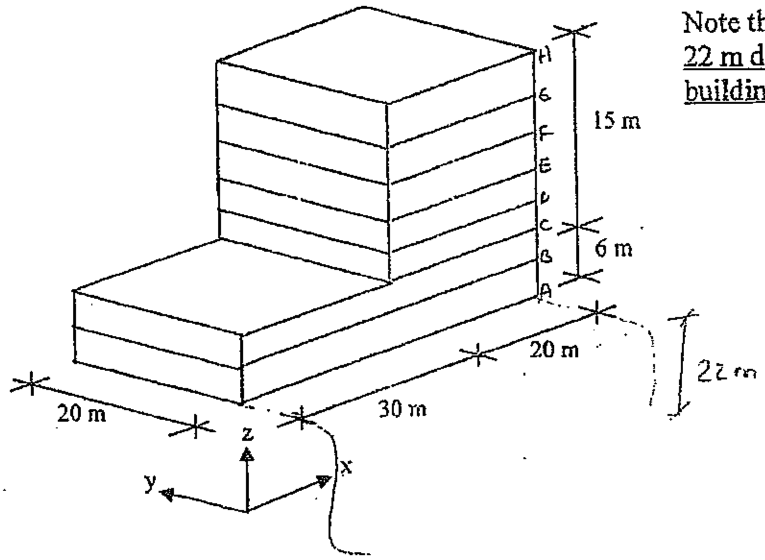
$V = \frac{\Sigma W \times A(T_1)}{R} = \frac{1355 \times 0.3568}{3} = 161.18$ t

Story	W_i(t)	H_i(m)	$W_i \cdot H_i$	$\frac{W_i \cdot H_i}{\Sigma W_i \cdot H_i}$	F_i(t)
1	274	3	822	0.0679	10.942
2	274	6	1644	0.1308	21.884
3	274	9	2466	0.2037	32.826
4	274	12	3288	0.2716	43.768
5	259	15	3885	0.3209	51.713
Σ	1355		12105	1 ✓	161.18 ✓

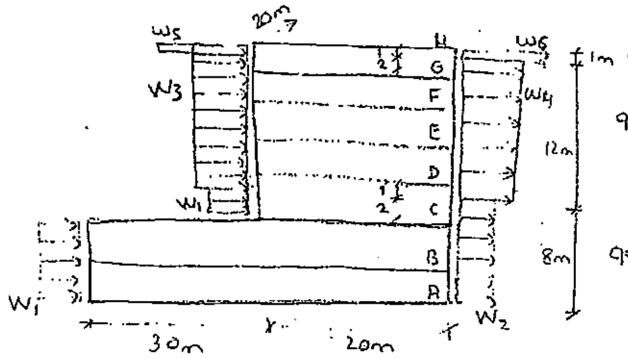


Name:..Student No:....................Section:..........

2. (18 Points) For the hotel building shown below, calculate the **wind induced** forces on the floor levels for **x and y-directions**. (Considering the wind forces on the roof, also). Assume 3 m high floors.

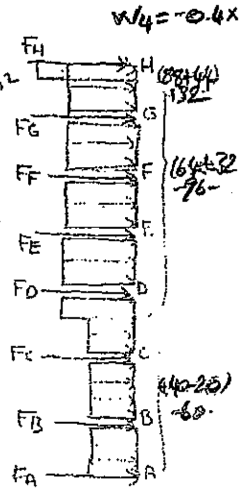


Note that, <u>the hotel is located close to a 22 m deep cliff on the longer side of the building.</u>



Wind forces for x-dir

$W_1 = 0.8 \times 50 = 40$ kgf/m²
$W_2 = -0.4 \times 50 = -20$ kgf/m²
$W_3 = 0.8 \times 80 = 64$ kgf/m²
$W_4 = -0.4 \times 80 = -32$ kgf/m²
$W_5 = 0.8 \times 110 = 88$ kgf/m²
$W_6 = -0.4 \times 110 = -44$ kgf/m²



$F_H = \frac{132 \times 1 \times 2.5 - 96 \times 2 \cdot 1}{3 \times 1000} \times 20 = 3.48\,t$

$F_G = \frac{132 \times 1 \times 0.5 + 96 \times 2 \times 2}{3 \times 1000} \times 20\,t$
$96 \times \frac{3}{2} \times 20 / 1000 = 5.88\,t$

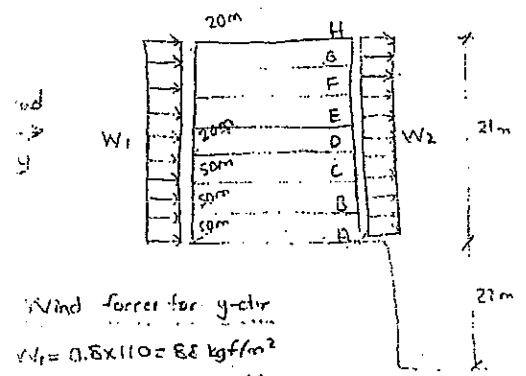
$F_F = F_E = 96 \times \frac{3 \times 20}{1000} = 5.76\,t$

$F_D = 96 \times \frac{3 \times 20}{2\,1000} + \frac{96 \times 1 \times 2.5 \cdot 60 \times 2 \times 1}{3 \times 1000} \times 20$
$= 5.28\,t$

$F_C = \frac{96 \times 1 \times 0.5 + 60 \times 2 \times 2}{3 \times 1000} \times 20$
$+ 60 \times \frac{3}{2} \times \frac{20}{1000} = 3.72\,t$

$F_B = 60 \times 3 \times \frac{20}{1000} = 3.6\,t$

$F_A = 60 \times \frac{3}{2} \times \frac{20}{1000} = 1.8\,t$



Wind forces for y-dir

$W_1 = 0.8 \times 110 = 88$ kgf/m²
$W_2 = -0.4 \times 110 = -44$ kgf/m²

$F_H = (\overset{132}{88+44}) \times \frac{3}{2} \times \frac{20}{1000} = 3.96\,t$

$F_G = F_F = F_E = F_D = 132 \times 3 \times \frac{20}{1000} = 7.92\,t$

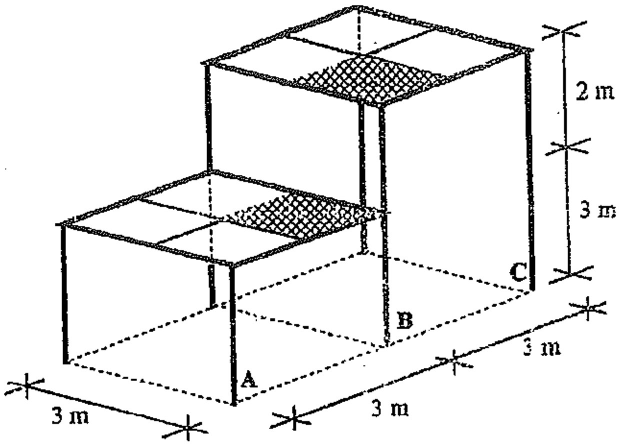
$F_C = (132 \times \frac{3}{2} \times 20 + 132 \times \frac{3}{2} \times 50)/1000 = 13.86\,t$

$F_B = 132 \times 3 \times 50/1000 = 19.8\,t$

15

Name:.......................................Student No:...................Section:...........

3. (10 Points) Calculate the **snow load** acting on column **B**.



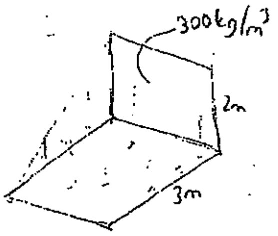
Assume that;

a. the snow load is 75 kgf/m² and 300 kg/m³
b. load is distributed between columns based on tributary area.

From 75 kgf/m²

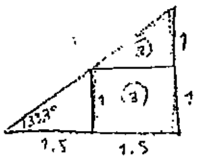
$F_1 = 2 \times \frac{75 \times 3 \times 3}{4} = 337.5$ kgf

From 300 kg/m³



$F_2 = 1 \times 1.5 \times \frac{1}{2} \times \frac{3}{2} \times 300 = 337.5$ kgf

$F_3 = 1.5 \times 1 \times 300 \times \frac{3}{2} = 675$ kgf

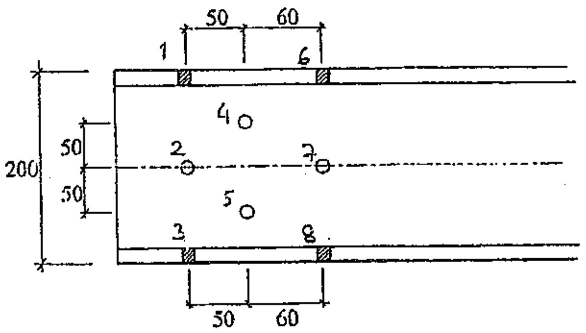


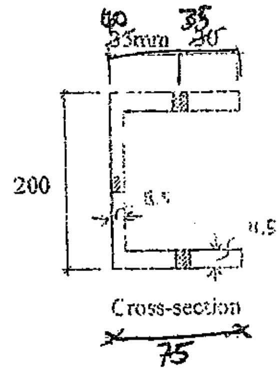
Snow load on column B

$F = F_1 + F_2 + F_3 = 337.5 + 337.5 + 675 = 1350$ kgf.

Name:..Student No:....................Section:.........

4. (18 Points) A C200 section is utilized as a tension member in a truss, with the connection pattern as shown in the sketch. Calculate the maximum allowable tension that can be carried by the member. (St37, TS648 specifications, M16 turned bolts)





$F = 32.2\ cm^2$

$d_i = 1.6 + 0.1 = 1.7\ cm$

Path 1-2-3

$A_n = 32.2 - 2\times1.7\times1.15 - 1.7\times0.85 = 26.845\ cm^2$

Path 1-4-5-3

1.332

$A_n = 32.2 - 2\times1.7\times1.15 - 2\times1.7\times0.85 + 2\times\dfrac{5^2}{4\left(5+4-\frac{0.85}{2}-\frac{1.15}{2}\right)}\times0.85 = 26.728\ m^2$

8

Path 1-4-2-5-3

$A_n = 32.2 - 2\times1.7\times1.15 - 3\times1.7\times0.85 + 2\times\dfrac{5^2}{4\times8}\times0.85 + 2\times\dfrac{5^2}{4\times5}\times0.85 = 27.408\ cm^2$

$0.85\times A_g = 0.85\times32.2 = 27.37\ cm^2$

$A_{n_{min}} = 26.728\ cm^2$

$\sigma_{all} = 1.44\ t/cm^2$

$P_{all} = A_n\times\sigma_{all} = 26.728\times1.44 = 38.49\ t$

Dr. Çetin Yılmaz
Dr. Ahmet Türer

SOLUTIONS

Nov 7, 2004

CE485
TERM TEST 1
PART 1 (Closed Book)
(Time allowed: 45 minutes for 41 points)

Name:.......................................Student No:....................Section:...........

1. (5 points) Indicate the following values:

   a) Approximate total production of steel in last year..16 million tons in Turkey

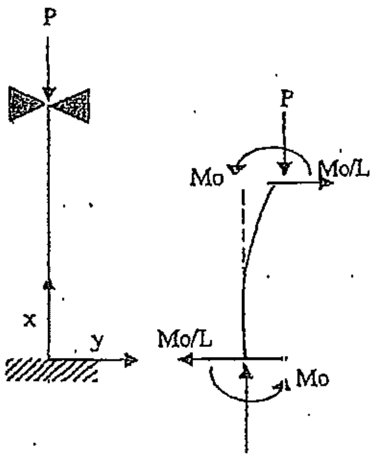
   b) Modulus of elasticity for St52.. 2.1×10^{6} kg/cm^2.........

   c) Shear modulus for St37.... 807 t/cm^2.......................

   d) Ultimate strength for St33.... 3300 kg/cm^2.......................

   e) Ultimate strain for St52.. $(100\sim200)\varepsilon_y \rightarrow 0,17\sim0,35$

2. (6 Points) To determine the Euler buckling load for the column fixed at one end pinned at the end as shown in the figure, the governing differential equation is;



$$\frac{d^2y}{dx^2}+k^2y=\frac{M_o}{EI}\left(1-\frac{x}{L}\right)$$

$$k^2=\frac{P}{EI}$$

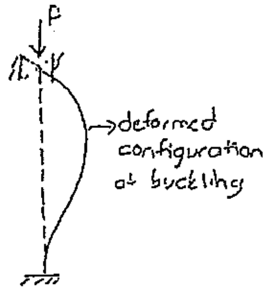
a) What is the critical force for this case?

b) Deformed configuration at buckling.

c) Boundary conditions for the solution of differential equation.

a) $P_{cr}=\dfrac{\pi^2EI}{(0{,}7L)^2}$

b)



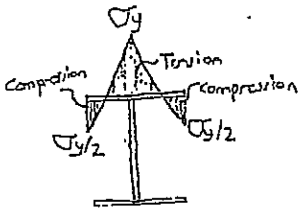
c) Boundary conditions;

at $x=0$ $y=0$
$x=0$ $y'=0$

at $x=L$ $y=0$
$x=L$ $y''=0$

Name:...Student No:...................Section:...........

3. (3 Points) Indicate the possible residual stress distribution for an I section and what are the reasons for residual stress.



Reasons for residual stress:

— Unequal cooling

— Welding.

— Flame cutting.

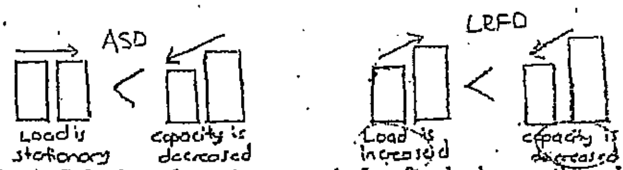
4. (3 Points) In tension members, in calculating net area, hole diameter is increased by 1 mm. Why?

Because of damage around the hole during punching or drilling operations and stress concentration around the hole.

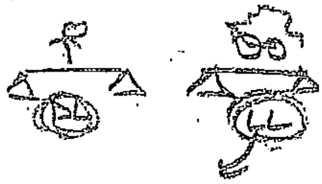
5. (3 Points) Describe two major differences between ASD and LRFD

a) Stress vs Force

1, b)



6. (3 Points) Calculate the **return period** of a design earthquake if a building will be in service for 50 years and probability of survival is 90% during its service life.



$0.90 = \left(1 - \frac{1}{R}\right)^{N} \Rightarrow 0.90 = \left(1 - \frac{1}{R}\right)^{50} \Rightarrow R = 475$ years.

7. (3 Points) Calculate the factor of safety needed if the occasional understrength is 10% and occasional overload is 35%. (Hint: R - ΔR = P + ΔP)

$\frac{1+0.35}{1-0.10}$

$FS = \frac{P+\Delta P}{R-\Delta R} = \frac{1+0.35}{1-0.10} = 1.5$

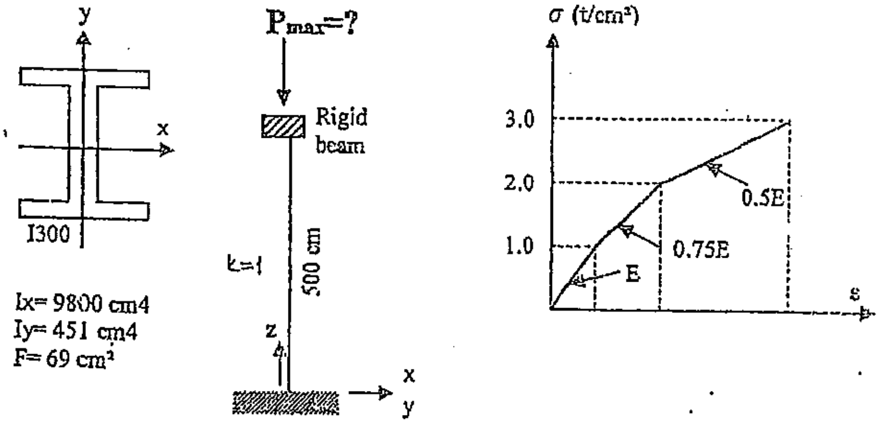
$F.S = \frac{\text{load}}{\text{strength}}$

Name:...Student No:....................Section: ....

8. (3 Points) State if DL+LL or DL only or LL only should be used for serviceability checks. Briefly describe reasoning behind it.

LL only. Because DL is stationary and has no effect on serviceability. Static DL deflection can be calculated and applied initially in the opposite direction during construction.

9. (12 points) Obtain the maximum load that can be carried by the column shown below if the stress-strain relationship is given as follows. ($E=2000 t/cm^2$)



$$i_x = \sqrt{\frac{I_x}{F}} = \sqrt{\frac{9800}{69}} = 11{,}92 \text{ cm} \qquad \lambda_x = \frac{1\times 500}{11{,}92} = 41{,}95$$

$$i_y = \sqrt{\frac{I_y}{F}} = \sqrt{\frac{451}{69}} = 2{,}56 \text{ cm} \qquad \lambda_y = \frac{1\times 500}{2{,}56} = 195{,}31$$

$\lambda_y > \lambda_x$ so take $\lambda_y = 195{,}31$

– Assume σ_{cr} is in between 0–1 (t/cm^2); ($E = 2000\ t/cm^2$)

$$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2 \times 2000}{(195{,}31)^2} = 0{,}517\ t/cm^2 \quad \text{(in the range)}$$

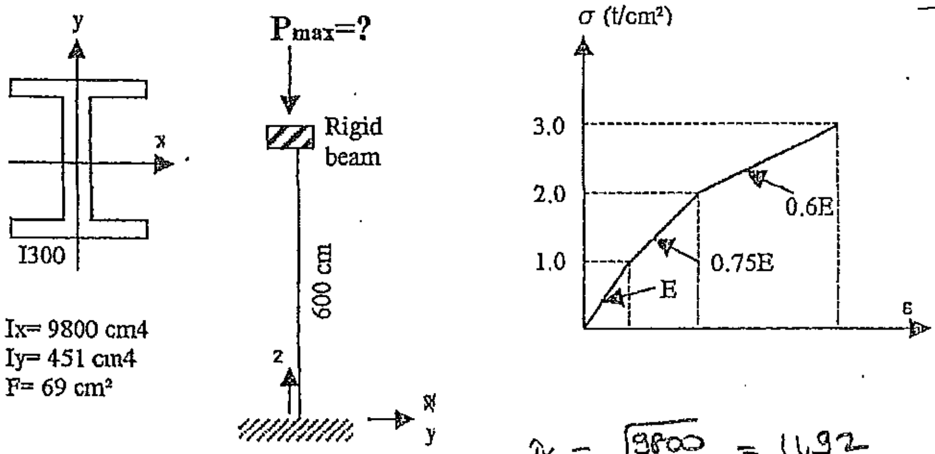
$$P_{cr} = \sigma_{cr} \times F = 0{,}517 \times 69 = 35{,}7\ t$$

Name:...Student No:....................Section:...........

8. (3 Points) State if DL+LL or DL only or LL only should be used for serviceability checks. Briefly describe reasoning behind it.

9. (12 points) Obtain the maximum load that can be carried by the column shown below if the stress-strain relationship is given as follows. (E=2000t/cm^2)

$\frac{t/cm^2}{cm^2}$



I_x, I_y

$k=1$

$$i_x = \sqrt{\frac{9800}{69}} = 11{,}92 \qquad i_y = \sqrt{\frac{451}{69}} = 2{,}56$$

$$\lambda_x = \frac{1 \cdot 600}{11{,}92}$$

$$P_{cr} = \frac{\pi^2 EI}{L^2} \rightarrow \sigma_{cr} = \frac{\pi^2 E}{\lambda^2}$$

$$\lambda_y = \frac{1 \cdot 600}{2{,}56} = 234$$

take greater one

$0 < \sigma < 1{,}0$

$$\sigma_{CR} = \frac{\pi^2 \cdot 2000}{234^2} = 0{,}36$$

$$P_{cr} = \frac{\pi^2 \cdot 2000 \cdot 451}{1 \cdot (600)^2} = 24{,}7$$

$0{,}36 \cdot 69 = 24{,}84$

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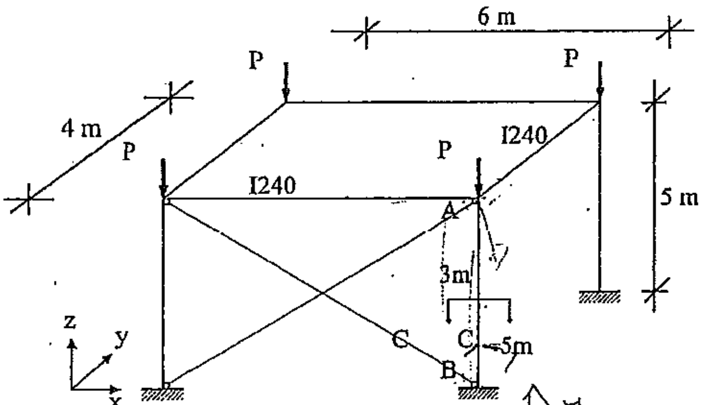
Dr. Çetin Yılmaz

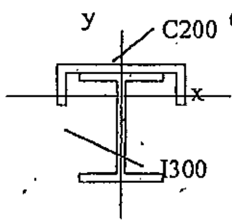
CE485
SUPPLEMENTARY EXAM
PART 2 (Open Book)
(Duration: 90 minutes; 70 points)

Name:.... ....Student No:. ...Section:...........

1. (25 Points) For column AB shown in the sketch, calculate the allowable load according to TS648, if St52 steel is used.

$I_{300} \to I_x = 9800$
$I_y = 451$

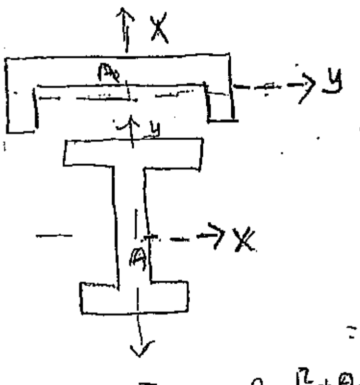


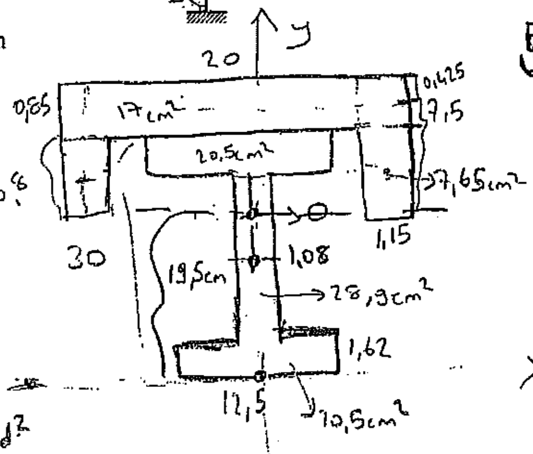


Section C-C

$C_{200} \Rightarrow I_x = 1910$
$I_y = 11,8$

$I_{240} \to I_x = 4250$
$I_y = 221$





$$\bar{y} = \frac{20,5\cdot 0,81 + 28,9\cdot 15 + 20,5\cdot(30-0,81) + 17\cdot 30,425 + 7,65\cdot 2\cdot 26,675}{69 + 32,2}$$

$\bar{y} = 19,5\,cm$

$\bar{x} = 0$

$i_x = I_{x_{I300}} + I_{y_{C200}} + A_1 d^2 + A_2 d^2$

$+ 20,5\cdot 9,69^2 + 17\cdot 10,925^2 + 7,65\cdot 2\cdot 6,1^2 = 22\,908\,cm$

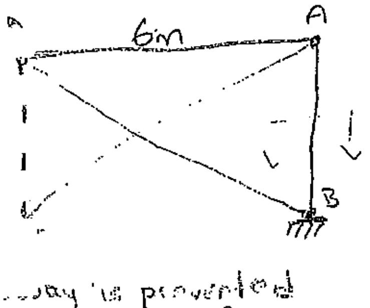
$\Rightarrow I_x = 9800 + 11,8 + 20,5\cdot 19,5^2 + 28,9\cdot 4,5^2$

$I_y = 451 + 1910 = 2361$

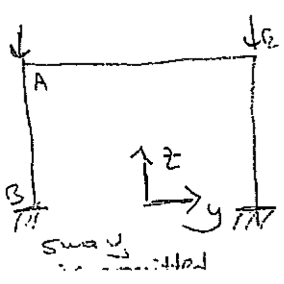
or column AB $\to$ $G_B = 1$

from table $\to k = 0,72$

$G_A = \frac{I_{AB}/L_{AB}}{I_{AD}/L_{AD}} \Rightarrow \frac{22908/500}{4250/600} = 6,46$



...way is prevented



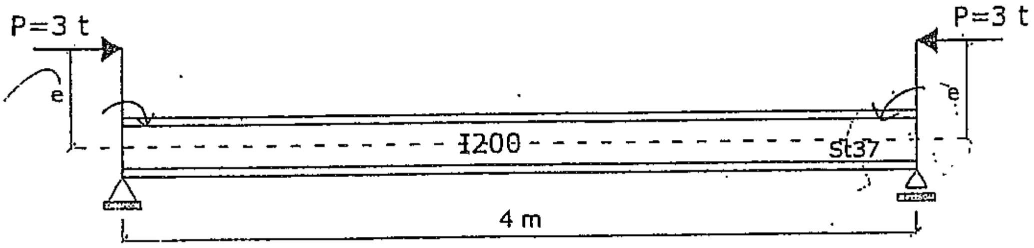
sway ...

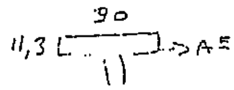
$G_B = 1$

$G_A = \frac{2361/500}{4250/400} = 0,44$ from table 1,25

Name:........ ....Student No:....................Section:...........

2. (25 Points) Find the maximum eccentricity e_{max} of the compressive loading that the following beam can support safely according to TS648. Assume that the flexural effect due to beam self-weight is negligible.





$I\,200 \Rightarrow I_x = 2140$

$I_y = 117$

$F = 33,4$

$i_y = 1,87$

$i_x = 8,00$

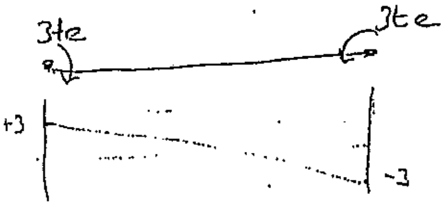
$\frac{3t}{33,4} = 0,0898 = \sigma_{eb}$

$\lambda = \frac{400}{1,87} = 213,9 \xrightarrow{\text{from table}} \sigma_{bem} = 0,1827$

$\frac{\sigma_{eb}}{\sigma_{bem}} = \frac{0,0898}{0,1827} = 0,49 > 0,15 \Rightarrow$ 2 equations will be checked.

$\sigma_e = \frac{8290}{\left(\frac{400}{8}\right)^2} = 3,316$

$\Rightarrow C_m = 1 - 0,3 \cdot \frac{0,0898}{3,316} = 0,992$



$\frac{M_A}{M_B} = -1 \Rightarrow C_b = 1,75 + 1,05(-1) + 0,3 \cdot 1^2 = 1$

$C_b = 1$

$\sigma_b = \frac{300 \cdot e\ tcm}{2140} = 0,14\,e\ tcm$

$\sigma_{B_1} = \frac{840 \cdot 1}{400 \cdot \frac{20}{10,17}} = 1,067 < 1,44$ o.k.

$\sigma_{B_2} = \left[\frac{2}{3} - \frac{2,4 \cdot (213,9)^2}{30.000 \cdot 1}\right] \cdot 2,4 = 1,3 < 1,4$ o.k. → Bigger

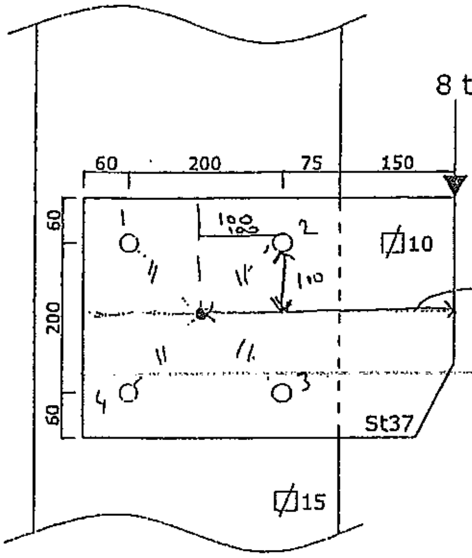
$\frac{0,089}{0,1827} + \frac{0,99 \cdot 0,14e}{\left(1 - \frac{0,089}{3,316}\right) \cdot 1,3} \leq 1,0$ $\boxed{e < 4,68\ cm}$ → safer.

$\Rightarrow \frac{0,0898}{0,6 \cdot 2,4} + \frac{0,14e}{1,3} < 1,0 \Rightarrow e < 8,706$

then $e_{max} < 4,68\,cm$

Name:….. …………Student No:. .Section:…………

3. (20 Points) Four identical turned bolts are to be used in the following bracket connection. Find the minimum diameter of each bolt based on TS648. Assume EY Loading.



St 37 turned bolts $\Rightarrow$ $\tau_{em} = 1,4$; $\sigma_{ex} = 2,8$; $\sigma_{çem} = 1,12$

$\rightarrow 150 + 75 + 100 = 325\,mm$

$T = 8t \cdot 32,5\,cm = 260\,t\cdot cm$

$J = \sum A r^2 \rightarrow \sum r^2 = 4 \cdot 10 \cdot \sqrt{2} = 799,99\,cm^2$
$\approx 800\,cm^2$

$1,4 = \frac{8}{4A} \Rightarrow \underline{1,428\,cm^2}$

$1 \cdot R \cdot 4$ $\quad \frac{8}{4R} = 2,8 \quad \frac{2}{R} = 2,8 \quad R = 1,4\,cm$ $\qquad A \frac{T \cdot x}{A \cdot r^2} \Rightarrow \frac{T \cdot x}{r^2} = N''$

2t per bolt

$\Rightarrow N_x' = 0 \qquad N_y'' = \frac{T \cdot x}{J} = \frac{260\,t\cdot cm \cdot 10\,cm}{800} = 3,25\,t$

$N_y' = 2t\,(-) \quad N_x'' = \frac{T \cdot y}{J} = N_y'' = 3,25\,t$

$\downarrow (-)$

$\Rightarrow N_x = N_x' + N_x'' = 0 \quad 4,596\,t$

$N_y = N_y' + N_y'' = -2 - 4,596 = -6,596\,t\,(\downarrow)$

$N = \sqrt{(3,25)^2 + (5,25)^2} \quad 6,039\,t$

	x	y	N_x'	N_y'	$N_x'' =$	N_y''	N_x	N_y	
1	−10	+10	0	−2t	+3,25	−3,25	3,25	−5,25	⇒ 6,17 t
2	+10	+10	0	−2t	+3,25	+3,25	3,25	1,25	
3	−10	−10	0	−2t	−3,25	−3,25	3,25	−5,25	⇒
4	+10	−10	0	−2t	−3,25	+3,25	3,25	1,25	

$6,17t = N$ for 1 & 3 bolts $\Rightarrow$

then $6.17\,t = A \cdot 1,4\,t/cm^2$

$A = \frac{6,17}{1,4} = \underline{4,407\,cm^2} \Rightarrow 4,407 = \pi \cdot \frac{R^2}{4} =$

$4,407 > 1,428$

$R = 2,36\,cm \cong 24\,mm\,\phi$

bolts are needed

5. (5 Points) Obtain σ_{cr} for λ=75, if the stress-strain curve is defined as follows:

	Stress (t/cm^2)	Strain
0	0	0
1	1.0	0.0004
2	2.0	0.0011
3	2.5	0.0017
4	3.0	0.0030
5	3.0	∞

Assuming σ_{cr} → between 1 & 0

$$\frac{\pi^2 \cdot (1-0)/(0{,}0004)}{75^2} = 4{,}38 \rightarrow \text{not ok} > 1{,}0$$

between 2 & 1

$$\frac{\pi^2 (2-1)/(0{,}0011-0{,}0004)}{75^2} = 2{,}50 \rightarrow \text{not ok} > 2{,}0$$

between 3 & 2

$$\frac{\pi^2 (2{,}5-2)/(0{,}0017-0{,}0011)}{75^2} = 1{,}46 < 2{,}0$$

then the σ_{cr} is (2,0) from table.

6. (4 points) List three of the advantages and disadvantages of welded connections compared to bolted connections.

Advantages

1. more Continious structures are made.
2. Applicable on site, no need pre produced materials
3. stronger connections, rigidity increase.

Disadvantages

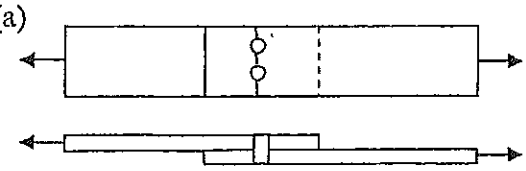
1. More rigidity.
2. special equipment is needed
3. Accidents may happen

7. (4 points) Indicate the circumstances under which you would strongly recommend bolted connections as opposed to welding.

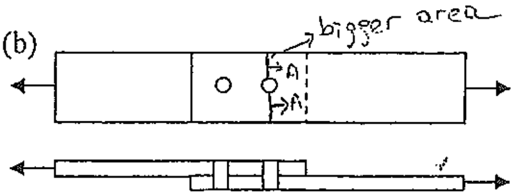
if the buckling, and elasticity of the structure is needed, bolted connections are more healty then welded connections,

8. (4 points) Given the following two alternative lap joints with the same capacity, which one would you prefer? Explain why?

(a)



(b)



b is preferable due to more tension areas on the planes.

Dr. Çetin Yılmaz
Dr. Uğur Polat

CE485
TERM TEST 2
PART 1 (Closed Book)
(Time allowed: 25 minutes for 25 points)

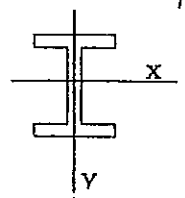
Dec 11, 2005

Name:...Student No:....................Section:...........

1. (5 Points) Obtain the critical load for the column below, if the material property is given as follows.

$\sigma = 35\varepsilon^{1/2}$ (t/cm²)

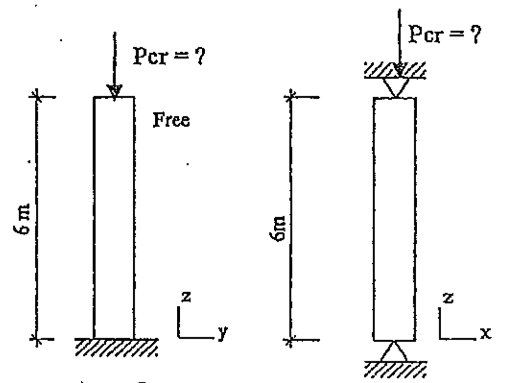
$\sigma = 35\sqrt{\varepsilon}$ 9.35



IPE300

Ix = 8360 cm⁴
Iy = 604 cm⁴
A = 53.8 cm²

ix = 12.5
iy = 3.35



k = 2
Lef = 12m
$\lambda_x = 96$
$\frac{1200}{12.5}$

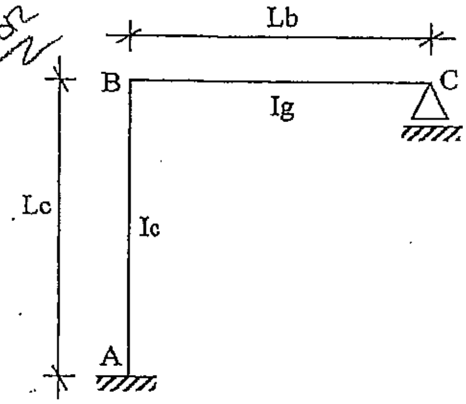
k = 1
Leff = 6m
$\lambda_y = 179$

$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2}$ $\lambda = \frac{L_{eff}}{i}$

$\sigma_{cr} = \frac{\pi^2 (2100)}{(179)^2} = 0.646$ t/cm²

$P_{cr} = 0.646 (53.8) = 34.8$ t

2.



a) (3 Points) What will be the effective length for the member AB for various values of Lc, Ic, Lb, Ig. Explain the results by a plot between $\frac{Lc/Ic}{Lb/Ig}$ and "k".

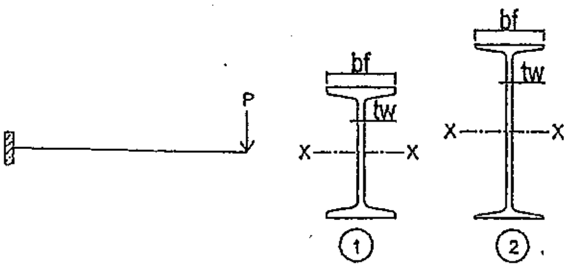
b) (3 Points) Do the same for member BC.

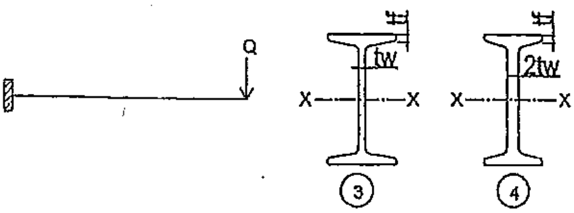
3. (4 Points) What is the reason for residual stresses in steel members? What kind of stresses will be produced in an I beam.

→ uneven cooling after hotrolling
→ cold bending or cambering during fabrication
→ punching of holes and cutting during fabrication
→ cooling after welding

4) a) (5 Points) There are two alternatives (sections 1 and 2) for a cantilever with a tip load of P. The sections 1 & 2 have the same inertia about the horizontal axis x-x. Plot on the same diagram the applied load P versus vertical tip deflection diagram for the sections. Explain if there is a difference in the ultimate load carried in each case

b) (5 Points) Discuss the same for the cantilever with a tip load of Q and sections 3 and 4.





Dr. Çetin Yılmaz
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CE485
TERM TEST 2
PART 1 (Closed Book)
(Time allowed: 25 minutes for 26 points)

Dec 19, 2004

Name:..Student No:....................Section:...........

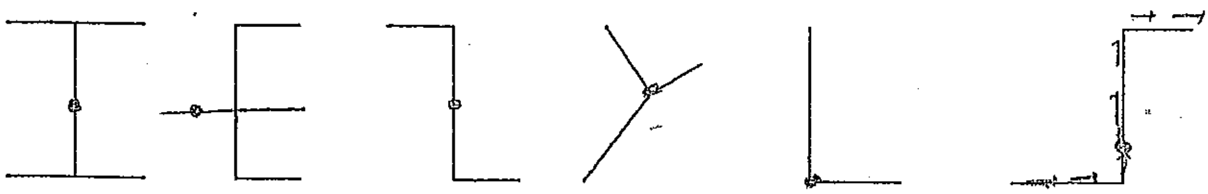
1. (3 Points) Clearly explain the reason/significance of terms A and B.

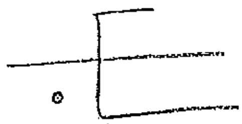
$$\frac{\sigma_{eb}}{\sigma_{bcm}} + \frac{\overset{A}{C_{mx}}\sigma_{bx}}{\underset{B}{\left(1-\frac{\sigma_{eb}}{\sigma'_{ex}}\right)}\sigma_{Bx}} + \frac{C_{mx}\sigma_{by}}{\left(1-\frac{\sigma_{eb}}{\sigma'_{ey}}\right)\sigma_{By}} \le 1.0$$

Cm : end moment / span moment / lateral support condition

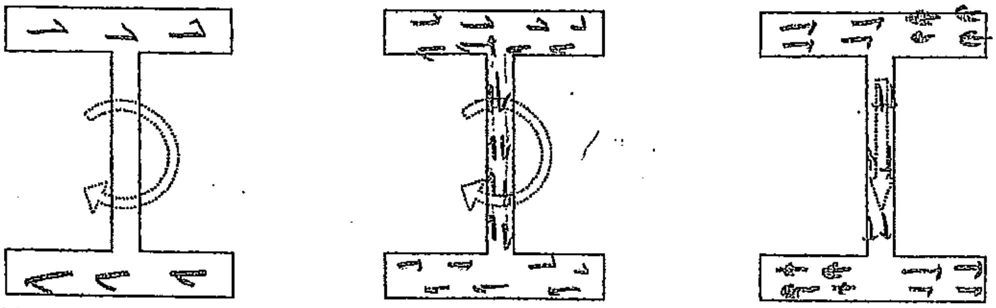
used for reducing slenderness ratio effect

2. (3 Points) Locate the shear center and draw the shear flow for vertical shear applied at the shear center.



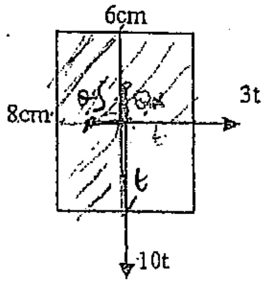


3. (3 Points) Show the distribution along the thickness and along the length.



Warping Shear Stress   Pure Torsional Shear Stress   Shear Flow

4. (3 points) Calculate the maximum absolute shear stress.



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

$I_x = \frac{1}{12} \cdot 6 \cdot 8^3 = 256\ cm^4$

$I_y = \frac{1}{12} \cdot 8 \cdot 6^3 = 144\ cm^4$

$\tau_x \quad \frac{3 \cdot 8(3)(1,5)}{144 \cdot (8)} = 0,094$

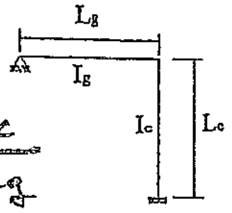
$\boxed{\tau_x = \frac{V_x \cdot Q_y}{I_y \cdot t}}$

$y \quad \frac{10 \cdot 6(4)(2)}{256(6)} = 0,3125$

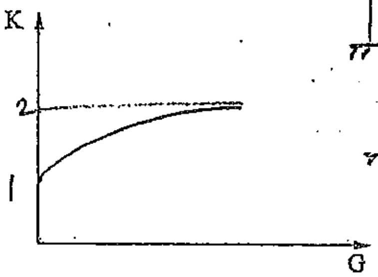
$\tau_y = \frac{V_y \cdot Q_x}{I_x \cdot t}$

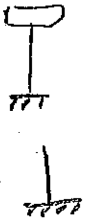
26

5. (3 Points) Give a sketch for effective length factor vs. G for the given column. (Hint: Modify permit stiffness of the beam.)



$\frac{I_c/L_c}{I_g/L_g}$

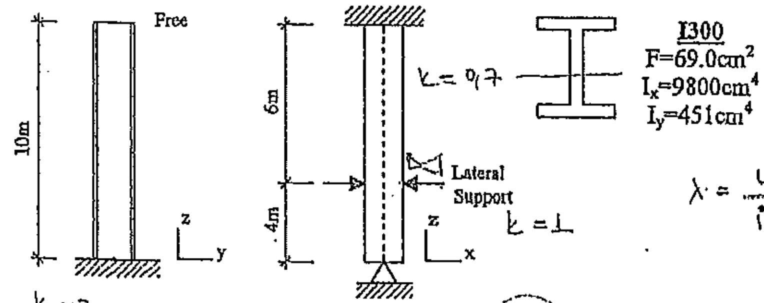




$I_g = \infty \quad k = 1$

$I_g = 0 \quad k = 2$

6. (3 Points) Determine the maximum slenderness ratio of the column shown.



I300
F=69.0cm^2
I_x=9800cm^4
I_y=451cm^4

$i_x = 11.9$
$i_y = 2.56$

$\lambda = \frac{L}{i}$

k = 2
l_{eff} = 20m

$\lambda_x = \frac{2000}{11.9} = 168$

$L_1 = 6 \times 0.7 = 4.3 / 2.56 = 168$
$L_2 = 1.4 = 4 / 2.56 = 156$

$\lambda_{max} = 168$

7. (5 Points) Obtain σ_{cr} for λ=75 if the stress-strain curve is defined as follows:

Stress (t/cm^2)	Strain
0	0
1.0	0.0004
2.0	0.0011
2.5	0.0017
3.0	0.0030
3.0	∞

$\sigma_{cr} = 2\ t/cm^2$

$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2}$

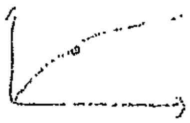
0-1 → 2500
σ_{cr} = 4.38 ✗

1-2 → E = 1428
σ_{cr} = 2.5 ✗

$\sigma = 2$

2-2.5 → E = 833
σ_{cr} = 1.46 ✗

2.5-3 → E = 384
σ_{cr} = 0.67 ✗



8. (3 Points) Write down the condition for a built-up member to be in group 1.

a) x-x axis is the neutral axis
b) all the main segments are the same rolled section
c) e is the same for all main segments if there are more than two sections.

Dec 19, 2004

Q1(20)	
Q2(25)	
Q3(25)	
Q4(6)	

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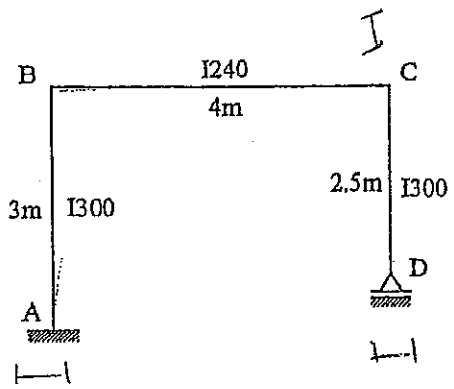
CE485
TERM TEST 2
PART 2 (Open Book)
(Time allowed: 110 minutes for 76 points)

Name:..Student No:....................Section:...........

1. (20 points) Calculate the slenderness ratio for members AB, BC and CD. Out-of-plane supports are provided at A, B, C and D. Members are oriented in their stronger axes.



permitted case

I240 $I_x = 4250$ $I_y = 221$
I300 $I_x = 9800$ $I_y = 451$

$$G_B = \frac{9800/300}{4250/400} = 3.07$$ $K_{AB} = 1,55$

$G_A = 1.$

$G_D = 10$

$$G_C = \frac{9800/250}{4250/400} = 3,688$$ $K_{CD} = 2,4$

$$\lambda_{AB} = \frac{1,55(300)}{11,9} = 39,07$$

$$\lambda_{CD} = \frac{2,4(250)}{11,9} = 50,42$$

Dr. Çetin Yılmaz
Dr. Can Balkaya

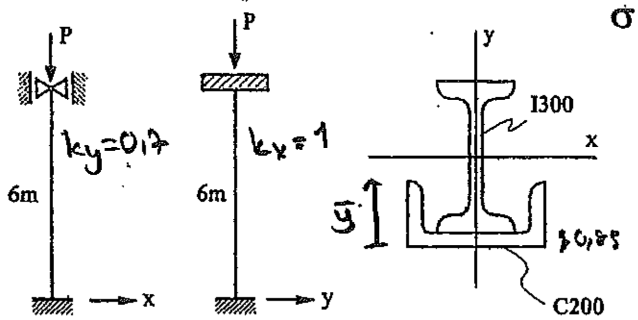
Q1(35)	35
Q2(30)	15
Q3(35)	15
TOTAL	65

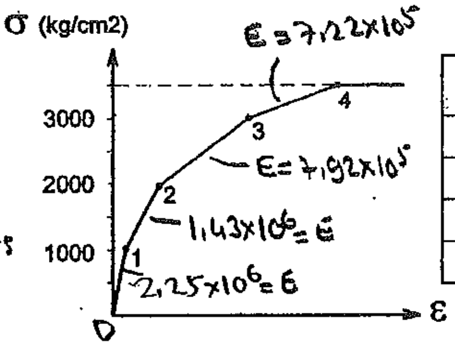
CE485
SUPPLEMENTARY EXAM
(Open Book)
(Time allowed: 120 minutes)

Name:....... ....Student No:.. ...Section: ....

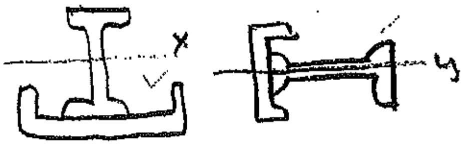
1. (35 Points) Calculate P_{all} if the stress-strain diagram of the steel used is as given in the sketch, using the factor of safety as F.S.=2.0

35





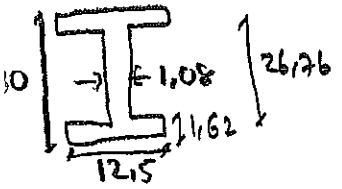
	Stress (kgf/cm²)	Strain
1	900	0.0004
2	1900	0.0011
3	2850	0.0023
4	3500	0.0032



$$\bar{y} = \frac{F_I \times 15{,}85 + F_C \times 2{,}01}{F_I + F_C} = \frac{69 \times 15{,}85 + 32{,}2 \times 2{,}01}{69 + 32{,}2}$$

$\bar{y}$: 11,44 cm from bottom

I 300:



$I_x = 9800\,cm^4$
$I_y = 451\,cm^4$
$i_x = 11,9\,cm$
$i_y = 2,56\,cm$
$F = 69\,cm^2$

$$I_{xx} = (I_x)_I + F_I(15{,}85 - 11{,}44)^2 + (I_y)_C + F_C(11{,}44 - 2{,}01 - e_y)^2$$

$$I_{xx} = 9800 + 69(15{,}85 - 11{,}44)^2 + (148) + 32{,}2(11{,}44 - 2{,}01)^2$$

$$I_{xx} = 14153{,}3\,cm^4$$

$F = 32,2\,cm^2$
$I_x = 1910\,cm^4$
$I_y = 148\,cm^4$
$i_x = 7,7\,cm$
$i_y = 2,14\,cm$

$$I_{yy} = (I_y)_I + (I_x)_C$$

$$I_{yy} = 451 + 1910 = 2361\,cm^4$$

$$i_x = \sqrt{\frac{I_{xx}}{F_{Total}}} = \sqrt{\frac{14153{,}3}{101{,}2}} = 11{,}83\,cm \qquad \lambda_x = \frac{k_x \cdot L}{i_x} = \frac{1 \times 600}{11{,}83} = 50{,}72$$

$$i_y = \sqrt{\frac{I_{yy}}{F_{Total}}} = \sqrt{\frac{2361}{101{,}2}} = 4{,}83\,cm \qquad \lambda_y = \frac{k_y \cdot L}{i_y} = \frac{0{,}7 \times 600}{4{,}83} = 86{,}96$$

$\lambda_y > \lambda_x$ so $\lambda_{max} = \lambda_y = 86{,}96$

If σ is between 0-1 points:

$$\sigma = \frac{\pi^2 \times 2{,}25 \times 10^6}{86{,}96^2} = 2936\,kgf/cm^2 \quad X$$

If σ is between 1-2 points

$$\sigma = \frac{\pi^2 \times 1.43 \times 10^6}{86.96^2} = 1866.3 \ kgf/cm^2$$ (between 1000 - 2000 kgf/cm²)

is ok

$$\sigma_{all} = \frac{\sigma}{F.S} = \frac{1866.3}{2} = 933.18 \ kgf/cm^2$$

$$P_{all} = \sigma_{all} \times F_{Total} = 933.18 \times 101.2 = \underline{94.438 \ t}$$

Name:.... ....Student No:. ...Section: ....

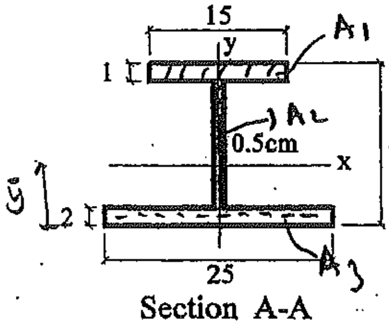
2. (30 Points) Calculate

(15)

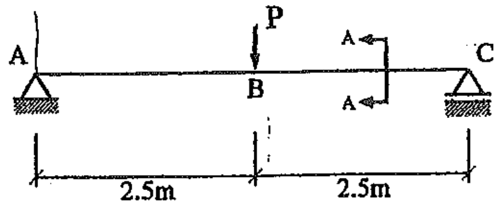
a) The yield and the ultimate (plastic) moment capacities of the given section.

b) P_{ult} if the beam is laterally supported throughout its length.

The beam is made of St52 steel.



Section A-A



$F_T = 1\times15 + 0{,}5\times(30-2-1) + 2\times25 = 78{,}5\,cm^2$

$$\bar{y} = \frac{(15\times1)\times(30-0{,}5) + 0{,}5\times27\times(13{,}5+2) + (2\times25)\times(1)}{78{,}5}$$

$\bar{y} = 8{,}94\,cm$ from bottom

$$I_{xx} = \frac{1}{12}25\,2^3 + 2\times25\times(8{,}94-1)^2 + \frac{1}{12}0{,}5\times27^3 + 0{,}5\times27\times(15{,}5-8{,}94)^2 +$$

$$\frac{1}{12}15\times1^3 + 1\times15\times(29{,}5-8{,}94)^2$$

$I_{xx} = 10911{,}87\,cm^4$

$M_{max} = 1{,}25P$

a) $\sigma_y = \frac{M.c}{I} = 3{,}6 = \frac{M_x(30-8{,}94)}{10911{,}87} \Rightarrow M_y = 1865{,}27\,t.cm$

$F_{compression} = F_{tension} = \frac{F_T}{2}$

$2\times25\times(\bar{y}-2) + (\bar{y}-2)\times0{,}5\times\frac{(\bar{y}-2)}{2} = \frac{78{,}5}{2}$

$50\bar{y} - 100 + (\bar{y}^2 - 4\bar{y} + 4)\times0{,}25 = 39{,}25 \Rightarrow$

$0{,}25\bar{y}^2 - \bar{y} + 1 + 50\bar{y} - 100 = 39{,}25$

$0{,}25\bar{y}^2 + 49\bar{y} + 138{,}25$

$\bar{y}^2 + 196\bar{y} + 553 = 0$

$\bar{y} = 2{,}78$ from bottom

$M_{pl} = 1\times15\times(30-2{,}78) +$
$26{,}22\times0{,}5\times13{,}11 + 2\times25\times0{,}78$
$-0{,}78\times0{,}5\times0{,}39$

$M_{pl} = 541{,}02\,t\,cm$

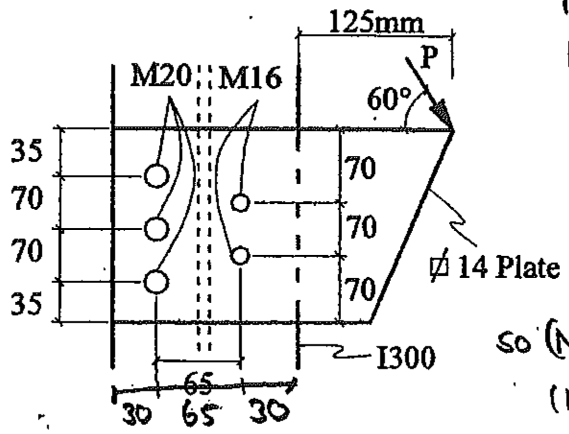
b) $M_{ult} = M_{pl} = 541{,}02 = M_{max} = 1{,}25P$

$P = 432{,}81\,t$

Name:..... ....Student No: _ ...Section:...........

3. (35 Points) In the connection shown, three M20 and two M16 turned bolts are used (placed symmetrically about the web of I300) to connect a 14mm thick plate to the flange of an I300 section, both of which are made of St37. Calculate the allowable load P for EY loading condition. (Consider only bolt failure).

15



$N_x = P.\cos 60 = P/2 \rightarrow$ $\qquad A_{20} = \frac{\pi \times 2^2}{4} = 3{,}14\ cm^2$

$N_y = P.\sin 60 = \frac{P\sqrt{3}}{2} \downarrow$ $\qquad A_{16} = \frac{\pi \times 1{,}6^2}{4} = 2{,}01\ cm^2$

Total Bolt area $= B_T = 3 \times A_{20} + 2 \times A_{16} = 13{,}44\ cm^2$

Coefficents for the distribution of N_x and N_y

$k_{A_{20}} = \frac{A_{20}}{B_T} = 2{,}34$ $\qquad k_{A_{16}} = \frac{A_{16}}{B_T} = 0{,}15$

so $(N'_x)_{20} = k_{A20} \times N_x = 2{,}34 \times \frac{P}{2} = 1{,}17P \rightarrow$

$(N'_x)_{16} = k_{A16} \times N_x = 0{,}15 \times \frac{P}{2} = 0{,}075P \rightarrow$

$(N'_y)_{20} = k_{A20} \times N_y = 2{,}34 \times \frac{P\sqrt{3}}{2} = 2{,}03P \downarrow$

$(N'_y)_{16} = k_{A16} \times N_y = 0{,}15 \times \frac{P\sqrt{3}}{2} = 0{,}13P \downarrow$

$\bar{x} = \frac{3 \times A_{20} \times 3 + 2 \times A_{16} \times 9{,}5}{B_T}$

$\bar{x} = \frac{3 \times 3{,}14 \times 3 + 2 \times 2{,}01 \times 9{,}5}{13{,}44}$ ✓

$\bar{x} = 4{,}94\ cm = 49{,}4mm$ from leftside

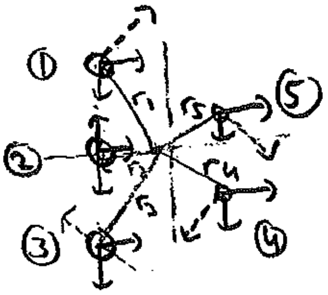
$\bar{y} = \frac{1 \times A_{20} \times [3{,}5 + 10{,}5 + 17{,}5] + 1 \times A_{16} \times [7 + 14]}{B_T}$

$\bar{y} = 10{,}5cm = 105mm$ from bottom ✓

$\bar{J} = \Sigma F_i r_i^2 = A_{20} \times 1 \times (r_1^2 + r_3^2) + A_{20} \times (r_2^2) + A_{16} \times 1 \times (r_4^2 + r_5^2)$

$\bar{J} = 3{,}14 \times 1 \times (7{,}26^2 + 7{,}26^2) + 3{,}14 \times (1{,}94^2) + 2{,}01 \times 1 \times (3{,}83^2 + 3{,}83^2)$

$\bar{J} = 401{,}79\ cm^4$ 417,6



bolt ⑤ and ① are most critical ones ✓

$(N''_x)_{20} = \frac{P/2 \times 3{,}14}{401{,}79} \times 7{,}26 = 0{,}028P$ for ① and ③

$(N''_x)_{20} = \frac{P/2 \times 3{,}14}{401{,}79} \times 1{,}94 = 0{,}007P$ for ②

$(N''_x)_{16} = \frac{P/2 \times 2{,}01}{401{,}79} \times 3{,}83 = 0{,}009P$ for ④ and ⑤

$(N''_y)_{20} = \frac{P\sqrt{3}/2 \times 3{,}14}{401{,}79} \times 7{,}26 = 0{,}049P$ for ① and ③

$(N''_y)_{20} = \frac{P\sqrt{3}/2 \times 3{,}14}{401{,}79} \times 1{,}94 = 0{,}013P$ for ②

$(N''_y)_{16} = \frac{P\sqrt{3}/2 \times 2{,}01}{401{,}79} \times 3{,}83 = 0{,}016P$ for ④ and ⑤

For, bolt 5

$N_x = N'_x + N''_x = 0{,}075P + 0{,}009P = 0{,}084P$

$N_y = N'_y + N''_y = 0{,}13P + 0{,}016P = 0{,}146P$

$N_5 = \sqrt{N_x^2 + N_y^2} = 0{,}168P$

For bolt 1

$N_x = N'_x + N''_x = 1{,}17P + 0{,}028P = 1{,}198P$

$N_y = N'_y - N''_y = 2{,}03P - 0{,}049P = 1{,}981P$

$N_1 = \sqrt{N_x^2 + N_y^2} = 2{,}315P$

so bolt 1 is most critical

$N_{em} = \frac{\pi \times 2^2}{4} \times \tau_{em} = 3{,}14 \times 1{,}4 = 4{,}396t$

$N_{ez} = d \times t_{min} \times \sigma_{ez} = 2 \times 1{,}4 \times 2{,}8 = 7{,}84t$

So N_{em} is more critical

$2{,}315P = 4{,}396 \Rightarrow P \cong 1{,}9t$

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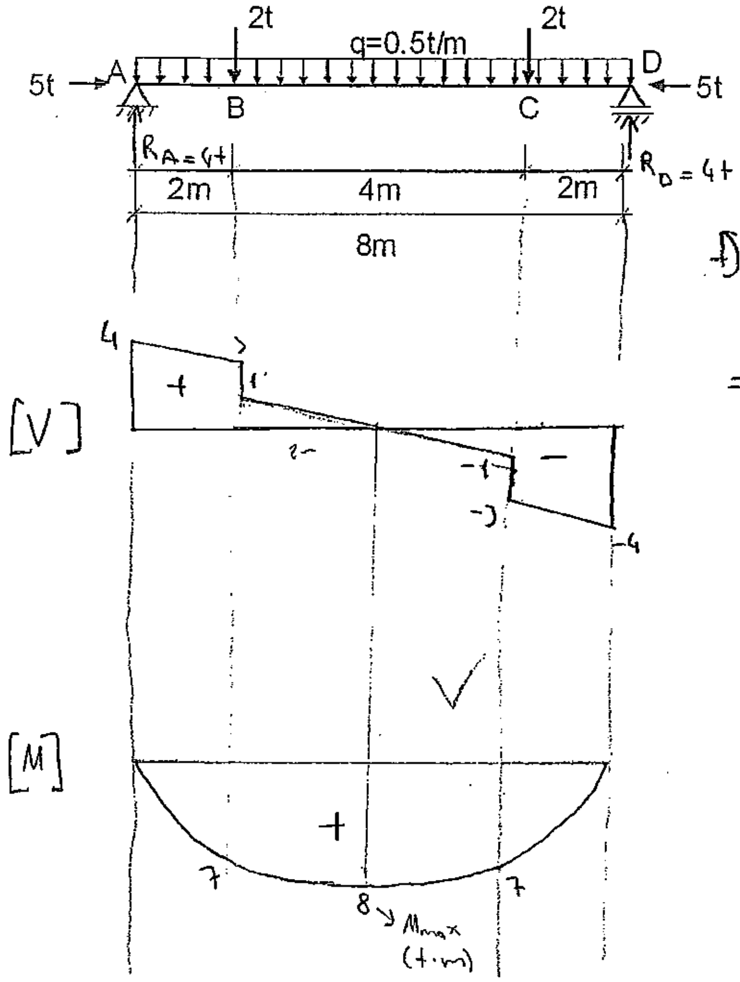
Dr. Can Balkaya

Q1(40)	
Q2(30)	
Q3(30)	
Total	

CE485
EXTRA EXAMINATION FOR GRADUATION
(Open Book)
(Time allowed: 120 minutes)

Name:. ....Student No: ....Section:. ......

1. (40 Points) Design an economical I-section beam shown. Use St37, EY loading, TS648 specifications. Lateral supports exist only at A and D.



$+\uparrow \Sigma F_y = 0 \Rightarrow$

$R_A + R_D - 2 - 2 - 0,5 \cdot 8 = 0$

$\Rightarrow R_A + R_D = 8t$ ✓

$\circlearrowright \Sigma M_A = 0 \Rightarrow$

$R_D \cdot 8 - 2 \cdot 2 - 2 \cdot 6 - 0,5 \cdot 8 \cdot 4 = 0$

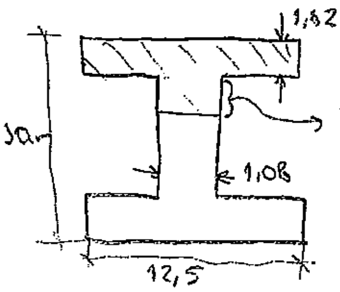
$\Rightarrow R_D = 4t \quad \Rightarrow R_A = 4t$

$\sigma_{bem} < \frac{M_{max}}{W}$

$1,44 < \frac{800}{W} \Rightarrow W < \frac{800}{1,44}$

$\Rightarrow W_{min} = 555,56 \text{ cm}^3$

TRY I 300



$\frac{30 - 2 \cdot 1,62}{6} = 4,46\text{cm}$

$F_c = 12,5 \cdot 1,62 + 1,08 \cdot 4,46 = 25,1 \text{ cm}^2$

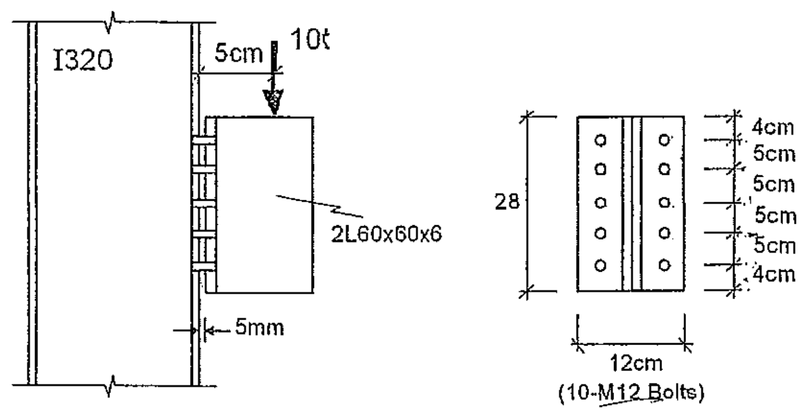
$I_y = \frac{1}{12} \cdot 4,46 \cdot 1,08^3 + \frac{1}{12} \cdot 1,62 \cdot 12,5^3 = 264,14 \text{ cm}^4$

$i_y = \sqrt{\frac{264,14}{25,1}} = 3,24 \text{ cm}$

Name:. ....Student No: .....Section:...........

2. (30 Points) Check whether the bolted connection shown is satisfactory or not. Steel type is St37 and bolts are 4D turned bolts. Use EY loading and TS648 specifications.

30



0

shear check:

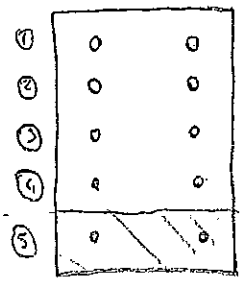
$n\cdot F_1\cdot\tau = P$, $F_1 = \pi\cdot 6^2 = 1,13\,cm^2$

$10\cdot 1,13\cdot\tau = 10t \Rightarrow \tau = 0,885\,t/cm^2 < \tau_{em} = 1,4\,t/cm^2 \Rightarrow$ OK!

combined shear & tension check:

$M = 10\cdot 5 = 50\,t\cdot cm$

assume NA is between 4 & 5:



$12\cdot y\cdot\frac{y}{2} = 2\cdot 1,13(9-y)+2\cdot 1,13(14-y)+2\cdot 1,13(19-y)+2\cdot 1,13(24-y)$

$6y^2 = 149,16 - 9,04y$

$\Rightarrow y = 4,3\,cm$ it is between rows 4 & 5 $\Rightarrow$ OK!

$c_t = 24 - 4,3 = 19,7\,cm$

$$I = \frac{1}{3}\cdot 12\cdot 4,3^3 + 2\cdot 1,13\underbrace{(9-4,3)}_{4,7}^2 + 2\cdot 1,13\underbrace{(14-4,3)}_{9,7}^2 + 2\cdot 1,13\underbrace{(19-4,3)}_{14,7}^2 + 2\cdot 1,13\underbrace{(24-4,3)}_{19,7}^2 = 1969,5\,cm^4$$

maximum tension is at row 1:

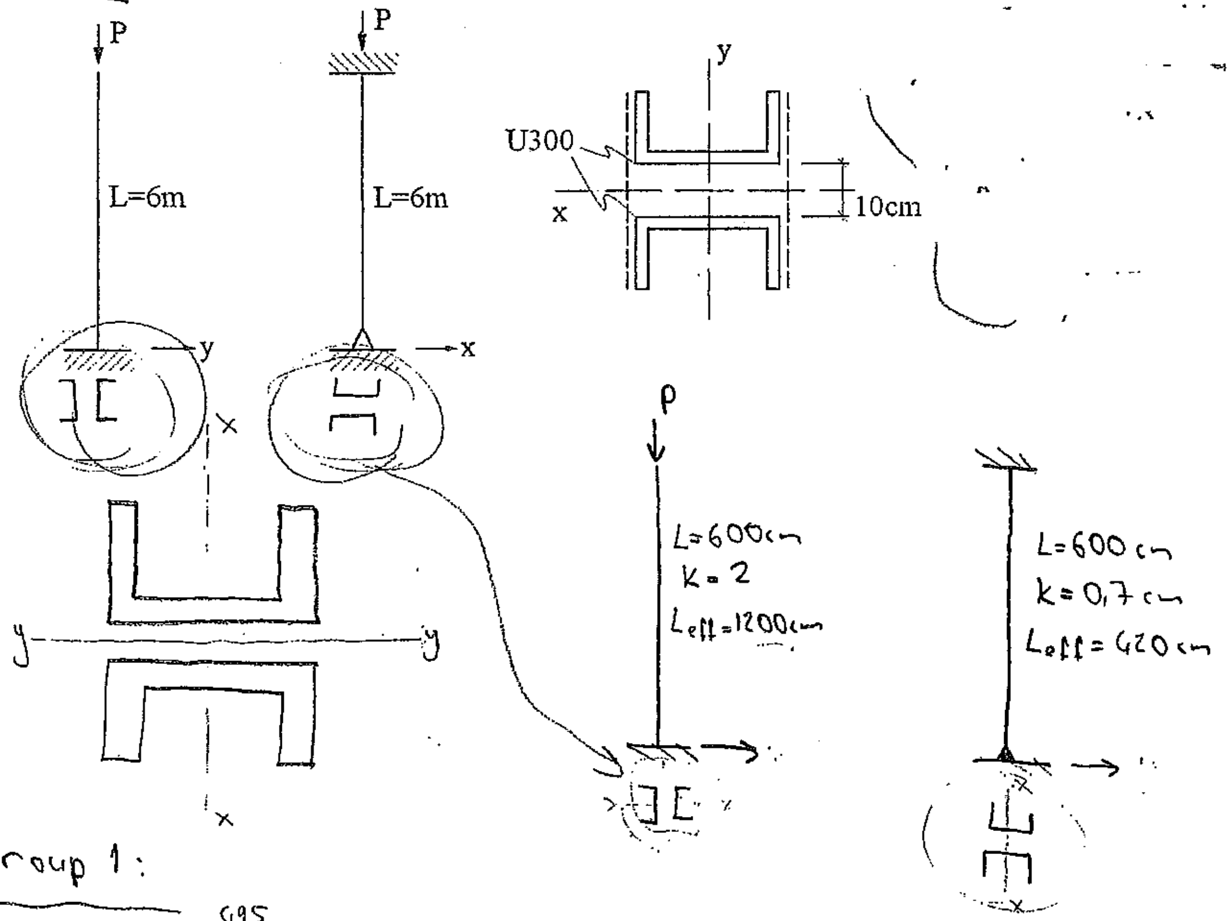
$\sigma = \frac{50\cdot 19,7}{1969,5} = 0,5 < \sigma_{em} = 1,12 \Rightarrow$ OK! safe

interaction between shear & tension:

$\left(\frac{0,5}{1,12}\right)^2 + \left(\frac{0,885}{1,4}\right)^2 = 0,6 < 1 \Rightarrow$ OK! safe

Name:. ....Student No:. ...Section:....... ....

3. (30 points) For the given built-up column made of 2-U300 channel sections,
a) Calculate the allowable axial compressive load if batten spacing is 80cm.
b) Design the batten plates
St52, EY loading.



group 1:

$$I_{yy} = 2\cdot \overset{495}{I_y} + 2\cdot 58{,}8(2{,}7+5)^2 = 7962{,}5$$

$$\Rightarrow i_y = \sqrt{\frac{7962{,}5}{2\cdot 58{,}8}} = 8{,}23\,cm$$

$$\lambda_y = \frac{1200}{8{,}23} = 145{,}8 \;,\; \lambda_1 = \frac{80}{2{,}9} = 27{,}6 \Rightarrow \lambda_{yi} = \sqrt{145{,}8^2 + \frac{2}{2}\cdot 27{,}6^2} = 148{,}4$$

$$\lambda_x = \frac{420}{11{,}7} = 36 \qquad \lambda_{max} = 148{,}4$$

$$\Rightarrow \sigma_{bem} = 0{,}3754\ t/cm^2$$

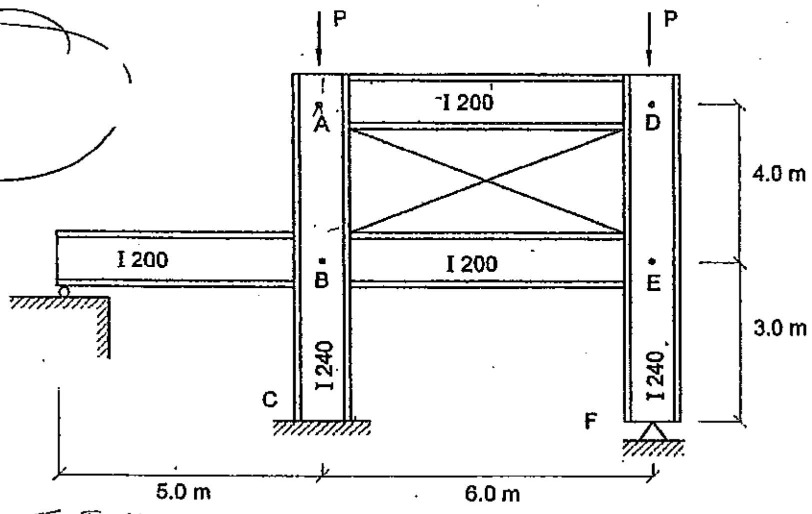
$$\Rightarrow P_{all} = 0{,}3754\cdot 2\cdot 58{,}8 = 44{,}14\ t$$

Dr. Çetin Yilmaz
Dr. Can Balkaya

CE 485
SUPPLEMENTARY EXAM
(Time allowed: 120 minutes)
USE ST37 STEEL AND EY LOADING IN ALL PROBLEMS

Name:. ……………….Student No:… ….Section:………..

1. (30 Pts) Find the allowable load, P, that might be applied to the frame made of St37 I-profiles. Consider only in-plane stresses and stability.



I200 $I_x = 2140 \text{ cm}^4$ I240 $I_x = 4250 \text{ cm}^4$

$G = \dfrac{\Sigma I_c/L_c}{\Sigma I_g/L_g}$

k values from nomogram on TS648

A-B (sway prevent)

$G_A = \dfrac{4250/4}{2140/6} = 2{,}98$

$G_B = \dfrac{4250/4 + 4250/3}{2140/6 + 1{,}5 \times 2140/5} = \dfrac{2479{,}17}{998{,}67} = 2{,}48$

k = 0,88 $L_{eff} = 0{,}88 \times 4 = 3{,}52 \text{ m}$

B-C (sway permit)

$G_B = \dfrac{4250/4 + 4250/3}{2140/6 + 0{,}5 \times \frac{2140}{5}} = \dfrac{2479{,}17}{560{,}67} = 4{,}34$

k = 1,65 $L_{eff} = 4{,}95 \text{ m}$

$G_C = 1$

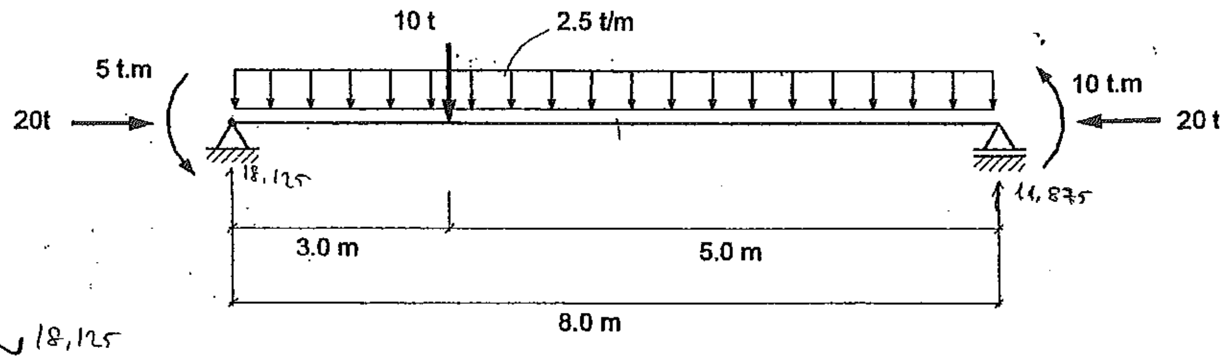
Name:........................ .. ..Student No:... ..... ,Section:...........

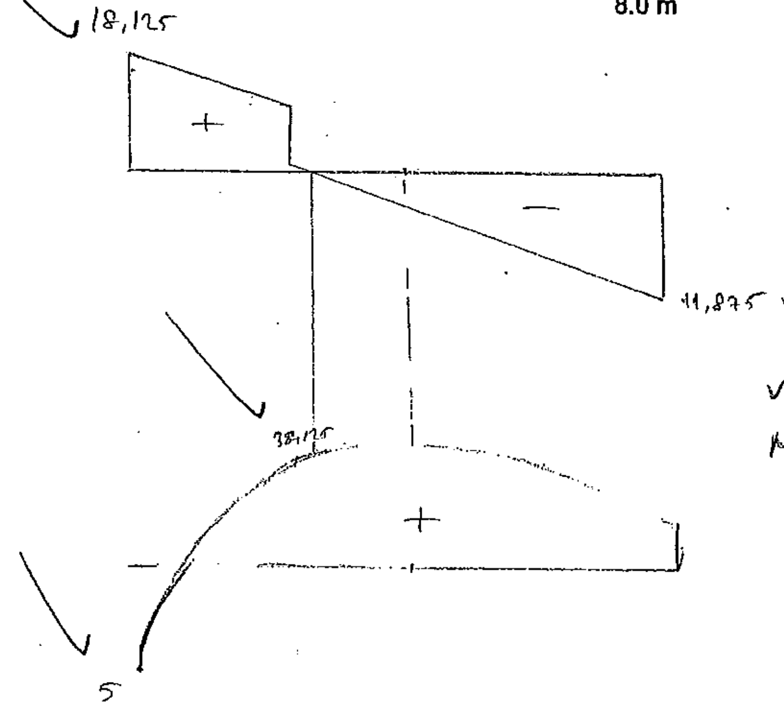
2. (40 Points) Design the beam-column using an IPBv section. Lateral supports exist only at the end Check midspan deflection:

$$\delta \cong \frac{5}{48} \cdot \frac{L^2}{EI} \cdot \left[M_S - 0.1 \cdot (M_1 + M_2) \right]$$

Where M_s is the midspan moment, M_1 and M_2 are the end moments. The sign of M_1 or M_2 is positive if it is acting in the same direction as the span moment.

L/300





$V_{max} \cong 18,125$ t

$M_{max} \cong 38,2$ t.m

$L_{eff} = 8$ m

$\sigma_{cr} = \frac{M}{W} + \frac{P}{F} = \frac{3820}{W} + \frac{20}{F} < 1,44$

TRY IPBV - 300 ;

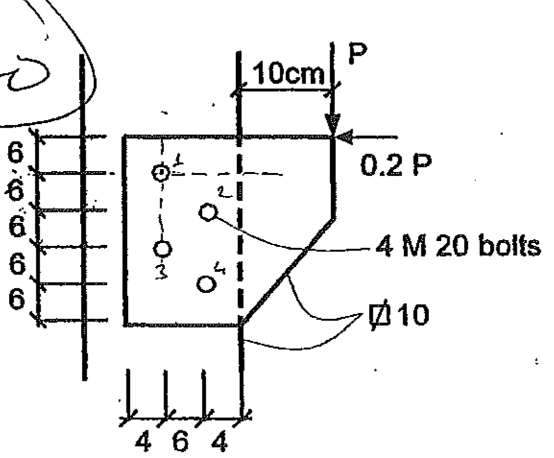
$\frac{3820}{3480} + \frac{20}{303} \leq 1,44$ ✓

??

Name:……………………………………………………Student No:……….. ..Section:………..

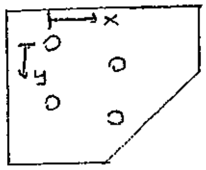
20

3. (30 Points) Find P_{all} for the bracket connection. Dimensions are in cm.



$$\bar{x} = \frac{0+6+0+6}{4} = 3\,cm$$

$$\bar{y} = \frac{0+6+12+18}{4} = 9\,cm$$



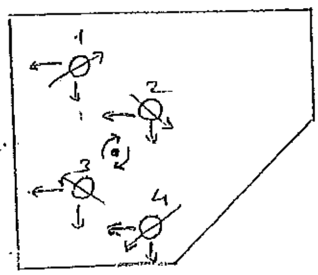
$$T = P\times(10+7) - 0.2P\times 15 = \underline{14P}$$

$$\frac{P}{4} = 0.25P \qquad \frac{0.2P}{4} = 0.05P$$

$N'_{1x} = 0.05P\ (\leftarrow)$ $N'_{2x} = 0.05P\ (\leftarrow)$ $N'_{3x} = 0.05P\ (\leftarrow)$ $N'_{4x} = 0.05P\ (\leftarrow)$

$N'_{1y} = 0.25P\ (\downarrow)$ $N'_{2y} = 0.25P\ (\downarrow)$ $N'_{3y} = 0.25P\ (\downarrow)$ $N'_{4y} = 0.25P\ (\downarrow)$

Shear stresses;



most critical connection is # 4.

- Polar Moment of inertia;

$$J = 3^2+9^2+3^2+3^2+3^2+3^2+3^2+9^2 = \underline{216\ cm^4}$$

$N''_{1x} = \frac{T\cdot y}{J}$ → (distance from centroid to bolt) $\Longrightarrow$ $N''_{4x} = \frac{14.P\times 9}{216} = \underline{0.583P}$

$N''_{1y} = \frac{T\cdot x}{J}$

$N''_{4y} = \frac{14P\times 3}{216} = \underline{0.194P}$

Total shear force; $N''_{4x} + N'_{4x} = \underline{0.633P}$

$N''_{4y} + N'_{4y} = \underline{0.444P}$

$$N_T = \sqrt{(0.633P)^2 + (0.444P)^2} = \underline{0.773P}$$

$$\pi r^2 = \underline{3.14\ cm^2}$$

$\tau_{em} = 1.4\ t/cm^2$ $0.773P = 1.4\times 3.14 \Longrightarrow P = 5.68\ t$

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shape factor $= \boxed{\dfrac{M_p}{M_y}}$

Jan 04, 2006

CE485
FINAL
PART 1 (Closed Book)
(Duration: 45 minutes; 30 points)

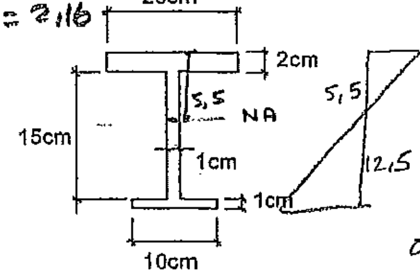
plastic
For rectangular section
$\boxed{M_p = 1{,}5\, M_{yield}}$

(Do not ask any question, for any missing information make a reasonable assumption)

Name:..Student No:.......................Section:............

1. (6 points) Obtain yield (My) $\frac{Mc}{I}$ and plastic moment capacity (Mp) for the section shown in the sketch for (σy=3.6 t/cm²)

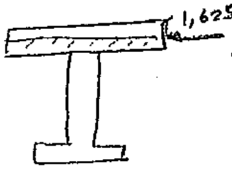
$\sigma_{all} = 2{,}16$



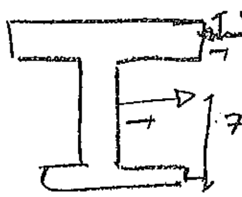
$\bar{y} = \dfrac{20(2)(1) + (15)(1)(9{,}5) + (1)(10)(17{,}5)}{65} = 5{,}5$

$I_x = \frac{1}{12} \cdot 1 \cdot 15^3 + (15)(1)(4)^2 + \frac{1}{12} \cdot 20 \cdot 2^3 + 40(4{,}5)^2$
$+ \frac{1}{12} \cdot 10 \cdot 1^3 + 10(12)^2 \Rightarrow \boxed{2785{,}42 \text{ cm}^4}$

$\sigma = \dfrac{Mc}{I}$ $M = \dfrac{3{,}6(2785{,}42)}{12{,}5} = 802{,}2$ tcm.



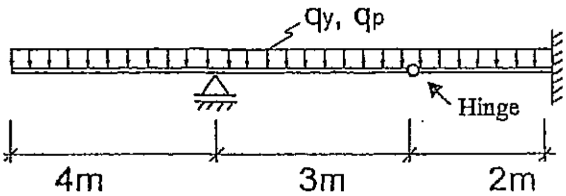
$20y = 20(2-y) + 25$
$(y = 1{,}625)$
$F = (32{,}5)(3{,}6) = 117$



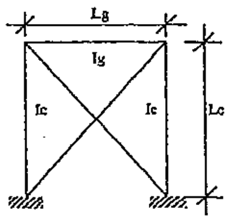
$M_p = 117(18 - 0{,}8125 - 7{,}8125)$
$M_p = 1096{,}875$ tcm

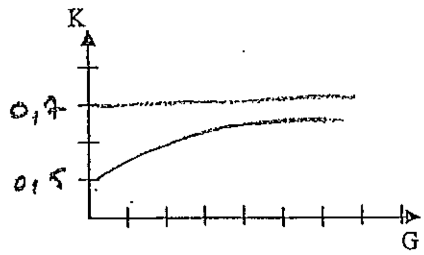
2. (6 Points) If the section above is utilized in the following (assuming it is fully laterally supported), what would be the yield (q_y) and ultimate (q_p) load for the beam.

6



3. (3 points) Give a sketch for effective length factor vs. G for the given columns, if $G = \dfrac{Ic/Lc}{Ig/Lg}$





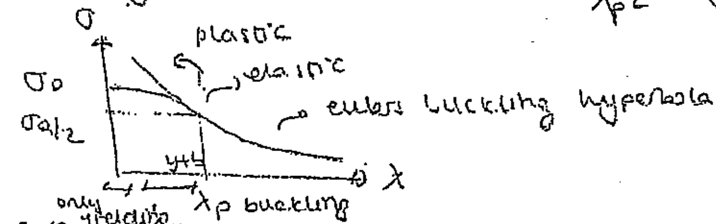
braced

$I_g = \infty$ $k = 0{,}5$

$I_g = 0$ $k = 0{,}7$

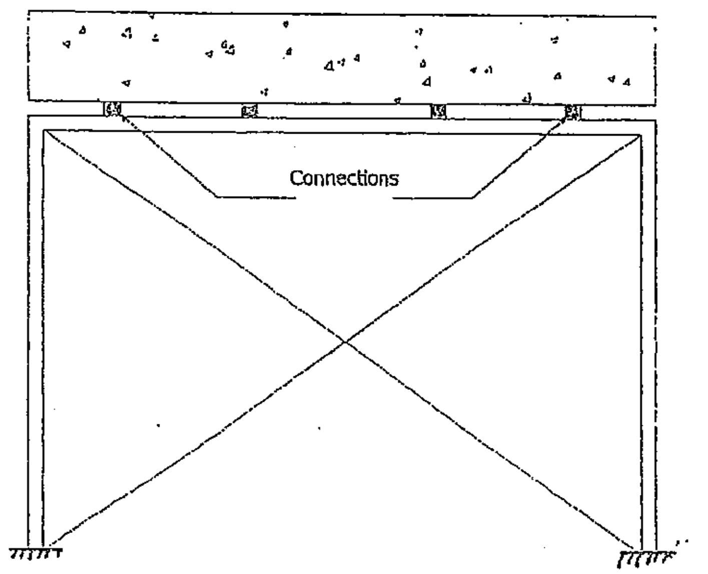
4. (4 points) Explain the critical slenderness ratio λ_p as stated in TS648. What will be its value for a steel with yield stress $\sigma y = 3.5$ t/cm²? What would you say for the type ~~of buckling~~ of $\lambda < \lambda_p$

$\sigma_y = 3,5$ t/cm² $\Rightarrow$ $\sigma_a/_2 = \dfrac{\pi^2 E}{\lambda_p^2}$ $\quad \lambda_p = 131,4$



5. (3 points) Indicate the following values:
   a. Poisson's ratio for St52
   b. Ultimate strain for St37
   c. Ultimate strength for St70

6. (8 points) A heavy platform is supported by a braced steel portal frame as shown. The frame has a negligible mass compared to that of the platform. The platform is attached to the girder by 4 identical connections. The structure is located in a seismic region and the total base shear (EQ load) is calculated as 20 tons based on a response modification factor of R=4. What would you take the design load for each connection? Please explain why?



37

Jan 04, 2006

Dr. Çetin Yılmaz
Dr. Uğur Polat

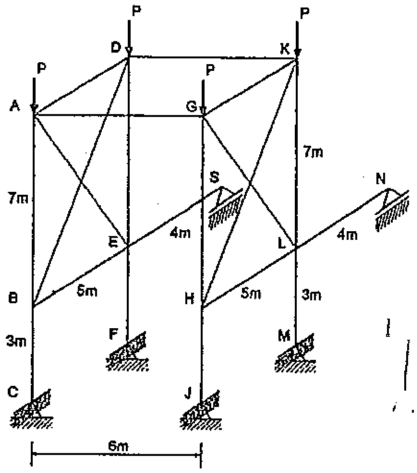
CE485
FINAL
PART 2 (Open Book)
(Duration: 120 minutes; 80 points)

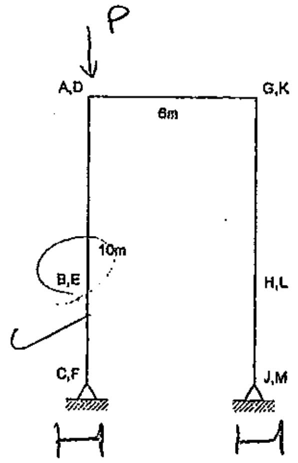
Q1(20)	
Q2(20)	
Q3(20)	
Q4(20)	
Total	

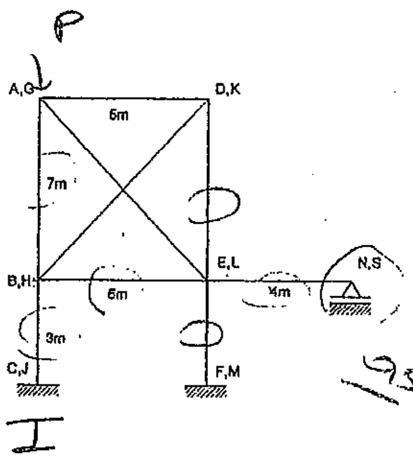
(Do not ask any question, for any missing information make a reasonable assumption)

Name:...Student No:...................Section:...........

(20 Pts.) 1. In the frame shown below, all columns are I300 and all beams are I240, made of St52. Utilizing TS648 specifications and EY loading conditions, calculate the allowable load, P. Remember that you should orient the columns in the most efficient way.







I_x

$k_x = 2$

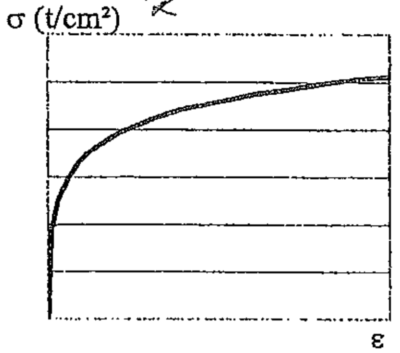
I_y

Name:..Student No:..........................Section:............

(20 Pts) 2. Calculate the allowable stress for a column having the maximum slenderness ratio of λ=80 and factor of safety of 2.0. The stress-strain relationship of steel is given related to the natural logarithm of strain as follows:

$\sigma = \ln(80 \cdot \varepsilon^{0.6} + 1)$

$\frac{1}{x} \cdot 80, 0{,}6 \cdot \varepsilon^{-0{,}4}$



$\frac{d\sigma}{d\varepsilon} = E$ $\quad E = \frac{80 \cdot 0{,}6 \cdot \varepsilon^{-0{,}4}}{80\,\varepsilon^{0{,}6}+1}$

$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2}$

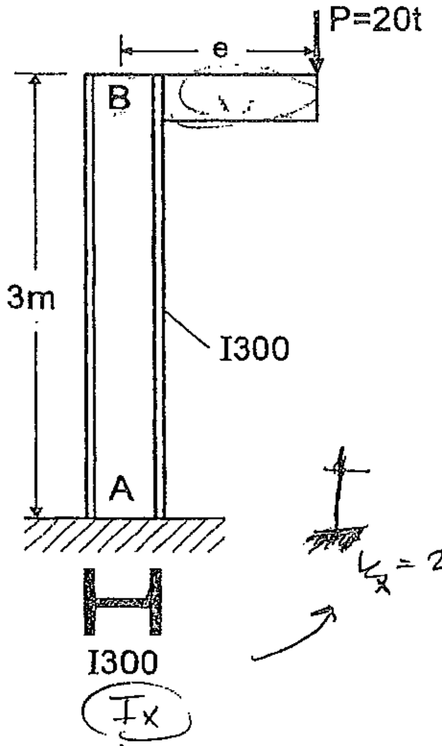
$\sigma_{all} = \frac{\sigma_{cr}}{2}$

σ	ε
0,07	
0	0

E =

Name:..Student No:....................Section:...........

(20 Pts) 3. The steel column AB is fixed at the base (A) and restrained laterally at top (B). It is subjected to a point load of P = 20.0 tons at the top as shown. Find the maximum value of "e" for which the column will be able to carry the given load safely based on TS648. Use St37 and EY Loading.



20t

20e

$c_m = 0,6$

$\sigma_{eb} = 20 / A$

$\sigma_{bem} =$

$\sigma_b = 1$

M_c

$e =$

$k_x = 2$

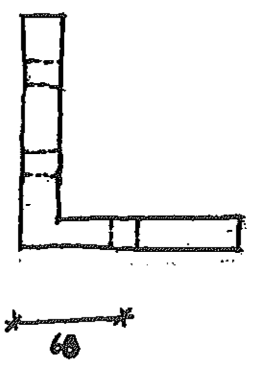
I_x

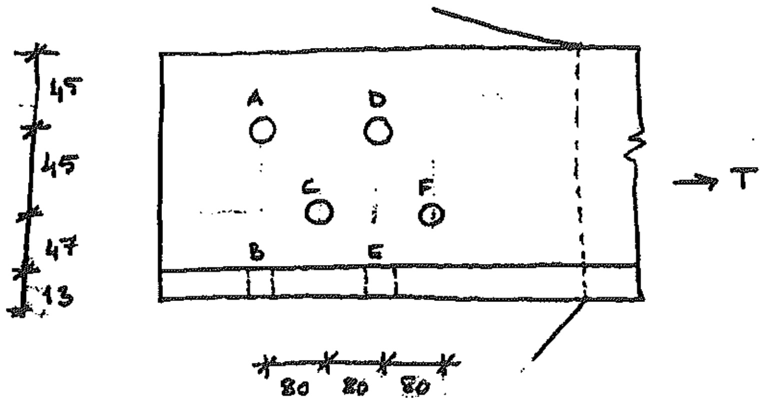
$k_y = 0,7$

I_y

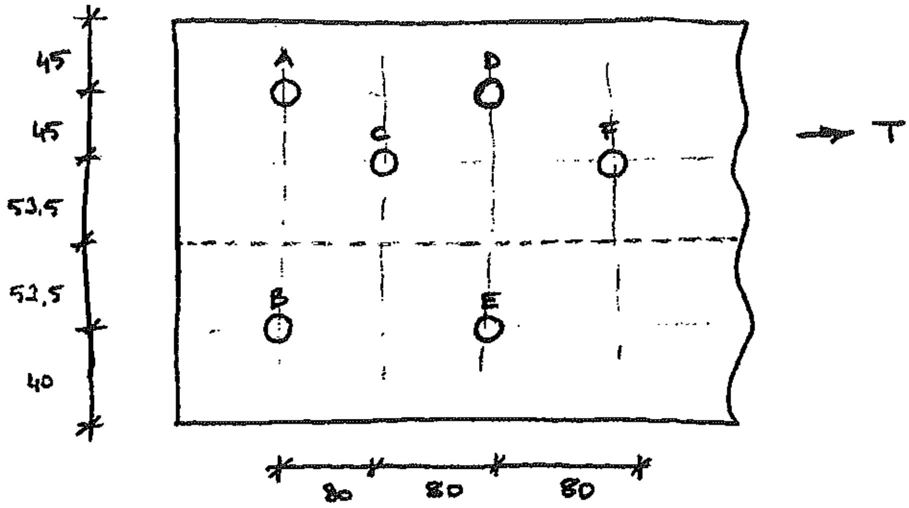
388 Rcc

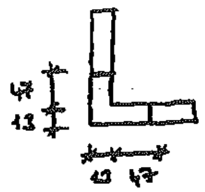
L 150 x 100 x 13





M19 bolts are used. St37 steel. T_{all} = ?





$47 + 13 + 47 = 107$ mm

$107/2 = 53{,}5$ mm

$A_{gross} = 150 \times 13 + 87 \times 13 = 3081 \text{ mm}^2 \rightarrow 0{,}85 A_{gross} = 2618{,}9 \text{ mm}^2$

$A_{AB} = 3081 - 2 \times 20 \times 13 = 2561 \text{ mm}^2$

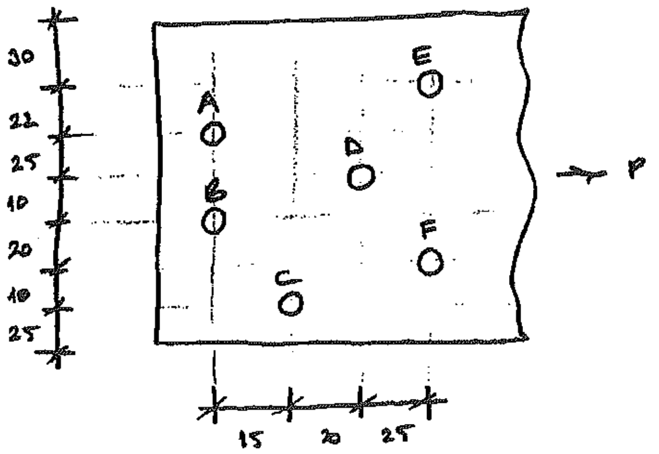
$A_{ACB} = 3081 - 3 \times 20 \times 13 + \frac{80^2}{4 \times 45} \times 13 + \frac{80^2}{4 \times 107} \times 13 = 2957.6 \text{ mm}^2$

Note: $A_{ACB} = A_{ACE} = A_{DCB} = A_{DCE} = A_{DFE}$

$A_n = A_{AB} = 25.61 \text{ cm}^2$

$T_{all} = 25.61 \times 1.44 = 36.88$ t

Determine the allowable tensile load P for the tension member shown which is produced using M14 turned bolts. The plate is 7mm thick and is made of St52 steel. All dimensions are in mm



$A_{gross} = 142 \times 7 = 994 \ mm^2 \quad \rightarrow \quad 0.85 \, A_{gross} = 844.9 \ mm^2$

$A_{AB} = 994 - 2 \times (14+1) \times 7 = 784 \ mm^2$

$A_{ABC} = 994 - 3 \times (14+1) \times 7 + \frac{15^2}{4 \times 30} \times 7 = 692.1 \ mm^2$

$A_{EDFC} = 994 - 4 \times (14+1) \times 7 + \frac{25^2}{4 \times 47} \times 7 + \frac{25^2}{4 \times 30} \times 7 + \frac{45^2}{4 \times 10} \times 7 = 988.1 \ mm^2$

$A_{ADFC} = 994 - 4 \times (14+1) \times 7 + \frac{35^2}{4 \times 25} \times 7 + \frac{25^2}{4 \times 30} \times 7 + \frac{45^2}{4 \times 10} \times 7 = 1050.6 \ mm^2$

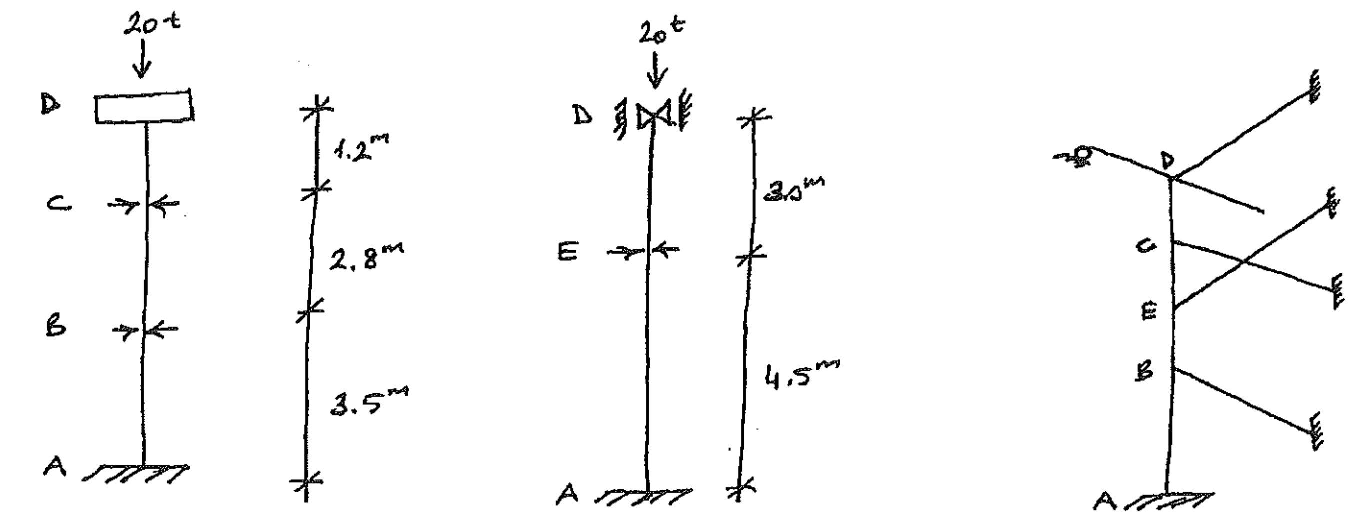
$A_{EDBC} = 994 - 4 \times (14+1) \times 7 + \frac{25^2}{4 \times 47} \times 7 + \frac{35^2}{4 \times 10} \times 7 + \frac{15^2}{4 \times 30} \times 7 = 824.8 \ mm^2$

$A_{min} = A_{ABC} = 6.921 \ cm^2$

$T_{all} = A_{min} \times \sigma_{cem} = 6.921 \times 2.16 = 14.95 \ t$

$A_{ADBC} = 994 - 4 \times (14+1) \times 7 + \frac{35^2}{4 \times 25} \times 7 + \frac{35^2}{4 \times 10} \times 7 + \frac{15^2}{4 \times 30} \times 7 = 887.3 \ mm^2$

$A_{EDBFC} = 994 - 5 \times (14+1) \times 7 + \frac{25^2}{4 \times 47} \times 7 + \frac{35^2}{4 \times 10} \times 7 + \frac{60^2}{4 \times 20} \times 7 + \frac{45^2}{4 \times 10} \times 7 = 1376 \ m$



Design column AD for an axial load of 20 t. Use I-section and St 37 steel. Lateral supports are provided at points B, C and E.

Calculate effective lengths:

$k_{AB} = 0.8 \rightarrow L_{eff} = 0.8 \times 3.5 = 2.8^m$

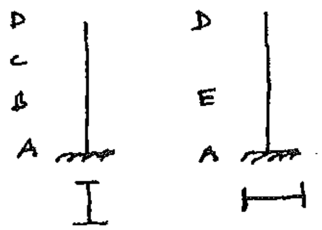
$k_{BC} = 1.0 \rightarrow L_{eff} = 1.0 \times 2.8 = \underline{\underline{2.8^m}}$

$k_{CD} = 2.0 \rightarrow L_{eff} = 2.0 \times 1.2 = 2.4^m$

$k_{AE} = 0.8 \rightarrow L_{eff} = 0.8 \times 4.5 = \underline{\underline{3.6^m}}$

$k_{ED} = 1.0 \rightarrow L_{eff} = 1.0 \times 3.0 = 3.0^m$

Since column AED has a greater L_{eff} →



$$F = \frac{P}{\sigma_{all}} = \frac{20^t}{1.44^{t/cm^2}} = 13.89 \ cm^2$$

Choose I 120 → $F = 14.2 \ cm^2$, $i_x = 4.81 \ cm$, $i_y = 1.23 \ cm$

$$\lambda_x = \frac{360}{4.81} = 74.84 \qquad \lambda_y = \frac{280}{1.23} = 227.64$$

$\lambda_{max} = 227.64$

$\sigma_{bem} = 0.16 \ t/cm^2$

$P = 0.16 \times 14.2 = 2.27^t < 20^t$ ✗

Let's assume $\lambda = \lambda_p = \sqrt{\dfrac{2\pi^2 E}{\sigma_a}} = \sqrt{\dfrac{2\pi^2 \times 2100}{2.4}} = 131.4$

$\lambda = \dfrac{L_{eff}}{i} \rightarrow \lambda_x = 131.4 = \dfrac{360}{i_x} \rightarrow i_x = 2.74\ cm$

$\lambda_y = 131.4 = \dfrac{280}{i_y} \rightarrow i_y = 2.13\ cm$

choose I 240 $\rightarrow$ $F = 46.1\ cm^2$, $i_x = 9.59\ cm$, $i_y = 2.20\ cm$

$\lambda_x = \dfrac{360}{9.59} = 37.54 \qquad \lambda_y = \dfrac{280}{2.20} = 127.27$

$\lambda_{max} = 127.27$

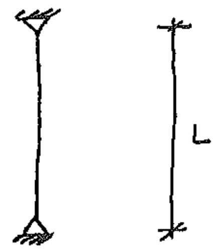
$\sigma_{bem} = 0.51\ t/cm^2$

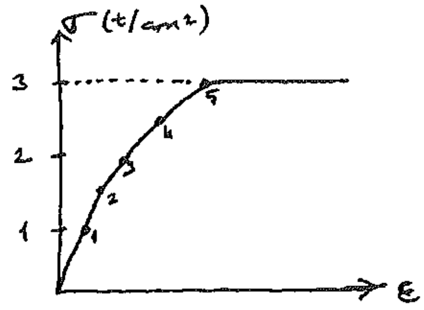
$P = 0.51 \times 46.1 = 23.5^t > 20^t$ ✓

The chosen section is I 240

Note: $\lambda < \lambda_p$ inelastic buckling — λ_p: critical slenderness ratio

$\lambda > \lambda_p$ elastic buckling





	ε	σ (t/cm²)
1	0.0005	1.0
2	0.001	1.6
3	0.002	2.0
4	0.0045	2.5
5	0.012	3.0

$k_x = k_y$

Using a factor of safety of 2.0, find the allowable compressive load for I 400 section for i) L = 700 cm
ii) L = 250 cm

$L_{eff} = k \times L = 1.0 \times L = L$

$E_{01} = 2000$ t/cm², $E_{12} = 1200$ t/cm², $E_{23} = 400$ t/cm², $E_{34} = 200$ t/cm², $E_{45} = 66.7$ t/

I 400 → $F = 118$ cm², $i_x = 15.7$ cm, $i_y = 3.13$ cm

i) $\lambda_x = \frac{700}{15.7} = 44.6$ $\quad \lambda_y = \frac{700}{3.13} = 223.6 \rightarrow \lambda_{max} = 223.6$

Assume σ_{cr} is in bw 0-1 ($E = 2000$ t/cm²)

$$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2 \times 2000}{223.6^2} = 0.39 \text{ t/cm}^2 \rightarrow 0 < \sigma_{cr} < \sigma_1 = 1.0 \text{ t/cm}^2,$$

$$\sigma_{all} = \frac{0.39}{2} = 0.195 \text{ t/cm}^2 \rightarrow P_{all} = 0.195 \times 118 = 23.0^t$$

ii) $\lambda_x = \frac{250}{15.7} = 15.9$ $\quad \lambda_y = \frac{250}{3.13} = 79.9 \rightarrow \lambda_{max} = 79.9$

Assume σ_{cr} is in bw 1-2 ($E = 1200$ t/cm²)

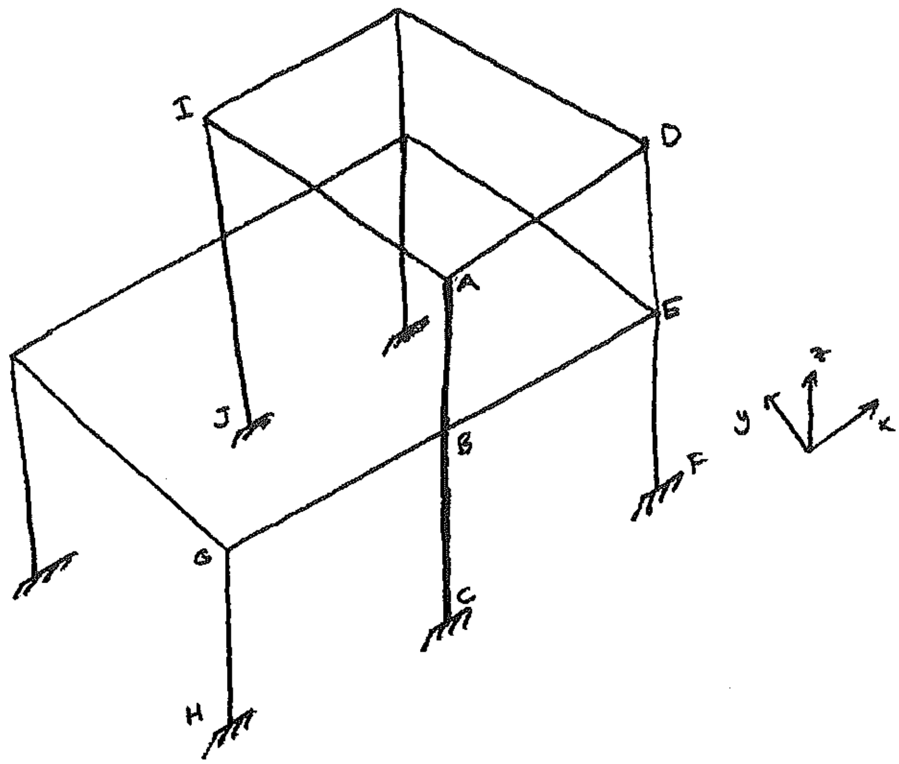
$$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2 \times 1200}{79.9^2} = 1.86 \text{ t/cm}^2 > \sigma_2 = 1.6 \text{ t/cm}^2 \quad X$$

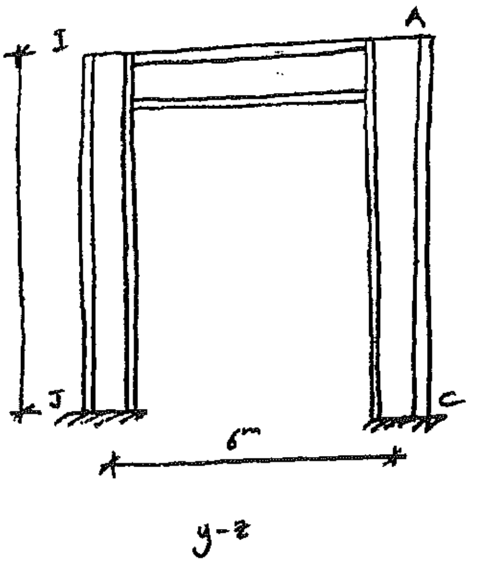
Assume σ_{cr} is in bw 2-3 ($E = 400$ t/cm²)

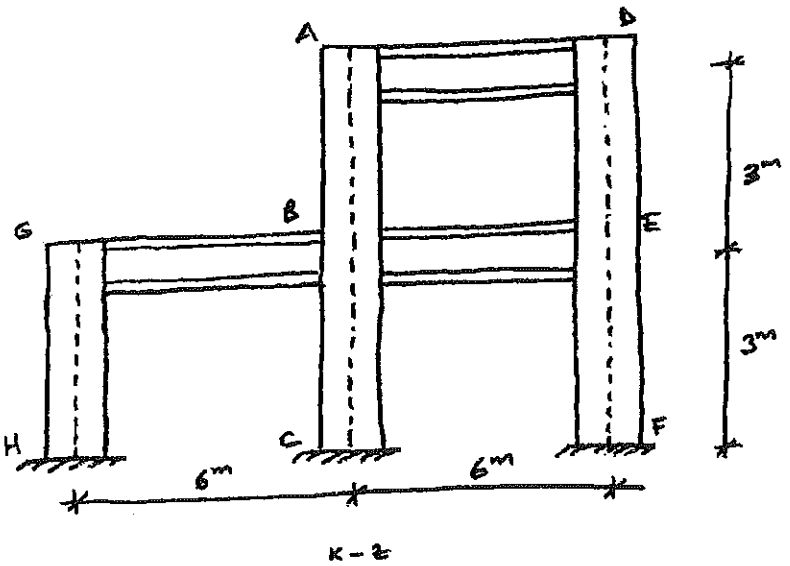
$$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2 \times 400}{79.9^2} = 0.62 \text{ t/cm}^2 < \sigma_2 = 1.6 \text{ t/cm}^2 \quad X$$

Then, $\sigma_{cr} = 1.6$ t/cm²

$$\sigma_{all} = \frac{1.6}{2} \text{ t/cm}^2 = 0.8 \text{ t/cm}^2 \rightarrow P_{all} = 0.8 \times 118 = 94.4t$$







In the frame shown, all columns are IPN 200 and all beams are IPN 240. The orientations of the members are shown in the figure. Using TS648 and St 37, calculate the maximum allowable compression load that the column ABC can carry.

	A(cm²)	Ix (cm⁴)	Iy (cm⁴)	ix (cm)	iy (cm)
IPN 200	33.4	2140	117	8.00	1.87
IPN 240	46.1	4250	221	9.59	2.20

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)}$$

For columns; pinned → $G = 10$
fixed → $G = 1$

$$P = \frac{\sigma_{acm} \times A}{w} \qquad \lambda = \frac{kL}{i}$$

$\sigma_{acm} = 1.44\ t/cm^2$ for st 37 steel

y-z plane

ABC (permitted):

$$G_C = 1$$

$$\left.\begin{aligned} G_C &= 1 \\ G_A &= \frac{2140/600}{4250/600} = 0.5 \end{aligned}\right\} k = 1.22$$

$$\lambda = \frac{1.22 \times 600}{8.00} = 92$$

$$w = 1.81$$

$$P = \frac{1.44 \times 33.4}{1.81} = 26.6\ t$$

x-z plane

BC (permitted):

$$\left.\begin{aligned} G_C &= 1.0 \\ G_B &= \frac{2\times\ 117/300}{2\times\ 4250/600} = 0.06 \end{aligned}\right\} k = 1.18$$

$$\lambda = \frac{1.18 \times 300}{1.87} = 189$$

$$w = 6.20$$

$$P = \frac{1.44 \times 33.4}{6.20} = 7.8\ t$$

AB (permitted):

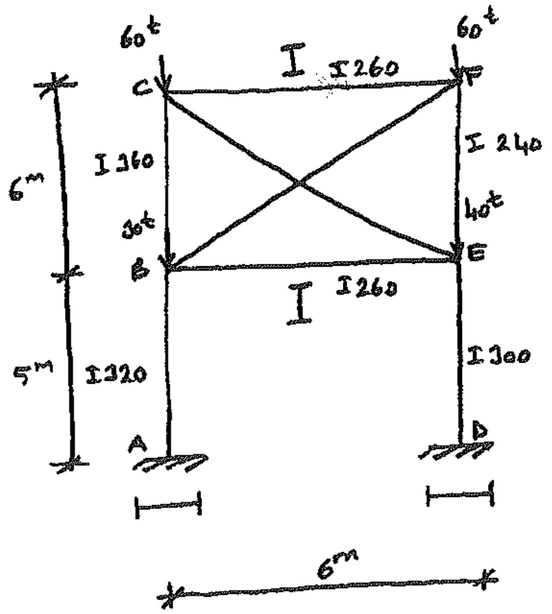
$$\left.\begin{aligned} G_B &= 0.06 \\ G_A &= \frac{117/300}{4250/600} = 0.06 \end{aligned}\right\} k = 1.05$$

$$\lambda = \frac{1.05 \times 300}{1.87} = 168$$

$$w = 4.90$$

$$P = \frac{1.44 \times 33.4}{4.90} = 9.8\ t$$

$$P_{all} = 7.8\ t$$



Check AB and FE columns. St 52 steel
EY loading.

I 300 : $I_y = 9800\ cm^4$, $I_z = 451\ cm^4$

I 360 : $I_y = 19610\ cm^4$, $I_z = 818\ cm^4$

I 260 : $I_y = 5740\ cm^4$, $I_z = 288\ cm^4$

I 320 : $I_y = 12510\ cm^4$, $I_z = 555\ cm^4$

$i_y = 12.70\ cm$, $i_z = 2.67\ cm$

$A = 77.7\ cm^2$

I 240 : $I_y = 4250\ cm^4$, $I_z = 221\ cm^4$

$i_y = 9.59\ cm$, $i_z = 2.20\ cm$

$A = 46.1\ cm^2$

Column AB : sidesway permitted (unbraced)

$$G_A = 1.0$$

$$G_B = \frac{\frac{12510}{500} + \frac{19610}{600}}{\frac{5740}{600}} = 6.03$$

$$K = 1.75$$

$$\lambda = \frac{1.75 \times 500}{12.70} = 69$$

$$w = 1.68$$

$$\sigma_{cem} = 2.16\ t/cm^2$$

$$P_{all} = \frac{2.16 \times 77.7}{1.68} = 100 \text{ tons}$$

$100\,t > (60+30 = 90\,t)$ ✓

Column FE : sidesway prevented (braced)

$$G_E = \frac{\frac{9800}{500} + \frac{4250}{600}}{\frac{5740}{600}} = 2.79$$

$$G_F = \frac{\frac{4250}{600}}{\frac{5740}{600}} = 0.74$$

$$K = 0.81$$

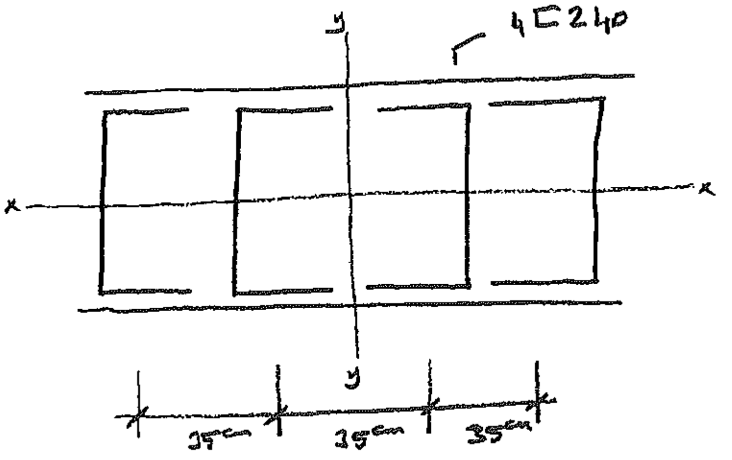
$$\lambda = \frac{0.81 \times 600}{9.59} = 51$$

$$w = 1.39$$

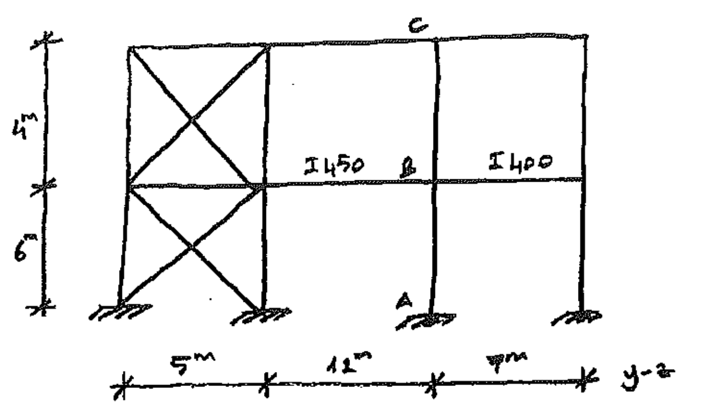
$$\sigma_{cem} = 2.16\ t/cm^2$$

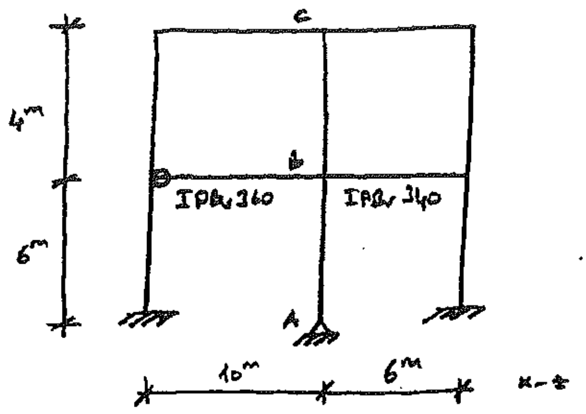
$$P_{all} = \frac{2.16 \times 46.1}{1.39} = 71.6 \text{ tons}$$

$71.6\,t > (60\,t)$ ✓



Calculate the allowable axial load that can be carried by the built-up column AB. St 37 Steel. EY Loading. Batten spacing is given as 60 cm.





	F(cm²)	i_x / i_y(cm)	I_x (cm⁴)	I_y (cm⁴)
[240	42.3	2.42	3600	248
I 450			45890	1730
I 400			29210	1160
IPBv 360			84870	19520
IPBv 340			76370	19710

For column AB;

$$\Sigma I_{xx} = 4 \times 3600 = 14400 \text{ cm}^4$$

$$\Sigma I_{yy} = 4 \times I_y + 2 \times F \times \left(e + \frac{e}{2}\right)^2 + 2 \times F \times \left(\frac{e}{2}\right)^2$$

$$= 4 \times 248 + 2 \times 42.3 \times \left(35 + \frac{35}{2}\right)^2 + 2 \times 42.3 \times \left(\frac{35}{2}\right)^2$$

$$= 260079.5 \text{ cm}^4$$

$$i_y = \sqrt{\frac{\Sigma I_{yy}}{\Sigma F}} = \sqrt{\frac{260079.5}{4 \times 42.3}} = 39.21 \text{ cm}$$

$$i_x = \sqrt{\frac{14400}{4 \times 42.3}} = 9.22 \text{ cm}$$

y-z direction : side-sway prevented

$$G_A = 1$$

$$G_B = \frac{\frac{14400}{600} + \frac{14400}{400}}{\frac{45850}{1200} + \frac{29210}{700}} = 0.75 \quad \Bigg\} \quad k_x = 0.76 \quad \rightarrow \quad \lambda_x = \frac{0.76 \times 600}{9.22} = 48.81$$

x-z direction : side-sway permitted

$$G_A = 10$$

$$G_B = \frac{\frac{260079.5}{600} + \frac{260079.5}{400}}{\frac{84870}{1000} + \frac{76070}{600}} = 5.11 \quad \Bigg\} \quad k_y = 2.58 \quad \rightarrow \quad \lambda_y = \frac{2.58 \times 600}{39.21} = 39.48$$

$$\lambda_1 = \frac{s_1}{i_1} = \frac{60}{2.42} = 24.79$$

$$\lambda_{yi} = \sqrt{\lambda_y^2 + \frac{m}{2}\lambda_1^2}$$

$$= \sqrt{39.48^2 + \frac{4}{2} \times 24.79^2} = 52.8$$

$$\rightarrow \quad \lambda_{max} = \lambda_{yi} = 52.8 \quad \rightarrow \quad \omega = 1.29 \quad \rightarrow \quad P_{all} = \frac{1.44 \times 4 \times 42.3}{1.29}$$

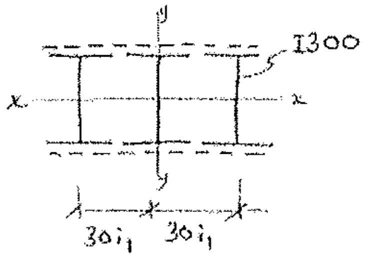
$$= 189^t \quad \checkmark$$

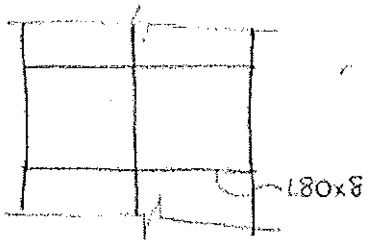
1.) For the built-up column shown below,

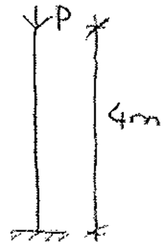
a) Find P_{all}

b) Check battens

Column end conditions are the same in both directions, St 37, EY Loading







Solution

I300

$I_x = 9800 \ cm^4$

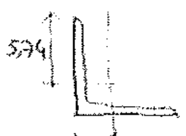
$I_y = 451 \ cm^4$

$F = 69 \ cm^2$

$i_x = 11,9 \ cm$

$i_y = 2,56 \ cm$

L80×8



$I_x = I_y = 72,3 \ cm^4$

$F = 12,3 \ cm^2$

$s_1 \leq 50 i_1 = 50 \times 2,56 = 128 \ cm$

$\frac{400}{4} = 100 \ cm$ so choose $s_1 = 100 \ cm$ for $n = 3$

a) $e = 30 i_1 = 30 \times 2,56 = 76,8 \ cm$

$$I_{yy} = 3 I_y + 2F(e)^2 = 3 \times 451 + 2 \times 69 \times (76,8)^2 = 815310 \ cm^4$$

$$i_y = \sqrt{\frac{815310}{3 \times 69}} = 62,76 \ cm \quad , \quad \lambda_y = \frac{2 \times 400}{62,76} = 12,75 \quad , \quad \lambda_1 = \frac{100}{2,56} = 39,06$$

$$\left.\begin{array}{l} \lambda_x = \dfrac{2 \times 400}{11,9} = 67,23 \\ \lambda_y = \sqrt{12,75^2 + \frac{3}{2} \times 39,06^2} = 49,51 \end{array}\right\} \lambda_{max} = 67,23 \Rightarrow \sigma_{bcm} = 1 \ t/cm^2$$

$$P_{all} = 1 \times 3 \times 69 = 207 \ t$$

b) $e > 20 i_1$

$$Q_i = \frac{3 \times 69 \times 1,44}{80} \left[1 + 0,05\left(\frac{76,8}{2,56} - 20\right)\right] = 5,59 \ t$$

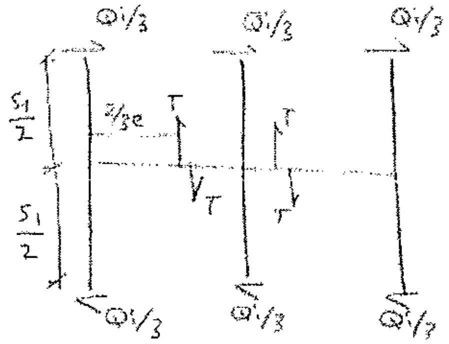
$$T = \frac{5,59 \times 100}{2 \times 76,8} = 3,64 \ t \quad \text{for 2 battens}$$

$T = 1,82 \ t$ for one batten

$$\sigma = \frac{T \cdot \frac{2}{3} e \cdot 5,74}{I_x} = \frac{1,82 \times \frac{2}{3} \times 76,8 \times 5,74}{72,3}$$

$$= 7,4 \ t/cm^2 >> 1,44 \ t/cm^2$$

Greater section should be used.



TRY 75x10

$I_x = I_y = 71{,}4\ cm^4$; $i_x = i_y = 2{,}25\ cm$, $i_1 = 1{,}45\ cm$, $F = 14{,}1\ cm^2$

$e = 20 i_1 = 20 . 1{,}45 = 29\ cm$

y-y direction

$$I_{yy} = 8 . 71{,}4 + 4 . 14{,}1 \left(29 + \frac{29}{2}\right)^2 + 4 . 14{,}1 \left(\frac{29}{2}\right)^2$$

$$= 119152{,}2\ cm^4$$

$$i_y = \sqrt{\frac{119152{,}2}{8 . 14{,}1}} = 32{,}5\ cm$$

$$\lambda_y = \frac{1{,}6 . 1000}{32{,}5} = 49{,}23 \quad ; \quad \lambda_1 = \frac{70}{1{,}45} = 48{,}28$$

$$\lambda_{yi} = \sqrt{49{,}23^2 + \frac{4}{2} 48{,}28^2} = 84{,}18$$

x-x direction

$$I_{xx} = 8 . 71{,}4 + 8 . 14{,}1 . \left(\frac{29}{2}\right)^2 = 24287{,}4\ cm^4$$

$$i_x = \sqrt{\frac{24287{,}4}{8 . 14{,}1}} = 14{,}67\ cm$$

$$\lambda_x = \frac{0{,}85 . 1000}{14{,}67} = 57{,}94 \qquad \lambda_1 = \frac{S_1}{i_1} = \frac{50}{1{,}45} = 34{,}48$$

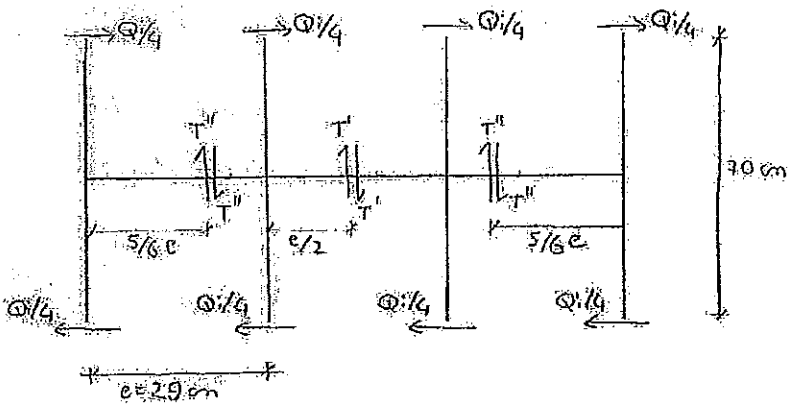
$$\lambda_{xi} = \sqrt{57{,}94^2 + \frac{2}{2} 34{,}48^2} = 67{,}42$$

$\lambda_{max} = \lambda_{yi} = 84{,}18 \longrightarrow \omega = 1{,}67$ ($\sigma_{bcm} = 0{,}8623\ t/cm^2$)

$$P_{max} = \frac{1{,}44 . 8 . 14{,}1}{1{,}67} = 97{,}3\ t > 80\ t \ \checkmark$$ USE L75x10

o Batten Design

Battens paralel to x-x axis



$e = 20 i_1 = 29\ cm$

$$Q_i = \frac{F . \sigma_{aem}}{80} = \frac{8 . 14{,}1 . 1{,}44}{80}$$

$$= 2{,}03\ t$$

$$T' = 0{,}4 \cdot \frac{Q_i \cdot s_1}{e} = 0{,}4 \cdot 2{,}03 \cdot \frac{70}{29} = 1{,}96\ t$$

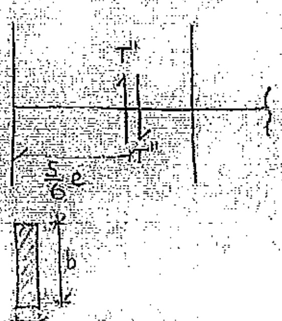
$T_1' = 0{,}98\ t \Rightarrow$ in a single batten

$$T'' = 0{,}3 \cdot \frac{2{,}03 \cdot 70}{29} = 1{,}47\ t \Rightarrow T_1'' = 0{,}735\ t$$

Shear stress

$$\tau_{max} = \frac{3}{2} \frac{T_1'}{b \cdot t} \Rightarrow 0{,}831 = \frac{3}{2} \cdot \frac{0{,}98}{b \cdot t} \Rightarrow b \cdot t = 1{,}77\ cm^2$$

Moment created by T''



$$M = \frac{5}{6} e \cdot T'' = \frac{5}{6} \cdot 29 \cdot 0{,}735 = 17{,}763\ tcm$$

$$\sigma_{cem} = \frac{M \cdot c}{I} = \frac{M\left(\frac{b}{2}\right)}{\frac{1}{12} t \cdot b^3} \Rightarrow 1{,}44 = 17{,}763 \cdot \frac{6}{tb^2}$$

$$\Rightarrow t \cdot b^2 = 74{,}01\ cm^3$$

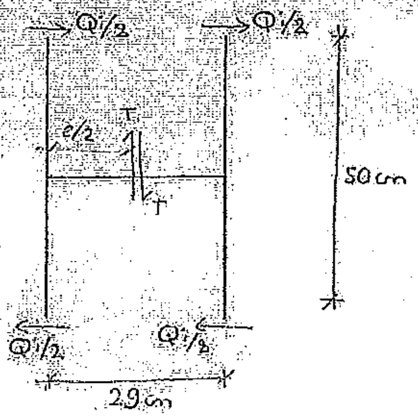
Choose $t = 0{,}5$ cm $\longrightarrow b = 12{,}17$ cm $\sim b = 12{,}5$ cm

USE 125x5 mm rectangular plates

Check shear

$b \cdot t = 0{,}5 \cdot 12{,}5 = 6{,}25\ cm^2 > 1{,}77\ cm^2$ ✓

Battens parallel to y-y axis



$e = 29$ cm

$$Q_i = \frac{F \cdot \sigma_{cem}}{80} = \frac{8{,}141 \cdot 1{,}44}{80} = 2{,}03\ t$$

$$T = \frac{Q_i}{(4)} \frac{s_1}{e} = \frac{2{,}03 \cdot 50}{4 \cdot 29} = 0{,}875\ t$$

4 battens

$T = 0{,}875\ t \rightarrow$ in a single batten

Shear stress

$$\tau_{max} = \frac{3}{2} \cdot \frac{T}{bt} \Rightarrow 0{,}831 = \frac{3}{2} \cdot \frac{0{,}875}{b \cdot t} \Rightarrow b \cdot t = 1{,}58\ cm^2$$

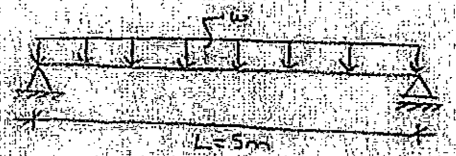
Axial stress

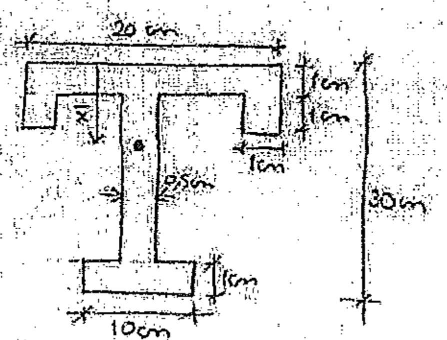
$$M = T \cdot \frac{e}{2} = 0{,}875 \cdot \frac{29}{2} = 12{,}69\ tcm$$

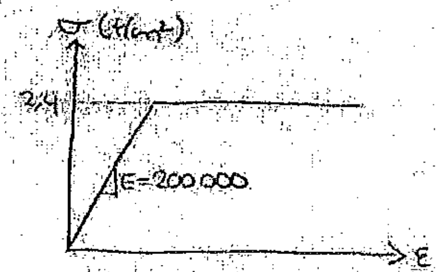
$$\sigma_{cem} = \frac{M \cdot c}{I} \Rightarrow 1{,}44 = M \cdot \frac{6}{tb^2} \Rightarrow tb^2 = 52{,}86\ cm^3$$

Choose $t = 0{,}5$ cm $\rightarrow b = 10{,}28$ cm $\sim b = 10{,}5$ cm USE 105x5 rectangular plates

CE485
RECITATION-1







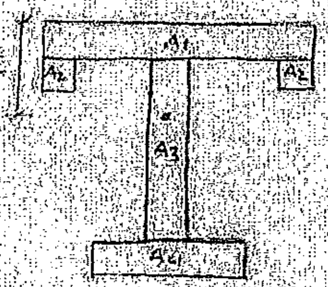
Stress-strain diagram

Calculate the allowable load w using,

a) Allowable stress design using FS=1,67

b) Ultimate strength design using FS=2,00

($\sigma_y = 2{,}4$ t/cm²)



cross-sectional area:

$$A_T = A_1 + 2A_2 + A_3 + A_4$$

$$= 20\times1 + 2\times1\times1 + 28\times0{,}5 + 10\times1 = 46\,cm^2$$

Centroid:

$$\bar{x} = \frac{20\times1\times0{,}5 + 2\times1\times1\times1{,}5 + 28\times0{,}5\times15 + 10\times1\times29{,}5}{46} = 11{,}26\ cm$$

Moment of inertia about bending axis

$$I = \frac{1}{12}\times20\times1^3 + 20\times1\times(11{,}26-0{,}5)^2 + 2\times\frac{1}{12}\times1\times1^3 + 2\times1\times1\times(11{,}26-1{,}5)^2$$

$$+ \frac{1}{12}\times0{,}5\times28^3 + 0{,}5\times28\times(15-11{,}26)^2 + \frac{1}{12}\times10\times1^3 + 10\times1\times(29{,}5-11{,}26)^2$$

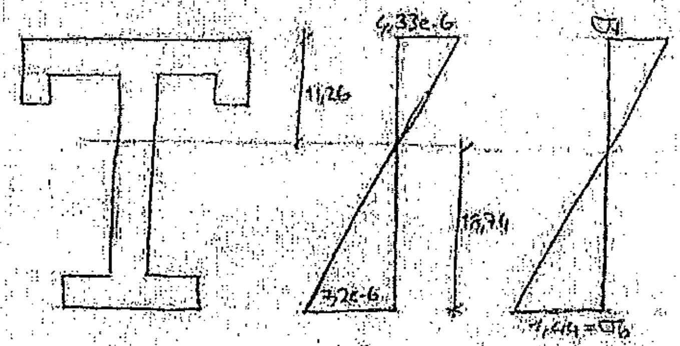
$$I = 6946{,}2\ cm^4$$

a) calculating w_{all} by allowable stress design.

$$\sigma_{all} = \frac{\sigma_y}{1{,}67} = 1{,}44\ t/cm^2 \quad , \quad M_{max} = \frac{w_{all}\cdot L^2}{8} = \frac{w_{all}\cdot 500^2}{8} = 31250\,w_{all} = M_{all}$$

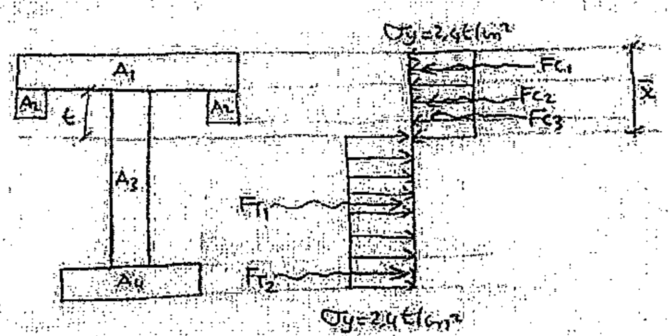
$$\sigma_{all} = \frac{M_{all}\cdot y}{I} \Rightarrow 1{,}44 = \frac{31250\,w_{all}\cdot(30-11{,}26)}{6946{,}2}$$

$$\Rightarrow w_{all} = 0{,}017\ t/cm = 1{,}7\ t/m$$



$$\frac{\sigma_t}{11{,}26} = \frac{\sigma_b}{18{,}74} \Rightarrow \sigma_t = 0{,}865\ t/cm^2 < \sigma_y = 1{,}44\ t/cm^2$$

b) calculating wall by ultimate strength design.



All the section has yielded ($\sigma=\sigma_y$ everywhere). Because steel is assumed to have the same stress-strain diagram in tension and compression

$\Sigma F_C = \Sigma F_T$

$F_{C_1} + F_{C_2} + F_{C_3} = F_{T_1} + F_{T_2}$

$A_1 \times \sigma_y + 2A_2 \times \sigma_y + t \times 0{,}5 \times \sigma_y = (28-t) \times 0{,}5 \times \sigma_y + A_4 \times \sigma_y$

$A_1 + 2A_2 + 0{,}5t = 14 - 0{,}5t + A_4 \Rightarrow t = 14 + 10 - 20 - 2$

$t = 2\ cm$

$\bar{x} = t + 1 = 3\ cm$

<u>Compression Forces</u>

$F_{C_1} = A_1 \times \sigma_y = 20 \times 2{,}4 = 48t$

$F_{C_2} = 2A_2 \times \sigma_y = 2 \times 2{,}4 = 4{,}8t$

$F_{C_3} = t \times 0{,}5 \times \sigma_y = 2 \times 0{,}5 \times 2{,}4 = 2{,}4t$

$\Sigma F_C = 55{,}2t$

<u>Tension Forces</u>

$F_{T_1} = (28-2) \times 0{,}5 \times 2{,}4 = 31{,}2t$

$F_{T_2} = A_4 \times \sigma_y = 10 \times 2{,}4 = 24t$

$\Sigma F_T = 55{,}2t$

$\Sigma F_C = \Sigma F_T$ ✓

<u>Moment arms with respect to the centroid</u>

$x_{C_1} = 3 - 0{,}5 = 2{,}5\ cm$ $\qquad x_{T_1} = \frac{28-2}{2} = 13\ cm$

$x_{C_2} = 3 - 1{,}5 = 1{,}5\ cm$ $\qquad x_{T_2} = 26{,}5\ cm$

$x_{C_3} = 1\ cm$

$M_p = (48 \times 2{,}5) + (4{,}8 \times 1{,}5) + (2{,}4 \times 1) + (31{,}2 \times 13) + (24 \times 26{,}5)$

$= 1171{,}2\ tcm$

$= 11{,}71\ tm$

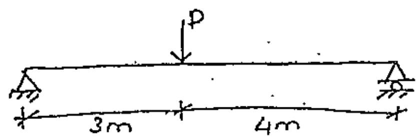
$M_{all} = \frac{11{,}71}{2} = 5{,}855\ tm$

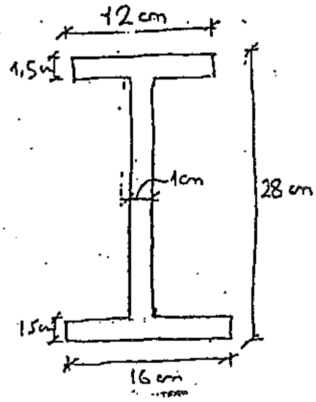
$w_{all} = \frac{8 \cdot M_{all}}{L^2} = \frac{8 \times 5{,}855}{5^2} = 1{,}87\ t/m$

CE485 - FUNDAMENTALS OF STEEL DESIGN

RECITATION-1

Exercise 1





Calculate the allowable load P using

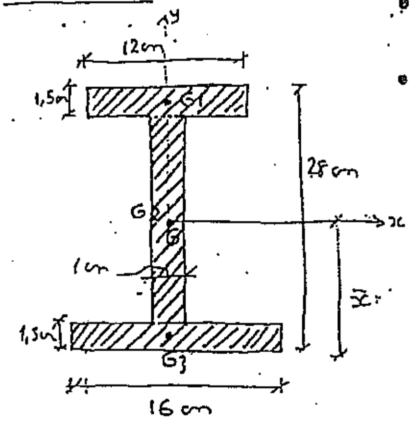
a) Allowable stress design using $F_s = 1{,}67$

b) Ultimate strength design using $F_s = 2{,}00$

$\sigma_y = 2{,}4\ t/cm^2$

c) Shape factor about binding axes

Solution



- Cross-sectional area;

$$A_t = A_1 + A_2 + A_3 = 1{,}5.12 + (28-1{,}5-1{,}5).1 + 1{,}5.16 = 18 + 25 + 24 = 67\ cm^2$$

- Centroid;

$$\bar{x} = \frac{A_1.x_1 + A_2.x_2 + A_3.x_3}{A_t} = \frac{18.(28-\frac{1{,}5}{2}) + 25.14 + 24.\frac{1{,}5}{2}}{67} = \frac{858{,}5}{67}$$

$$\Rightarrow \bar{x} = 12{,}8\ cm$$

- Moment of Inertia (about bending axis)

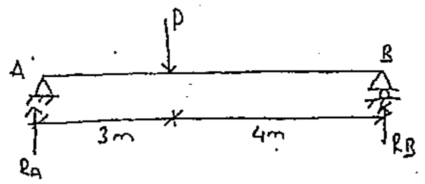
$$I_x = \frac{1}{12}.12.1{,}5^3 + 1{,}5.12.(28-12{,}8-\frac{1{,}5}{2})^2 \longrightarrow 3761{,}82\ cm^4$$

$$+\frac{1}{12}.1.25^3 + 1.25.(14-12{,}8)^2 \longrightarrow 1338{,}83\ cm^4$$

$$+\frac{1}{12}.16.1{,}5^3 + 1{,}5.16.(12{,}8-\frac{1{,}5}{2})^2 \longrightarrow 3489{,}36\ cm^4$$

$$\Rightarrow I_x = 8590{,}01\ cm^4$$

a) Calculate the yield moment capacity (elastic analysis)



$$\sigma_y = \frac{M_y.c}{I_x} \Rightarrow M_y = \frac{\sigma_y.I_x}{c} = \frac{2{,}4.8590{,}01}{(28-12{,}8)} = 1356{,}32\ tcm = 13{,}56\ tm$$

max moment that the section can carry.

$$\sigma_{max} = \frac{M_{max}.c}{I_x} \leq \sigma_{allowable}$$

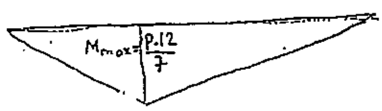
$$\sigma_{all} = \frac{\sigma_y}{F_s} \Rightarrow M_{all} = \frac{M_y}{F_s} = \frac{13{,}56}{1{,}67} = 8{,}12\ tm$$

$$\Sigma M_B = 0 \Rightarrow R_A.7 - P.4 = 0 \Rightarrow R_A = \frac{P.4}{7}$$

$$\Sigma F\uparrow = 0 \Rightarrow R_A + R_B - P = 0 \Rightarrow R_B = \frac{P.3}{7}$$

$$M_{all} = M_{max} = \frac{P_{all}.12}{7} \Rightarrow P_{all} = \frac{7.M_{max}}{12} = \frac{7.8{,}12}{12} = 4{,}74\ t.$$

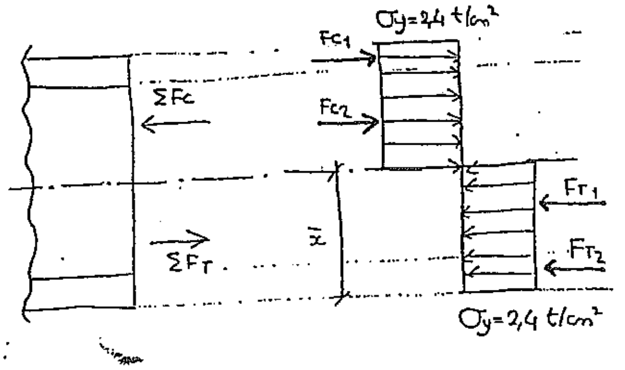
max P force.



b) Ultimate strength design;

For plastic moment capacity, we need the location of the shifted neutral axis. Let $\bar{x}$ be the distance between neutral axis and the bottom of the section.

$A_{comp} = A_{tens}$ (Force equilibrium on the cross-section)



$$1{,}5.16 + (\bar{x}-1.5).1 = \frac{67}{2} \Rightarrow \bar{x} = 11\ cm$$

Proving the force equilibrium;

Force in; top flange: $F_{c1} = A_{c1}.\sigma_y = 1{,}5.12.2{,}4 = 43{,}2t$ $\Big\}\ \Sigma F_c = 80{,}4t$

web in compr: $F_{c2} = A_{c2}.\sigma_y = 15{,}5.1.2{,}4 = 37{,}2t$

web in tension: $F_{T1} = A_{T1}.\sigma_y = 9{,}5.1.2{,}4 = 22{,}8t$ $\Big\}\ \Sigma F_T = 80{,}4t$

bottom flange: $F_{T2} = A_{T2}.\sigma_y = 1{,}5.16.2{,}4 = 57{,}6t$

$\Sigma F_c = \Sigma F_T = 80{,}4t$ ✓

Distances to neutral axis;

For $F_{c1} \rightarrow x_{c1} = 16{,}25\ cm$

$F_{c2} \rightarrow x_{c2} = 7{,}75\ cm$

$F_{T1} \rightarrow x_{T1} = 4{,}75\ cm$

$F_{T2} \rightarrow x_{T2} = 10{,}25\ cm$

Plastic moment capacity (ultimate)

$$M_p = F_{c1}.x_{c1} + F_{c2}.x_{c2} + F_{T1}.x_{T1} + F_{T2}.x_{T2}$$

$$= 43{,}2.16{,}25 + 37{,}2.7{,}75 + 22{,}8.4{,}75 + 57{,}6.10{,}25$$

$$M_p = 1689\ tcm = 16{,}89\ tm$$

$$M_{all} = \frac{M_p}{F_s} = \frac{16{,}89}{2{,}00} = 8{,}445\ tm$$

$$P_{all} = \frac{7.M_{all}}{12} = \frac{7.8{,}445}{12} = 4{,}926\ t$$

c) Shape factor $= \frac{W_{pl}}{W_{el}}$ (section modulus) $= \frac{M_{pl}}{M_{el}} = \varphi$

$$\varphi = \frac{16{,}89\ tm}{13{,}56} = 1{,}25$$

$Z_p = \frac{M_p}{\sigma_y} = \frac{1689}{2{,}4} = 703{,}75\ cm^3$ (plastic section modulus)

$Z = \frac{I_x}{y} = \frac{8590{,}01}{15{,}2} = 565{,}13\ cm^3$ (elastic section modulus)

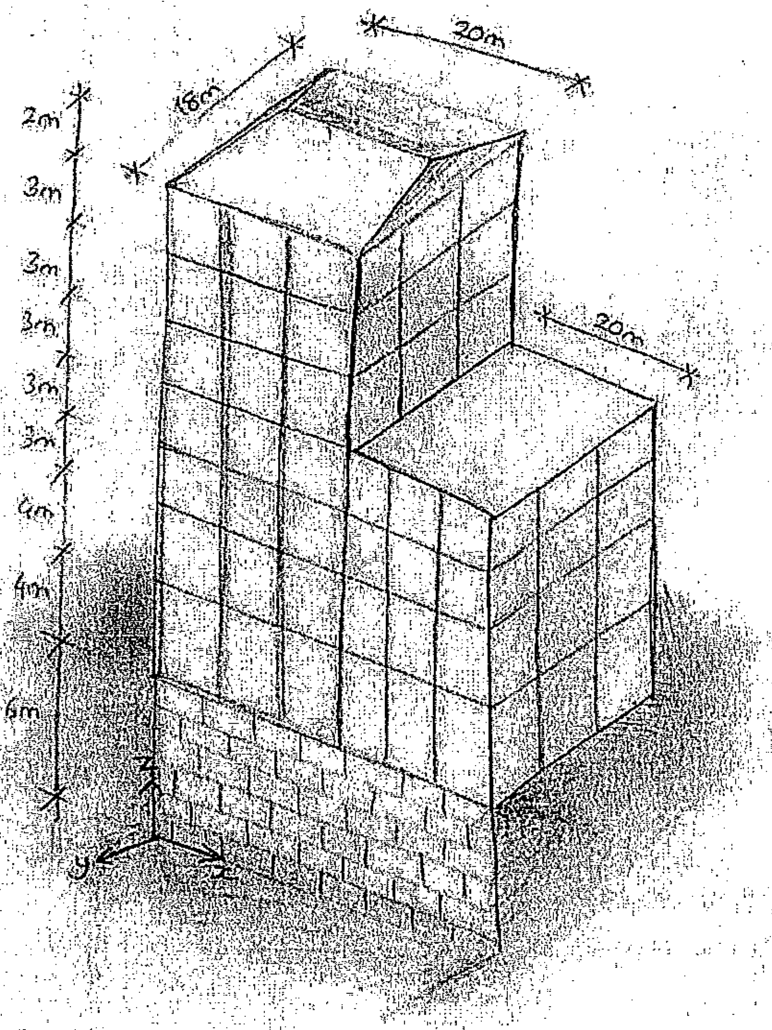
$$\varphi = \frac{703{,}75}{565{,}13} = 1{,}25$$

1

2/2

CE485 – RECITATION

WIND LOAD CALCULATION



Calculate most critical equivalent static wind force acting on each story and draw story shear diagram.

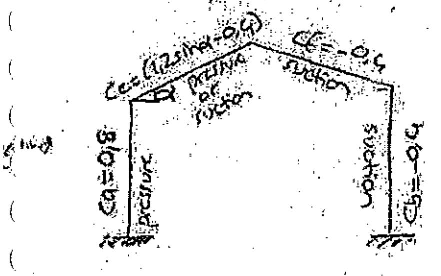
SOLUTION

– Determine most critical direction

H, m	q, kg/m²
0 – 8	50
9 – 20	80
21 – 100	110
>100	130

Wind pressure increases with height. When wind blows in +x direction effective height of the building should be taken 25 m (including roof).
On the other hand when wind blows in –y direction effective height of the building is 31 m (including roof.)
Since in –y direction the effective height is 31 m, wind pressure is critical when wind blows in this direction.

– Wind pressure



$\alpha = \tan^{-1}\left(\frac{2}{10}\right)$

$\Rightarrow \alpha = 11{,}3°$

$C_e = (1{,}2\sin 11{,}3 - 0{,}4)$
$= -0{,}165$ (suction)

For $0 < H < 8m \Rightarrow W_{a_1} = 0{,}8 \times 50 = 40\ kg/m^2$, $W_{b_1} = -0{,}4 \times 50 = -20\ kg/m^2$

$9 < H < 20m \Rightarrow W_{a_2} = 0{,}8 \times 80 = 64\ kg/m^2$, $W_{b_2} = -0{,}4 \times 80 = -32\ kg/m^2$

$21 < H < 100m \Rightarrow W_{a_3} = 0{,}8 \times 110 = 88\ kg/m^2$, $W_{b_3} = -0{,}4 \times 110 = -44\ kg/m^2$

$W_e = -0{,}165 \times 110 = 18{,}15\ kg/m^2$

$W_f = -0{,}4 \times 110 = -44\ kg/m^2$

22/02/06

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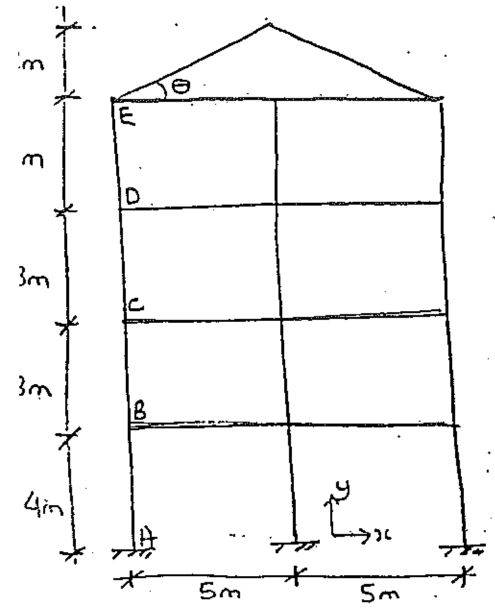
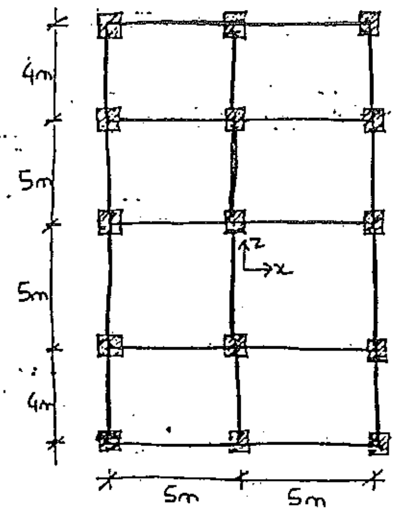
CE485
RECITATION
WIND LOAD CALCULATION (2)





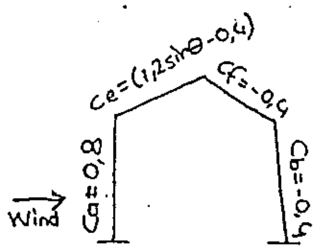
Calculate equivalent static wind force acting on each storey and draw storey shear diagram for wind blowing in x direction.

Wind pressure

$\theta = \tan^{-1}\left(\frac{2}{5}\right) = 21{,}8^\circ$

$0 < H < 8 \rightarrow q_1 = 50\ kg/m^2$

$8 < H < 20 \rightarrow q_2 = 80\ kg/m^2$



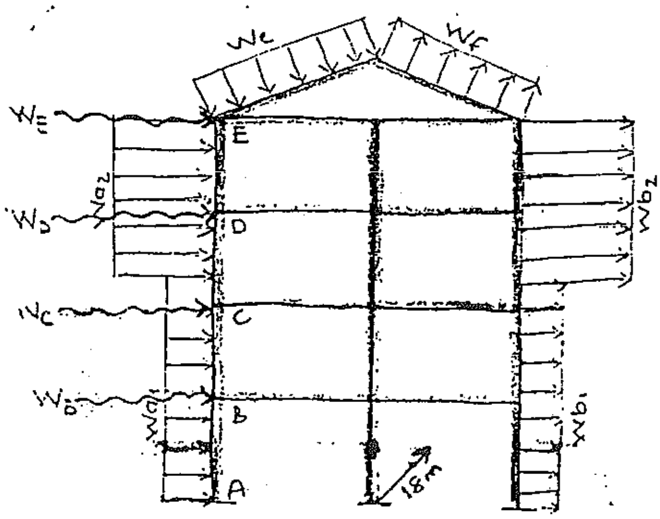
$C_a = 0{,}8$

$C_e = (1{,}2\sin 21{,}8 - 0{,}4) = 0{,}0456$

$C_f = -0{,}4$

$C_b = -0{,}4$

Equivalent static wind pressure (+pressure, −suction)



$W_{a_1} = c_a \cdot q_1 = 0{,}8 \times 50 = 40\ kg/m^2$

$W_{a_2} = c_a \cdot q_2 = 0{,}8 \times 80 = 64\ kg/m^2$

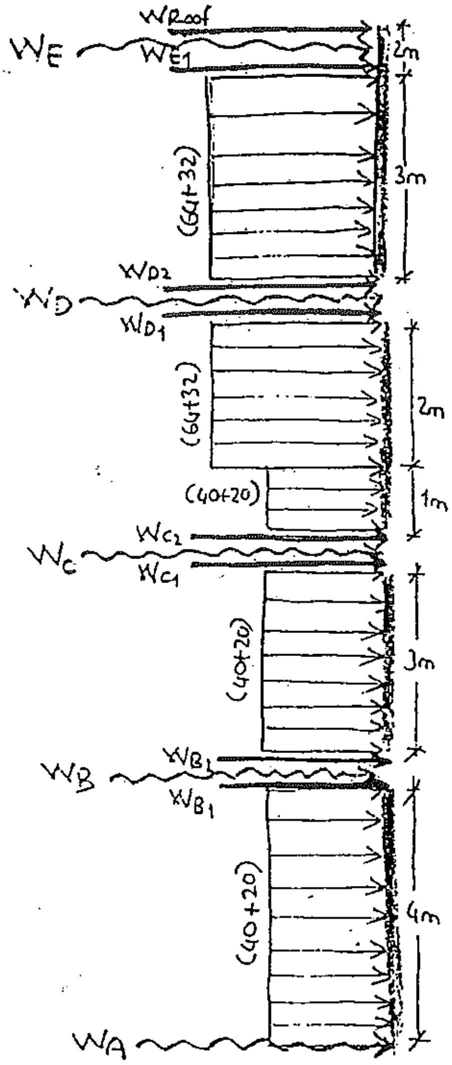
$W_{b_1} = c_b \cdot q_1 = -0{,}4 \times 50 = -20\ kg/m^2$

$W_{b_2} = c_b \cdot q_2 = -0{,}4 \times 80 = -32\ kg/m^2$

$W_e = c_e \cdot q_2 = 0{,}0456 \times 80 = 3{,}65\ kg/m^2$

$W_f = C_f \cdot q_2 = -0{,}4 \times 80 = -32\ kg/m^2$

Wind force at each story



$$W_{Roof} = 18 \times 2 \times (3{,}65 + 32) = 1283{,}4\ kg = 1{,}283\ t$$

$$W_{E_1} = W_{D_2} = 18 \times \frac{3}{2} \times (64+32) \times \frac{1}{1000} = 2{,}592t$$

$$W_E = W_{E_1} + W_{Roof} = 2{,}592 + 1{,}283 = 3{,}875\ t$$

$$W_{D_1} = \frac{18 \times (96 \times 2 \times 2 + 60 \times 0{,}5)}{3} \times \frac{1}{1000} = 2{,}484\ t$$

$$W_D = W_{D_1} + W_{D_2} = 2{,}592 + 2{,}484 = 5{,}076\ t$$

$$W_{C_2} = \frac{18 \times (96 \times 2 \times 1 + 60 \times 1 \times 2{,}5)}{3} \times \frac{1}{1000} = 2{,}052\ t$$

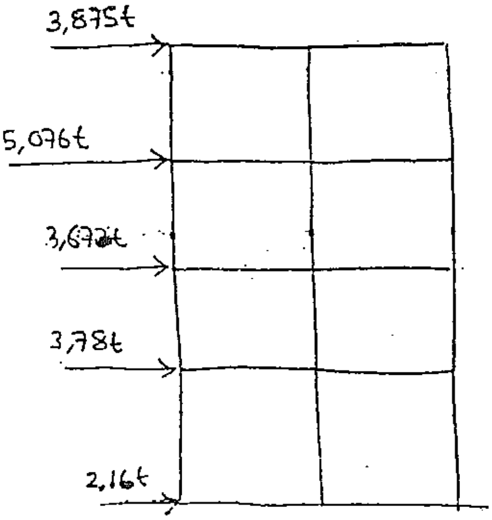
$$W_{C_1} = W_{B_2} = 18 \times \frac{3}{2} \times 60 \times \frac{1}{1000} = 1{,}62\ t$$

$$W_C = W_{C_1} + W_{C_2} = 2{,}052 + 1{,}62 = 3{,}672\ t$$

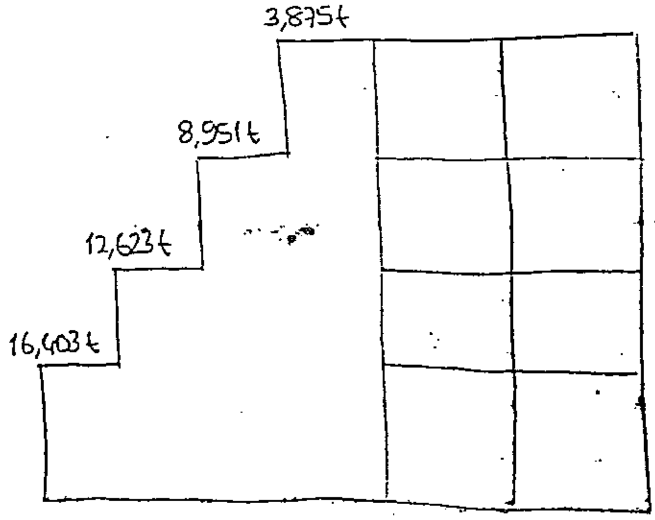
$$W_{B_1} = W_A = 18 \times \frac{4}{2} \times 60 \times \frac{1}{1000} = 2{,}16t$$

$$W_B = W_{B_1} + W_{B_2} = 1{,}62 + 2{,}16 = 3{,}78t$$

$$W_A = 2{,}16t$$



Wind load acting on each story



Story shear diagram

Story	w_i (t)	H_i (m)	$w_i H_i$	F_{fi}
1	396.56	5	1982.805	0.115
2	333.28	9	2999.529	0.174
3	301.64	12	3619.692	0.210
4	301.64	15	4524.615	0.263
5	226.25	18	4072.572	0.237
		Total	17199.21	1.0000

Story	m_i (t)	d_{fi}	F_{fi}	$m_i\, d_{fi}^2$	$F_{fi}\, d_{fi}$
1	387.62	5.302e-6	0.115	1.11E-08	6.11E-07
2	324.54	1.002e-5	0.174	3.35E-08	1.75E-06
3	292.90	1.239e-5	0.210	4.63E-08	2.61E-06
4	292.90	1.401e-5	0.263	5.92E-08	3.69E-06
5	221.89	1.495e-5	0.237	5.06E-08	3.54E-06
		Total	1.0000	2.01E-07	1.22E-05

$$T_1 = 2\pi\left[\frac{2.01e-7}{1.22e-5}\right]^{1/2} = 0.806s$$

$S(T_1)$=? (The soil type must be known; Z1)
Characteristic periods of the soil are required (See 2006TEQ)
T_A = 0.1sec, T_B = 0.3sec for soil type Z1 and $T_1 > T_B$

$$S(T_1) = 2.5\cdot\left(\frac{T_B}{T_1}\right)^{0.8} = 1.134$$

Now calculate $A(T_1)$, the Spectral Acceleration Coefficient:

$$\mathrm{A}(\mathrm{T}_1) = \mathrm{A}_0 \cdot \mathrm{I} \cdot \mathrm{S}(\mathrm{T}_1)$$

A_0 = 0.4 (For earthquake region I)
I = 1.5 (For hospital buildings)

$A(T_1)$ = 0.4 x 1.5 x 1.134 = **0.68g**

$R_a(T_1)$ = R (Since $T > T_A$) and R = 4 for R/C frame structure possessing normal ductility. Thus,

$R_a(T_1) = 4$

$$V_t = \frac{1559.38 \times 0.68}{4} = 265.19t$$

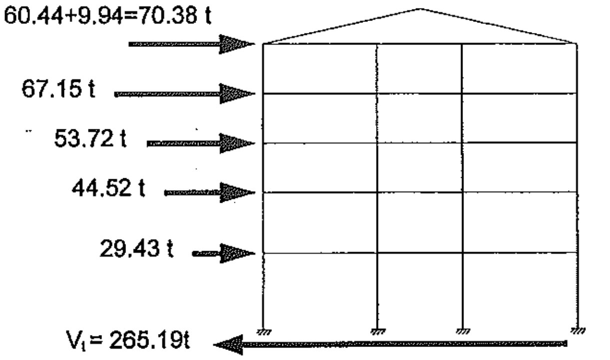
Check $\dfrac{\mathrm{W}\cdot\mathrm{A}(\mathrm{T}_1)}{\mathrm{R}_a(\mathrm{T}_1)} \geq 0.1\cdot\mathrm{A}_0\cdot\mathrm{I}\cdot\mathrm{W}$ or its simplified form: $\dfrac{\mathrm{S}(\mathrm{T}_1)}{\mathrm{R}_a(\mathrm{T}_a)} \geq 0.1$

$$\frac{1.134}{4} = 0.284 > 0.1 \quad \text{OK}$$

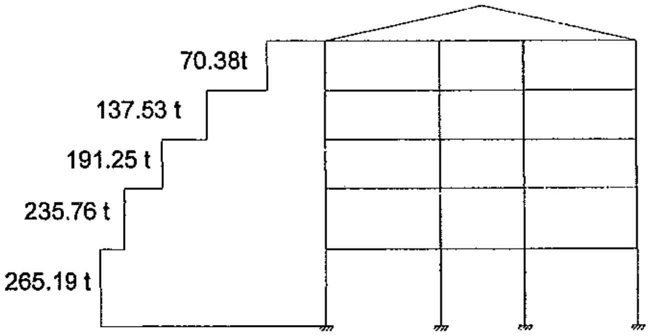
$$\mathrm{V}_t = \Delta\mathrm{F}_n + \sum \mathrm{F}_i \qquad \Delta F_n = 0.0075xNxVt = 0.0075x5x265.19 = 9.94t$$

$V_t - \sum F_i = 265.19 - 9.94 = 255.25$ this load is distributed to the floors

STORY	W_i (T)	H_i (M)	W_iH_i (T.M)	$W_iH_i/\Sigma W_iH_i$	F_i (T)
1	396.56	5	1982.81	0.115	29.43
2	333.28	9	2999.53	0.174	44.52
3	301.64	12	3619.69	0.210	53.72
4	301.64	15	4524.62	0.263	67.15
5	226.25	18	4072.57	0.237	60.44
TOTAL			17199.21	1.0000	**255.25 = V_t✓**



Earthquake Loads Acting on Each Story

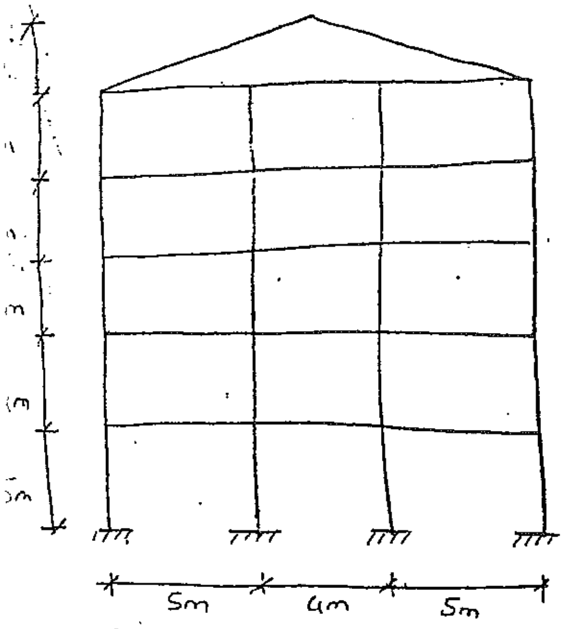


Story Shear Force Diagram

CE 485

RECITATION (EQ) 3

01/03/06



Compute the EQ force acting on each story and draw the story shear diagram, calculate total base shear.

Assume ; $g = 1{,}2\ t/m^2$

$q = 0{,}2\ t/m^2$ (office)

$g_{roof} = 0{,}2\ t/m^2$

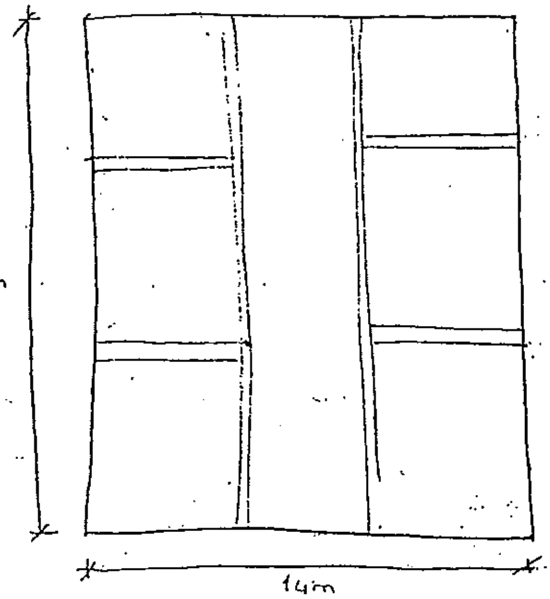
Altitude = 800 m

EQ Zone = 2

Soil type = Z1

R/C → nominal ductile frames

Snow load region = II



Plan view

TOTAL WEIGHT OF THE BUILDING

Snow Load

Since the building is 800 m high from the sea level and in 2. Region, $P_{k0} = 0,85\ kN/m^2 = 85\ kg/m^2$ (for TS 498).

The snow load is modified according to the slope of the roof. When roof slope is less than 30°, m=1 and we have to take snow load (P_k) as P_{k0}. If it is greater than 30°, $m = 1 - \frac{\alpha - 30}{40}$.

Slope of the roof : $\alpha = \tan^{-1}\left(\frac{2}{7}\right) = 16° < 30° \rightarrow m = 1$

$P_k = m P_{k0} = 1.85 = 85\ kg/m^2$

Total snow load is evaluated according to plan view of the roof.

$LL_s = 0,085 . 20 . 14 = 23,8\ t$ //

Live Load for each story

Since it is an office building and $q = 0,2\ t/m^2$, we can write LL on each story as LL (except roof) $= 0,2 . 20 . 14 = 56\ t$ //

According to EQ Code LL can be reduced by multiplying with 0,3 for office buildings (Table 6.7)

$W_{qi} = 56 \times 0,3 = 16,8\ t \quad (i = 1,2,3,4)$

$W_{qs} = 23,8 . 0,3 = 7,18\ t$

Story weights

The weight per unit area is given as $1,2\ t/m^2$

$W_i = \underbrace{1,2 . 20 . 14}_{DL} + \underbrace{16,8}_{LL} = 352,8\ t \quad i = 1,2,3,4$

$W_s = 1,2 . 20 . 14 + \underbrace{0,2 . 20 . 14}_{\text{weight of roof}} + \underbrace{7,18}_{\text{snow load}} = 399,2\ t$

01/03/06

EQ Calculations

$$V_t = \frac{W \cdot A(T_1)}{R_a(T_1)} \geq 0{,}1 \cdot A_0 \cdot I \cdot W$$

$V_t \rightarrow$ Total base shear

$W \rightarrow$ Total building weight

$A_0 \rightarrow$ Effective ground acceleration coefficient $\rightarrow$

EQ Zone	A_0
1	0,4
2	0,3
3	0,2
4	0,1

$I \rightarrow$ Building importance factor

$R_a(T) \rightarrow$ EQ load reduction factor

$$A(T_1) = A_0 \cdot I \cdot \overbrace{S(T_1)}^{\text{spectrum coeff}}$$

$\hookrightarrow$ spectral acc. coeff

$H_N = 18\,m$ (up to top story under roof)

$T_1 = C_t \cdot H_N^{3/4}$ where $C_t = 0{,}07$ for R/C frame structures
0,08 for steel structures
0,05 for others

First natural vibration period of structure

$T_1 = 0{,}07 \cdot 18^{3/4} = 0{,}61$ sec.

For soil type Z1 ; $T_A = 0{,}10\,s$, $T_B = 0{,}30\,s$ — spectrum characteristic periods

	T_A	T_B
Z1	0,1	0,3
Z2	0,15	0,4
Z3	0,15	0,6
Z4	0,2	0,9

If $T_1 > T_B \rightarrow S(T) = 2{,}5 \cdot \left(\frac{T_B}{T_1}\right)^{0{,}8} = 1{,}42$

$T_A < T_1 \leq T_B \rightarrow S(T) = 2{,}5$

$0 \leq T \leq T_A \rightarrow S(T) = 1 + 1{,}5 \cdot \frac{T}{T_A}$

$A_0 = 0{,}3$ for EQ Region 2

$I = 1{,}0$ for office buildings.

$A(T_1) = 0{,}3 \cdot 1{,}0 \cdot 1{,}42 = 0{,}4269$

$T > T_A \rightarrow R_a(T_1) = R = 4 \rightarrow$ for R/C normal (nominal) ductility

$\frac{S(T_1)}{R_a(T_1)} \geq 0{,}1$ required $\rightarrow \frac{1{,}42}{4} > 0{,}1$ ✓

$$V_t = \frac{W \cdot A(T_1)}{R_a(T_1)} = \frac{(4.352{,}8 + 399{,}2) \cdot 0{,}426}{4} = 192{,}8\,t \rightarrow \text{Total Base shear}$$

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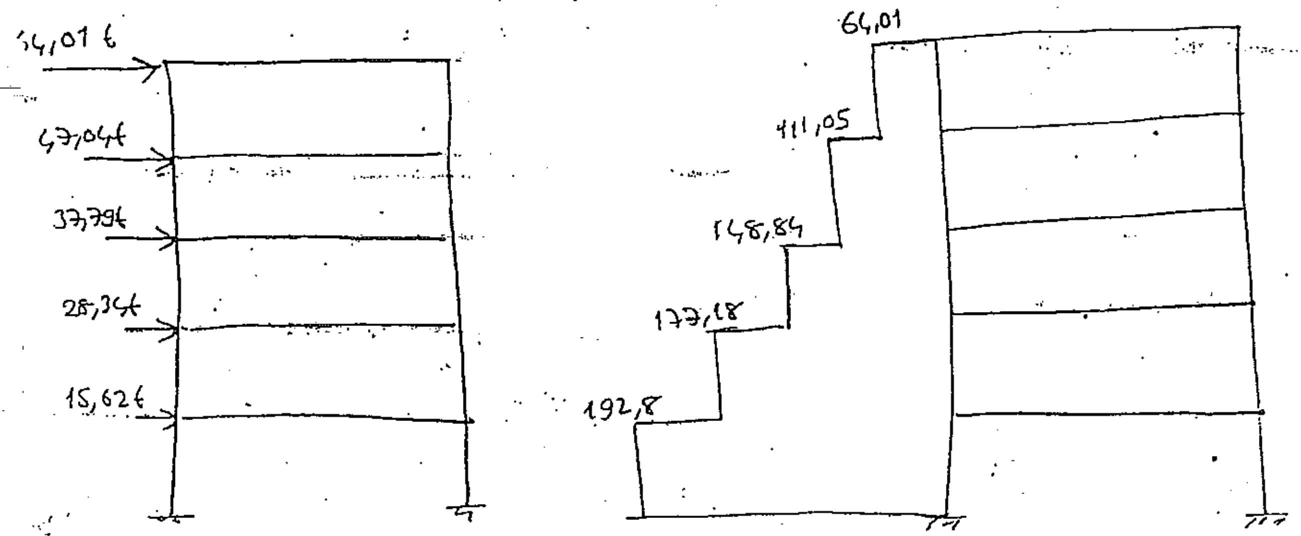
07/02/06

To find EQ acting on each story ∴ use the formula below;

$$F_i = V_t \cdot \frac{W_i \cdot H_i}{\Sigma W_i \cdot H_i}$$

W_i; each story weight.
H_i; each story height.

Story	W_i	H_i	$W_i \cdot H_i$	$\frac{W_i \cdot H_i}{\Sigma W_i \cdot H_i}$	F_i (tons)
1	352,8	5	1764,0	0,081	15,62
2	352,8	9	3175,2	0,147	28,34
3	352,8	12	4233,6	0,196	37,79
4	352,8	15	5292,0	0,244	47,04
5	399,2	18	7185,6	0,332	64,01
		Σ	21650,4	1,00 ✓	192,87 ✓



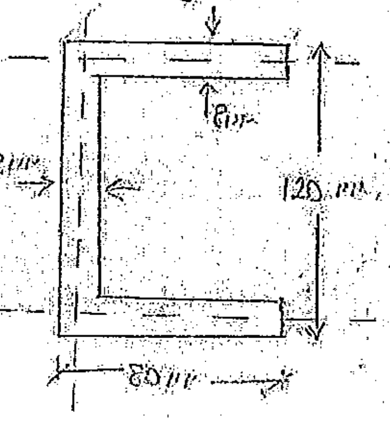
Story shear diagram

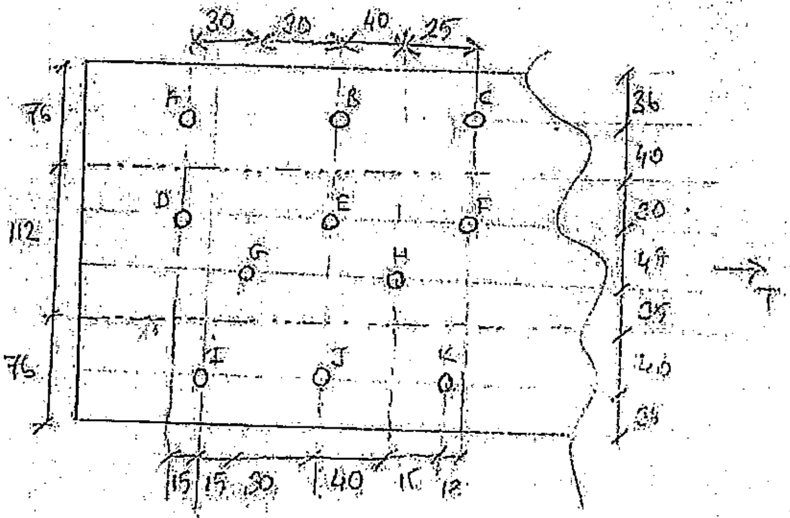
4/4

01/09/06

CE 489 RECITATION 4

Calculate the maximum tensile load that can be carried by the member shown considering plate tension St 37, M16 bolts





Hole diameter = bolt diameter + 1 mm

$A_{net} = A_{gross} - n d_i t + \sum \frac{s^2 t}{4g}$

n = # of holes

s = spacing btw bolts parallel to tensile force

g = distance btw bolts perpendicular to tensile force

t = plate thickness

$P = A_g \sigma_{t}$

$A_{net} \sigma_t \leq 0.85 A_g \sigma_t$

$A_{net} \leq 0.85 A_g$

$A_g = (76 + 112 + 76)(8) = 2112$ mm²

$A_{net} = 0.85 A_g = 1795.2$ mm²

- 1 holed path(s) ⟹ $A_{n_1} = (76+112+76-17) \times 8 = 1976$ mm²
- 2 holed path(s) ⟹ $A_{n_2} = (76+112+76-2\times17)\times 8 = 1840$ mm²

3 holed path(s)

path BEJ $\longrightarrow A_n = (264 - 3\times17)\times8 = 1704$ mm²

path ADG = BEG $\longrightarrow (264-3\times17)\times8 + \frac{30^2}{4\times47}\times8 = 1742.3$ mm²

path CFH $\longrightarrow (264-3\times17)\times8 + \frac{25^2}{4\times47}\times8 = 1730.6$ mm²

path CFK $\longrightarrow (264-3\times17)\times8 + \frac{10^2}{4(47+35+40)} \times 8 = 1705.6$ mm²

4 holed path(s)

path ADGI $\longrightarrow (264-4\times17)\times8 + \frac{30^2}{4\times47}\times8 + \frac{15^2}{4\times75}\times8 = 1612.3$ mm² (smaller than BEGJ, equal to BEGI, smaller than BEHJ, equal to BEHK, smaller than CFHJ)

path CFHK $\longrightarrow (264-4\times17)\times8 + \frac{25^2}{4\times47}\times8 + \frac{15^2\times8}{4\times75} = 1600.6$ mm²

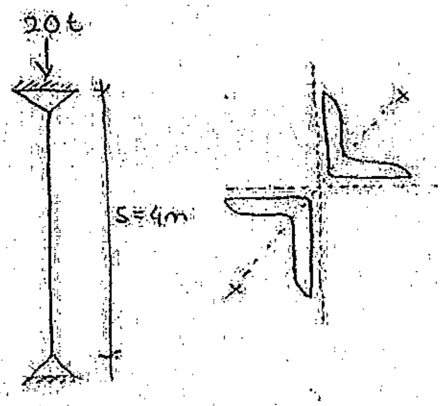
$\sigma_{gen} = \sigma_{all} = 1.44$ t/cm² for St 37

$A_{net} = 1600.6 < 0.85 A_g$ (OK)

$P = \sigma_{gen} A_{net} = 1.44 \times 16 = 23.04$ t.

CE 485
RECITATION
BUILT-UP MEMBERS

1)



Design the column using two equal-angle-leg section.
$S_{plane} = S_{out of plane} = 4m$. St 37 , EY Loading.
($s_1 = 20$ m)

$$S_{kx} = \frac{S_{plane} + S_{out\ of\ plane}}{2} = \frac{400 + 400}{2} = 400 \text{ cm}$$

$$\lambda_x = \frac{S_{kx}}{i_x}$$

TRY L80x10

$F = 15{,}1 \text{ cm}^2$, $i_x = 3{,}03$ cm , $i_{min} = 1{,}54$ cm

$$\lambda_x = \frac{400}{3{,}03} = 132{,}01 \longrightarrow \sigma_{all} = 0{,}4758 \text{ t/cm}^2$$

$$P_{all} = 2 \times 15{,}1 \times 0{,}4758 = 14{,}36 \text{ t} < 20 \text{ t} \quad \times$$

TRY L100x8

$F = 15{,}5 \text{ cm}^2$, $i_x = 3{,}85$ cm , $i_{min} = 1{,}96$ cm

$$\lambda_x = \frac{400}{3{,}85} = 103{,}9 \longrightarrow \sigma_{all} = 0{,}7014 \text{ t/cm}^2$$

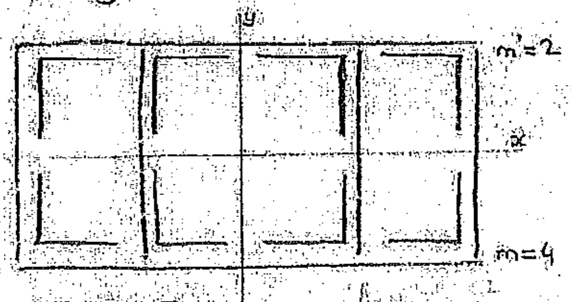
$$P_{all} = 2 \times 15{,}5 \times 0{,}7014 = 21{,}74 \text{ t} > 20 \text{ t} \quad \checkmark$$

Check $\lambda_1 = \frac{s_1}{i_1} = \frac{20}{1{,}96} = 10{,}2 < 50 \quad \checkmark$

USE L100x8

CE485 - RECITATION - BUILT-UP MEMBERS

2) Designing the built-up column made of 8 equal-leg angle sections for an axial comp. load of 80 tons. $k_x = 0,85$, $k_y = 1,6$, St37, EY loading. Optimum spacing between main members is used. $S = 10$ m. $s_{1x} = 70$ cm, $s_{1y} = 50$ cm



TRY 70x7

$I_x = I_y = 42,4 \text{ cm}^4$, $i_x = i_y = 2,12$ cm, $i_1 = 1,37$, $F = 9,4 \text{ cm}^2$

$e \leq 20\, i_1 \Rightarrow e = 20 \cdot 1,37 = 27,4$ cm

y-y direction

$$I_{yy} = 8\, I_y + 4F\left(e + \frac{e}{2}\right)^2 + 4F\left(\frac{e}{2}\right)^2$$

$$= 8 \cdot 42,4 + 4 \cdot 9,4\left(27,4 + \frac{27,4}{2}\right)^2 + 4 \cdot 9,4\left(\frac{27,4}{2}\right)^2$$

$$= 70910,64 \text{ cm}^4$$

$$i_y = \sqrt{\frac{I_{yy}}{\Sigma F}} = \sqrt{\frac{70910,64}{8 \cdot 9,4}} = 30,71 \text{ cm}$$

$$\lambda_y = \frac{k_y \cdot S}{i_y} = \frac{1,6 \cdot 1000}{30,71} = 52,1 \quad , \quad \lambda_1 = \frac{70}{1,37} = 51,1$$

$$\lambda_{yi} = \sqrt{\lambda_y^2 + \frac{m}{2}\lambda_1^2} = \sqrt{52,1^2 + \frac{4}{2} \cdot 51,1^2} = 89,1$$

x-x direction

$$I_{xx} = 8 \cdot I_x + 8F\left(\frac{e}{2}\right)^2 = 8 \cdot 42,4 + 8 \cdot 9,4\left(\frac{27,4}{2}\right)^2 = 14453,49 \text{ cm}^4$$

$$i_x = \sqrt{\frac{I_{xx}}{\Sigma F}} = \sqrt{\frac{14453,49}{8 \cdot 9,4}} = 6,53 \text{ cm}$$

$$\lambda_x = \frac{k_x \cdot S}{i_x} = \frac{0,85 \cdot 1000}{6,53} = 130,2 \quad , \quad \lambda_1 = \frac{50}{1,37} = 36,5$$

$$\lambda_{xi} = \sqrt{\lambda_x^2 + \frac{m}{2}\lambda_1^2} = \sqrt{130,2^2 + \frac{2}{2} \cdot 36,5^2} = 135,2$$

$$\lambda_{max} = \lambda_{xi} = 135,2 \longrightarrow \omega = 3,17 \quad (\sigma_{bem} = 0,4549 \text{ t/cm}^2)$$

$$P_{max} = \frac{\sigma_a \cdot \Sigma F}{\omega} = \frac{1,44 \cdot 8 \cdot 9,4}{3,17} = 34,16 \text{ t} < 80 \text{ t} \quad \text{NOT OK}$$

TRY 75x10

$I_x = I_y = 71{,}4\ cm^4$, $i_x = i_y = 2{,}25\ cm$, $i_1 = 1{,}45\ cm$, $F = 14{,}1\ cm^2$

$e = 20 i_1 = 20 \cdot 1{,}45 = 29\ cm$

y-y direction

$$I_{yy} = 8 \cdot 71{,}4 + 4 \cdot 14{,}1 \left(29 + \frac{29}{2}\right)^2 + 4 \cdot 14{,}1 \left(\frac{29}{2}\right)^2$$

$$= 119152{,}2\ cm^4$$

$$i_y = \sqrt{\frac{119152{,}2}{8 \cdot 14{,}1}} = 32{,}5\ cm$$

$$\lambda_y = \frac{1{,}6 \cdot 1000}{32{,}5} = 49{,}23 \quad , \quad \lambda_1 = \frac{70}{1{,}45} = 48{,}28$$

$$\lambda_{yi} = \sqrt{49{,}23^2 + \frac{4}{2} 48{,}28^2} = 84{,}18$$

x-x direction

$$I_{xx} = 8 \cdot 71{,}4 + 8 \cdot 14{,}1 \cdot \left(\frac{29}{2}\right)^2 = 24287{,}4\ cm^4$$

$$i_x = \sqrt{\frac{24287{,}4}{8 \cdot 14{,}1}} = 14{,}67\ cm$$

$$\lambda_x = \frac{0{,}85 \cdot 1000}{14{,}67} = 57{,}94 \qquad \lambda_1 = \frac{s_1}{i_1} = \frac{50}{1{,}45} = 34{,}48$$

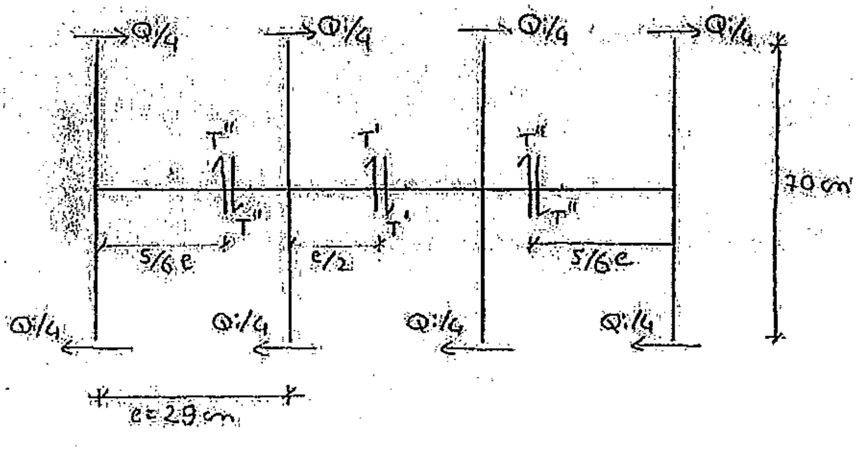
$$\lambda_{xi} = \sqrt{57{,}94^2 + \frac{2}{2} 34{,}48^2} = 67{,}42$$

$$\lambda_{max} = \lambda_{yi} = 84{,}18 \longrightarrow \omega = 1{,}67 \quad (\sigma_{bcm} = 0{,}8623\ t/cm^2)$$

$$P_{max} = \frac{1{,}44 \cdot 8 \cdot 14{,}1}{1{,}67} = 97{,}3\ t > 80\ t \ \checkmark \qquad \text{USE } L\,75\text{x}10$$

o Batten Design

Battens parallel to x-x axis



$e = 20 i_1 = 29\ cm$

$$Q_i = \frac{F \cdot \sigma_{acm}}{80} = \frac{8 \cdot 14{,}1 \cdot 1{,}44}{80}$$

$$= 2{,}03\ t$$

$T' = 0{,}4 \cdot \frac{Q_i \cdot s_1}{e} = 0{,}4 \cdot 2{,}03 \cdot \frac{70}{29} = 1{,}96\ t$

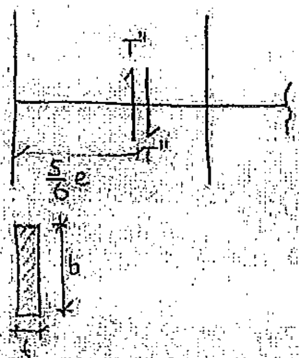
$T_1' = 0{,}98\ t \Rightarrow$ in a single batten

$T'' = 0{,}3 \cdot \frac{2{,}03 \cdot 70}{29} = 1{,}47\ t \Rightarrow T_1'' = 0{,}735\ t$

Shear stress

$\tau_{max} = \frac{3}{2}\,\frac{T_1'}{b \cdot t} \Rightarrow 0{,}831 = \frac{3}{2} \cdot \frac{0{,}98}{b \cdot t} \Rightarrow b \cdot t = 1{,}77\ cm^2$

Moment created by T''



$M = \frac{5}{6} e \cdot T'' = \frac{5}{6} \cdot 29 \cdot 0{,}735 = 17{,}763\ tcm$

$\sigma_{cem} = \frac{M \cdot c}{I} = \frac{M\left(\frac{b}{2}\right)}{\frac{1}{12} t \cdot b^3} \Rightarrow 1{,}44 = 17{,}763 \cdot \frac{6}{t b^2}$

$\Rightarrow t \cdot b^2 = 74{,}01\ cm^3$

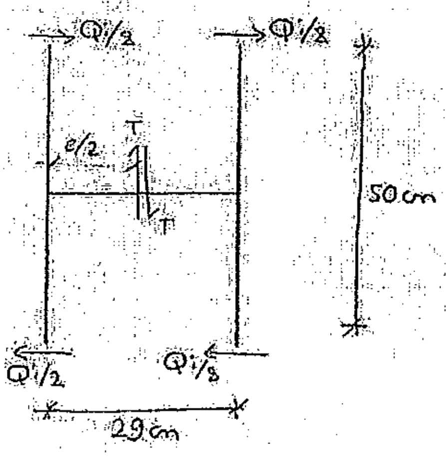
Choose $t = 0{,}5$ cm $\longrightarrow b = 12{,}17$ cm $\sim b = 12{,}5$ cm

USE 125x5 mm rectangular plates

Check shear

$b \cdot t = 0{,}5 \cdot 12{,}5 = 6{,}25\ cm^2 > 1{,}77\ cm^2$ ✓

Battens parallel to y-y axis



$e = 29$ cm

$Q_i = \frac{F \cdot \sigma_{cem}}{80} = \frac{8.141 \cdot 1{,}44}{80} = 2{,}03\ t$

$T = \frac{Q_i}{(4)}\,\frac{s_1}{e} = \frac{2{,}03 \cdot 50}{4 \cdot 29} = 0{,}875\ t$

4 battens

$T = 0{,}875\ t \rightarrow$ in a single batten

Shear stress

$\tau_{max} = \frac{3}{2} \cdot \frac{T}{bt} \Rightarrow 0{,}831 = \frac{3}{2} \cdot \frac{0{,}875}{b \cdot t} \Rightarrow b \cdot t = 1{,}58\ cm^2$

Axial stress

$M = T \cdot \frac{e}{2} = 0{,}875 \cdot \frac{29}{2} = 12{,}69\ tcm$

$\sigma_{cem} = \frac{M \cdot c}{I} \Rightarrow 1{,}44 = M \cdot \frac{6}{t b^2} \Rightarrow t b^2 = 52{,}86\ cm^3$

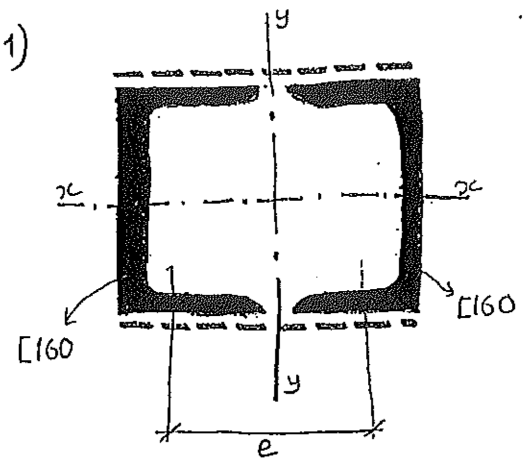
Choose $t = 0{,}5$ cm $\rightarrow b = 10{,}28$ cm $\sim b = 10{,}5$ cm   USE 105x5 rectangular plates

20/04/05

CE 485 - RECITATION

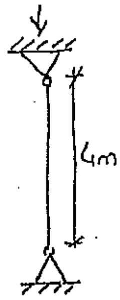
COLUMN DESIGN
BUIT-UP COLUMNS

1)



The built-up column shown is made of two [160 sections with batten spacing of 70 cm. St 52, EY loading.

a)



Compute "e" such that the column has equal slenderness ratios with respect to x-x and y-y axes.

Section Properties

$F = 24,0\ cm^2$

$I_x = 925\ cm^4 \qquad i_x = 6,21\ cm$

$I_y = 85,3\ cm^4 \qquad i_y = 1,89\ cm$

b) Assuming the buckling length about x-x axis to be 3,0 m and about y-y axis to be 4,0 m and e=20 cm, compute the allowable axial compressive load.

c) If battens are 150x5 mm rectangular plates, check their safety for e=25 cm.

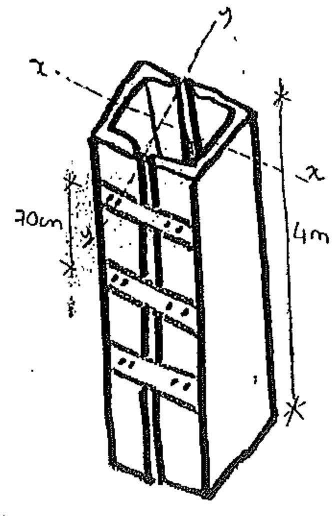
d) Design battens and batten spacing if e=30 cm

Grup I

a) $\lambda_x = \dfrac{S_{kx}}{i_x} = \dfrac{400}{6,21} = 64,41$ ($= \lambda_{yi}$ for the given condition.)

$\lambda_{yi} = \sqrt{\lambda_y^2 + \frac{m}{2}\lambda_1^2} \quad \rightarrow \lambda_1 = \dfrac{S_1}{i_1} = \dfrac{70}{1,89} = 37,03 \leq 50$

$I_{yg} = \Sigma I_y + \Sigma F_i \left(\frac{e}{2}\right)^2 = 2I_{y_1} + 2F \cdot \left(\frac{e}{2}\right)^2 = 2 \cdot 85,3 + 2 \cdot 24 \cdot \left(\frac{e}{2}\right)^2 = 170,6 + 12e^2$



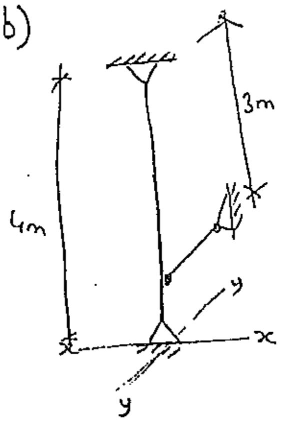
$i_y = \sqrt{\dfrac{I_{yy}}{F}} = \sqrt{\dfrac{170,6 + 12e^2}{48}} = \sqrt{3,55 + 0,25e^2}$

$\lambda_y = \dfrac{S_{ky}}{i_y} = \dfrac{400}{\sqrt{3,55 + 0,25e^2}}$

$\lambda_{yi}^2 = \dfrac{400^2}{3,55 + 0,25e^2} + \dfrac{2}{2}(37,03)^2 = \lambda_x^2 = 64,41^2$

$\dfrac{400^2}{3,55 + 0,25e^2} = 64,41^2 - 37,03^2 \Rightarrow e = 14,7\ cm$

b)



$S_{kx} = 300\ m$

$S_{ky} = 400\ m$

$e = 20\ cm$

$\lambda_x = \dfrac{S_{kx}}{i_x} = \dfrac{300}{6,21} = 48,3$

$\lambda_{yi} = \sqrt{\lambda_y^2 + \dfrac{m}{2}\lambda_1^2}$

$\lambda_1 = 37,03 < 50$ ✓

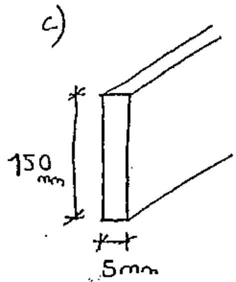
$\lambda_y^2 = \dfrac{400^2}{(1,89)^2 + \left(\frac{20}{2}\right)^2} = 1544,82$

$\lambda_{yi} = \sqrt{1544,82 + \frac{2}{2}\cdot(37,03)^2} = 54 \rightarrow$ critical.

$\lambda = 54 \xrightarrow{\text{Table 8}} \sigma_{ben} = 1512,8\ kgf/cm^2$

$P_{all} = 2 \times 24 \times \sigma_{ben} = 72,6\ t.$

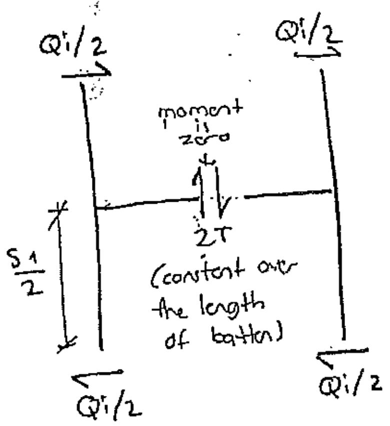
c)



$e = 25\ cm$ (check whether $e > 20 i_1$) → smaller i (i_x, i_y)

$20 i_1 = 20 . 1,89 = 37,8\ cm \rightarrow e = 25\ cm < 20 i_1 \rightarrow$ no increase in Q_i shearing force

$Q_i = \dfrac{F . \sigma_{cen}}{80} = \dfrac{2.24.2,16}{80} = 1,296\ t$ ↳ total shear on cross section



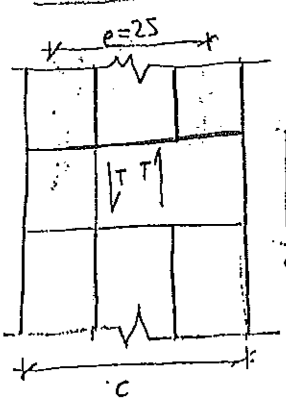
$\dfrac{Q_i}{2} . S_1 = T . \dfrac{e}{2} \Rightarrow \dfrac{1,296}{2} . 70 . \dfrac{2}{25} = T$

$T = 3,63\ t$ (acting on two battens)

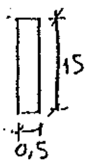
Check shear in batten plate ($\tau_{all} = \dfrac{\sigma_{cen}}{\sqrt{3}} = 1,247\ t/cm^2$)

$\tau = \dfrac{3}{2} . \dfrac{T}{bt} = \dfrac{3}{2} . \dfrac{(3,63/2)}{15 . 0,5} = 0,363\ t/cm^2 < 1,247\ t/cm^2$ ✓

Check axial stress



$\sigma = \dfrac{M . c}{I} = \dfrac{\left(3,63/2 \cdot \frac{e}{2}\right) . 7,5}{0,5 . \frac{15^3}{12}} = 1,21\ t/cm^2 < 2,16 \rightarrow$ SAFE ✓



?/

d) Designing batters and batten spacing.

$e = 30$ cm

The greatest distance is obtained by substituting $\lambda_1 = 50$

$\frac{s_1}{i_1} = 50 \Rightarrow s_1 = 50 \cdot 1{,}89 = 94{,}5$ cm $\longrightarrow$ use $s_1 = 90$ cm

$Q_i' = \frac{F \cdot \sigma_{cem}}{80} = 1{,}296\,t$ $\quad (e < 20 i_1)$

$\frac{Q_i'}{2} \cdot s_1 = T \cdot \frac{e}{2} \Rightarrow T = \frac{1{,}296}{2} \cdot 90 \cdot \frac{2}{30} = 3{,}89\,t$ (total force in 2 batters
$T_1 = 1{,}945\,t$ on single batten)

for $\tau = \tau_{all}$

$1{,}247 = \frac{3}{2} \cdot \frac{1{,}945}{b \cdot t} \Rightarrow b \cdot t = 2{,}34$ cm^2

for $\sigma = \sigma_{cem}$

$\sigma = \frac{Mc}{I} = \frac{M \cdot b/2}{\frac{1}{12} t \cdot b^3} \Rightarrow b^2 t = \frac{6 \cdot (1{,}945 \cdot 15)}{2{,}16} = 81$ cm^3 $\quad \frac{e}{2}$

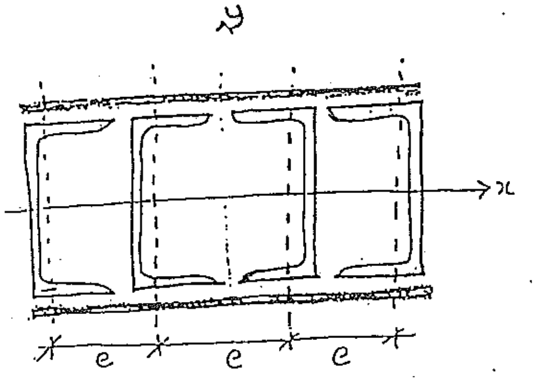
if $t = 0{,}5$ cm , $b = 12{,}7$ cm

$\Rightarrow$ USE 5x130 mm plates ($F = 6{,}5$ cm^2 ✓)

16/11/2005 1/3

CE485
RECITATION
BUILT-UP COLUMNS

(17)



Calculate the allowable axial compres... load that can be carried by the built-... column shown made of four [220 secti... Batten spacing, $s_1 = 70$ cm, effective length factors $k_x = 0{,}7$, $k_y = 3{,}0$, clear length $L = 5$ m, St 37, EY loading.

$e = 20$ cm

Section Properties

$F = 37{,}4 \text{ cm}^2$

$I_x = 2690 \text{ cm}^4$ $\quad i_x = 8{,}48$ cm

$I_y = 197 \text{ cm}^4$ $\quad i_y = 2{,}30$ cm

SOLUTION

Effective lengths: $s_{kx} = k_x \cdot L = 0{,}7 \times 5{,}0 = 3{,}5$ m

$s_{ky} = k_y \cdot L = 3{,}0 \times 5{,}0 = 15{,}0$ m

Radius of gyrations: $i_x = i_{x_1} = 8{,}48$ cm

$$i_y = \sqrt{\frac{\Sigma I_{yy}}{\Sigma F}} = \sqrt{\frac{4 \times I_{y_1} + 2 \times F \times \left(\frac{e}{2}\right)^2 + 2 \times F \times \left(e + \frac{e}{2}\right)^2}{4F}} = \sqrt{i_{y_1}^2 + \frac{e^2/4 + 9e^2/4}{2}}$$

$i_y = 22{,}5$ cm

Imaginary slenderness ratios:

$$\lambda_{yi} = \sqrt{\lambda_y^2 + m/2 \cdot \lambda_1^2}$$

$m = 4$ (member groups contained by the axes parall... to the y-axis)

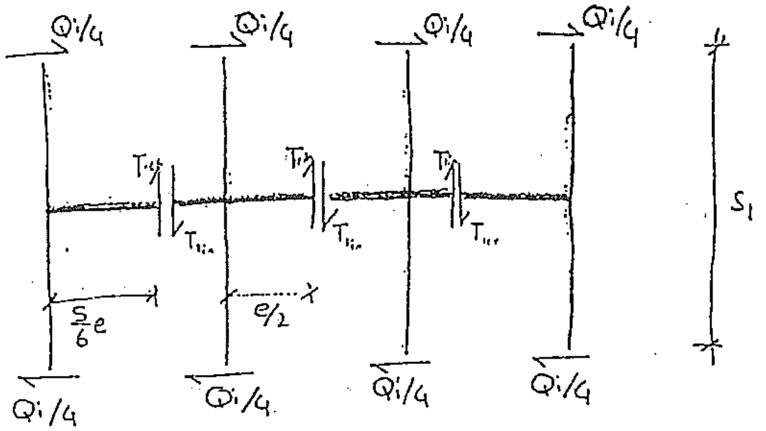
$$\lambda_y = \frac{s_{ky}}{i_y} = \frac{1500}{22{,}5} = 66{,}7 \quad , \quad \lambda_1 = \frac{s_1}{i_1} = \frac{70}{2{,}30} = 30{,}4$$

$$\left.\begin{array}{l} \lambda_{yi} = \sqrt{66{,}7^2 + \frac{4}{2} \cdot 30{,}4^2} = 79{,}3 \\ \lambda_x = \frac{s_{kx}}{i_x} = \frac{350}{8{,}48} = 41{,}3 \end{array}\right\} \lambda_{max} = \lambda_{yi} = 79{,}3 \longrightarrow \sigma_{em} = 905{,}4 \text{ kgf/cm}^2$$

$$P_{all} = \sigma_{em} \cdot 4F = 4 \times 37{,}4 \times 0{,}9054 = 135{,}4 \text{ t}$$

Design the batten plates for the above column. St37.

$$Q_i = \frac{F.\sigma_{cem}}{80} = \frac{(4\times 37,4)\times 1,44}{80} = 2,69\ t$$



→ e and s_1 are unknown. First choose s_1 and then e.

$\lambda_{all} = \frac{S_1}{i_1} = 50 \Rightarrow (S_1)_{max} = 50\times 2,3 = 115$ cm → USE $S_1 = 115$ cm

→ Equate $e = 20i$, since for $e > 20i$, Q_i needs to be magnified.

$e = 20\times 2,3 = 46$ → USE $e = 45$ cm

→ Now, the design shear of battens can be calculated.

For the inner struts;

$T_h\left(\frac{e}{2}+\frac{e}{6}\right) = s_1.\frac{Q_i}{4} \Rightarrow T_h = 0,375.\frac{Q_i.s_1}{e}$ (In TS648 $T_h = 0,4\frac{Q_i.s_1}{e}$)

$T_h = 0,4.\frac{2,69.115}{45} = 2,75\ t$ (carried by two battens on each side)

$T_{1in} = \frac{2,75}{2} = 1,375\ t$ on single interior batten

→ Maximum shear stress on the cross-section of the batten is;

$\tau_{max} = \frac{3}{2}.\frac{T_1}{b.t} \Rightarrow 0,831 = \frac{3}{2}.\frac{1,375}{b.t} \Rightarrow b.t = 2,48\ cm^2$

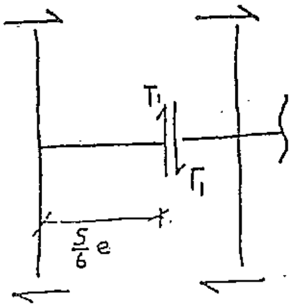
$\tau_{max} \rightarrow \frac{\sigma_{cem}}{\sqrt{3}}$

→ On the exterior battens

$T = 0,3.\frac{Q_i.S_1}{e} = 0,3.\frac{2,69\times 115}{45} = 2,06\ t$

$T_{1ex} = \frac{2,06}{2} = 1,03\ t$

→ Moment created by this shear is;



$$M = \frac{5}{6} . e . T_{1ex} = \frac{5}{6} . 45 . 1,03 = 38,63 \text{ tcm}$$

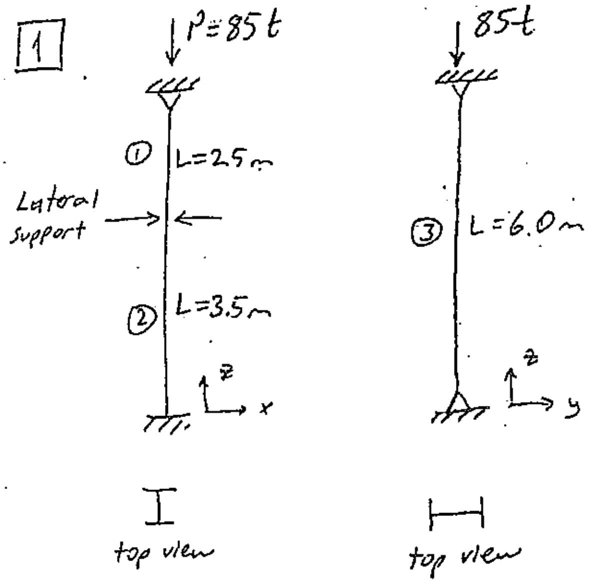
$$\sigma_{com} = \frac{Mc}{I} \Rightarrow 1,44 = \frac{38,63 . b/2}{\frac{1}{12} . t . b^3} \Rightarrow b^2 t \cong 170 \text{ cm}^3$$

Choose $t = 0,5$ cm $\Rightarrow$ $b \cong 18$ cm

· USE 180 x 5 rectangular plates with $s_1 = 115$ cm and $e = 45$ cm

CE 485 RECITATION 5 - Compression Members - Effective lengths, allowable stresses, etc.

1



Determine safety of the column shown under the given loading. The section is I300, St37 steel.

Effective lengths:

① : $2.5 \times 1.0 = 2.5$ $(k=1.0)$

② : $3.5 \times 0.7 = 2.45$ $(k=0.7)$

③ : $6.0 \times 1.0 = 6.0$ $(k=1.0)$

Section Properties (I300)

$F = 69.0\,cm^2$

$i_x = 11.9\,cm$

$i_y = 2.56\,cm$

Calculate slenderness ratios:

$$\lambda_y = \frac{250\,(cm)}{2.56\,(cm)} = 97.7 \quad \text{(Left figure)}$$

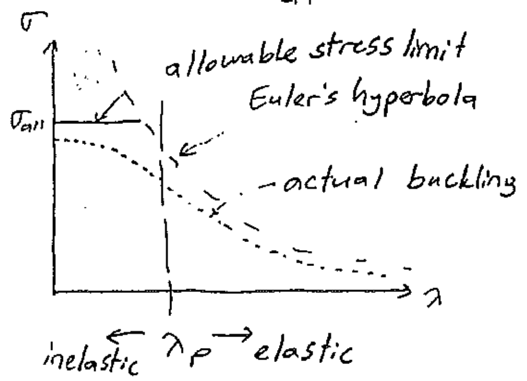
$$\lambda_x = \frac{600}{11.9} = 50.4 \quad \text{(Right figure)}$$

Calculate Euler's critical buckling stress:

$$\sigma_{cr} = \frac{\pi^2 E}{\lambda_{max}^2} = \frac{\pi^2 \times 2100}{97.7^2} = 2.17\,t/cm^2 \quad (>1.44\,!!)$$

Calculate the limit of elastic buckling:

$$\lambda_p = \sqrt{\frac{2\pi^2 E}{\sigma_{a}}}_{y\,2.4} = 131.4$$



What is the actual compressive stress?

$$\sigma = \frac{P}{F} = \frac{85}{69} = 1{,}23\, t/cm^2$$

---

if $\lambda < 20 \rightarrow \sigma_{bem} = \dfrac{\sigma_a}{1.67}$ (Buckling unimportant)

$20 < \lambda < \lambda_p \rightarrow \sigma_{bem} = \left(1 - 0.5\left(\dfrac{\lambda}{\lambda_p}\right)^2\right)\dfrac{\sigma_a}{n}$ (inelastic)

$\lambda > \lambda_p \rightarrow \sigma_{bem} = \dfrac{2}{5}\dfrac{\pi^2 E}{\lambda^2}$ (elastic)

Since the critical slenderness ratio of this column is $97.7 < \lambda_p$, inelastic type of buckling is possible.

$$\sigma_{bem} = \left(1 - 0.5\left(\frac{97.7}{131.4}\right)^2\right)\frac{2{,}4}{n}$$

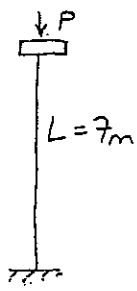
n: Factor of safety given by $\quad n = 1.5 + 1.2\left(\dfrac{\lambda}{\lambda_p}\right) - 0.2\left(\dfrac{\lambda}{\lambda_p}\right)^3$ for $20 \leq \lambda \leq \lambda_p$

$\Rightarrow n = 2.31$

$\Rightarrow \sigma_{bem} = 0{,}752\ t/cm^2$

$\sigma > \sigma_{bem}$ Therefore this column is UNSAFE under the given loading.

$\boxed{2}$



Calculate the allowable axial compressive load for the column made of IPBv220 section & St52.

Supports are the same in both directions.

<u>Section properties:</u>

$F = 149\, cm^2$

$i_x = 9.89\, cm$

$i_y = 5.79\, cm$

$k = 1.0,\ \lambda = \dfrac{1.0 \times 700}{5.79} = 120{,}9$

($\lambda_p = 107.3$ for St52)

$\lambda > \lambda_p \Rightarrow$ elastic buckling

$$\sigma_{bem} = \frac{2}{5}\frac{\pi^2 E}{\lambda^2} = 0{,}567\ t/cm^2$$

$P_{all} = 0{,}567 \times 149 = 84.5$ tons

CE 485
RECITATION
COMPRESSION MEMBERS

Question

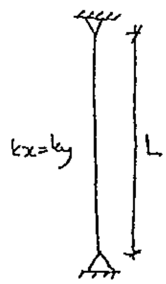
Using a factor of safety of 2,0, design an IPBv section whose stress-strain curve is given, for;

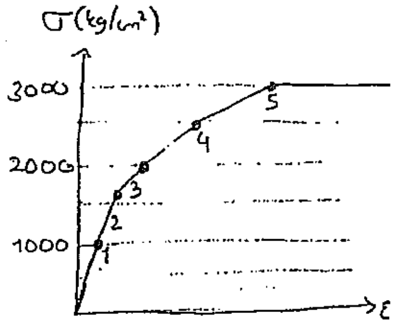
a) L= 1200 cm,

b) L= 250 cm

to carry an axial compressive load of 70 t.

(Hint: You may draw $\sigma_{cr}-\lambda$ curve)





	Strain	Stress
1	0,0005	1000
2	0,0010	1600
3	0,0020	2000
4	0,0045	2500
5	0,0120	3000

Solution

Let's find modulus of elasticity in each curve; (E is found in terms of t/cm^2)

– σ_{cr} is in 0-1;

$$E = \frac{\sigma_1 - \sigma_0}{\varepsilon_1 - \varepsilon_0} = \frac{1}{0{,}0005} = 2000 \ t/cm^2$$

– σ_{cr} is in 1-2;

$$E = \frac{\sigma_2 - \sigma_1}{\varepsilon_2 - \varepsilon_1} = \frac{1{,}6 - 1{,}0}{0{,}001 - 0{,}0005} = 1200 \ t/cm^2$$

– σ_{cr} is in 2-3;

$$E = \frac{\sigma_3 - \sigma_2}{\varepsilon_3 - \varepsilon_2} = \frac{2{,}0 - 1{,}6}{0{,}002 - 0{,}001} = 400 \ t/cm^2$$

– σ_{cr} is in 3-4;

$$E = \frac{\sigma_4 - \sigma_3}{\varepsilon_4 - \varepsilon_3} = \frac{2{,}5 - 2{,}0}{0{,}0045 - 0{,}002} = 200 \ t/cm^2$$

– σ_{cr} is in 4-5;

$$E = \frac{\sigma_5 - \sigma_4}{\varepsilon_5 - \varepsilon_4} = \frac{3{,}0 - 2{,}5}{0{,}0120 - 0{,}0045} = 66{,}7 \ t/cm^2$$

(2)

$$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2} \Rightarrow \lambda = \sqrt{\frac{\pi^2 E}{\sigma_{cr}}}$$

For each E values we define λ;

– For region 1-2 $(1{,}0 \leq \sigma_{cr} \leq 1{,}6)$

$$\lambda_2 = \sqrt{\frac{\pi^2 \times 1200}{1{,}6}} = 86{,}0 \longrightarrow \text{min}$$

$$\lambda_1 = \sqrt{\frac{\pi^2 \times 1200}{1{,}0}} = 108{,}8 \longrightarrow \text{max}$$

– For region 2-3 $(1{,}6 \leq \sigma_{cr} \leq 2{,}0)$

$$\lambda_3 = \sqrt{\frac{\pi^2 \times 400}{2{,}0}} = 44{,}4 \longrightarrow \text{min}$$

$$\lambda_2 = \sqrt{\frac{\pi^2 \times 400}{1{,}6}} = 49{,}7 \longrightarrow \text{max}$$

– For region 3-4 $(2{,}0 \leq \sigma_{cr} \leq 2{,}5)$

$$\lambda_4 = \sqrt{\frac{\pi^2 \times 200}{2{,}5}} = 28{,}1 \longrightarrow \text{min}$$

$$\lambda_3 = \sqrt{\frac{\pi^2 \times 200}{2{,}0}} = 31{,}4 \longrightarrow \text{max}$$

– For region 4-5 $(2{,}5 \leq \sigma_{cr} \leq 3{,}0)$

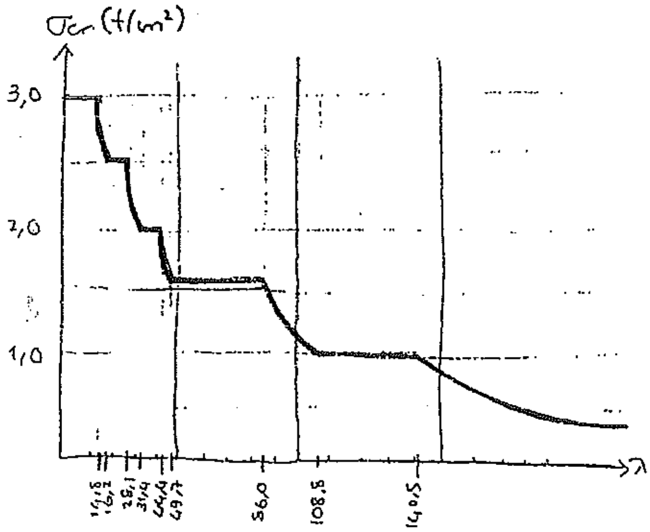
$$\lambda_5 = \sqrt{\frac{\pi^2 \times 66{,}7}{3{,}0}} = 14{,}8 \longrightarrow \text{min}$$

$$\lambda_4 = \sqrt{\frac{\pi^2 \times 66{,}7}{2{,}5}} = 16{,}2 \longrightarrow \text{max}$$

– For region 0-1 $(0 \leq \sigma_{cr} \leq 1{,}0)$

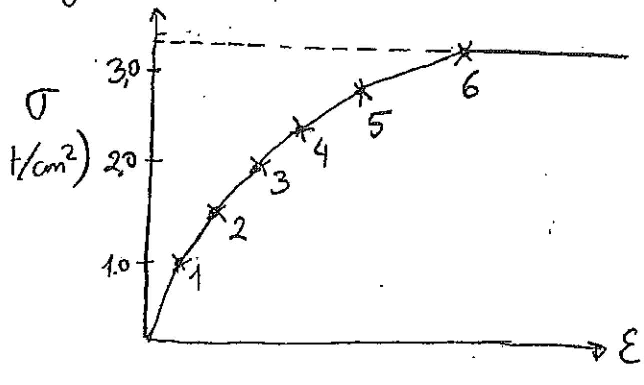
$$\lambda_1 = \sqrt{\frac{\pi^2 \times 2000}{1{,}0}} = 140{,}5$$

	σ_{cr}	λ
0		
1	1,0	140,5
	1,0	108,8
2	1,6	86,0
	1,6	49,7
3	2,0	44,4
	2,0	31,4
4	2,5	28,1
	2,5	16,2
5	3,0	14,8



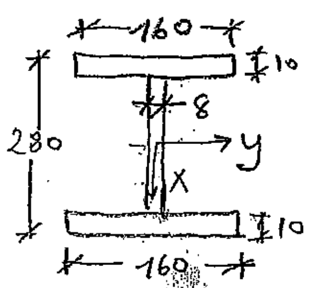
CE 485
RECITATION #5

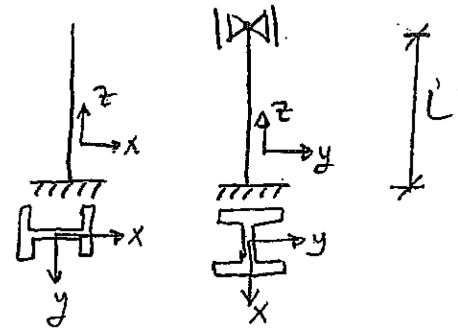
Determine the P_{cr} for the given column whose $\sigma-\varepsilon$ distribution is given as follows:



Point	ε	σ (t/cm²)
1	0,0005	1.0
2	0,0009	1,6
3	0,0013	2,0
4	0,0021	2,4
5	0,0040	3,0
6	0,0120	3,5

structural system :





Carry out your calculations for

a) L = 2500mm

b) L = 4500mm

(all dimensions are in mm)

- Calculate the mechanical properties of the section

$$I_y = \frac{1}{12} \times 8 \times 260^3 + 2 \times \left[\frac{1}{12} \times 160 \times 10^3 + 160 \times 10 \times (140-5)^2 \right] = 70064000 \text{ mm}^4$$

$$I_x = \frac{1}{12} \times 260 \times 8^3 + 2 \times \frac{1}{12} \times 10 \times 160^3 = 6837760 \text{mm}^4$$

$$A = 260 \times 8 + 2 \times 160 \times 10 = 5280 \text{ mm}^2$$

$$i_x = \sqrt{I_x/A} = 36 \text{ mm} \quad ; \quad i_y = \sqrt{I_y/A} = 115,2 \text{ mm}$$

a) L = 2500mm ; $\lambda = \frac{k \times L}{i}$; k = effective length ratio

$$\left.\begin{array}{l} \lambda_x = \frac{k_x \cdot L}{i_x} = \frac{0,7 \times 2500}{36} = 48.61 \\ \lambda_y = \frac{k_y \cdot L}{i_y} = \frac{2,0 \times 2500}{115,2} = 43,4 \end{array}\right\} \quad \lambda_{cr} = \max[\lambda_x ; \lambda_y] = \underline{48.61 = \lambda_x}$$

↳ buckling in x-direction is more critical !!!

- Assume σ_{cr} is between 2 & 3 → $1.6 < \sigma_{cr} < 2.0$

$E = \frac{2-1.6}{0.0013-0.0009} = 1000\ t/cm^2$; $\sigma_{cr} = \frac{\pi^2 E}{\lambda_{max}^2} = \frac{\pi^2 \times 1000}{48.61^2} = 4.18\ t/cm^2$

(wrong assumption! σ_{cr} is out of range 2-3)

- Assume σ_{cr} is between 3 & 4 → $2.0 < \sigma_{cr} < 2.4$

$E = \frac{2.4-2}{0.0021-0.0013} = 500\ t/cm^2 \rightarrow \sigma_{cr} = \frac{\pi^2 E}{\lambda_{max}^2} = \frac{\pi^2 \times 500}{48.61^2} = 2.09\ t/cm^2 \geq 2.0\ t/cm^2$ ✓

$< 2.4\ t/cm^2$ ✓

↳ assumption is correct $\underline{\sigma_{cr} = 2.09\ t/cm^2}$; $P_{cr} = A \cdot \sigma_{cr} = 52.8 * 2.09 = \underline{110.35\ t}$

b) $L = 4500mm$, $\lambda_{max} = \lambda_x$ (as found in part a))

$\lambda_x = \frac{k_x \cdot L}{i_x} = \frac{0.7 \times 4500}{36} = 87.5$

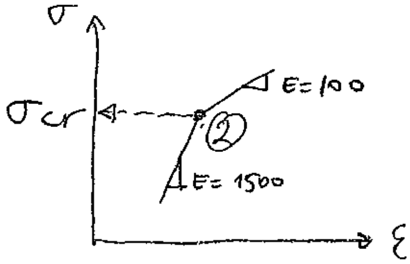
- Assume σ_{cr} is between 1&2 → $1.0\ t/cm^2 < \sigma_{cr} < 1.6\ t/cm^2$

$E = \frac{1.6-1.0}{0.0009-0.0005} = 1500\ t/cm^2 \rightarrow \sigma_{cr} = \frac{\pi^2 \times 1500}{87.5^2} = 1.93\ t/cm^2$

(wrong assumption!! σ_{cr} is out of range 1-2)

- Assume σ_{cr} is between 2&3 → $1.6\ t/cm^2 < \sigma_{cr} < 2.0\ t/cm^2$

$E = 1000\ t/cm^2 \rightarrow \sigma_{cr} = \frac{\pi^2 * 1000}{87.5^2} = 1.29\ t/cm^2$ (wrong assumption!! σ_{cr} is out of range 2-3)

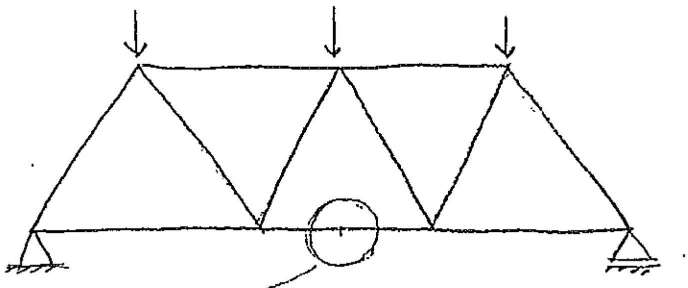


$\sigma_{cr} = 1.6\ t/cm^2$

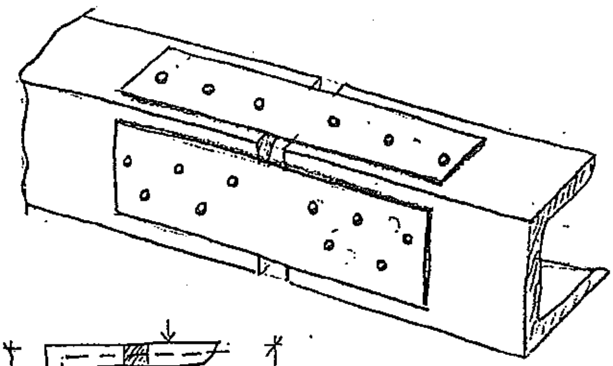
$P_{cr} = 1.6 * 52.8 = \underline{\underline{84.48\ t}}$

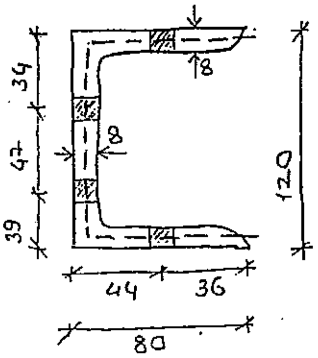
CE 485
RECITATION
TENSION MEMBERS (4)

08/03/06

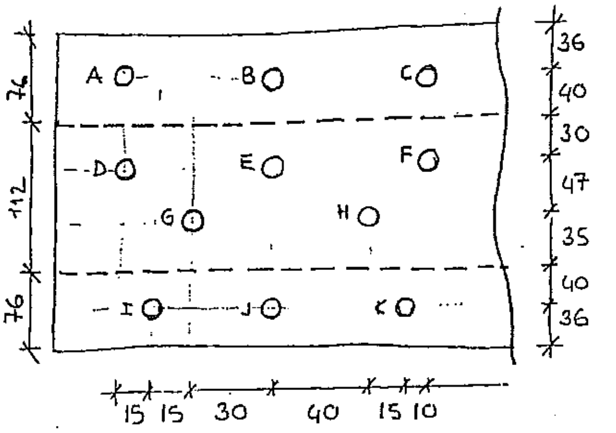


Calculate the maximum tensile load that can be carried by the member shown considering plate tension St37, M16 bolts.





opened through centerline →



Hole diameter = Bolt diameter + 1 (mm)

$d_1 = 16 + 1 = 17$ mm

$$A_{net} = A_{gross} - n.d_1.t + \sum \frac{s^2.t}{4g}$$

$n \rightarrow$ number of holes

$s \rightarrow$ spacing between bolts parallel to tensile force

$g \rightarrow$ distance between bolts perpendicular to tensile force

$t \rightarrow$ plate thickness

2/2

Calculate the minimum net area (for [120 → $A_g = 17\ cm^2$)

1) Check $0.85 A_g$

$A_g = 80 \times 8 \times 2 + 104 \times 8 = 2112\ mm^2$

$0{,}85 A_g = 1795{,}2\ mm^2$

2) 1-holed path (I or K)

$A_{n1} = A_g - n.d_1.t = 2112 - 1 \times 17 \times 8 = 1976\ mm^2$

3) 2-holed path (AD or CF)

$A_{n2} = 2112 - 2 \times 17 \times 8 = 1840\ mm^2$

4) 3-holed path (BEJ)

$A_{n3} = 2112 - 3 \times 17 \times 8 = 1704\ mm^2$

5) 4-holed path

BEGI
ADGI $\longrightarrow A_{n41} = 2112 - 4 \times 17 \times 8 + \left(\frac{30^2}{4 \times 47} + \frac{15^2}{4(35+40)}\right) \times 8 = 1612{,}3\ mm^2$

CFHK $\longrightarrow A_{n42} = 2112 - 4 \times 17 \times 8 + \left(\frac{25^2}{4 \times 47} + \frac{15^2}{4(35+40)}\right) \times 8 = 1600{,}6\ mm^2$

BEGJ $\longrightarrow A_{n43} = 2112 - 4 \times 17 \times 8 + \left(\frac{30^2}{4 \times 47} + \frac{30^2}{4(35+40)}\right) \times 8 = 1630{,}3\ mm^2$

BEHJ $\longrightarrow A_{n44} = 2112 - 4 \times 17 \times 8 + \left(\frac{40^2}{4 \times 47} + \frac{40^2}{4 \times 75}\right) \times 8 = 1678{,}75\ mm^2$

$(A_{net})_{min} = A_{n42} = 1600{,}6\ mm^2$

Allowable axial tensile load

$P_{all} = \sigma_{all} \times (A_{net})_{min}$ $\quad\left(\sigma_{all} = \sigma_{cem} = \frac{\sigma_a}{FS} = \frac{2{,}4}{1{,}67} = 1{,}44\ t/cm^2\right)$

$= 1{,}44 \times 16$

$P_{all} = 23{,}04\ t$ //

CE388 – Homework 4 – Solutions

Q1.

i. HEA 550
ii. HEB 550

Q2.

s (m)	σ_B (t/cm^2)	σ_B (t/cm^2)	σ_B (t/cm^2)
-	**IPE500**	**HEM500**	**Built-up**
50	1.44	1.44	1.44
100	1.44	1.44	1.44
150	1.44	1.44	1.44
200	1.44	1.44	1.44
250	1.44	1.44	1.44
300	1.44	1.44	1.44
350	1.44	1.44	1.44
400	1.44	1.44	1.44
450	1.44	1.44	1.44
500	1.44	1.44	1.44
550	1.44	1.44	1.44
600	1.44	1.44	1.44
650	1.44	1.44	1.44
700	1.34	1.44	1.44
750	1.25	1.44	1.44
800	1.18	1.44	1.44
850	1.11	1.44	1.38
900	1.05	1.44	1.31
950	0.99	1.44	1.25
1000	0.94	1.44	1.22
1050	0.90	1.44	1.18
1100	0.86	1.44	1.14
1150	0.82	1.44	1.09
1200	0.78	1.44	1.05
1250	0.75	1.44	1.00
1300	0.72	1.44	0.95
1350	0.70	1.44	0.90
1400	0.67	1.44	0.85
1450	0.65	1.44	0.81
1500	0.63	1.44	0.78

CE388 - FUNDAMENTALS OF STEEL DESIGN

2011-2012 Spring Term

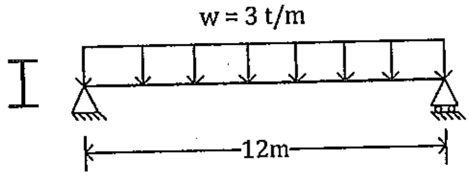
Homework IV

Due date: 3 May 2012

Submit your homework at class time or alternatively to Özkan Kale before 11:59am. Fifty percent penalty applies to homeworks submitted on 3 May 2012 between 11:59am and 17:00pm. Homeworks submitted thereafter will receive no credit.

1. For the two cases shown below determine the lightest I-section rolled shape that satisfies bending and shear provisions of TS648.

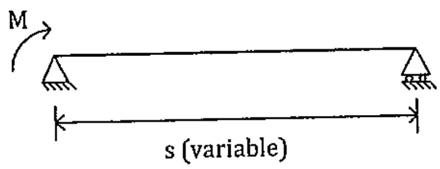
    i. Beam is continuously supported laterally
    ii. Beam is supported laterally at the ends only



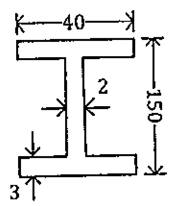
St37 steel
EY Loading

Note: Neglect self weight of the beam.

2. For a beam under the following loading condition plot σ_B (allowable bending stress according to TS648 provisions) versus s (unbraced length). The beam has lateral supports at the ends only. St37 Steel, EY Loading. Plot s such that it varies between 0cm to 1500cm. Calculate σ_B values for every 50cm of s (i.e. for s = 0, 50, 100, 150, ..., 1500cm). Tabulate your results.



    i. IPE 500 section
    ii. HEM 500 section
    iii. Built-up section d = 150cm, b_f = 40cm, t_f = 3cm, t_w = 2cm



CE 388 - HW2 - Solutions

2) Let's assume $a = 36.6$ cm

$I = 43322.53$ cm^4

$A = 210.6$ cm^2

$i = 14.34$ cm

$\lambda = 87.9$, $n = 2.24$, $\sigma_{bem} = 0.831 t/cm^2 \rightarrow P_{all} = 175$ t ✓

Answer: $a = 36.6$ cm //

3) HEA 400

$A = 159$ cm^2

$J_y = 45070$ cm^4 , $i_y = 16.8$ cm

$J_z = 8564$ cm^4 , $i_z = 7.34$ cm

Weak direction is critic.

$\lambda = 41 < \lambda_p = 131$

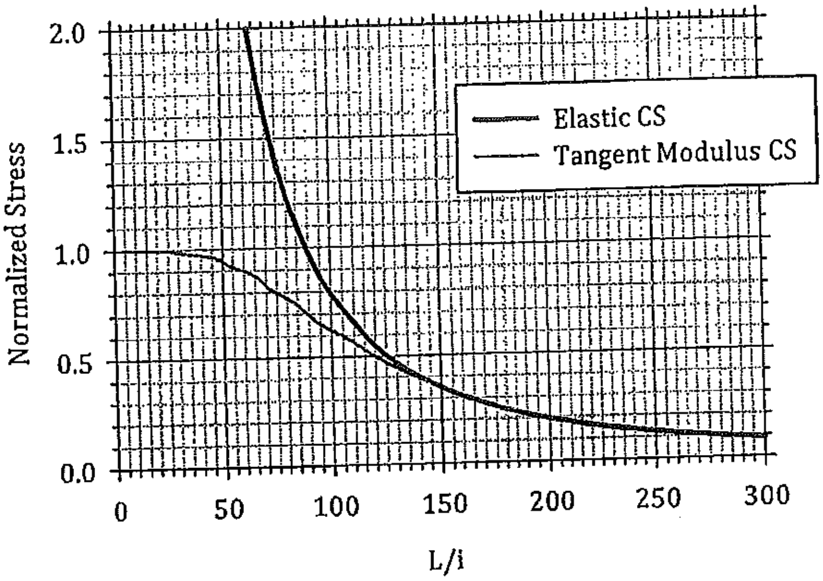
$n = 1.87$, $\sigma_{bem} = 1.22 t/cm^2 \rightarrow P_{all} = 194$ t //

CE 388 – HW2 – Solutions

1)

a) $E = 20000 \text{ MPa}$

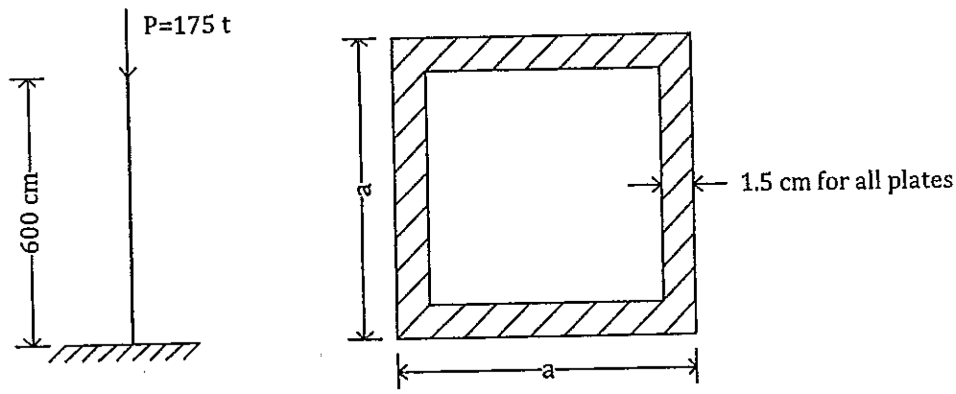
b)



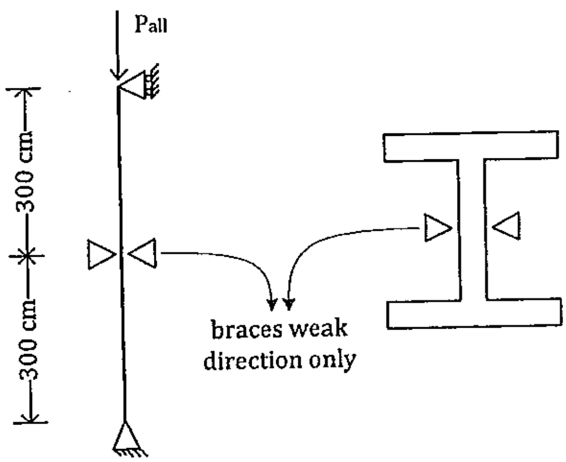
CE388 - FUNDAMENTALS OF STEEL DESIGN

2011-2012 Spring Term

2. For the box column shown below determine the value of "a" such that the column can safely carry a load of 175 tons according to TS 648 Provisions. EY Loading, St37 Steel. Use recommended K values.



3. For the HEA 400 column shown below determine the allowable load P (P_{all}) according to TS 648 Provisions. EY Loading, St37 Steel.



CE388 - FUNDAMENTALS OF STEEL DESIGN

2011-2012 Spring Term

Homework II

Due date: 29 March 2012

Submit your homework at class time or alternatively to course instructors before 11:59am. Fifty percent penalty applies to homeworks submitted on 29 March 2012 between 11:59am and 17:00pm. Homeworks submitted thereafter will receive no credit.

1. Consider a pin ended I-section column constructed with this type of steel. Assuming that the material fully yields at 250 MPa (σ_y=250 MPa), calculate the following:

   a) Determine the initial elastic modulus (E) for this type of steel.

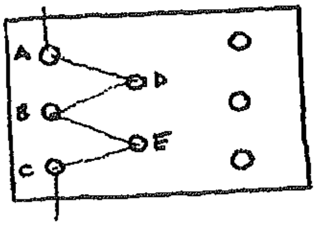
   b) On a single graph plot normalized stress (critical stress divided by the yield stress) versus slenderness (L/i) for the following cases:

      Case 1: Consider the elastic critical stress
      Case 2: Consider the tangent modulus critical stress
      Your (L/i) values should change between zero and 300 for case 1 and between 80 and 300 for case 2.

Strain	Stress (MPa)
0	0
0.0001	20
0.0002	40
0.0003	60
0.0004	80
0.0005	100
0.0006	119
0.0007	137
0.0008	154
0.0009	169
0.0010	183
0.0011	196
0.0012	207
0.0013	217
0.0014	226
0.0015	233
0.0016	239
0.0017	244
0.0018	247
0.0019	249
0.0020	250
0.0021	250
0.0022	250
0.0030	250

2) <u>Assumption-1 :</u>

hole diameter = 16 + 1 = 17 mm



$\rightarrow \frac{T_{max}}{2}$

t = 10 mm (upper or lower plate)

$A_{gross} = 23.8\ cm^2$

$A_{ADBEC} = 22.7\ cm^2$ (most critical path)

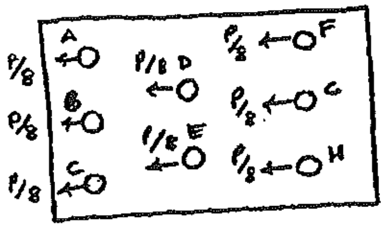
$0.85\ A_{gross} > A_{ADBEC}$ ✓

$\sigma_{gem} = 1.44\ t/cm^2$

$T_{max} = 65.38\ t$

<u>Assumption-2 :</u>

hole diameter = 16 + 1 = 17 mm



$\rightarrow \frac{T_{max}}{2} = P$

t = 10 mm (upper or lower plate)

$A_{gross} = 23.8\ cm^2$

$A_{FGH} = 22.9\ cm^2 \rightarrow$ Tension load $= P = \frac{T_{max}}{2}$ in this path

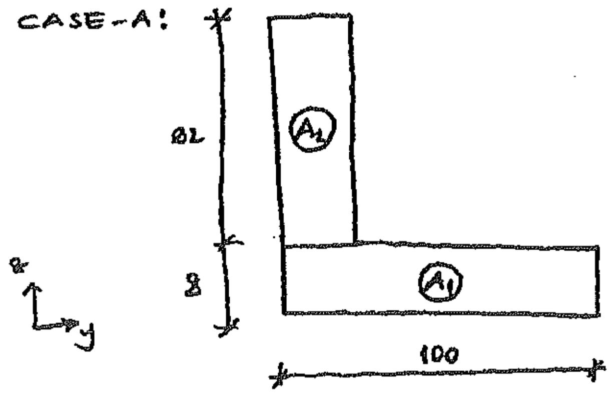
$0.85\ A_{gross} > A_{FGH}$ ✓

$\sigma_{gem} = 1.44\ t/cm^2$

$T_{max} = 65.95\ t$

CE 388 – HW1 – Solutions

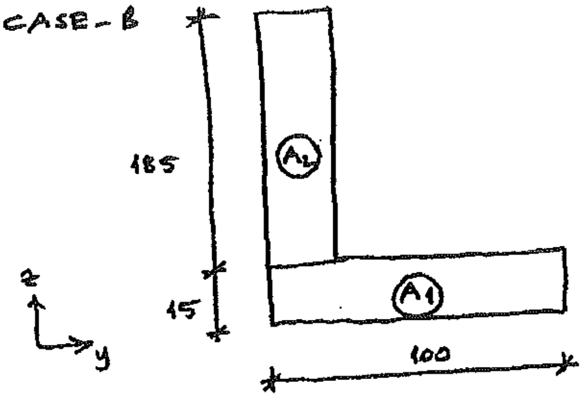
1) CASE–A:



$A_1 = 80.0\ cm^2$

$A_2 = 73,6\ cm^2$

CASE–B



$A_1 = 150.0\ cm^2$

$A_2 = 277.5\ cm^2$

J_{yz}: Product of inertia $\rightarrow$ $\tan 2\alpha = -\dfrac{2 J_{yz}}{(I_y - I_z)}$

$J_u = J_y \cos^2\alpha - 2 J_{yz} \sin\alpha \cos\alpha + J_z \sin^2\alpha$

$J_v = J_z \cos^2\alpha + 2 J_{yz} \sin\alpha \cos\alpha + J_y \sin^2\alpha$

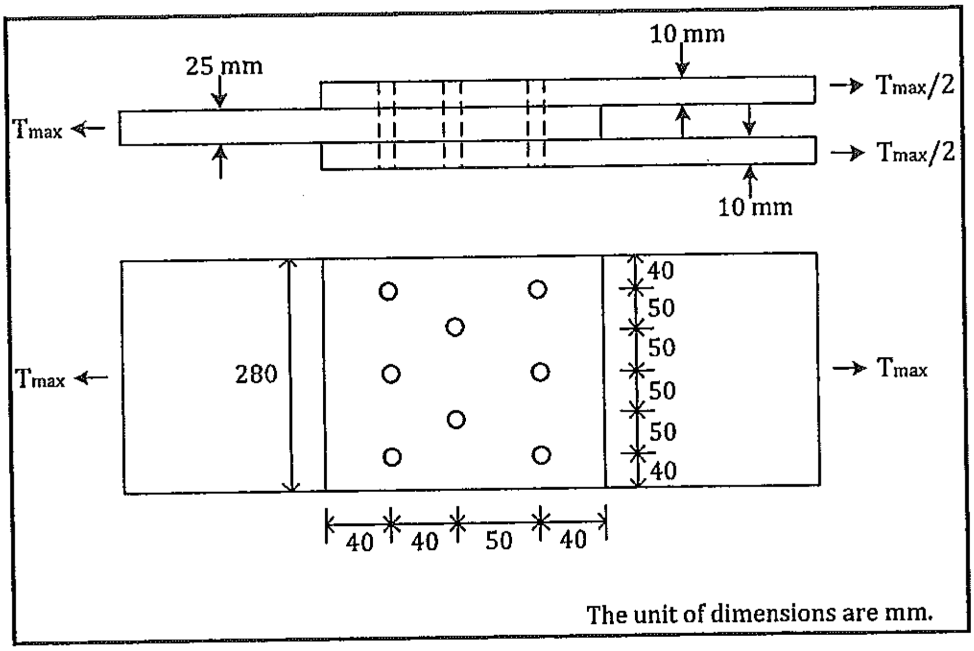
		CASE A		CASE B	
	Unit	Calculated	Tabulated	Calculated	Tabulated
y_s	cm	2,80	2,74	2,24	2.22
z_s	cm	2,80	2,74	7,24	7,16
J_y	cm^4	148.17	144.84	1767,95	1758,00
J_z	cm^4	148,17	144.84	306,07	299,10
i_y	cm	3.11	3,06	6.43	6,40
i_z	cm	3,11	3,06	2,68	2,64
α	°	45	45	14,75	14,57
J_u	cm^4	236.34	230.19	1876.96	1865,00
J_v	cm^4	60,00	59,5	197,06	193,10
i_u	cm	3.92	3,85	6,63	6.59
i_v	cm	1,98	1,96	2,15	2.12

*) The main reason for the differences between calculated and tabulated values is rounded shapes of tabulated sections.

CE388 - FUNDAMENTALS OF STEEL DESIGN

2011-2012 Spring Term

2. A tension member constructed by connecting 3 plates as shown below. Determine the maximum tension load permitted according to TS648 Specification. St37 Steel. All bolts have 16 mm diameter.



CE388 - FUNDAMENTALS OF STEEL DESIGN

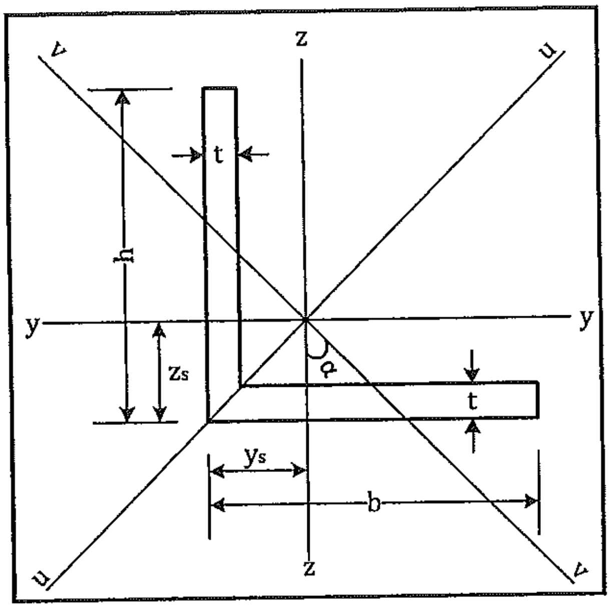
2011-2012 Spring Term

Homework I

Due date: 15 March 2012

Submit your homework at class time or alternatively to Özkan Kale before 11:59am. Fifty percent penalty applies to homeworks submitted on 15 March 2012 between 11:59am and 17:00pm. Homeworks submitted thereafter will receive no credit.

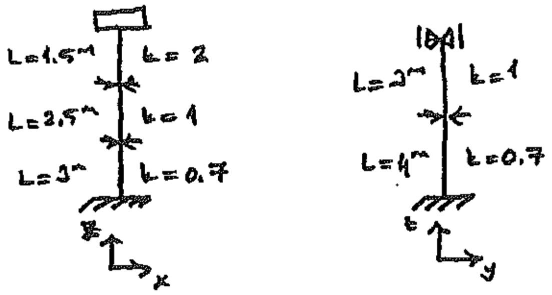
1. For the angle sections shown below calculate the following properties and compare it with the tabulated values. Comment on your findings.



Case	h (mm)	b (mm)	t (mm)
A	100	100	8
B	200	100	15

a. Location of centroid (y_s and z_s)
b. Moment of inertia with respect to z and y axes (J_y and J_z)
c. Radius of gyration with respect to z and y axes (i_y and i_z)
d. Moment of inertia with respect to the principal axes u and v (J_u and J_v)
e. Radius of gyration with respect to u and v axes (i_u and i_v)
f. Angle α between z-z and v-v axes

(3)

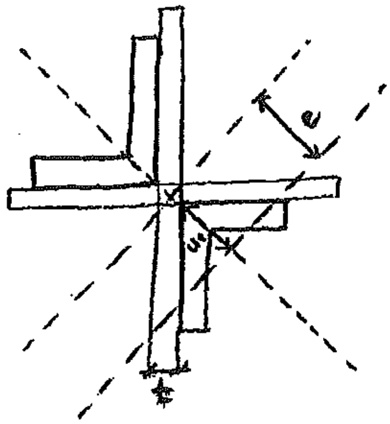


$(kL)_{max} = 3m$ $(kL)_{max} = 3m$

Since the effective lengths are same about x and y axes, the slenderness ratio does not depend on the orientation of the cross-section

$$\lambda_{max} = \frac{300}{i_y} = \frac{300}{2.56} = 117.2 \rightarrow P = 41.2^t$$

(4)



$$e = u_1 + t/\sqrt{2}$$

$$i = \frac{\sqrt{i_\omega^2 + e^2}}{1.15}$$

$$\lambda_1 = \frac{s_1}{i_1} \le 50$$

$$\lambda_1 = \frac{50 i_1}{i_1} = 50 \checkmark$$

L 30x30x3 $\rightarrow$ $e = 1.181 + 0.3/\sqrt{2} = 1.393cm$, $i = \frac{\sqrt{0.58^2 + 1.393^2}}{1.15} = 1.31 cm$

$e_y + \frac{t}{2} = 0.835 + \frac{0.3}{2} = 0.985 cm$, $i_x = \frac{\sqrt{0.90^2 + 0.985^2}}{1.15} = 1.16 cm$

L 40x40x4 $\rightarrow$ $e = 1.583 + 0.4/\sqrt{2} = 1.866 cm$, $i = \frac{\sqrt{0.77^2 + 1.866^2}}{1.15} = 1.76 cm$

$e_y + \frac{t}{2} = 1.12 + \frac{0.4}{2} = 1.320 cm$, $i_x = \frac{\sqrt{1.21^2 + 1.320^2}}{1.15} = 1.56 cm$

L 70x70x8 $\rightarrow$ $e = 2.84 + 0.8/\sqrt{2} = 3.406 cm$, $i = \frac{\sqrt{1.35^2 + 3.406^2}}{1.15} = 3.19 cm$

$e_y + \frac{t}{2} = 2.01 + \frac{0.8}{2} = 2.41 cm$, $i_x = \frac{\sqrt{2.10^2 + 2.41^2}}{1.15} = 2.78 cm$

⋮

$e > e_y + \frac{t}{2}$ and $i > i_x$

(1.4) (1.15)

Answers of HW-2

(1) **AB : sway-permitted**

$$G_A = 10$$

$$G_B = \frac{\frac{29210}{600} + \frac{9800}{500}}{\frac{4250}{500} + \frac{162}{400} \times 0.5} = 7.85$$

$k \cong 2.8$

$$\lambda = \frac{2.8 \times 600}{15.7} = 107$$

$$\omega = 2.11$$

$$P_{AB} = 80^{t}$$ ✓

BC : sway-permitted

$$G_B = 7.85$$

$$G_C = \frac{\frac{9800}{500}}{\frac{5740}{900} \times 0.5} = 6.15$$

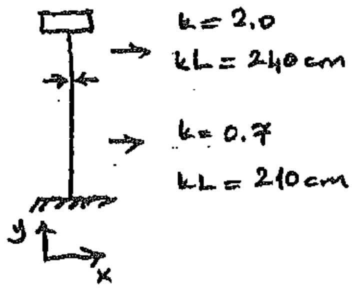
$k \cong 2.55$

$$\lambda = \frac{2.55 \times 500}{11.9} = 107$$

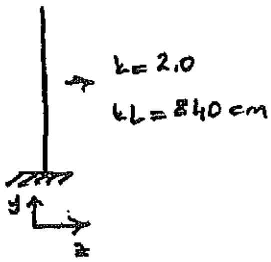
$$\omega = 2.11$$

$$P_{BC} = 46.6^{t}$$ ✓

(2)



$$\lambda = \frac{240}{4.66} = 51.5$$



$$\lambda = \frac{840}{24.1} = 34.6$$ → Assume σ_{cr} is between C and D. Then,

$$\sigma_{cr} = 2.12 \ t/cm^2$$

$$P_{cr} = 2.12 \times 156 = 331^{t}$$ ✓

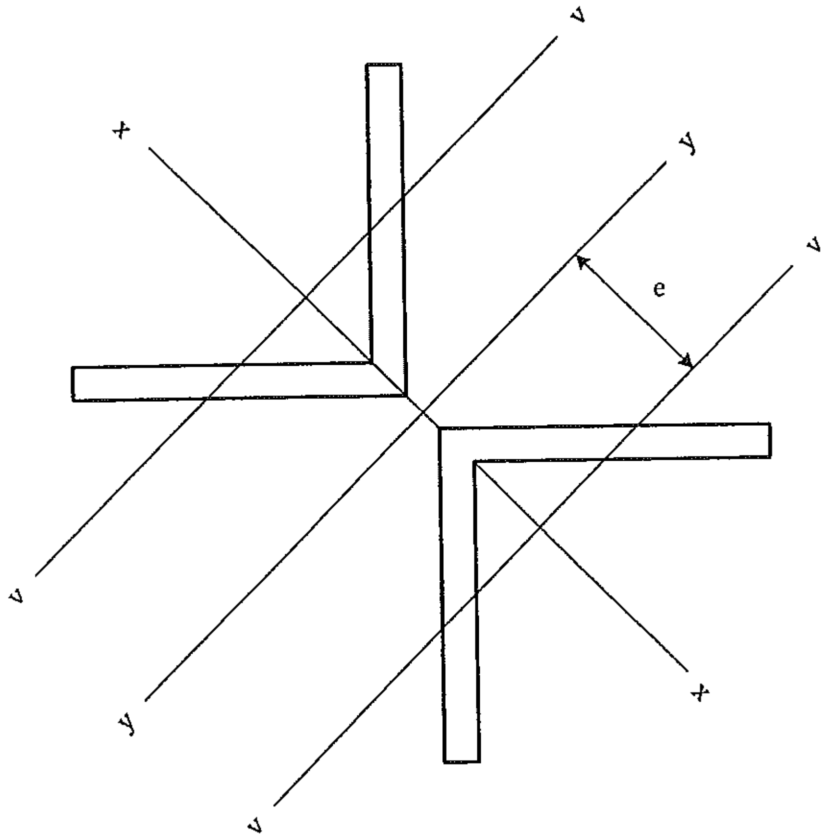
Add) You can also consider the recommended values of k.

CE388 - FUNDAMENTALS OF STEEL DESIGN

2012-2013 Fall Term

4. Show that Group II equal leg columns are not likely to buckle around y-y axis using ratios between weak and strong axis radius of gyration as well as ratios between eccentricity (distance between outer corner where two equal legs combine together and geometric center point). Use average of 5 different angle sizes to find ratios. Assume that the two equal leg angles are connected together using plates located at $s_1=50*i_1$ and the plate thicknesses are equal to the leg thicknesses. Assume the actual radius of gyration around the y-y axis of the Group II column will be 15% smaller than the radius of gyration assuming two angles work fully composite.

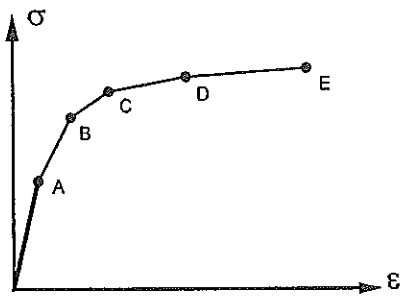
$$i = \frac{\sqrt{i_v^2 + e^2}}{1.15}$$

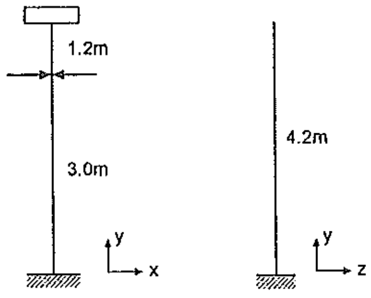


CE388 - FUNDAMENTALS OF STEEL DESIGN

2012-2013 Fall Term

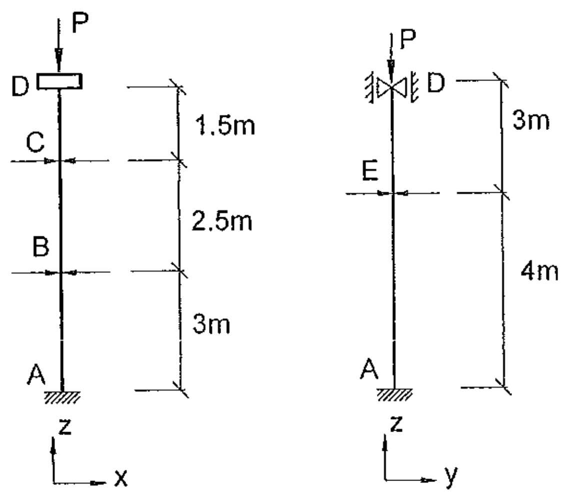
2. Calculate the critical load carrying capacity of the column shown below, which is made of IPE 600 section. The stress-strain diagram for the steel is also provided.





	Stress (t/cm²)	Strain
A	0.4	0.0002
B	0.8	0.0006
C	1.6	0.0016
D	2.4	0.0030
E	4.8	0.0080

3. Calculate the maximum allowable axial compressive load (P) for the column AD shown below which is made of I300 section. Use St37 steel. Lateral supports are provided at points B, C and E.



CE388 - FUNDAMENTALS OF STEEL DESIGN

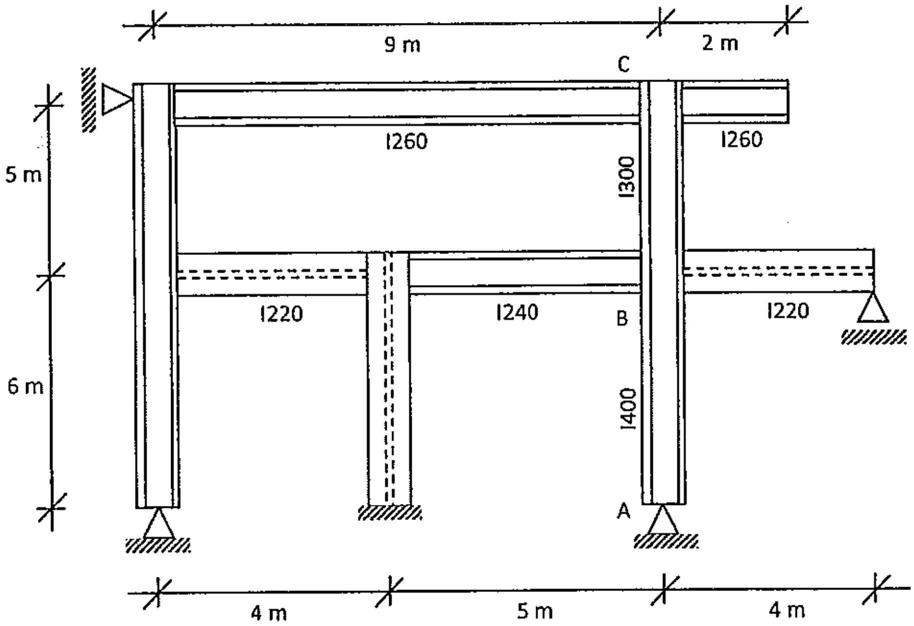
2012-2013 Fall Term

Homework II

Due date: 23rd November 2012

Submit your homework at class time or alternatively to Özkan Kale before 11:59AM. Fifty percent penalty applies to homeworks submitted on 23st November 2012 between 11:59AM and 17:00PM. Homeworks submitted thereafter will receive no credit.

1. Calculate the allowable axial load carrying capacities of columns AB and BC. St37 Steel, EY loading.



Answers of HW-1

(1)

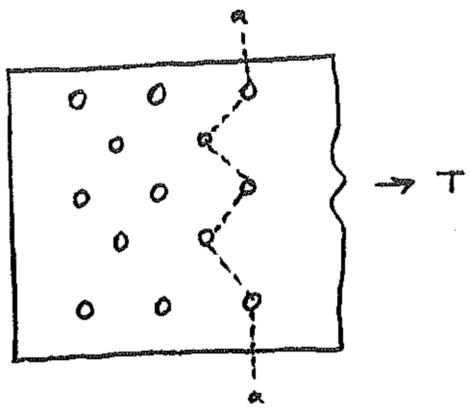
$A = 13.6 \text{ cm}^2$

$0.85\, A_g = 11.56 \text{ cm}^2$

$A_n = 13.6 - (1.8 + 0.1) \times 1.0 = 11.7 \text{ cm}^2$

$T_{all} = 2 \times 11.56 \times 1.44 = 33.3^t$

(2)



$A_g = 46 \times 1.2 = 55.2 \text{ cm}^2$

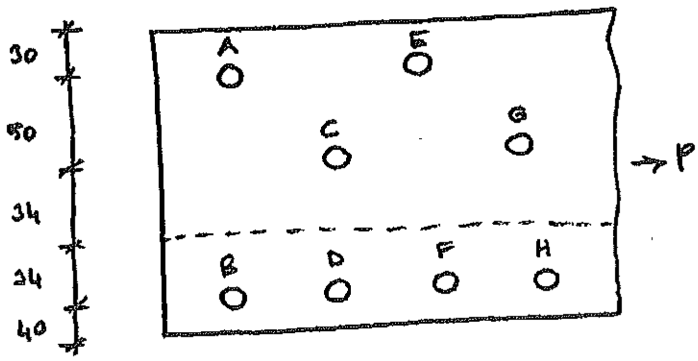
$0.85\, A_g = 46.92 \text{ cm}^2$

For path a-a,

$A = 46.58 \text{ cm}^2$

$T_{all} = 46.58 \times 1.44 = 67.1^t$

(3)



$A_g = 188 \times 12 = 2256 \text{ mm}^2$

$0.85\, A_g = 1917.6 \text{ mm}^2$

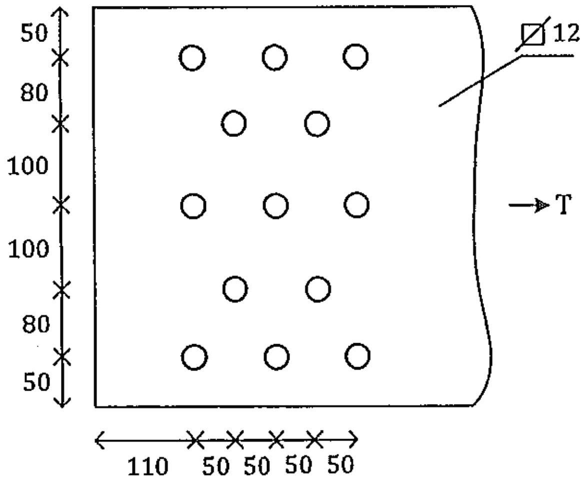
$A_{ECD} = 1794 \text{ mm}^2$

$P_{all} = 2 \times 17.94 \times 1.44 = 51.7^t$

CE388 - FUNDAMENTALS OF STEEL DESIGN

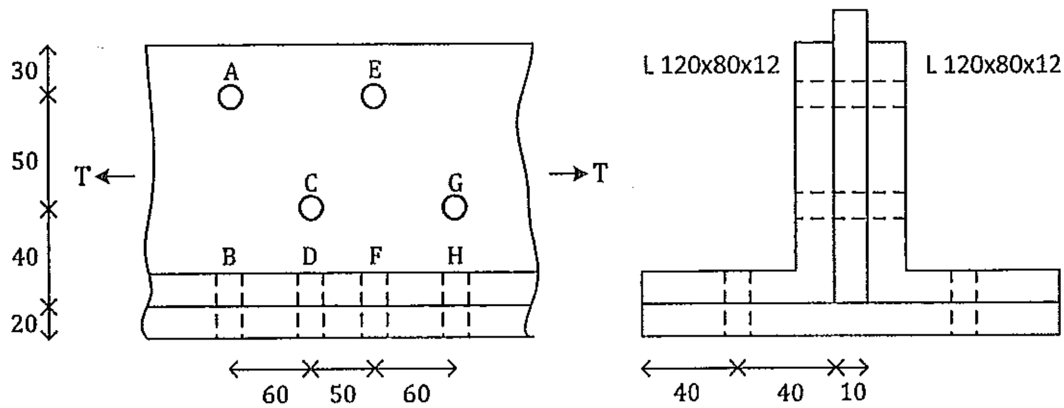
2012-2013 Fall Term

2. Determine the maximum tension load permitted according to TS648 Specification for the 12mm thick plate. St37 Steel. All bolt holes are 20 mm in diameter.



The unit of dimensions are mm.

3. A tension member composed of 2L120x80x12 sections is connected to each other using a T shaped batten plate as shown in the figure below. St37 Steel and M16 bolts are utilized (assume +1mm for the hole diameters). Calculate the allowable tensile load for this member. (Assume that the batten plates do not fail under T load).



The unit of dimensions are mm.

CE388 - FUNDAMENTALS OF STEEL DESIGN

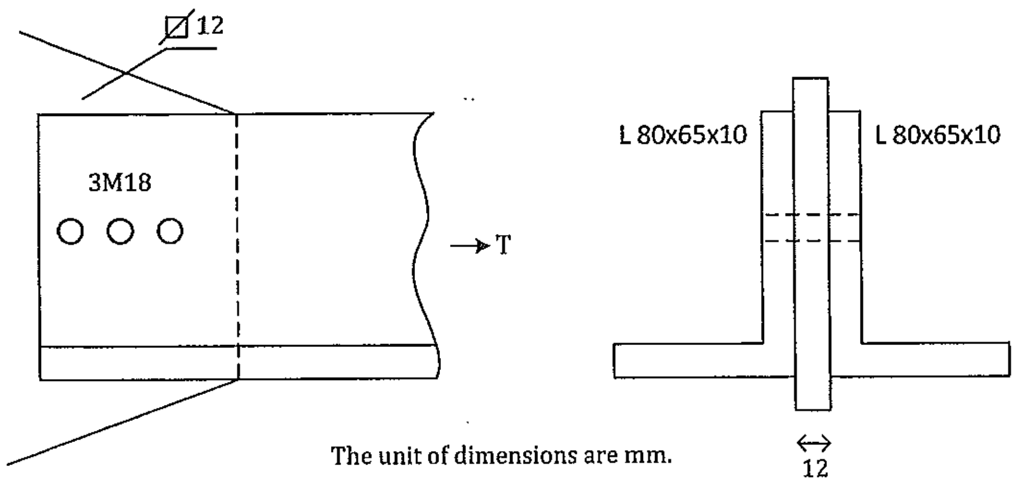
2012-2013 Fall Term

Homework I

Due date: 13rd November 2012

Submit your homework at class time or alternatively to Özkan Kale before 11:59am. Fifty percent penalty applies to homeworks submitted on 13rd November 2012 between 11:59am and 17:00pm. Homeworks submitted thereafter will receive no credit.

1. Determine the allowable tensile force for the double L80x65x10 angle section given using TS648 specifications. St37 Steel. All bolt holes are ϕ19 mm in diameter. (Ignore connection plate and bolt capacities and just consider double angle tensile load capacity in your allowable T force calculations).



The unit of dimensions are mm.

4) Uniform Torsion (St. Venant's Torsion)

$$K_t = (J) = \frac{1}{3}\sum b_i \cdot t_i^3 = \frac{1}{3}\left[2.12{,}5.1{,}62^3 + (30 - 2.1{,}62).1{,}08^3\right] = 46{,}67 \text{ cm}^4$$

$$\tau_4 = \frac{T_{max}.t}{K_t} = \frac{5.1{,}62}{46{,}67} = 0{,}174 \text{ t/cm}^2 \text{ at flange}$$

$$\tau_5 = \frac{T_{max}.t}{K_t} = \frac{5.1{,}08}{46{,}67} = 0{,}116 \text{ t/cm}^2 \text{ at web}$$

Therefore;

$$\tau_{\substack{flange\\total}} = \tau_2 + \tau_3 + \tau_4 = 0{,}023 + 0{,}013 + 0{,}174 = 0{,}21 \text{ t/cm}^2$$

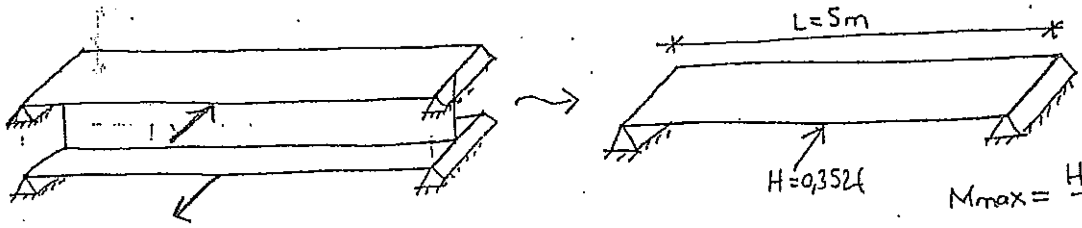
$$\tau_{\substack{web\\total}} = \tau_1 + \tau_5 = 0{,}077 + 0{,}116 = 0{,}193 \text{ t/cm}^2$$

Axial Stresses

1) Due to P;

$$\sigma_{max} = \frac{M_{max}}{W_x} = \frac{625}{653} = 0{,}957 \text{ t/cm}^2$$

2) Due to H;



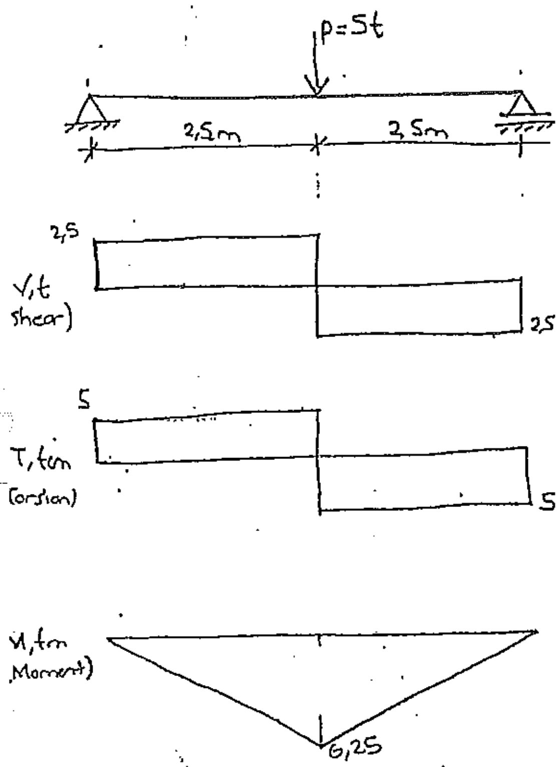
$$M_{max} = \frac{H.L}{4} = 44 \text{ tcm}$$

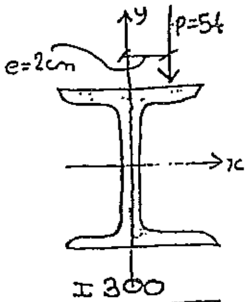
$$\sigma_{max} = \frac{M_{maxH}}{W_y/2} = \frac{44}{72{,}2/2} = 1{,}219 \text{ t/cm}^2 \rightarrow \text{due to warping only!}$$

approximate

$$\therefore \sigma_{max_{total}} = 0{,}957 + 1{,}219 = 2{,}176 \text{ t/cm}^2$$

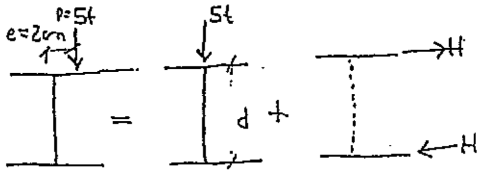
CE485
RECITATION
TORSION





I 300
$h = 300$ mm
$b = 125$ mm
$t_w = 10{,}8$ mm
$t_f = 16{,}2$ mm
$F = 69{,}1\ cm^2$
$I_x = 9800\ cm^4$
$I_y = 451\ cm^4$
$S_x = Q_x = 381\ cm^3$
$W_x = 653\ cm^3$
$W_y = 72{,}2\ cm^3$

Calculate the axial and shear stresses for the beam shown made of St 37. EY Loading. The beam is laterally supported.



$P.e = H.d$
$5.2 = H.(h - t_f)$
$10 = H.(30 - 1{,}62) \Rightarrow H = 0{,}352t$

Shear Stresses

1) Vertical Shear (calculate at midpoint of web)

$$\tau_1 = \frac{V}{h.t_w} = \frac{2{,}5}{30.1{,}08} = 0{,}077\ t/cm^2$$

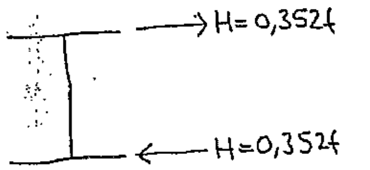
2) Horizontal shear (in the middle of the flange due to vertical load)

$$Q = 1{,}62.\frac{12{,}5}{2}.\left(15 - \frac{1{,}62}{2}\right) = 143{,}674\ cm^3$$

$$\tau_2 = \frac{V.Q_x}{I_x.t_f}$$

$$= \frac{2{,}5.143{,}674}{9800.1{,}62} = 0{,}023\ t/cm^2$$

3) Warping Shear (calculate at midpoint of flange)



$$V_H = \frac{H}{2} = 0{,}176t$$

$$\tau_3 = \frac{3}{2}\frac{V_H}{b.t_f} = \frac{3}{2}.\frac{0{,}176}{12{,}5.1{,}62} = 0{,}013\ t/cm^2$$

1/2

$$I_y = \frac{1}{3} \times 0{,}5 \times \left[5{,}75^3 + 2{,}25^3\right] \times 2 + 0{,}5 \times 14 \times 2{,}0^2 + \frac{1}{12} \times 14 \times 0{,}5^3$$

$I_y = 95{,}3\ \mathrm{cm}^4$

$I_z = I_x + I_y = 630{,}3$

distance between the centroid of the weld and the column face is

$$e = \underbrace{1}_{\text{space between column face \& beam end}} + 8 - 2{,}25 = 6{,}75\ \mathrm{cm}$$

shear stress due to shear force, V: (V acts at the column face)

$$\tau_V = \frac{7}{\underbrace{2 \times 15}_{\text{weld at both sides of the beam}}} = 0{,}23\ \mathrm{t/cm^2} \quad \text{(in y-direction)}$$

Shear stress due to torque created by eccentricity of V

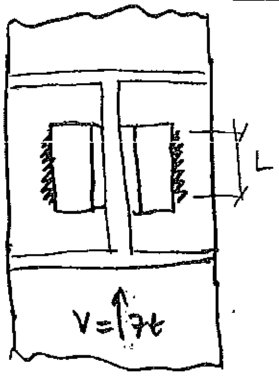
$$\tau_y = \frac{(3{,}5 \times 6{,}75) \times 5{,}75}{630{,}3} = 0{,}22\ \mathrm{t/cm^2} \qquad \tau_x = \frac{(3{,}5 \times 6{,}75) \times 7{,}5}{630{,}3} = 0{,}28\ \mathrm{t/cm^2}$$

Resultant stress:

$$\tau_R = \sqrt{(0{,}23 + 0{,}22)^2 + 0{,}28^2} = 0{,}53\ \mathrm{t/cm^2} < 0{,}75\ \mathrm{t/cm^2} \quad \text{OK}$$

(4 mm weld may be more suitable)

Angle to column flange connection



Try $t = 4$ mm weld

$2 \times 0{,}4 \times L \times 0{,}75 = 7$ tons

$\Rightarrow L = 11{,}7$ cm

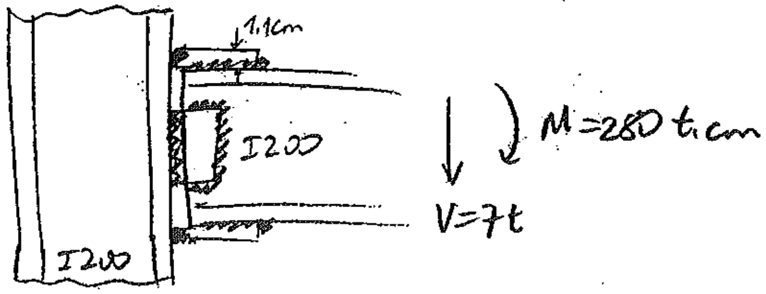
Increase the length of line weld by $2a$

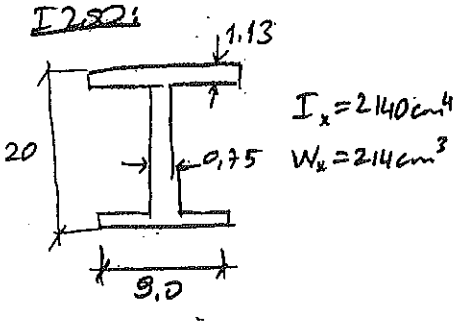
$L = 11{,}7 + 2 \times 0{,}4 = 12{,}5$ cm

$\Rightarrow L = 12{,}5$ cm weld required

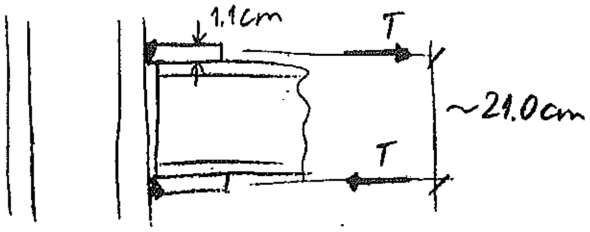
CE 485 RECITATION - WELDING

Design the beam to column connection shown.





Butt welded plate-to column connection:



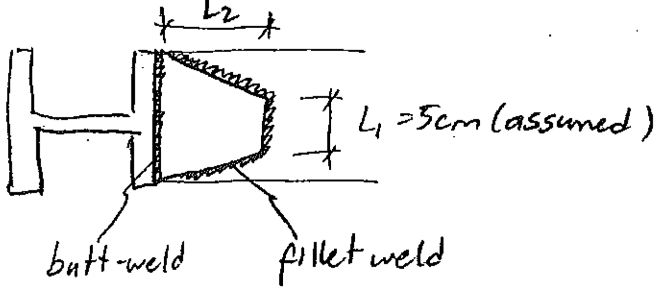
$$T = \frac{M}{21} = \frac{280}{21} = 13.3t$$

$$13.3 = (9 \times t) \times (\sigma_{all})_{butt}$$

$$13.3 = 9 \times t \times 1.4$$

$\rightarrow t = 1.06cm \quad \sim t = 1.1cm$ butt-weld

Plate to beam flange connection

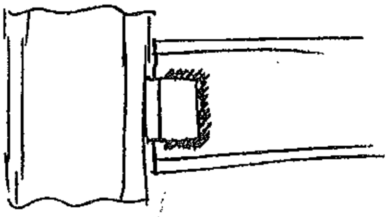


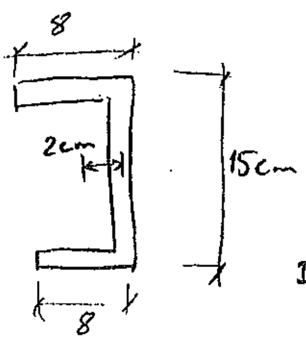
$$(5+2L_2) \times (\tau_{all})_{fillet} \times 0.4 = 13.3$$

↑ t= Assume 4mm weld

$\Rightarrow L_2 = 13.66 \sim 20cm$

Shear connection to beam web (with 2 angles)





Try a=5 mm and the stem area of weld →

$$A = 2 \times 8 \times 0.5 + 1 \times 14 \times 0.5$$

$$A = 15cm^2$$

$$\bar{x} = \frac{2 \times 8 \times 0.5 \times (4-0.25)}{15} = 2.0 cm$$

$$I_x = 2 \times 8 \times 0.5 \times (7.5-0.25)^2 + \frac{1}{12} \times 0.5 \times 14^3 = 535cm^4$$

2/2

Allowable tensile force :

$P_{all} = (3.80)(1.12) = 4.26$ tons > 2.45 t. OK.

Shear force :

$V = \frac{38}{10} = 3.8$ tons/bolt.

Allowable shear force :

$V_{all} = (3.80)(1.4) = 5.32$ tons > 3.8 t. OK.

Combined Stress

At shank :

$\sigma_c = \frac{2.45}{3.80} = 0.64$ t/cm^2. $\sigma_{cem} = 1.12$ t/cm^2.

$\tau = \frac{3.8}{3.80} = 1.00$ t/cm^2. $\tau_{em} = 1.4$ t/cm^2.

Using the elliptical interaction equation :

$$(\frac{0.64}{1.12})^2 + (\frac{1.0}{1.4})^2 = 0.33 + 0.51 = 0.84 < 1.0. \text{ OK.}$$

Maximum compression contact stress :

$$\sigma_b = \frac{(304)(8.33)}{13,300} = 0.19 \text{ t/cm}^2 < \sigma_{bem} = 1.44 \text{ t/cm}^2$$

Now check the connection according to DIN 1050.6.2.

$$\sigma = \sqrt{\sigma_c^2 + 3\tau^2} = \sqrt{0.64^2 + (3)(1.0)^2} = 1.843 \text{ t/cm}^2.$$

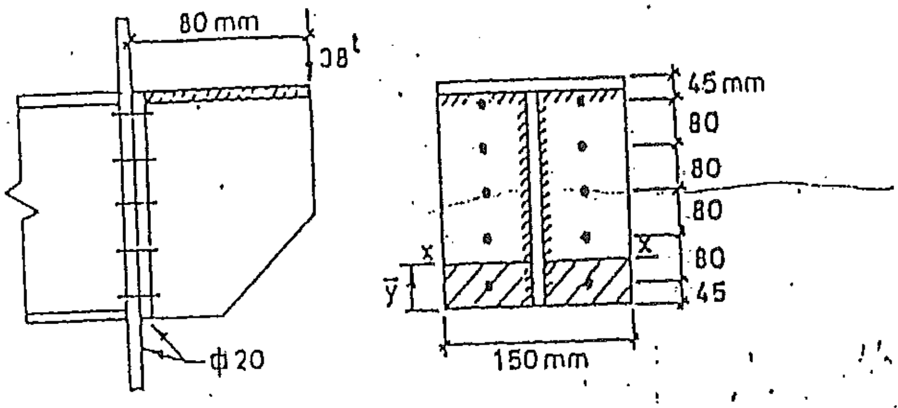
This must be less than $0.75\sigma_a$.

$0.75\sigma_a = (0.75)(2.4) = 1.8$ t/cm^2.

σ is slightly larger than $0.75\sigma_a$ but the difference is negligible.

RECITATION

Example VI.10 :



St37, M22 Turned Bolts, 4D, EY Loading.
Using actual shank area rather than root area of the thread, determine if this connection is allowable to carry the load.

Assuming that the neutral axis is somewhere between the first and the second bolts from the bottom, determine the location of the neutral axis. Since below the neutral axis the section is in compression, the whole section resists. On the other hand, above the neutral axis only the bolts resist the tensile stresses.

Shank area of one bolt = 3.8 cm^2 .

$$(15)\bar{y}\,\frac{\bar{y}}{2} = (2)(3.80)\,\{(12.5 - \bar{y}) + (20.5 - \bar{y}) + (28.5 - \bar{y}) + (36.5 - \bar{y})\}$$

$\bar{y}$ = 8.33 cm. Assumption correct.

$$J_x = (1/3)(15)(8.33)^3 + (2)(3.80)\,\{(12.5 - 8.33)^2 + (20.5 - 8.33)^2 + (28.5 - 8.33)^2 + (36.5 - 8.33)^2\} = 13300\ cm^4$$

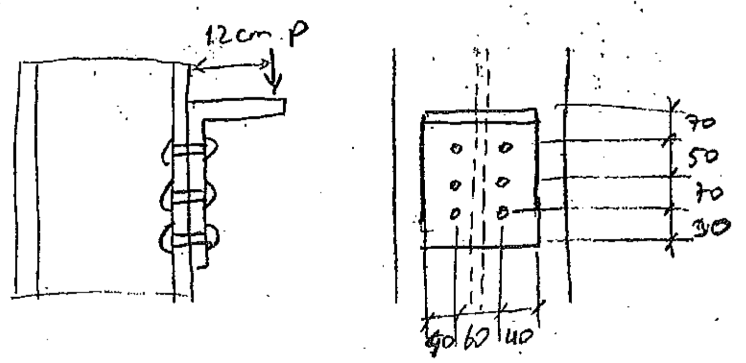
$M = (38)(8) = 304$ t-cm

Tension at top bolt :

$$P_T = \frac{MZ}{J_x}(F) = \frac{(304)(28.17)}{13300}(3.80) = 2.45 \text{ tons.}$$

CE485

(2)



(4D)

Ø16 turned bolts are used in the connection shown. Considering bolt failures only; calculate the allowable load P, using EY loading. Neglect initial tension in the bolts.

Shear check:

$n \cdot F_1 \cdot \tau = P$ n: # of bolts

$F_1 = \frac{\pi}{4} 16^2 \cong 200 mm^2 = 2 cm^2$

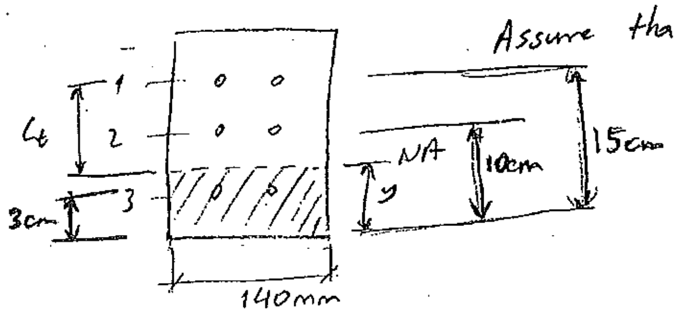
$n = 6$

$\tau = \frac{P}{12 \to 6 \times 2}$ (P in tons)

$\tau_{em} = 1.4 \ t/cm^2 \Rightarrow P_{all} = 12 \times 1,4 = 16,8 \ t$

Combined shear and tension

$M = P \times 12$ t.cm due to eccentricity of load



Assume that the N.A. is between rows 2 & 3

also the centroid (in elastic range)

$14 \times y \times (y/2) = 2 \times 2 \times (10 - y) + 2 \times 2 \times (15 - y)$

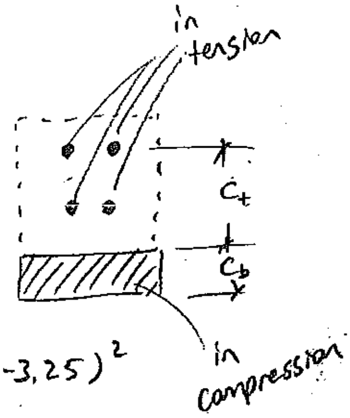
$7y^2 + 8y - 100 = 0$

$y = 3,25$ cm (betw 2&3 ok)

$c_t = 15 - 3,25 = 11,75$ cm

$I = \frac{1}{3} \times 14 \times 3,25^3 + 2 \times 2 \times (10 - 3,25)^2 + 2 \times 2 \times (15 - 3,25)^2$

$I = 895 \ cm^4$



Maximum tension is at row 1

$$\therefore \sigma_4 = \frac{Mc_t}{I} = \frac{12P \times 11,75}{895} = 0,1575P$$

$$\sigma_{tem} = 1.12\ t/cm^2 \Rightarrow P_{all} = \frac{1,12}{0,1575} = 7,1\ t < (P_{all})_{shear}$$

Therefore $P_{all} = 7.1t$.

Check interaction between shear and tension!

$$\sigma_4 = 1.12\ t/cm^2$$

$$\tau_{eq} = \frac{P}{12} = \frac{7.1}{12} = 0.6\ t/cm^2$$

$$\left(\frac{1,12}{1,12}\right)^2 + \left(\frac{0,6}{1.4}\right)^2 \not< 1.0 \quad \text{NO good}$$

$$\left(\frac{\frac{12P_{all} \times 11.75}{895}}{1.12}\right)^2 + \left(\frac{\frac{P_{all}}{12}}{1,4}\right)^2 = 1$$

$$0,0214 P_{all}^2 = 1 \Rightarrow \boxed{P_{all} = 6.8\ t}$$

Rivet shear stresses

$$1.149P = \tau_{em} = 1.4\,t/cm^2$$

$$\Rightarrow P_{all} = 1.22\,t$$

Rivet bearing stresses

$$\phi 14 \rightarrow \frac{1.149P \times 1.54}{1.13 \times 1.4} = \sigma_{ez} = 2.8\,t/cm^2$$

$$\Rightarrow P_{all} = 2.50\,t$$

$$\phi 12 \rightarrow \frac{0.941P \times 1.13}{1.13 \times 1.2} = 2.8$$

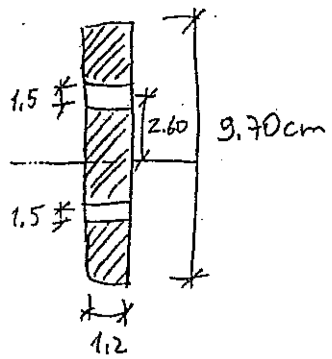
$$\Rightarrow P_{all} = 3.57\,t$$

Plate bearing stress $(t_{fl} < t_{plate}\ ;\ 1.13 < 1.2\,cm)$

$$\frac{1.149P \times 1.54}{1.13 \times (1.4)} = \sigma_{ez(plate)} = 2.1\,t/cm^2$$

$$P_{all} = 1.88\,t$$

Plate bending stress



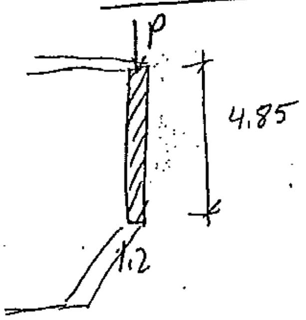
$$I_x \cong \frac{1}{12} \times 1.2 \times 9.70^3 - 2 \times 1.5 \times 1.2 \times 2.60^2$$

$$I_x = 66.93\,cm^4$$

$$\frac{M_{max} \cdot c}{I_x} = \sigma_{all} = 1.44\,t/cm^2$$

$$\frac{P \times (10 + \frac{9}{4}) \times \frac{9.70}{2}}{66.93} = 1.44 \Rightarrow P_{all} = 1.62\,t/cm^2$$

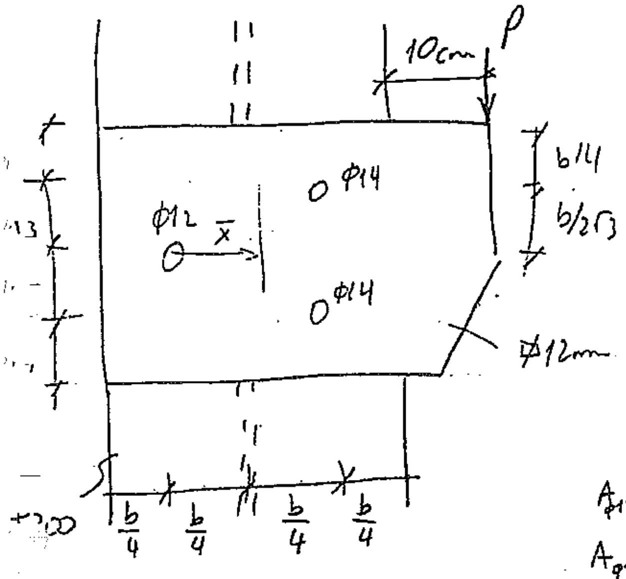
Plate shear stress



$$\tau_{max} = \frac{3}{2}\frac{P}{h \cdot t} \Rightarrow \frac{3}{2}\frac{P}{1.2 \times 4.85} = 0.831 \Rightarrow P_{all} = 3.22\,t$$

$P_{all} = 1.22\,t$ (from rivet shear stress)

CE 485 RECITATION - CONNECTIONS



Calculate P_{max}

St37, EY loading

(Rivets)

I200

$b = 90mm$

$t_{fl} = 11.3mm$

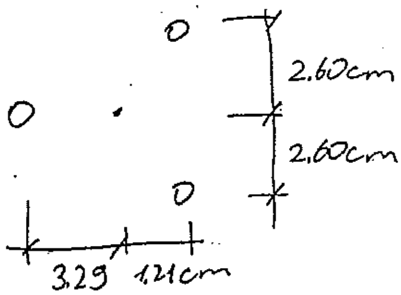
$A_{\phi 12} = 1.13cm^2$

$A_{\phi 14} = 1.54cm^2$

Locate the centroid

$$\bar{x} = \frac{2\times\frac{\pi}{4}1{,}4^2\times 9/2}{2\times\frac{\pi}{4}1{,}4^2+\frac{\pi}{4}1{,}2^2} = 3.29\text{ cm from the centroid of the left rivet.}$$

load eccentricity, $e = 10 + \frac{3}{4}\times 9 - 3.29 = 13.46cm$

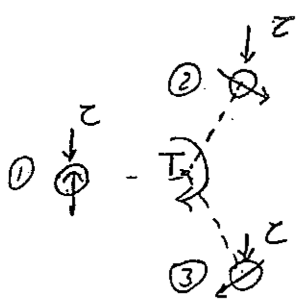


Torque, $T = 13.46P$ t.cm

$J = \Sigma A_i r_i^2 = \Sigma A_i (x_i^2 + y_i^2)$

$J = \frac{\pi}{4}1{,}2^2(3.29^2) + 2\frac{\pi}{4}1{,}4^2(2.60^2+1.21^2)$

$J = 37.56\ cm^4$



Stresses due to direct shear

$$\tau_i' = \frac{P}{\Sigma A} = \frac{P}{1.13+2\times 1.54} = \frac{P}{4.21}\ t/cm^2 = 0{,}238P$$

Stresses due to Torque

$$\tau_i'' = \frac{T.r_i}{J} \quad \tau_{x_i}'' = \frac{T.y_i}{J} \quad \tau_{y_i}'' = \frac{T.x_i}{J} \quad \tau_i = \sqrt{\Sigma\tau_{x_i}^2 + \Sigma\tau_{y_i}^2}$$

	x_i	y_i	τ_{x_i}''	τ_{y_i}''	τ_i'	τ_i
①	-3.29	0	0	1.179P	$\bar{0}$,238P	0.941P
②	1.21	2.60	0,932P	$\bar{0}$,434P	$\bar{0}$,238P	1.149P
③	1.21	-2.60	$\bar{0}$.932P	$\bar{0}$.434P	$\bar{0}$,238P	1.149P

all in (-) y - direction

Rivet NO	x_i (cm)	y_i (cm)	r_i (cm)	$N_{xi} = N'_{xi} + N''_{xi}$	$N_{yi} = N'_{yi} + N''_{yi}$	N_s
1	(-)5.6	(+)8.4	10.1	+0.04P +0.45P	-0.2P +0.30P	0.50P
2	4.4	(+)8.4	9.5	+0.04P +0.45P	-0.2P -0.23P	0.65P
3	1.4	-1.6	2.1	+0.04P -0.08P	-0.2P +0.07P	0.27P
4	(-)2.6	-6.6	7.1	+0.04P -0.35P	-0.2P +0.14P	0.32P
5	2.4	-8.6	8.9	+0.04P -0.46P	-0.2P -0.13P	0.53P

Forces acting on the rivets

net force

$J = \sum r_i^2 = 326.3\,cm^2$

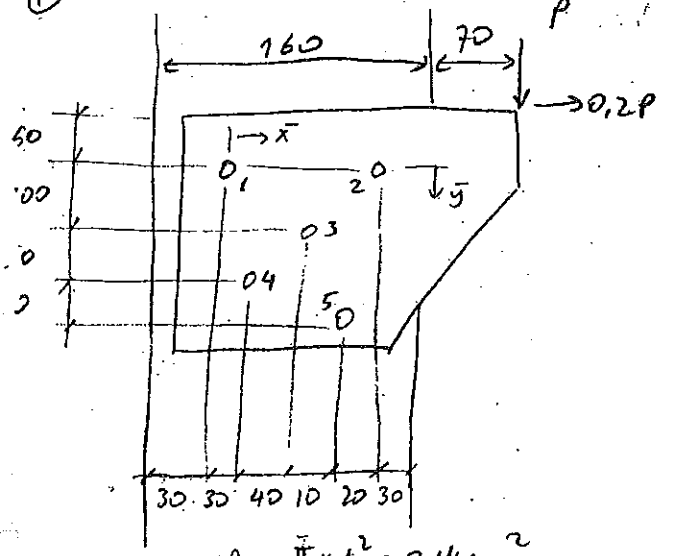
From table, the most critical rivet is number 2.

$$\left.\begin{aligned} \tau_{em} &= \frac{0.65P}{3.14} = 1.4 \Rightarrow P_{all} = 6.76\,t \\ \sigma_{ez} &= \frac{0.65P}{2.0 \times 1.0} = 2.8 \Rightarrow P_{all} = 8.62\,t \end{aligned}\right\} \text{Use smaller one} \therefore P_{all} = 6.76\,t$$

CE 485 RECITATION

- CONNECTIONS -

(1)



In the connection shown, ϕ20 rivets (St34) are used for connecting a 10 mm thick plate to the 12 mm thick web of a column, both of which are made of St37. Calculate the allowable load P under EY loading. (Ignore plate failures)

$A_1 = \frac{\pi}{4} \times 1^2 = 3{,}14 cm^2$

Find the centroid of the rivets,

$$\bar{y} = \frac{0+0+100+150+170}{5} = 84 mm$$

$$\bar{x} = \frac{0+30+70+80+100}{5} = 56 mm$$

Torque caused by the applied forces at the centroid of the rivets is

$$T = P \times [7{,}0 + (16-3-5{,}6)] + 0{,}2P \times [6{,}0 + 8{,}4]$$

$$T = 17{,}3\, P \ t.cm$$

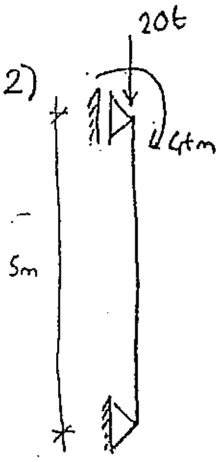
In direct shear, all rivets carry 1/5 of the applied load (Force is proportional with the area)

$N'_{xi} = 0{,}04P\ (\rightarrow)$ $\quad N'_{yi} = 0{,}2P\ (\downarrow)$ $\qquad N'_i = P \cdot \frac{A_i}{\Sigma A_i}$

Due to torque, $N''_{x1} = \frac{T \cdot y_1}{J}$, $N''_{yi} = \frac{T \cdot x_i}{J}$

General form: $N''_{xi} = A_i \frac{T \cdot y_i}{J}$

$N''_{yi} = A_i \frac{T \cdot x_i}{J}$ $\qquad J = \Sigma A_i r_i^2$

2) 

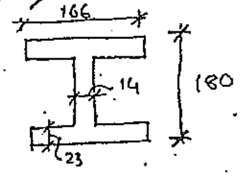
Design an economical IPBv section (made of St37, subjected to EY loading) for the beam-column shown below for the given loading. Lateral supports are provided only at the bottom and the top supports.

TRY IPBv160

$F = 91{,}7 \text{ cm}^2$

$I_x = 5100 \text{ cm}^4$, $I_y = 1760 \text{ cm}^4$, $W_x = 566 \text{ cm}^3$, $W_y = 212 \text{ cm}^3$

$i_x = 7{,}25 \text{ cm}$, $i_y = 4{,}26 \text{ cm}$



$$\left.\begin{aligned} \lambda_x &= \frac{500}{7{,}25} = 69{,}0 \\ \lambda_y &= \frac{500}{4{,}26} = 117{,}4 \end{aligned}\right\} \lambda_{max} = 117{,}4 \longrightarrow \sigma_{bem} = 0{,}5938 \text{ t/cm}^2$$

$$\sigma_{eb} = \frac{P}{F} = \frac{20}{91{,}7} = 0{,}218 \text{ t/cm}^2$$

$$\frac{\sigma_{eb}}{\sigma_{bem}} = \frac{0{,}218}{0{,}5938} = 0{,}367 > 0{,}15$$

$$C_m = 0{,}6 - 0{,}4 \cdot \left(\frac{M_1}{M_2}\right) = 0{,}6 - 0{,}4\left(\frac{0}{4}\right) = 0{,}6$$

$$\sigma_b = \frac{M_{max}}{W_x} = \frac{400}{566} = 0{,}707 \text{ t/cm}^2$$

$$\sigma_B = \frac{840 \cdot C_b}{s \cdot \frac{d}{F_d}} = \frac{840 \cdot 1{,}75}{500 \cdot \frac{18}{16{,}6 \cdot 2{,}3}} = 6{,}24 \text{ t/cm}^2 > 1{,}44 = \sigma_{all} \longrightarrow \sigma_B = 1{,}44 \text{ t/cm}^2$$

$$\sigma'_{ex} = \frac{\pi^2 E}{\lambda_x^2 \cdot 2{,}5} = \frac{\pi^2 \cdot 2100}{69^2 \cdot 2{,}5} = 1{,}74 \text{ t/cm}^2$$

Interaction Equations

$$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{C_m \cdot \sigma_b}{\left(1 - \frac{\sigma_{eb}}{\sigma'_{ex}}\right)\sigma_B} \leq 1{,}0$$

$$0{,}367 + \frac{0{,}6 \cdot 0{,}707}{\left(1 - \frac{0{,}218}{1{,}74}\right) \cdot 1{,}44} \leq 1{,}0$$

$$0{,}367 + 0{,}337 = 0{,}704 < 1{,}0 \rightarrow \text{OK}$$

and

If buckling does not occur

$$\frac{\sigma_{eb}}{0{,}6\sigma_a} + \frac{\sigma_b}{\sigma_B} \leq 1{,}0$$

$$\frac{0{,}218}{0{,}6 \cdot 2{,}4} + \frac{0{,}707}{1{,}44} = 0{,}642 < 1{,}0 \quad \text{OK}$$

Shear Check

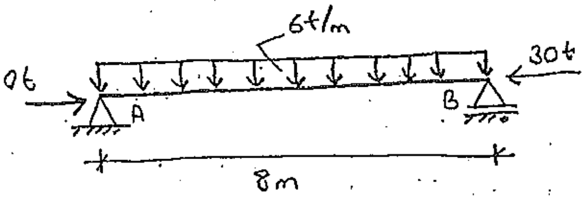
$$\tau_{av} = \frac{3}{2} \cdot \frac{4/5}{18 \cdot 1{,}4} = 0{,}0476 \leq \frac{\sigma_{cem}}{\sqrt{3}} = 0{,}831$$

USE IPBv160

CE 485 - RECITATION

BEAM - COLUMNS

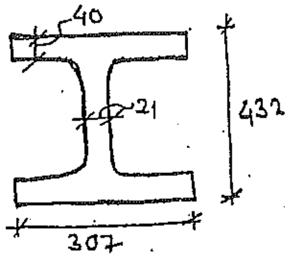
1)



Check safety of the beam-column shown below, which is made of IPBv400 and is laterally unsupported except at A and B. Use St52 and EY loading.

IPBv 400

$I_x = 104100$
$W_x = 4820$
$i_x = 17,9$
$I_y = 19340$
$W_y = 1260$
$i_y = 7,70$
$F = 326$



$$\lambda_{max} = \frac{S_{ky}}{i_y} = \frac{800}{7,7} = 103,9$$

$$\rightarrow \sigma_{ben} = 0,768 \ t/cm^2$$

$$\sigma_{eb} = \frac{P}{F} = \frac{30}{326} = 0,092 \ t/cm^2$$

$$\frac{\sigma_{eb}}{\sigma_{ben}} = \frac{0,092}{0,768} = 0,12 < 0,15 \quad \text{so check;}$$

$$\frac{\sigma_{eb}}{\sigma_{ben}} + \frac{\sigma_b}{\sigma_B} \leq 1,0$$

$$\sigma_b = \frac{M_{max}}{W_x} = \frac{6 \cdot 10^{-2} \cdot \frac{800^2}{8}}{4820} = 1,00 \ t/cm^2$$

$$\sigma_B = \frac{840 \cdot C_b}{s \cdot \frac{d}{F_B}} = \frac{840 \cdot 1,0}{800 \cdot \frac{43,2}{30,7 \cdot 4}} = 2,98 \ t/cm^2 > \sigma_{all} = 2,16 \ t/cm^2 \longrightarrow \sigma_B = 2,16 \ t/cm^2$$

(St52)

$$\frac{\sigma_{eb}}{\sigma_{ben}} + \frac{\sigma_b}{\sigma_B} = 0,12 + \frac{1}{2,16} = 0,12 + 0,46 = 0,58 < 1 \quad \text{SAFE}$$

OVERDESIGN → TRY A SMALLER SECTION

$0,2 < 1$ IPBv 400 / 360

↓

IPBv 320

$$\sigma_b = \frac{5 \times 10^2}{1740} = 0{,}287\ t/cm^2$$

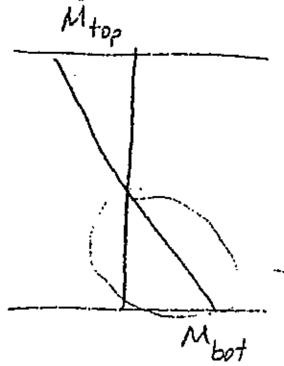
$$\sigma_B = \frac{840 \times 2{,}3}{500 \times \frac{42.5}{(2{,}3 \times 16{,}3)}} = 3{,}4 > 1{,}44 \rightarrow \sigma_B = 1{,}44\ t/cm^2$$

Interaction equation for $\frac{\sigma_{eb}}{\sigma_{bem}} < 0{,}15$

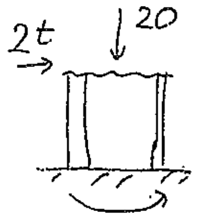
$$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{\sigma_b}{\sigma_B} \leqslant 1{,}0$$

$$0{,}113 + \frac{0{,}287}{1.44} = 0{,}318 << 1{,}0 \rightarrow \text{try smaller section}$$

BMD :



$M_{top} = M_{bot}$



$M_{bot} = 2 \times 2{,}5 = 5\ t.m$

$$\sigma_{eb} = \frac{20}{178} = 0.112\ t/cm^2$$

$$\lambda_x = \frac{1{,}20 \times 500}{19.6} = 30.6 \rightarrow \sigma_{bem} \cong 1{,}311\ t/cm^2$$

$$\frac{\sigma_{eb}}{\sigma_{bem}} = \frac{0{,}112}{1{,}311} = 0{,}085 < 0{,}15$$

$$C_b = 1{,}75 + 1{,}05\left(\frac{5}{5}\right) + 0{,}3\left(\frac{5}{5}\right)^2$$

$$\sigma_b = \frac{5 \times 10^2}{2750} = 0.182\ t/cm^2$$

$$\sigma_B = \frac{840 \times 2{,}3}{500 \times \frac{50}{(2{,}7 \times 18.5)}} = 3{,}86\ t/cm^2 > 1.44$$

No need to calculate σ_e' or C_m.

$\sigma_B = 1.44\ t/cm^2$

Interaction equation:

$$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{\sigma_b}{\sigma_B} \leq 1.0 \longrightarrow 0{,}085 + \frac{0{,}182}{1{,}44} = 0{,}211 \ll 1.0 \rightarrow \text{Try a smaller section}$$

Try I425

$I_x = 36970$ $\quad I_y = 1440$
$W_x = 1740$ $\quad W_y = 176$
$i_x = 16{,}7$ $\quad i_y = 3.30$
$F = 132$

$$\sigma_{eb} = \frac{20}{132} = 0{,}151\ t/cm^2 \qquad \lambda_x = \frac{1{,}2 \times 500}{16{,}7} = 35.9 \rightarrow \sigma_{bem} \cong 1{,}274\ t/cm^2$$

$$\frac{\sigma_{eb}}{\sigma_{bem}} = \frac{0{,}151}{1{,}274} = 0{,}119 < 0{,}15 \rightarrow \text{No need to calculate } \sigma_e' \text{ or } C_m.$$

$$\sigma_B = \frac{840 \times C_b}{s \cdot \frac{d}{F_b}} = \frac{840 \times 1.75}{300 \times \frac{34}{(3.3 \times 31)}} = 17.43 > 1.44 \Rightarrow \sigma_B = 1.44 \text{ t/cm}^2$$

$$\sigma_{ex}' = \frac{\pi^2 E}{\left(k \cdot \frac{s}{i_x}\right)^2} \cdot \frac{1}{2.5} = \frac{\pi^2 \times 2100}{\left(1.0 \times \frac{600}{14.0}\right)^2} \times \frac{1}{2.5} = 4.51 \text{ t/cm}^2$$

From table 10, $C_m = 1.0$

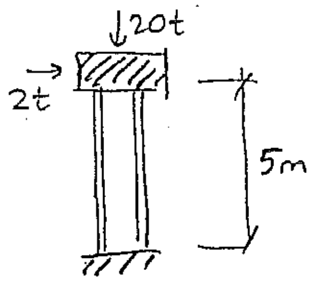
Check interaction formulae:

① $$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{C_{mx}\,\sigma_{bx}}{\left(1.0 - \frac{\sigma_{eb}}{\sigma_{ex}'}\right)\sigma_{Bx}} \leq 1.0$$

$$0.19 + \frac{1.0 \times 0.254}{\left(1.0 - \frac{0.231}{4.51}\right)1.44} = 0.376 \lll 1.0 \quad \checkmark \text{ OK}$$

② $$\frac{\sigma_{eb}}{0.60\,\sigma_a} + \frac{\sigma_b}{\sigma_B} \leq 1.0$$

$$\frac{0.231}{0.60 \times 2.4} + \frac{0.254}{1.44} = 0.337 \ll 1.0 \quad \checkmark \text{ OK}$$



Design the beam-column shown, using an I-section. St37, EY loading. Consider in-plane buckling only (column is supported against out of plane buckling)

TRY I500

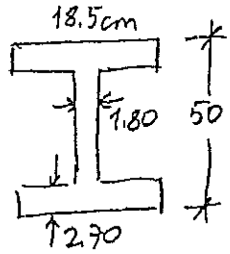
Section properties:

$I_x = 68740 \text{ cm}^4$ $\quad I_y = 2480 \text{ cm}^4$

$W_x = 2750 \text{ cm}^3$ $\quad W_y = 268 \text{ cm}^3$

$i_x = 19.6 \text{ cm}$ $\quad i_y = 3.72 \text{ cm}$

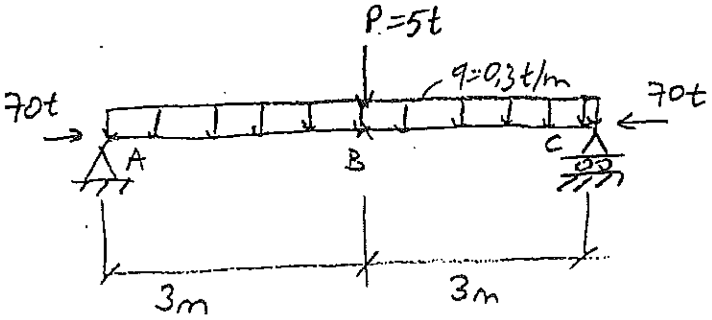
$F = 179 \text{ cm}^2$



22.12.2004

CE485 Recitation: BEAM-COLUMNS

15



Analyze the beam-column made of IPBv 300. Lateral supports exist at A, B & C. St37, EY loading.

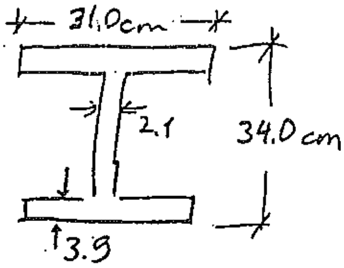
Section Properties

$F = 303 cm^2$

$I_x = 59200 cm^4$ $\quad I_y = 19400 cm^4$

$W_x = 3480 cm^3$ $\quad W_y = 1250 cm^3$

$i_x = 14.0 cm$ $\quad i_y = 8.00 cm$



First check whether $\frac{\sigma_{eb}}{\sigma_{bem}} > 0.15$ (Magnitude of axial load)

σ_{eb}: Axial stress under the axial load, only (eksenel basma)

$$\sigma_{eb} = \frac{70}{303} = 0{,}231 \ t/cm^2$$

σ_{bem}: Allowable compressive stress (from λ)

$$\lambda_y = \frac{k \cdot L_y}{i_y} = \frac{1.0 \times 300}{8.00} = 37.5 \quad \text{(About y-axis, the weaker)}$$

$$\lambda_x = \frac{k \cdot L_x}{i_x} = \frac{1.0 \times 600}{14.0} = 42.9 \quad \text{(About x-axis, the stronger)}$$

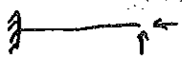
$\Rightarrow \sigma_{bem} \cong 1.205 \ t/cm^2$ for $\lambda = 42.9$

$$\frac{\sigma_{eb}}{\sigma_{bem}} = \frac{0{,}231}{1.205} = 0{,}19 > 0.15 \rightarrow \text{Two equations will be used.}$$

$$M_{max} = \frac{5 \times 6}{4} + \frac{0{,}3 \times 6^2}{8} = 8.85 \ t.m$$

$$\sigma_b = \frac{8.85 \times 10^2}{3480} = 0{,}254 \ t/cm^2$$

Check Interaction Equations



$C_m = 1 - 0.3 \frac{\sigma_{eb}}{\sigma_e'}$

$C_m = 1 - 0.3 \frac{0.391}{12.74} = 0.99$

① $\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{C_{mx}\,\sigma_{bx}}{\left(1 - \frac{\sigma_{eb}}{\sigma_{ex}'}\right)\sigma_{Bx}} \leq 1.0$

$0.852 + \frac{0.99 \times 0.109}{\left(1 - \frac{0.391}{12.74}\right) 1.44} = 0.930 < 1.0$ OK

② $\frac{\sigma_{eb}}{0.60\,\sigma_a} + \frac{\sigma_{bx}}{\sigma_{Bx}} \leq 1.0$

$\frac{0.391}{0.6 \times 2.4} + \frac{0.109}{1.44} = 0.347 < 1.0$ OK

Check Shear

$\tau_{max} = \frac{V_{max}}{h \cdot t} = \frac{\frac{3\,t.m}{5\,m} + \frac{0.6\,t/m \times 5\,m}{2}}{50 \times 1.8} = 0.023\ t/cm^2 \ll \tau_{all} = 0.831\ t/cm^2$ OK

$\boxed{\text{I500 is OK}}$

Check I475

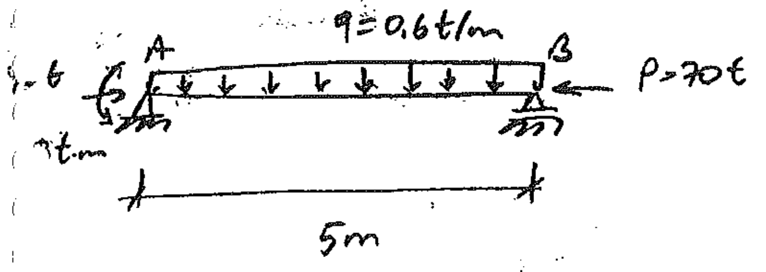
$\sigma_{eb} = \frac{70}{163} = 0.429\ t/cm^2$

$\lambda = \frac{500}{3.60} = 138.9 \rightarrow \sigma_{bem} = 0.4297\ t/cm^2$

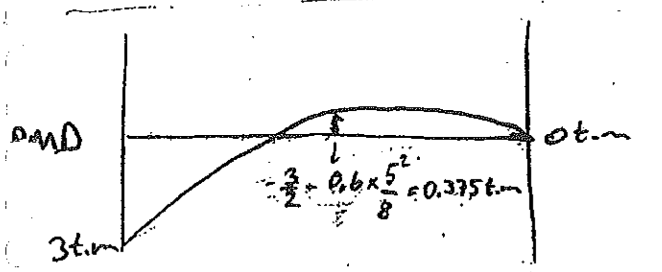
$\frac{\sigma_{eb}}{\sigma_{bem}} = 0.998$ → obviously this section will not satisfy the interaction equations, when $\frac{C_{mx}\,\sigma_{bx}}{\left(1 - \frac{\sigma_{eb}}{\sigma_{ex}'}\right)\sigma_{Bx}}$ term is added.

⇒ Most economical section is I500

CE485 RECITATION - BEAM-COLUMNS



Design member AB using an I-section and St37 steel. EY loading. Lateral supports exist at the ends only.



$P_{eq} = P + M \cdot B_x \cdot \frac{1}{\omega}$

1st trial: Assume $B_x = 0{,}09$; $\omega = 1$

$P_{eq} = 70 + 300 \times 0{,}09 = 97t$ → GO TO TABLE ($kL=5m$, $P_{eq}=97t$, I section)

Choose I550 ($P_{all}=115t$)

$\lambda_{max} = \frac{1.0\times 500}{4.02} = 124.4 \rightarrow \omega = 2.68$

$\Rightarrow P_{eq} = 70 + 300 \times 0.06 \times \frac{1}{2{,}68} = 76{,}7t$ → GO TO TABLE ($kL=5m$, $P_{all} > 76{,}7t$)

Choose I500 ($P_{all} = 83t$)

<u>Check I500</u>

$i_x = 19{,}6cm$

$i_y = 3{,}72cm$

$F = 179cm^2$

$W_x = 2750 cm^3$

$\sigma_{eb} = \frac{70}{179} = 0{,}391$

$\lambda = \frac{500}{3.72} = 134.4 \rightarrow \sigma_{bem} = 0{,}459\ t/cm^2$

$\frac{\sigma_{eb}}{\sigma_{bem}} = \frac{0{,}391}{0{,}459} = 0{,}852 > 0.15$

$\sigma_e' = \frac{8290}{\left(k\frac{S_x}{i_x}\right)^2} = \frac{8290}{\left(\frac{500}{19.6}\right)^2} = 12{,}74\ t/cm^2$

$\sigma_{B2} = \frac{840 C_b}{s\frac{d}{F_B}} = \frac{840\times 1.75}{500\times\frac{50}{18{,}5\times 2.7}} = 2{,}94 > 1.44 \rightarrow \sigma_{B2} = 1.44\ t/cm^2$

since $\sigma_B = max(\sigma_{B1}, \sigma_{B2})$, no need to check σ_{B1}

$\sigma_b = \frac{300}{2750} = 0{,}109\ t/cm^2$

Deflection Check $\left(\Delta \leq \frac{L}{200} \right)$

$$\Delta_1 = \frac{5}{384} \cdot \frac{qL^4}{EI_x} = \frac{5}{384} \cdot \frac{(0.012)(850)^4}{2100\,(24290)} = 1.599 \text{ cm}$$

$$\Delta_2 = \frac{Pa}{48EI}(3l^2 - 4a^2) = \frac{3(200)}{48(2100)(24290)} \cdot (3 \times 850^2 - 4 \times 200^2) = 0.492 \text{ cm}$$

$$\Delta_3 = \frac{4(400)}{48(2100)(24290)} \left(3 \times 850^2 - 4 \times 400^2\right) = \underset{1,0}{1.123} \text{ cm}$$

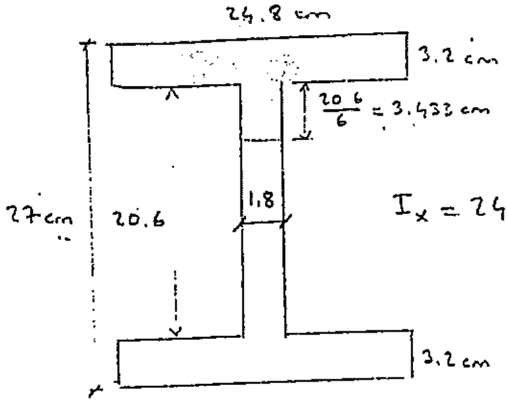
where Δ_1 is due to distributed load
Δ_2 is due to concentrated load of 3t } at midspan
Δ_3 is due to concentrated load of 4t

$$\Delta_T = \Delta_1 + \Delta_2 + \Delta_3 = 1.599 + 0.492 + \underset{1,0}{\cancel{1.123}} = \underset{3,1}{\cancel{3.214}} \text{ cm} < \Delta_{all} = \frac{850}{200} = 4.25 \text{ cm}$$

OK ✓

Therefore USE $\boxed{\text{IPB}_v\ 240}$

IPB$_v$ 240



$I_x = 24290 \text{ cm}^4$

Shear Check

$$\tau = \frac{V_{max}}{A_{web}} \leq \tau_{all} = 0.831 \text{ t/cm}^2$$

$A_{web} = h.t$

$$\tau = \frac{9.276}{\cancel{20.6} \; 27 \times 1.8} = \cancel{0.25} \; 0.19 \text{ t/cm}^2 \leq 0.831 \text{ t/cm}$$

OK ✓

Lateral Buckling Check

A B $\rightarrow$ $C_b = 1.75$

C – C $\rightarrow$ $C_b = 1.75 + 1.05\left(-\frac{16.152}{22.092}\right) + 0.3\left(-\frac{16.152}{22.092}\right)^2 = 1.143$

C D $\rightarrow$ $C_b = 1.75$

$$i_{yc} = \sqrt{\frac{\frac{3.2 \times (24.8)^3}{12} + \left(\frac{20.6}{6}\right) \times 1.8^3 \times \frac{1}{12}}{24.8 \times 3.2 + 1.8 \times \frac{20.6}{6}}} = 6.897 \text{ cm.}$$

A B :

$$\frac{S}{i_{yc}} = \frac{200}{6.897} = 28.99 < \sqrt{\frac{30000 \times 1.75}{2.4}} = 147.9$$

$$\sigma_B = \left[\frac{2}{3} - \frac{2.4\,(28.99)^2}{90000 \times 1.75}\right] 2.4 = 1.569 \text{ t/c}^2 > 1.44 \text{ t/c}^2 \quad \text{No Lat. Bucklin}$$

$$\sigma_B = \frac{840 \times 1.75}{200 \cdot \frac{27}{24.8 \times 3.2}} = 21.6 \text{ t/c}^2 \quad \text{No Lateral Buckling}$$

B – C :

$$\frac{S}{i_{yc}} = \frac{250 \text{ cm}}{6.897 \text{ cm}} = 36.247 < \sqrt{\frac{3000 \times 1.143}{2.4}} = 119.5$$

$$\sigma_B = \left[\frac{2}{3} - \frac{2.4\,(36.247)^2}{90000 \times 1.143}\right] \times 2.4 = 1.526 \text{ t/c}^2 > 1.44 \text{ t/c}^2 \quad \text{No Lat. Buck.}$$

$$\sigma_B = \frac{840 \times 1.143}{250 \cdot \frac{27}{24.8 \times 3.2}} = 11.29 \text{ t/c}^2 \quad \text{No Lat. Buck.}$$

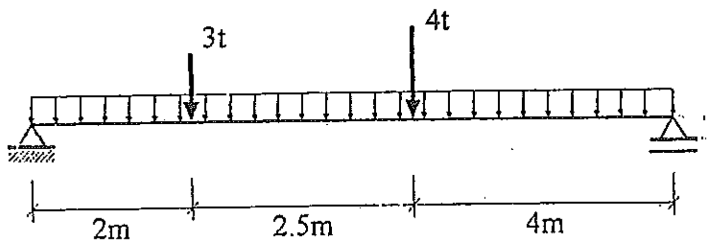
C – D :

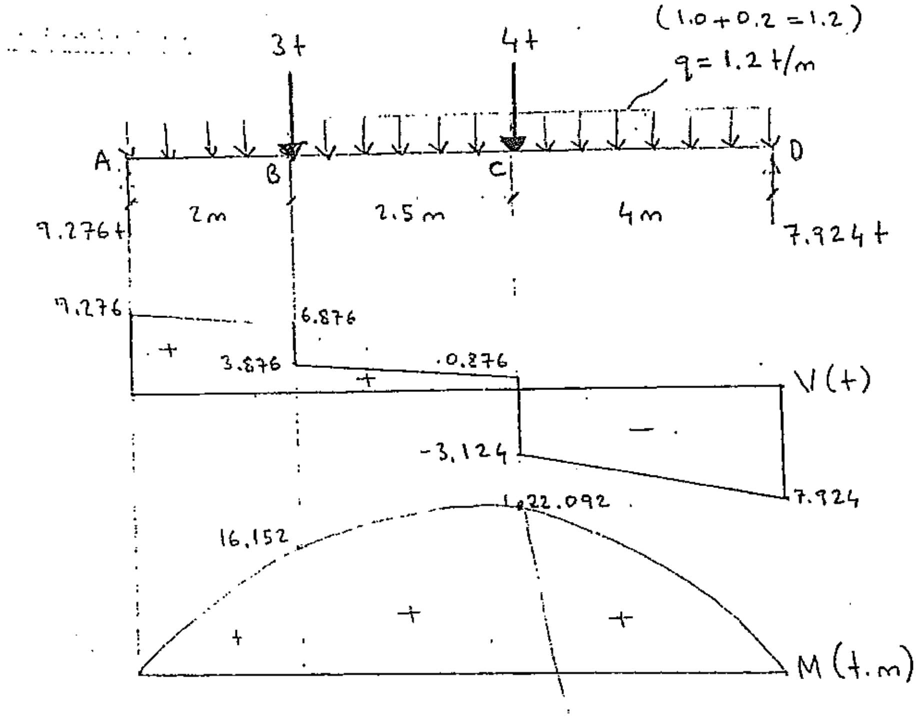
$$\frac{S}{i_{yc}} = \frac{400}{6.897} = 57.99 < 147.9$$

$$\sigma_C = \left[\frac{2}{3} - \frac{2.4\,(57.99)^2}{90000 \times 1.75}\right] \cdot 2.4 = 1.477 \text{ t/c}^2 > 1.44 \text{ t/c}^2 \quad \text{No Lat. Buck.}$$

$$\sigma_B = \frac{840 \times 1.75}{400 \cdot 27/24.8 \times 3.2} = 10.8 \text{ t/c}^2 \quad \text{No Lat. Buckling}$$

1) Design the beam in the figure using IPBv section. Check deflection $\Delta \leq L/200$. There are lateral supports at the beam-ends and under the concentrated loads. Distributed load on the beam is $q = 1.0 t/m^2$. St37, EY Loading.





* Let's assume the self weight of the beam as 0.2 t/m

check moment at the sides of this point

$M_{max} = 22.092$ t.m

$V_{max} = 9.276$ t

Stress Check

$$\sigma_{max} = \frac{M_{max}}{W_{reqd}} \leq \sigma_{all} \quad \text{where} \quad \sigma_{all} = 1.44\, t/c^2 \text{ for St37}$$

$$W_{reqd} = \frac{M_{max}}{\sigma_{all}} = \frac{2209.2}{1.44} = 1534.17\ cm^3 \quad \therefore \text{ Choose IPB}_v\, 240$$

$W_x = 1800\ cm^3 > 1534.17\ cm^3$

$G = 0.157\ t/m < G_{assumed}$

ok ✓

ateral buckling check at BC

$M_B = 19{,}5$ tm , $M_C = -9$ tm , $s = 400$ cm , $d = 36$ cm , $F_B = 27{,}89$ cm² $\left(\frac{M_C}{M_B} = +\right.$ double curvature$\left.\right)$

$c_b = 1{,}75 + 1{,}05\left(\frac{M_C}{M_B}\right) + 0{,}3\left(\frac{M_C}{M_B}\right)^2 = 1{,}75 + 1{,}05\left(\frac{9}{19{,}5}\right) + 0{,}3\left(\frac{9}{19{,}5}\right)^2 = 2{,}3 \leq 2{,}3$ ✓

$\sqrt{\frac{30\,000\, c_b}{\sigma_a}} = \sqrt{\frac{30\,000 \times 2{,}3}{3{,}6}} = 138{,}44$

$\frac{s}{i_y} = \frac{400}{3{,}7} = 108{,}1 < 138{,}44 \rightarrow \sigma_{B1} = \left[\frac{2}{3} - \frac{3{,}6 \times 108{,}1^2}{90000 \times 2{,}3}\right] 3{,}6 = 1{,}67$ t/cm²

$\sigma_{B2} = \frac{840 \times 2{,}3}{400 \times \frac{36}{27{,}89}} = 3{,}75$ t/cm²

$\sigma_{Bmax} = 3{,}74$ t/cm² $> \sigma_{Ball} = 0{,}6\,\sigma_a = 2{,}16$ t/cm² $\rightarrow$ take $\sigma_B = 2{,}16$ t/cm²

$\sigma_b = \frac{M_{max}}{W_x} = \frac{1950}{1090} = 1{,}789$ t/cm² $< 2{,}16$ t/cm² ✓

Lateral buckling check at CD

$M_C = -9$ tm , $M_D = 0$, $s = 300$ cm , $d = 36$ cm, $F_B = 27{,}89$ cm²

$c_b = 1{,}75 + 1{,}05\left(\frac{M_D}{M_C}\right) + 0{,}3\left(\frac{M_D}{M_C}\right)^2 = 1{,}75$

$\sqrt{\frac{30000\, c_b}{\sigma_a}} = \sqrt{\frac{30\,000 \times 1{,}75}{3{,}6}} = 120{,}76$

$\frac{s}{i_y} = \frac{300}{3{,}7} = 81{,}1 < 120{,}76 \rightarrow \sigma_{B1} = \left[\frac{2}{3} - \frac{3{,}6 \times 81{,}1^2}{90000 \times 1{,}75}\right] \times 3{,}6 = 1{,}86$ t/cm²

$\sigma_{B2} = \frac{840 \times 1{,}75}{300 \times \frac{36}{27{,}89}} = 3{,}8$ t/cm²

$\sigma_{Bmax} = 3{,}8$ t/cm² $> \sigma_{Ball} = 2{,}16$ t/cm² $\rightarrow$ take $\sigma_B = 2{,}16$ t/cm²

$\sigma_b = \frac{M_{max}}{W_x} = \frac{900}{1090} = 0{,}83$ t/cm² $< \sigma_{Ball} = 2{,}16$ t/cm² ✓

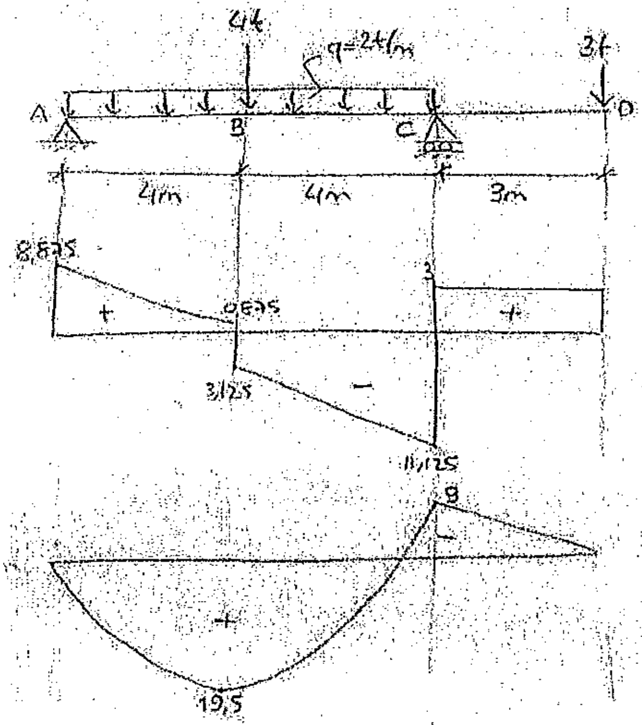
Shear Check

$V_{max} = 11{,}125$ t

$\tau_{av} = \frac{V_{max}}{h t} = \frac{11{,}125}{36 \times 1{,}3} = 0{,}24$ t/cm² $< \tau_{max} = \frac{\sigma_{acm}}{\sqrt{3}} = 1{,}247$ t/cm² ✓

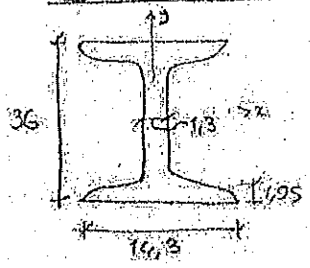
BEAM is safe

CE485
RECITATION
LATERAL BUCKLING



Check safety of the beam, which is made of I360, against lateral buckling and shear failure. St52, EY Loading. Lateral supports exist at A, B and C.

I360



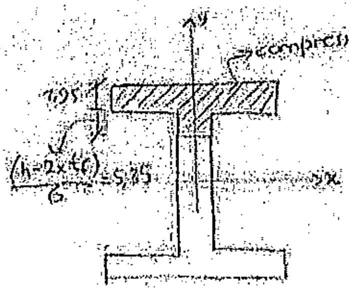
$I_x = 19610\ cm^4$, $I_y = 818\ cm^4$

$W_x = 1090\ cm^3$, $W_y = 114\ cm^3$

$F = 97,1\ cm^2$,

$i_x = 14,2\ cm$

$i_y = 2,9\ cm$



$A_b = 14,3 \times 1,95 + 5,35 \times 1,3 = 34,84\ cm^2$

$I_{yb} = \frac{1}{12} \times 1,95 \times 14,3^3 + \frac{1}{12} \times 5,35 \times 1,3^3$

$= 476,16\ cm^4$

$i_y = \sqrt{\frac{I_{yb}}{A_b}} = 3,7\ cm$

Lateral buckling check at AB

$M_A = 0$, $M_B = 19,5\ tm$, $s = 400\ cm$, $d = 36\ cm$, $F_B = 14,3 \times 1,95 = 27,89\ cm^2$

$$c_b = 1,75 + 1,05\left(\frac{M_A}{M_B}\right) + 0,3\left(\frac{M_A}{M_B}\right)^2 = 1,75$$

$$\sqrt{\frac{30\,000 \cdot c_b}{\sigma_a}} = \sqrt{\frac{30\,000 \times 1,75}{3,6}} = 120,76$$

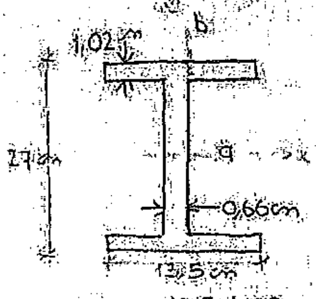
$$\frac{s}{i_y} = \frac{400}{3,7} = 108,1 < 120,76 \rightarrow \sigma_{B_1} = \left[\frac{2}{3} - \frac{\sigma_a \left(s/i_y\right)^2}{90000 \cdot c_b}\right]\sigma_a = \left[\frac{2}{3} - \frac{3,6 \times 108,1^2}{90000 \times 1,75}\right] \times 3,6 = 1,44\ t/cm^2$$

$$\text{Also} \quad \sigma_{B_2} = \frac{840 \cdot c_b}{s \cdot \frac{d}{F_B}} = \frac{840 \times 1,75}{400 \times \frac{36}{27,89}} = 2,85\ t/cm^2$$

$\sigma_{Bmax} = 2,85\ t/cm^2 > \sigma_{Ball} = 0,6\,\sigma_a = 2,16\ t/cm^2$ so take $\sigma_B = 2,16\ t/cm^2$

$$\sigma_b = \frac{M_{max}}{W_x} = \frac{19,5 \times 100}{1090} = 1,79\ t/cm^2 < \sigma_B = 2,16\ t/cm^2 \checkmark$$

Check Shear

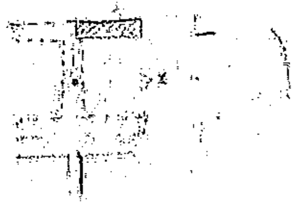


$$\tau_a = \frac{V_y \cdot Q_{ax}}{I_x \cdot t_w} \quad \text{or} \quad (\tau_a)_{av} = \frac{V_y}{h \cdot t_w}$$

$V_y = 0{,}26 \cdot 3m = 0{,}78\,t$

$V_x = 0{,}15 \cdot 3m = 0{,}45\,t$

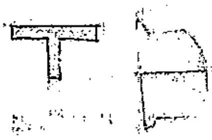
$$\tau_b = \frac{V_y \cdot Q_{bx}}{I_x \cdot t_f} + \frac{V_x \cdot Q_{by}}{I_y \cdot t_f}$$



$$Q_{bx} = \frac{13{,}5 \cdot 1{,}02}{2} \cdot \left(\frac{27}{2} - \frac{1{,}02}{2}\right) = 89{,}44 \ cm^3$$

$$Q_{by} = \frac{13{,}5 \cdot 1{,}02}{2} \cdot \left(\frac{13{,}5}{4}\right) = 23{,}24 \ cm^3$$

$$Q_{ax} = 13{,}5 \cdot 1{,}02 \cdot \left(\frac{27}{2} - \frac{1{,}02}{2}\right) + 0{,}66 \cdot \frac{\left(\frac{27}{2} - 1{,}02\right)^2}{2} = 230{,}3 \ cm^3$$



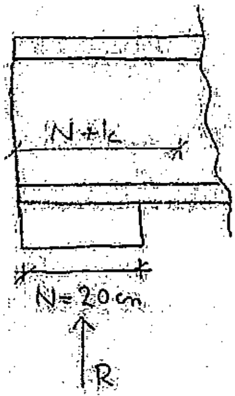
$$\tau_a = \frac{0{,}78 \cdot 230{,}3}{5790 \cdot 0{,}66} = 0{,}047 \ll \frac{\sigma_{em}}{\sqrt{3}} = \tau_{all} = 0{,}831 \ t/cm^2 \ \checkmark$$

$$\tau_b = \frac{0{,}78 \cdot 89{,}44}{5790 \cdot 1{,}02} + \frac{0{,}45 \cdot 23{,}24}{420 \cdot 1{,}02} = 0{,}036 \ll 0{,}831 \ t/cm^2 \ \checkmark$$

Check Web Crippling

Under concentrated loads: $\dfrac{P}{t_w \cdot (N + 2k)} \leq 0{,}75\,\sigma_a$

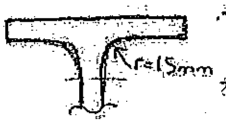
Under the supports: $\dfrac{R}{t_w \cdot (N + k)} \leq 0{,}75\,\sigma_a$



$$R = \frac{6 \cdot 0{,}30}{2} \cdot \cos(30^\circ) = 0{,}78\,t$$

Check

$$\frac{0{,}78}{0{,}66\,(20 + 2{,}52)} = 0{,}052 \ t/cm^2 \ll 0{,}75 \cdot 2{,}4 = 1{,}8 \ t/cm^2 \ \checkmark$$

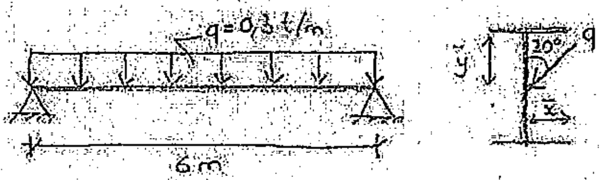


$k = t_f + r = 10{,}2 + 15 = 25{,}2 \ mm$

$t_w = 6{,}6 \ mm$

CE-485 RECITATION
BIAXIAL BENDING

Design the beam shown using an IPE section. St 37, EY loading. $\Delta_{all} \leq \frac{L}{200}$
The load passes through the centroid. The bearing plates at the supports are 20 cm long.



$$\sigma_{max} = \frac{M_x \cdot \bar{y}}{I_x} + \frac{M_y \cdot \bar{x}}{I_y}$$

$q_y = q \cdot \cos\alpha = 0{,}3 \cdot \cos 30 = 0{,}260\ t/m$

$q_x = q \cdot \sin\alpha = 0{,}3 \cdot \sin 30 = 0{,}150\ t/m$

$$M_x = \frac{q_y \cdot L^2}{8} = \frac{0{,}260 \cdot 10^{-2} \cdot 600^2}{8} = 117\ tcm$$

$$M_y = \frac{q_x \cdot L^2}{8} = \frac{0{,}150 \cdot 10^{-2} \cdot 600^2}{8} = 67{,}5\ tcm$$

- $\sigma_{max} = \frac{117 \cdot h/2}{I_x} + \frac{67{,}5 \cdot b/2}{I_y} = \frac{58{,}5h}{I_x} + \frac{33{,}75b}{I_y} \leq \sigma_{cem}$

- Deflection Requirement:

$$\Delta_{max} = \sqrt{\left[\frac{5}{384} \cdot \left(\frac{q_y \cdot L^4}{E \cdot I_x}\right)\right]^2 + \left[\frac{5}{384} \cdot \left(\frac{q_x \cdot L^4}{E \cdot I_x}\right)\right]^2}$$

$$= \sqrt{\left[\frac{5}{384} \cdot \left(\frac{0{,}26 \cdot 10^{-2} \cdot 600^4}{2100 \cdot I_x}\right)\right]^2 + \left[\frac{5}{384} \cdot \left(\frac{0{,}15 \cdot 10^{-2} \cdot 600^4}{2100 \cdot I_y}\right)\right]^2}$$

$$= \sqrt{4365114{,}8/I_x^2 + 1452815{,}8/I_y^2}$$

<u>TRY IPE 270</u>

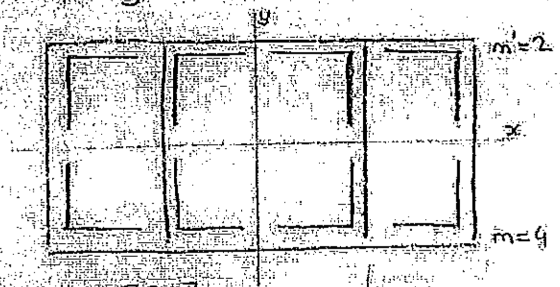
$I_x = 5790\ cm^4$, $I_y = 420\ cm^4$, $h = 270\ mm$, $b = 135\ mm$

$$\sigma_{max} = \frac{58{,}5 \cdot 27}{5790} + \frac{33{,}75 \cdot 13{,}5}{420} = 0{,}27 + 1{,}08 = 1{,}35 < 1{,}44\ t/cm^2\ ✓$$

$$\Delta_{max} = \sqrt{\left[\frac{5}{384} \cdot \frac{1{,}605 \cdot 10^5}{5790}\right]^2 + \left[\frac{5}{384} \cdot \frac{9{,}257 \cdot 10^4}{420}\right]^2} = 2{,}89\ cm < \frac{600}{200} = 3\ cm\ ✓$$

CE485 - RECITATION - BUILT-UP MEMBERS

2) Designing the built-up column made of 8 equal-leg angle sections for an axial comp. load of 80 tons. $k_x = 0,85$, $k_y = 1,6$, St37, EY loading. Optimum spacing between main members is used. $S = 10$ m. $s_{1x} = 70$ cm, $s_{1y} = 50$ cm



TRY 70x7

$I_x = I_y = 42,4 \text{ cm}^4$, $i_x = i_y = 2,12$ cm, $i_1 = 1,37$, $F = 9,4 \text{ cm}^2$

$e \leq 20\, i_1 \Rightarrow e = 20 \cdot 1,37 = 27,4$ cm

y-y direction

$$I_{yy} = 8 I_y + 4F\left(e + \frac{e}{2}\right)^2 + 4F\left(\frac{e}{2}\right)^2$$

$$= 8 \cdot 42,4 + 4 \cdot 9,4\left(27,4 + \frac{27,4}{2}\right)^2 + 4 \cdot 9,4\left(\frac{27,4}{2}\right)^2$$

$$= 70910,64 \text{ cm}^4$$

$$i_y = \sqrt{\frac{I_{yy}}{\Sigma F}} = \sqrt{\frac{70910,64}{8 \cdot 9,4}} = 30,71 \text{ cm}$$

$$\lambda_y = \frac{k_y \cdot S}{i_y} = \frac{1,6 \cdot 1000}{30,71} = 52,1 \quad , \quad \lambda_1 = \frac{70}{1,37} = 51,1$$

$$\lambda_{yi} = \sqrt{\lambda_y^2 + \frac{m}{2}\lambda_1^2} = \sqrt{52,1^2 + \frac{4}{2} \cdot 51,1^2} = 89,1$$

x-x direction

$$I_{xx} = 8 \cdot I_x + 8F\left(\frac{e}{2}\right)^2 = 8 \cdot 42,4 + 8 \cdot 9,4\left(\frac{27,4}{2}\right)^2 = 14453,49 \text{ cm}^4$$

$$i_x = \sqrt{\frac{I_{xx}}{\Sigma F}} = \sqrt{\frac{14453,49}{8 \cdot 9,4}} = 6,53 \text{ cm}$$

$$\lambda_x = \frac{k_x \cdot S}{i_x} = \frac{0,85 \cdot 1000}{6,53} = 130,2 \quad , \quad \lambda_1 = \frac{50}{1,37} = 36,5$$

$$\lambda_{xi} = \sqrt{\lambda_x^2 + \frac{m}{2}\lambda_1^2} = \sqrt{130,2^2 + \frac{2}{2} \cdot 36,5^2} = 135,2$$

$\lambda_{max} = \lambda_{xi} = 135,2 \longrightarrow \omega = 3,17$ ($\sigma_{bm} = 0,4543$ t/cm^2)

$$P_{max} = \frac{\sigma_a \cdot \Sigma F}{\omega} = \frac{1,44 \cdot 8 \cdot 9,4}{3,17} = 34,16\,t < 80\,t \quad \text{NOT OK}$$

Lateral buckling check at CD

$M_C = -9$ tm $\quad s = 300$ cm $\quad F_b = 27.89$ cm^2

$M_D = 0 \quad d = 36$ cm

$$c_b = 1.75 + 1.05\left(\frac{0}{9}\right) + 0.3\left(\frac{0}{9}\right)^2 = 1.75$$

$$\sqrt{\frac{30000 \times 1.75}{3.6}} = 120.76 \quad > \quad \frac{s}{i_y} = \frac{300}{3.7} = 81.1$$

$$\Rightarrow \sigma_{B1} = \left[\frac{2}{3} - \frac{3.6 \times 81.1^2}{90000 \times 1.75}\right] \times 3.6 = 1.86 \text{ t/cm}^2$$

$$\sigma_{B2} = \frac{840 \times 1.75}{300 \times \frac{36}{27.89}} = 3.8 \text{ t/cm}^2$$

$$\sigma_{B,max} = 3.8 \text{ t/cm}^2 > \sigma_{B,all} = 2.16 \text{ t/cm}^2 \rightarrow \sigma_B = 2.16 \text{ t/cm}^2$$

$$\sigma_b = \frac{M_{max}}{W_x} = \frac{900}{1090} = 0.83 \text{ t/cm}^2 < 2.16 \text{ t/cm}^2 \quad \checkmark$$

Shear check

$V_{max} = 11.125$ t

$$\tau = \frac{V_{max}}{h\,t} = \frac{11.125}{36 \times 1.3} = 0.24 \text{ t/cm}^2 < \tau_{max} = \frac{\sigma_{cem}}{\sqrt{3}} = 1.247 \text{ t/cm}^2$$

Beam is safe !

Lateral buckling check at AB

$M_A = 0$ $\quad s = 400$ cm $\quad F_b = 14.3 \times 1.95 = 27.89\ cm^2$

$M_B = 19.5$ tm $\quad d = 36$ cm $\quad$ ↳ area of flange

$$c_b = 1.75 + 1.05\left(\frac{0}{19.5}\right) + 0.3\left(\frac{0}{19.5}\right)^2 = 1.75$$

$$\sqrt{\frac{30000 \times 1.75}{3.6}} = 120.76 \quad > \quad \frac{s}{i_y} = \frac{400}{3.7} = 108.1$$

$$\Rightarrow \quad \sigma_{B_1} = \left[\frac{2}{3} - \frac{\sigma_a (s/i_y)^2}{30000\ c_b}\right]\sigma_a = \left[\frac{2}{3} - \frac{3.6 \times 108.1^2}{30000 \times 1.75}\right] \times 3.6 = 1.44\ t/cm^2$$

$$\sigma_{B_2} = \frac{840\ c_b}{s\ \frac{d}{F_b}} = \frac{840 \times 1.75}{400 \times \frac{36}{27.89}} = 2.85\ t/cm^2$$

$$\sigma_{B,max} = 2.85\ t/cm^2 > \sigma_{B,all} = 0.6\,\sigma_a = 2.16\ t/cm^2 \Rightarrow \sigma_B = 2.16\ t/cm^2$$

$$\sigma_b = \frac{M_{max}}{W_x} = \frac{19.5 \times 100}{1090} = 1.79\ t/cm^2 < \sigma_B = 2.16\ t/cm^2 \quad ✓$$

Lateral buckling check at BC

$M_B = 19.5$ tm $\quad s = 400$ cm $\quad F_b = 27.89\ cm^2$

$M_C = -9$ tm $\quad d = 36$ cm

$$C_b = 1.75 + 1.05\left(\frac{9}{19.5}\right) + 0.3 \times \left(\frac{9}{19.5}\right)^2 = 2.3$$

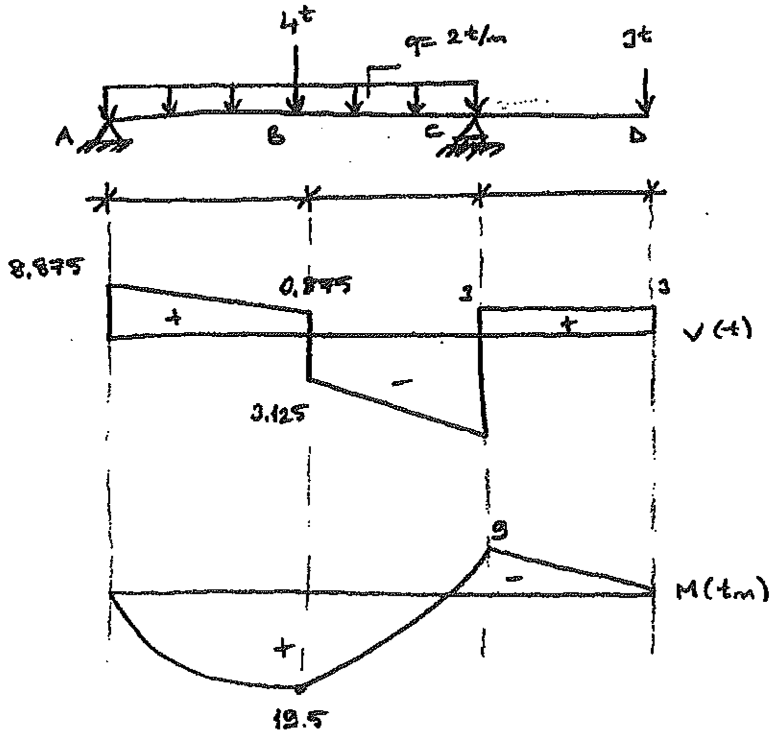
$$\sqrt{\frac{30000\ C_b}{\sigma_a}} = \sqrt{\frac{30000 \times 2.3}{3.6}} = 138.44 \quad > \quad \frac{s}{i_y} = \frac{400}{3.7} = 108.1$$

$$\Rightarrow \quad \sigma_{B_1} = \left[\frac{2}{3} - \frac{3.6 \times 108.1^2}{30000 \times 2.3}\right] \times 3.6 = 1.67\ t/cm^2$$

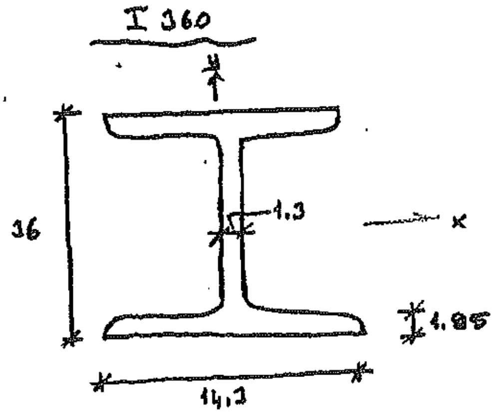
$$\sigma_{B_2} = \frac{840 \times 2.3}{400 \times \frac{36}{27.89}} = 3.75\ t/cm^2$$

$$\sigma_{B,max} = 3.75\ t/cm^2 > \sigma_{B,all} = 0.6\,\sigma_a = 2.16\ t/cm^2 \Rightarrow \sigma_B = 2.16\ t/cm^2$$

$$\sigma_b = \frac{M_{max}}{W_x} = \frac{1950}{1090} = 1.79\ t/cm^2 < 2.16\ t/cm^2 \quad ✓$$



Check safety of the beam, which is made of I 360, against lateral buckling and shear failure. St 52 steel, EY loading. Lateral supports exist at A, B and C.



$I_x = 19610$ cm^4

$I_y = 818$ cm^4

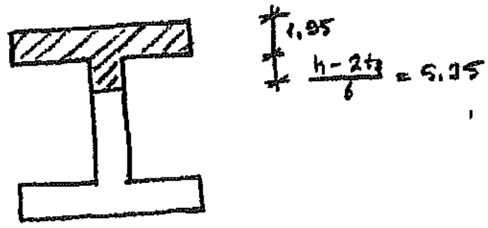
$W_x = 1090$ cm^3

$W_y = 114$ cm^3

$F = 97.1$ cm^2

$i_x = 14.2$ cm

$i_y = 2.9$ cm



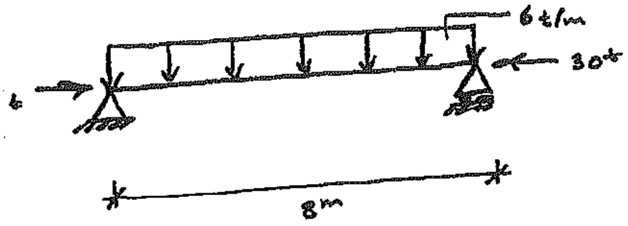
$F_b = 14,2 \times 1,95 + 5,35 \times 1,3$

$= 34,84$ cm^2

$I_{yb} = \frac{1}{12} \times 1,95 \times 14,2^3 + \frac{1}{12} \times 5,35 \times 1,3^3$

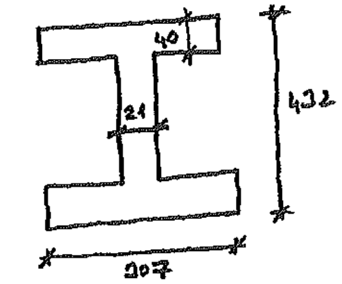
$= 476,16$ cm^4

$i_y = \sqrt{I_{yb}/F_b} = \sqrt{\frac{476,16}{34,84}} = 3,7$ cm



Check safety of beam-column which IPBv 400 and laterally supported at A and B. Use St 52 steel and EY Loading.

IPBv 400



$I_x = 104100\ cm^4$

$W_x = 4820\ cm^3$

$i_x = 17.9\ cm$

$I_y = 19340\ cm^4$

$W_y = 1260\ cm^3$

$i_y = 7.70\ cm$

$F = 226\ cm^2$

$$\lambda_{max} = \frac{800}{7.7} = 103.9$$

$$\rightarrow \sigma_{bcm} = 0.768\ t/cm^2$$

$$\sigma_{cb} = \frac{P}{F} = \frac{30}{226} = 0.092\ t/cm^2$$

$$\frac{\sigma_{cb}}{\sigma_{bcm}} = \frac{0.092}{0.768} = 0.12 < 0.15$$

$$\frac{\sigma_{cb}}{\sigma_{bcm}} + \frac{\sigma_b}{\sigma_B} \leq 1.0$$

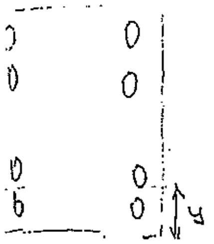
$$\sigma_b = \frac{M_{max}}{W_x} = \frac{0.06 \times 800^2/8}{4820} = 1.00\ t/cm^2$$

$$\sigma_B = \frac{840 \times C_b}{s \quad d/F_B} = \frac{840 \times 1.0}{800 \times \frac{43.2}{30.7 \times 4}} = 2.98\ t/cm^2 > \sigma_{all} = 2.16\ t/cm^2 \Rightarrow \sigma_B = 2.16\ t/cm^2$$

$$\frac{\sigma_{cb}}{\sigma_{bcm}} + \frac{\sigma_b}{\sigma_B} = 0.12 + \frac{1.00}{2.16} = 0.58 < 1.0$$

SAFE

$F_{12} \cong 1.13\,cm^2 \quad \tau = \frac{P/2}{8} = P/16 \Rightarrow P = 16\times1.4 = 22.4\,t$



$$2\times1.13(14-y+11-y+6-y) = y\times10\times y/2$$

$$5y^2+6.8y-70=0 \quad y = 4.48\,cm = c_b < 6cm \quad \text{OK}$$

$$c_t = 17-4.48 = 12.52\,cm$$

$$= 1/3\times10\times4.48^3 + 2\times1.13\times9.52^2 + 2\times1.13\times6.52^2 + 2\times1.13\times1.52^2$$

$$\cong 606\,cm^4$$

$$) = 8P$$

$$\sigma = \frac{8P\times12.52}{606} = 0.1653P$$

$$\sigma_{gem} = 1.12\,t/cm^2 \qquad P_{all} = 6.77\,t < 22.4 = (P_{all})_{shear}$$

$$P_{all} = 6.77\,t$$

$$\tau = \frac{6.77}{16} = 0.42\,t/cm^2$$

$$\left(\frac{1.12}{1.12}\right)^2 + \left(\frac{0.42}{1.4}\right)^2 <\neq 1.0 \quad \text{Not OK}$$

$$\left(\frac{\frac{8P\times12.52}{606}}{1.12}\right)^2 + \left(\frac{\frac{P}{16}}{1.4}\right)^2 = 1.0$$

$$0.022P^2 + 3.97\times10^{-6}P^2 = 1.0$$

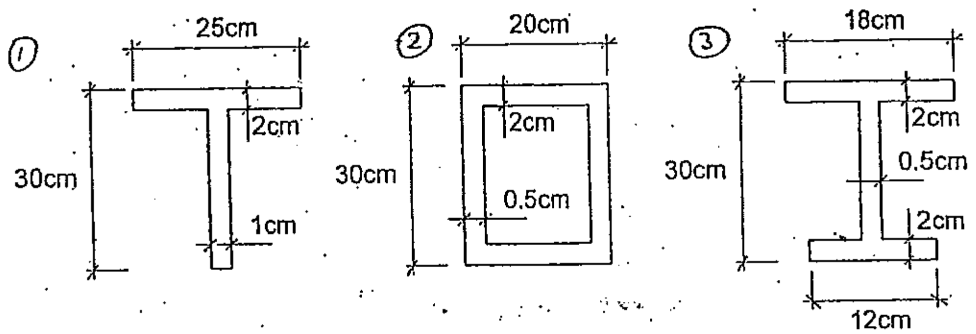
$$P_{all} \cong 6.74\,t \;//$$

Due March 10, 2004

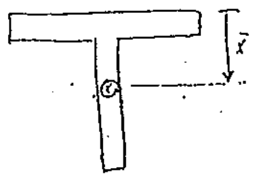
CE 485 - FUNDAMENTALS OF STEEL DESIGN

HOMEWORK 1

1. Calculate the yield moment capacity M_y, the section modulus (w), the plastic (ultimate) moment capacity M_p and the and plastic modulus (z) for the given sections, which are made of St37 steel (σ_y=2.4t/cm^2).



(1)



$$\bar{x} = \frac{24 \times 2 \times 1 + 30 \times 1 \times 15}{24 \times 2 + 30 \times 1} = 6.38\text{cm}$$

$$I = \frac{1}{12} \times 1 \times 30^3 + 1 \times 30 \times (15-6.38)^2 + \frac{1}{12} \times 24 \times 2^3 + 24 \times 2 \times (6.38-1)^2$$

$$I = 5884.5\text{cm}^4$$

$$W = \frac{I}{c} = \frac{5884.5}{(30-6.38)} = 249.1\text{ cm}^3$$

$$M_y = \sigma_y \cdot W = 2.4 \times 249.1 = 597.9\text{ t.cm}$$

— o —

For plastic case, $\bar{x} = ?$ $\Rightarrow A_c = A_t = \frac{A}{2} = \frac{78}{2} = 39\text{cm}^2$

Assume that $\bar{x} < 2\text{cm} \Rightarrow (\bar{x} \cdot 25) = 39\text{cm}^2 \Rightarrow \bar{x} = 1.56\text{ cm}$

$$M_p = (1.56 \times 25\text{cm}) \times \frac{1.56}{2} \times 2.4 + ((2-1.56) \times 25\text{cm}) \times \frac{(2-1.56)}{2} \times 2.4 + (1 \times 28\text{cm}) \times [\frac{28}{2} + (2-1.56)] \times$$

$$M_p = 1049.2\text{ t.cm}$$

$$z = \frac{M_p}{\sigma_y} = \frac{1049.2}{2.4} = 437.17\text{ cm}^3$$

(2) The section is symmetrical; $\bar{x} = 15\text{cm}$

$$I = \frac{1}{12} \times 20 \times 30^3 - \frac{1}{12} \times 19 \times 26^3 = 17171.3\text{ cm}^4$$

$$W = \frac{I}{c} = \frac{17171.3}{15} = 1144.8\text{ cm}^3 \Rightarrow M_y = \sigma_y \cdot W = 2.4 \times 1144.8 = 2747.4\text{ t.cm}$$

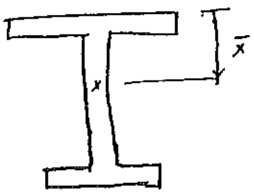
— o —

plastic N.A. passes through $\bar{x} = 15\text{cm}$, since the section is symmetrical!

$$M_p = 2 \times [(19 \times 2) \times 14 + (2 \times 0.5 \times 15) \times 7.5] \times 2.4 = 3093.6\text{ t.cm}$$

$$z = \frac{M_p}{\sigma} = \frac{3093.6}{2.4} = 1289\text{ cm}^3$$

(3)



$$A = 18\times2 + 26\times0.5 + 12\times2 = 73\ cm^2$$

$$\bar{x} = \frac{18\times2\times1 + 26\times0.5\times15 + 12\times2\times29}{73} = 12.70\,cm$$

$$I_x = \frac{1}{12}\times18\times2^3 + 18\times2\times(11.70)^2 + \frac{1}{12}\times0.5\times26^3 + 0.5\times26\times(13-10.70)^2$$
$$+ \frac{1}{12}\times12\times2^3 + 12\times2\times(16.30)^2 = 12125.7\ cm^4$$

$$W = \frac{I}{c} = \frac{12125.7}{(30-12.7)} = 700.9\ cm^3$$

$$M_y = \sigma_y \cdot W = 2.4\times700.9 = 1682.2\ t.cm$$

—o—

plastic neutral axis:

assume $\bar{x} > 2cm$

$$18\times2 + (\bar{x}-2)\times0.5 = \frac{73}{2} \Rightarrow \bar{x} = 3cm \quad (>2cm\ OK)$$

$$M_p = (18\times2)\times2\times2.4 + (1\times0.5)\times0.5\times2.4 + (25\times0.5)\times12.5\times2.4 + (12\times2)\times26\times2.4$$

$$M_p = 2046.0\ t.cm$$

$$Z = \frac{M_p}{\sigma_y} = \frac{2046}{2.4} = 852.5\ cm^3$$

November 1, 2014

CE 485
HOMEWORK-I SOLUTIONS

1) a) Compute the total dead load of the building.

Slab

$$\underbrace{(15\times 32\times 0{,}12\times 2{,}5)}_{R/C\ slab}+\underbrace{(15\times 32\times 0{,}035\times 2{,}3)}_{\text{Tapping + Levelling concrete}}+(15\times 32\times 0{,}02\times 2{,}0)=201{,}84\ t/story$$

Beams

$$(4\times 12+6\times 16+7\times 6+8\times 4)\times 0{,}07=15{,}26\ t/story$$

Columns

$3m \longrightarrow 24\times 0{,}25\times 3{,}0=18\ t/story$

$4m \longrightarrow 24\times 0{,}25\times 4{,}0=24\ t/story$

$5m \longrightarrow 24\times 0{,}25\times 5{,}0=30\ t/story$

Walls

3m interior $\longrightarrow [8\times(2{,}48\times 4)+2\times(32\times 2{,}48-10\times 2\times 1)]\times 0{,}25=49{,}52\ t/story$

3m exterior $\longrightarrow [2\times 32\times 2{,}48-10\times 2\times 1{,}5+2\times 15\times 2{,}48]\times 0{,}4=81{,}25\ t/story$

4m interior $\longrightarrow [8\times(3{,}48\times 4)+2\times(32\times 3{,}48-10\times 2\times 1)]\times 0{,}25=73{,}52\ t/story$

4m exterior $\longrightarrow [2\times 32\times 3{,}48-10\times 2\times 1{,}5+2\times 15\times 3{,}48]\times 0{,}4=118{,}85\ t/story$

5m interior $\longrightarrow [8\times(4{,}48\times 4)+2\times(32\times 4{,}48-10\times 2\times 1)]\times 0{,}25=97{,}52\ t/story$

5m exterior $\longrightarrow [2\times 32\times 4{,}48-10\times 2\times 1{,}5+2\times 15\times 4{,}48]\times 0{,}4=156{,}45\ t/story$

Roof

$15\times 32\times 0{,}22=105{,}6\ t$

$$\text{Total Dead Load}=W_T=6\times(201{,}84+15{,}26)+4\times(18+49{,}52+81{,}25)+(24+73{,}52+118{,}85)+(30+97{,}52+156{,}45)+105{,}6$$

$$W_T=1302{,}6+595{,}08+216{,}37+283{,}97+105{,}6$$

$$W_T=2503{,}62\ t$$

3a

Storey weights:

$W = W_{DL} + 0.3\,W_{LL}$ (0.3: LL reduction coefficient for office buildings)

$W_i = (1.2 \times 10 \times 25) + 0.3 \times 80 = 324^t$

Top storey weight:

$W_{top} = W_{DL} + 0.3 W_{LL} = (1.2 \times 10 \times 25) + 0.3(38.75) = 311.6^t$

$T_1 = C_T \cdot H_N^{3/4}$ since $H_N < 25\,m$ (excluding roof height)

$C_T = 0.07$ (R/C frames)

$T_1 = 0.07\,(22)^{3/4} = 0.71$ sec:

for Z2, $T_A = 0.15$ sec $T_B = 0.4$ sec.

$T_1 > T_B$ ∴ $S(T_1) = 2.5\,(T_B/T_1)^{0.8} = 1.58$

$A(T_1) = A_0 . I . S(T_1) = 0.4 \times 1.0 \times 1.58 = 0.63$

where $I = 1.0$ for offices

$A_0 = 0.4$ for EQ zone 1

$T > T_A$ ∴ $R_0(T_1) = R = 4$

$\frac{S(T_1)}{R_0(T_1)} \geq 0.1$ required, $\frac{1.58}{4} > 0.1$ ok.

Total weight of the building,

$\Sigma W = 6 \times 324 \times 311.6 = 2255.6^t$

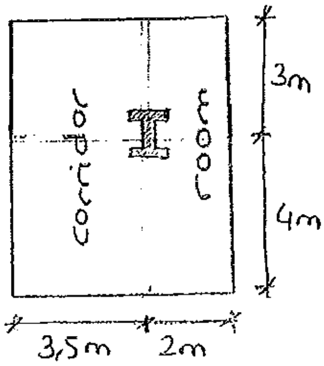
$V_t = \frac{W . A(T_1)}{R_0(T_1)} = \frac{2255.6 \times 0.63}{4} = 355.3^t$

ö-18

) Compute the reduced live load on a typical interior column for ... of the structure against gravitational loads.

From TS 498/1987 → Table 14 (for office buildings);

$q_{room} = 0{,}2\ t/m^2$, $q_{corridor} = 0{,}5\ t/m^2$



$A_c = 3{,}5 \times 7 = 24{,}5\ m^2$, $A_r = 2 \times 7 = 14\ m^2$

$LL = 24{,}5 \times 0{,}5 + 14 \times 0{,}2 = 15{,}05\ t$ (for 1-5

Snow load

Beşiktaş is in 2. snow load region and elevation is 70 m.

From TS 498/1987 → Table 11;

$P_{k0} = 75\ kg/m^2 = 0{,}075\ t/m^2$

$\theta = \tan^{-1}\left(\frac{2}{7{,}5}\right) = 14{,}93° < 30° \longrightarrow m = 1{,}0$

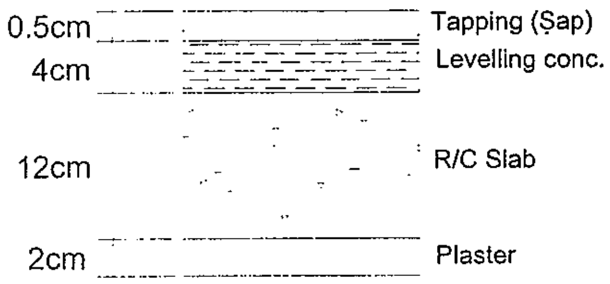
$P_k = m \cdot P_{k0} = 0{,}075\ t/m^2$

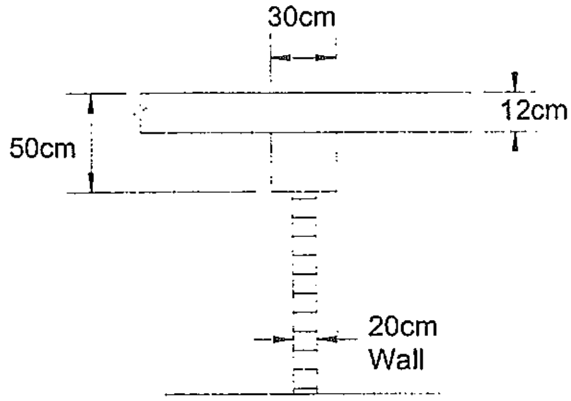
Snow load $= LL_s = 7 \times 5{,}5 \times 0{,}075 = 2{,}89\ t$

Reduced Live Load

From TS 498/1987 → Table 15;

Story	1	2	3	4	5	6
Reduction Coeff (β)	1	1	1	0,90	0,85	1
LL (t) on each floor	15,05	15,05	15,05	15,05	15,05	2,89
LL (t) on each column	73,63	58,58	43,53	28,48	14,93	2,89





1.

a) Compute the total dead load of the building.
b) Compute the <u>total</u> live load on the first floor columns. (Hint: Make live load reduction)

2.

a) Compute wind pressure/suction distribution and show on sketches for the wind <u>blowing from</u> z direction and x direction separately.
b) Compute and plot total wind forces for each floor and show on sketches.
c) Plot story shear diagrams for wind coming from z direction and x direction separately.

3.

a) Compute total base shear due to earthquake by assuming total floor dead load to be 1.2 ton/m^2 per story.
b) Compute and plot total earthquake forces acting on each floor and show on sketches.
c) Plot story shear diagram for earthquake loads.

HINTS:

- There is no attick room.
- Include column weights/masses in earthquake analysis and load calculation.
- q_{roof}=0.1t/m^2
- q_{wall20}=0.42t/m^2
- q_{wall10}=0.25t/m^2
- $q_{lev.conc.}$=2.2t/m^3
- Be careful about live load reduction.

(+1)

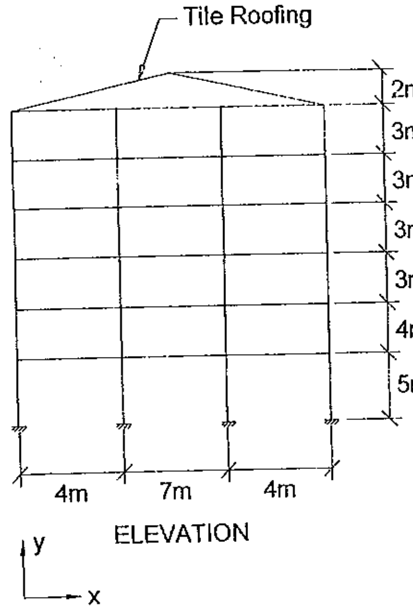
ō-12

Due November 1, 2004

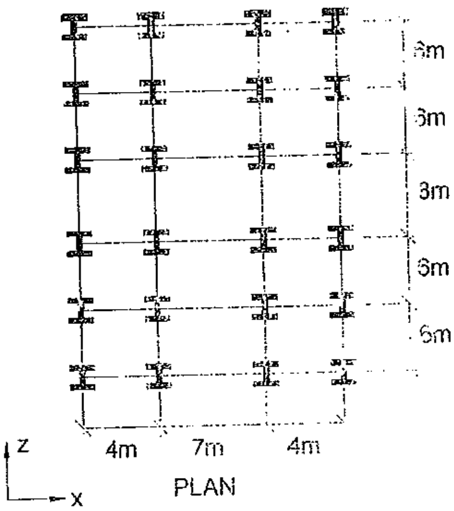
CE 485 - FUNDAMENTALS OF STEEL DESIGN

HOMEWORK 1

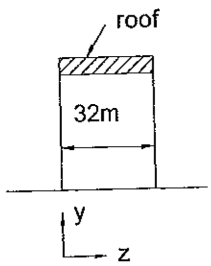
An office building is going to be built in Beşiktaş, Istanbul (Elevation=70m) on type Z3 soil. Its lateral load carrying system is made of steel frames with brick infill walls. The windows and the doors are shown on the plan. Column weights are expected to be 250 kg/m and beams are about 70 kg/m and the slab cross-section is as shown. Exterior walls are 25 cm thick, interior walls are 15 cm thick. There are no attic rooms.

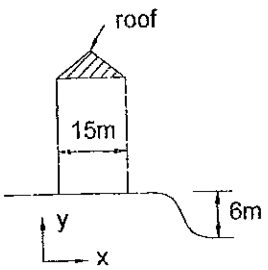


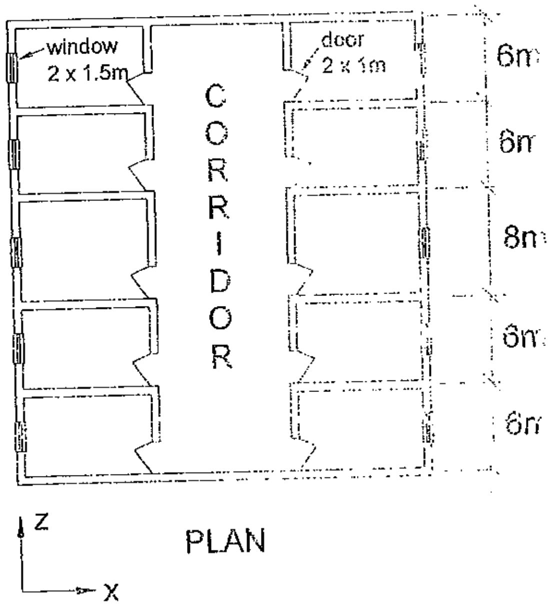
ELEVATION



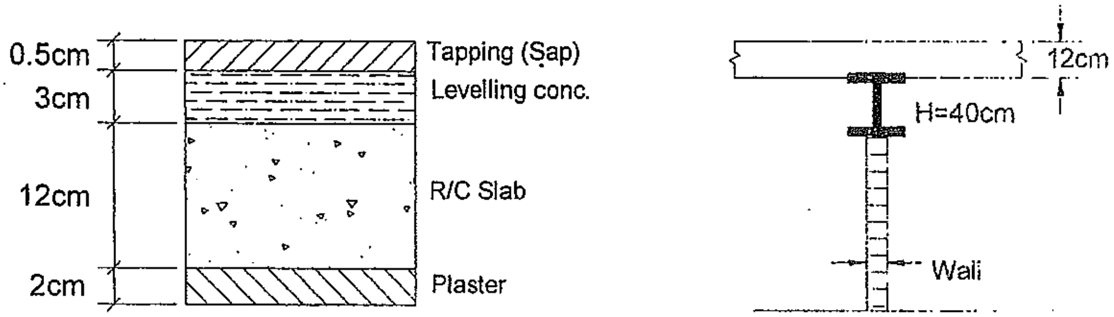
PLAN







PLAN



1.
  a) Compute the <u>total</u> dead load of the building.
  b) Compute the reduced live load on a typical interior column for the design of the structure against gravitational loads (list the reduced live load values in tabular format for all floors, for the selected column).

2.
  a) Compute wind pressure/suction distribution and show on sketches for the wind blowing in negative-x direction.
  b) Compute and plot wind force acting on each story and show on a sketch (negative-x direction).

3.
  a) Compute total base shear due to earthquake, assuming that story dead load is 1.20 ton/m^2 per story (includes all dead load for each story; roof load excluded).
  b) Compute and plot static equivalent earthquake loads acting on each story and show on sketch.

HINTS:

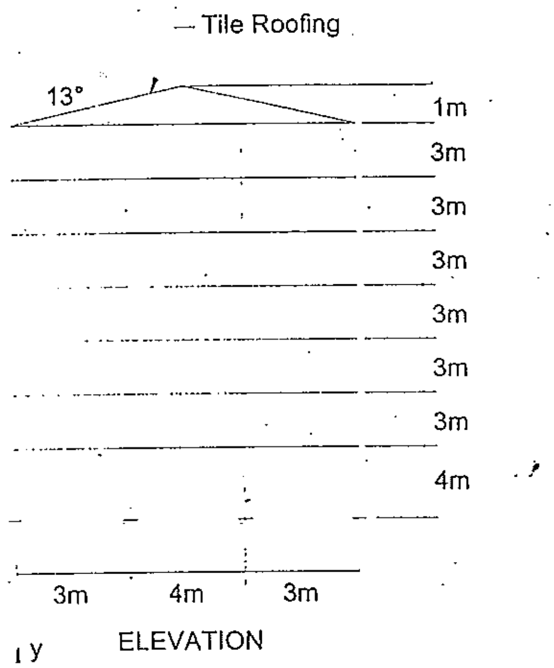
- q_{roof}=0.22t/m^2
- q_{wall25}=0.40t/m^2
- q_{wall15}=0.25t/m^2
- $q_{lev.conc.}$=2.3t/m^3

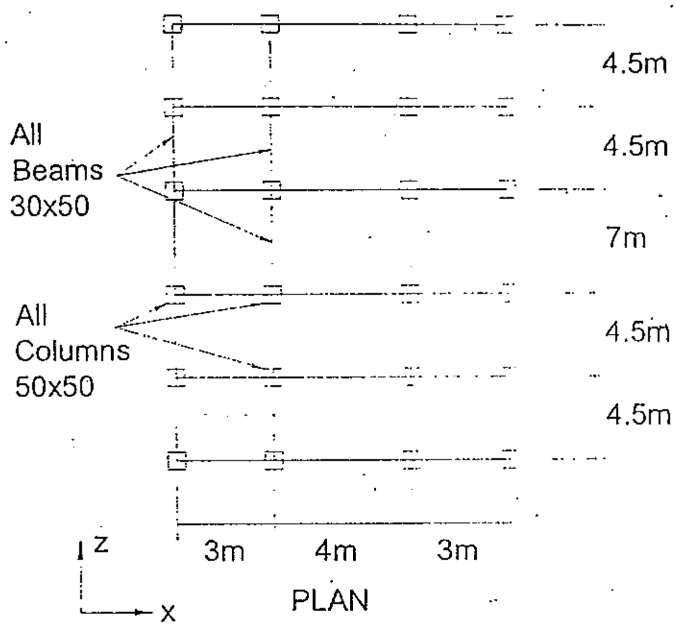
Due March 31, 2000

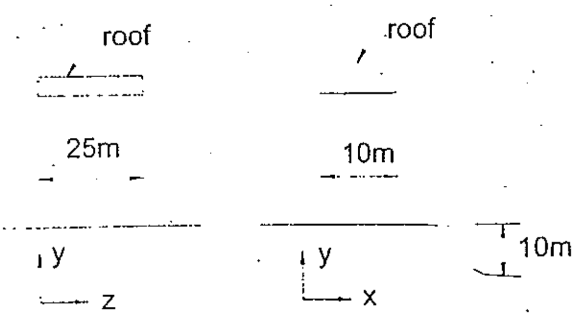
CE 485 - FUNDAMENTALS OF STEEL DESIGN

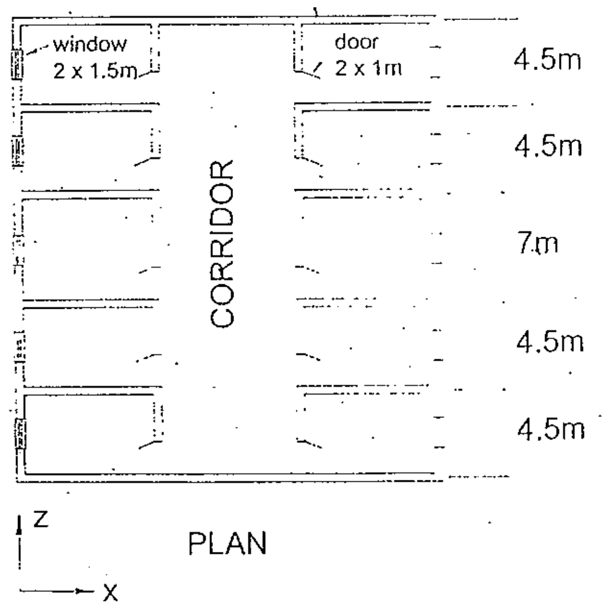
HOMEWORK 2

Assume that the office building given below has 12 cm thick slab. It is non-ductile frame with brick wall. The slab cross-section is given. The windows and doors are shown on the plan. All columns are 50x50cm and all beams are 30x50 cm. The structure is planned to be built in Erzincan, Çayırlı (H=1600m). The soil is very hard clay (Z2). Exterior walls are to be 20 cm, interior walls are to be 10 cm thick.









March 24, 2004

CE 485
HOMEWORK-2 SOLUTIONS

1) a) Compute the total dead load of the building.

Slab

$(14\times26\times0{,}12\times2{,}5)+(14\times26\times0{,}065\times2{,}2)=161{,}25$ t/story

Beams

$(4\times26+6\times14)\times0{,}0761=14{,}3$ t/story

Columns

$3m \longrightarrow 24\times0{,}256\times3{,}0=18{,}43$ t/story

$4m \longrightarrow 24\times0{,}256\times4{,}0=24{,}58$ t/story

$5m \longrightarrow 24\times0{,}256\times5{,}0=30{,}72$ t/story

Walls

3m interior $\longrightarrow \left[8\times(2{,}52\times4{,}0)+2\times(26\times2{,}52-10\times2\times1)\right]\times0{,}15=25{,}75$ t/story

3m exterior $\longrightarrow \left[2\times26\times2{,}52-10\times2\times1{,}5+2\times14\times2{,}52\right]\times0{,}30=51{,}48$ t/story

4m interior $\longrightarrow \left[8\times(3{,}52\times4{,}0)+2\times(26\times3{,}52-10\times2\times1)\right]\times0{,}15=38{,}35$ t/story

4m exterior $\longrightarrow \left[2\times26\times3{,}52-10\times2\times1{,}5+2\times14\times3{,}52\right]\times0{,}30=75{,}48$ t/story

5m interior $\longrightarrow \left[8\times(4{,}52\times4{,}0)+2\times(26\times4{,}52-10\times2\times1)\right]\times0{,}15=50{,}95$ t/story

5m exterior $\longrightarrow \left[2\times26\times4{,}52-10\times2\times1{,}5+2\times14\times4{,}52\right]\times0{,}30=99{,}48$ t/story

Roof

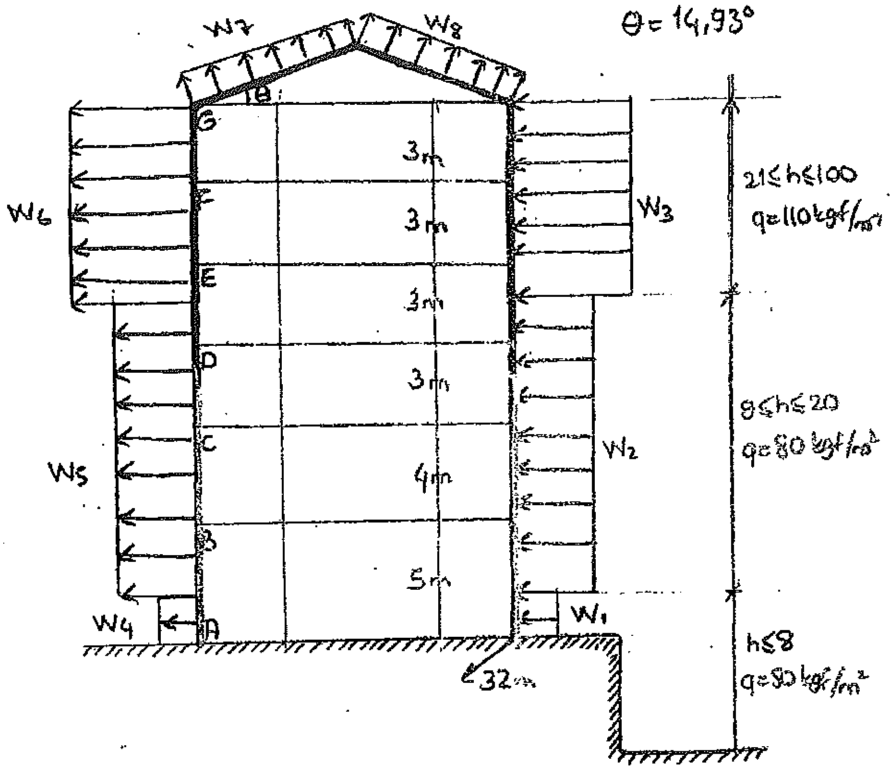
$14\times26\times0{,}25=91$ t

$$\text{Total Dead Load}=W_T=5\times(161{,}25+14{,}3)+2\times(18{,}43+25{,}75+51{,}48)+2\times(24{,}58+38{,}35+75{,}48)+1\times(30{,}72+50{,}95+99{,}48)+91$$

$$W_T=877{,}75+191{,}32+276{,}82+181{,}15+91=1618{,}04\text{ t}$$

5-7

2)a) Compute wind pressure/suction distribution and show on sketches for the wind blowing in negative x-direction.



$W_1 = 0{,}8 \cdot 50 = 40\ kgf/m^2$

$W_2 = 0{,}8 \cdot 80 = 64\ kgf/m^2$

$W_3 = 0{,}8 \cdot 110 = 88\ kgf/m^2$

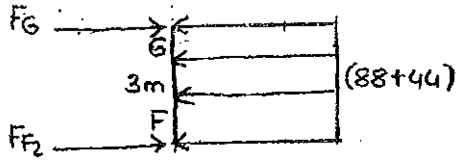
$W_4 = -0{,}4 \cdot 50 = -20\ kgf/m^2$

$W_5 = -0{,}4 \cdot 80 = -32\ kgf/m^2$

$W_6 = -0{,}4 \cdot 110 = -44\ kgf/m^2$

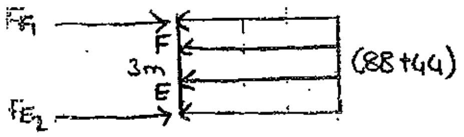
$W_7 = -0{,}4 \cdot 110 = -44\ kgf/m^2$

$W_8 = (1{,}2\sin\theta - 0{,}4) \cdot 10$
$= -10\ kgf/m^2$

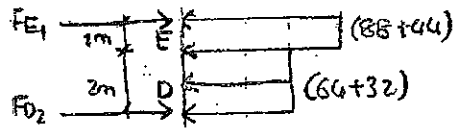


$F_G = F_{F_2} = 132 \times \frac{3}{2} \times 32 = 6336\ kgf = 6{,}34\ t$

$F_{roof} = (44 - 10) \times 32 \times 2 \times \frac{1}{1000} = 2{,}18\ t$

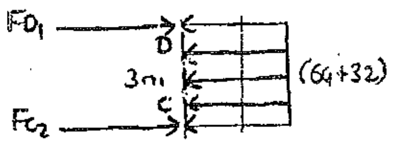


$F_{F_1} = F_{E_2} = 132 \times \frac{3}{2} \times 32 \times \frac{1}{1000} = 6{,}34\ t$

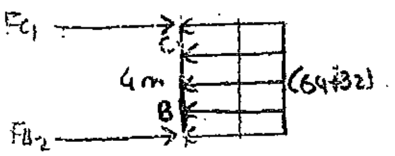


$F_{E_1} = 32 \times \frac{132 \times 1(m) \times 2{,}5(m) + 96 \times 2(m) \times 1(m)}{3(m) \times 1000} = 5{,}57\ t$

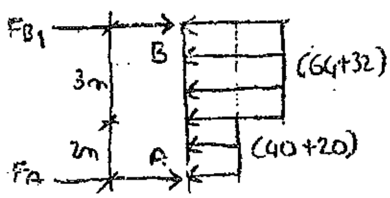
$F_{D_2} = 32 \times \frac{132 \times 1(m) \times 0{,}5(m) + 96 \times 2(m) \times 2(m)}{3(m) \times 1000} = 4{,}8\ t$



$F_{D_1} = F_{C_2} = 96 \times 32 \times \frac{3}{2} \times \frac{1}{1000} = 4{,}61\ t$



$F_{C_1} = F_{B_2} = 96 \times \frac{4}{2} \times 32 \times \frac{1}{1000} = 6{,}14\ t$



$F_{B_1} = 32 \times \frac{96 \times 3(m) \times 3{,}5(m) + 60 \times 2(m) \times 1(m)}{5(m) \times 1000} = 7{,}22\ t$

$F_A = 32 \times \frac{96 \times 3(m) \times 1{,}5(m) + 60 \times 2(m) \times 4(m)}{5(m) \times 1000} = 5{,}84\ t$

$P_A = 5{,}84\,t$

$P_B = F_{B_1} + F_{B_2} = 7{,}22 + 6{,}14 = 13{,}36\,t$

$P_C = F_{C_1} + F_{C_2} = 6{,}14 + 4{,}61 = 10{,}75\,t$

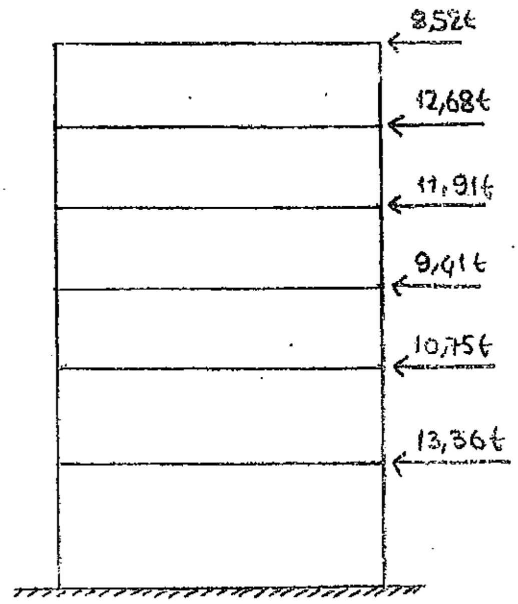
$P_D = F_{D_1} + F_{D_2} = 4{,}61 + 4{,}80 = 9{,}41\,t$

$P_E = F_{E_1} + F_{E_2} = 5{,}57 + 6{,}34 = 11{,}91\,t$

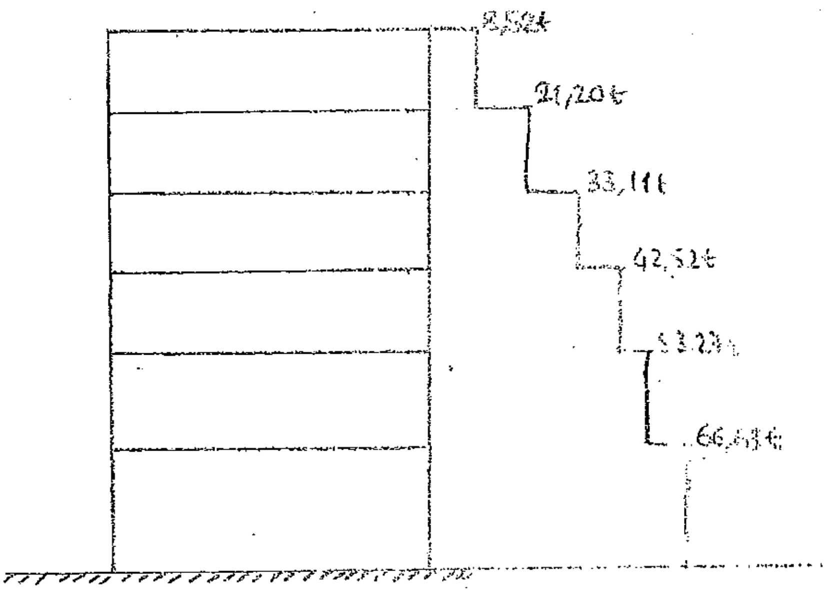
$P_F = F_{F_1} + F_{F_2} = 6{,}34 + 6{,}34 = 12{,}68\,t$

$P_G = F_G + F_{roof} = 6{,}34 + 2{,}18 = 8{,}52\,t$

b) Compute and plot wind force acting on each story and show on a sketch.



Wind load acting on each story



Story shear diagram

3)a) Compute total base shear due to earthquake, assuming that story dead load 1,20 t/m^2/story (includes all dead load for each story; roof load excluded)

Dead Load $= DL = 1{,}2 \times 15 \times 32 = 576$ t/story

Live Load $= LL = q_{corridor} \times 7 \times 32 + q_{room} \times 2 \times 4 \times 32$

$= 0{,}5 \times 7 \times 32 + 0{,}2 \times 2 \times 4 \times 32 = 137{,}6$ t/story

Snow Load $= LL_s = 15 \times 32 \times 0{,}075 = 36\,t$

Roof Load $= DL_{roof} = 0{,}22 \times 15 \times 32 = 105{,}6\,t$

$LL_{reduced} = (137{,}6 \times 5 + 36) \times 0{,}3 = 217{,}2\,t$

$\Sigma W = 576 \times 6 + 217{,}2 + 105{,}6 = 3778{,}8\,t$

Beşiktaş is in I. EQ zone; $(A_0 = 0,4)$

Soil Type : Z3

Importance factor → $I = 1,0$ (for office building)

$$T_1 = C_T \times H_N^{3/4} = 0,08 \times 21^{3/4} = 0,785 \text{ sec}$$

For soil type Z3 → $T_A = 0,15$ sec , $T_B = 0,60$ sec

Since $T_1 > T_B$ → $S(T_1) = 2,5 \left(\frac{T_B}{T_1}\right)^{0,8} = 2,5\left(\frac{0,60}{0,785}\right)^{0,8} = 2,02$

$$A(T_1) = A_0 \times I \times S(T_1) = 0,4 \times 1,0 \times 2,02 = 0,808 g$$

From EQ Code 6.5.1.3 (In the first and second seismic zones, structural systems of high ductility level shall be used for buildings with structural systems comprised of frames only)

$T_1 > T_A$ → $R_a(T_1) = R = 8$

$$V_t = \frac{\Sigma W \times A(T_1)}{R_a(T_1)} \geq 0,1 \times A_0 \times I \times \Sigma W$$

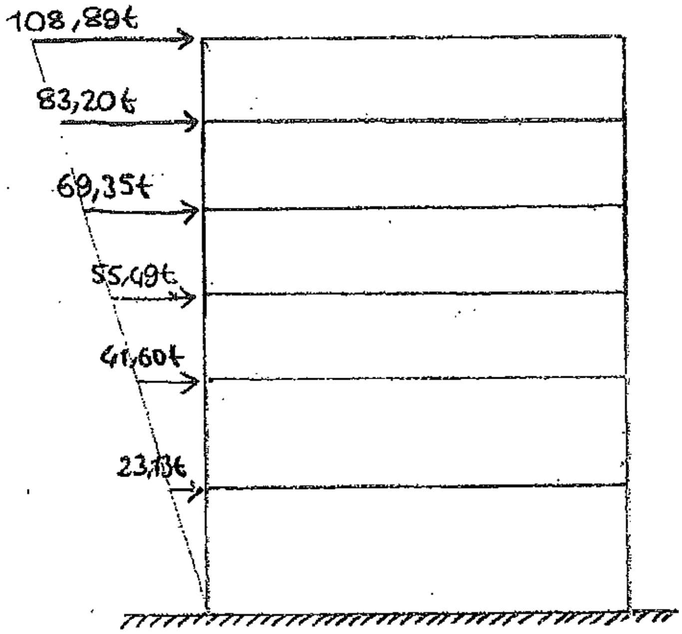
$$V_t = \frac{3778,8 \times 0,808}{8} = 381,66 t$$

Check : $0,1 \times A_0 \times I \times \Sigma W = 0,1 \times 0,4 \times 1,0 \times 3778,8 = 151,15 t < 381,66 t$ ✓

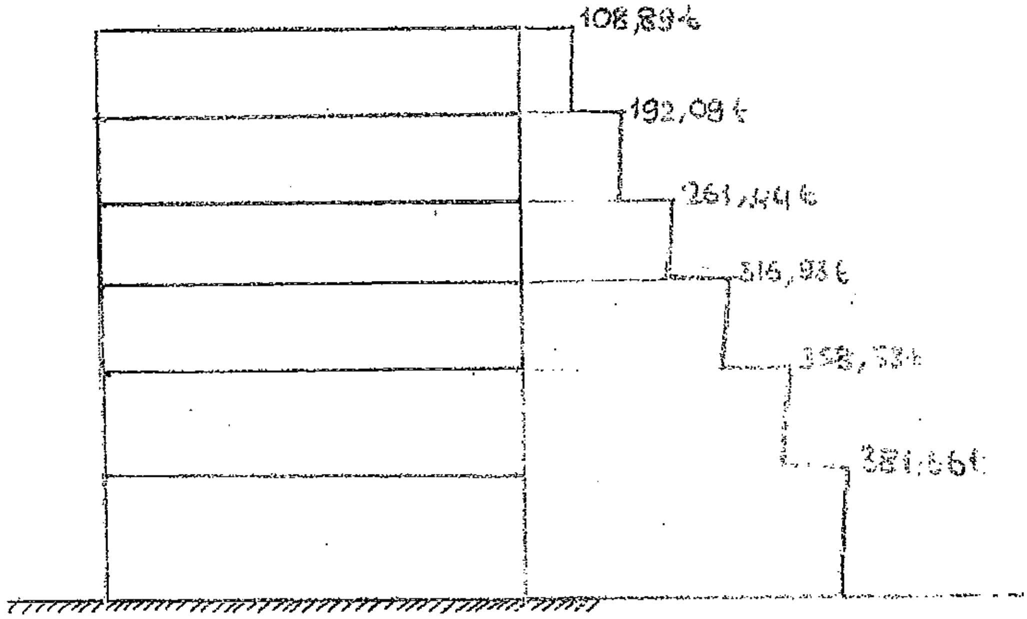
b) Compute and plot static equivalent earthquake loads acting on each story and show on sketch.

Story	Wi (t)	Hi (m)	Wi.Hi	$\frac{Wi.Hi}{\Sigma Wi.Hi}$	Fi (t)
1	617,28	5	3086,40	0,0606	23,13
2	617,28	9	5555,52	0,1090	41,60
3	617,28	12	7407,36	0,1454	55,49
4	617,28	15	9259,20	0,1817	69,35
5	617,28	18	11111,04	0,2180	83,20
6	692,4	21	14540,40	0,2853	108,89
Σ	3781,4		50959,92	1,000 ✓	381,66 ✓

$$F_i(t) = V_t \cdot \frac{W_i \cdot H_i}{\Sigma W_i \cdot H_i}$$



EQ loads acting on each story



Story shear force diagram