CVE 407: STRUCTURAL DESIGN II

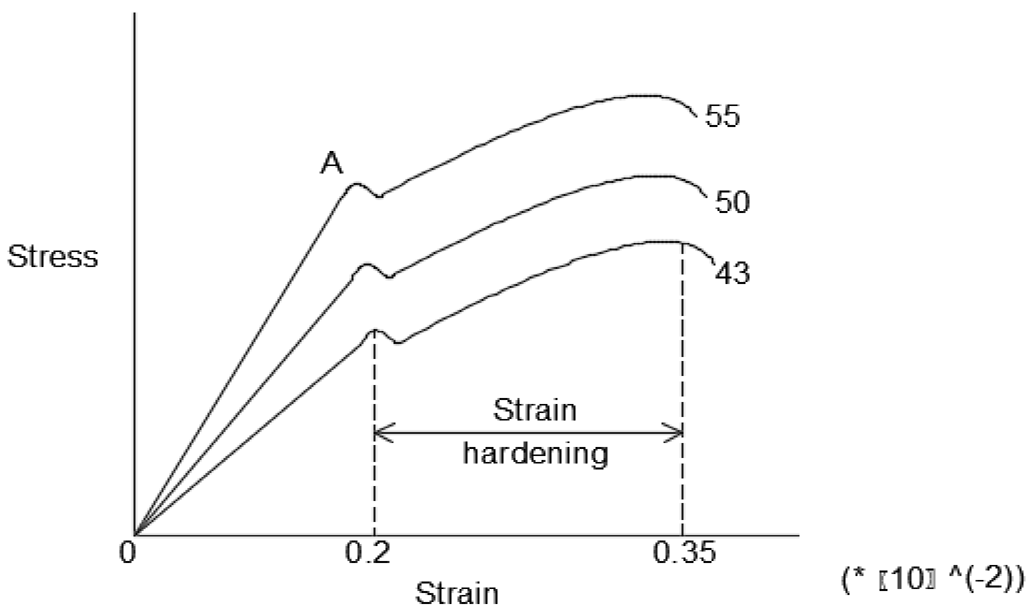
COURSE OUTLINE

- Design of steel structures: Loading of structures, materials. Design of axially loaded members, flexural members, design of connections – bolted, riveted and welded connections
- Limit state philosophy
- Design of structural elements in steel
- Design of special structures: bunkers, silos, chimneys, water tank, steel warehouse, steel bridge and pedestrian bridges

STRUCTURAL STEEL AND PROPERTIES

Grades of steel

- Grade 43
- Grade 50
- Grade 55



Properties of steel

- Durability
- Impact resistance
- Weld ability
- Strength

Design method

- Elastic design
- Plastic design
- Limit state design

Elastic design is a traditional method of design. It describes the steel to be perfectly elastic up to the yield point. In elastic theory, the section is sized so that the permissible stress will not be exceeded. The design is based on BS – 449

Plastic design takes into account the behavior of steel past the yield point and the theory is based on finding the load that can cause the structure to collapse. The working load is the collapsed load and it is divided by the factor of safety. This is also permitted by BS – 449

The **limit state design** takes into account the conditions that can make the structure structurally unfit. The design is based on the actual behavior of material and structure in use. There are two types of limit state design, namely:

- Serviceability limit state
- Ultimate limit state

DESIGN OF STRUCTURAL ELEMENTS

1.0 COLUMNS

They are compression members and primarily resist axial load. Most columns are subjected to axial load and moment.

Classification of cross-section

To prevent local buckling, limiting proportions for flanges and webs in axial compression are given in Table 7 of BS – 5590 Part 1. Check whether section is plastic, compact or semi-compact. Determine the effective length of the column. The depends on the end condition of the column (see Table 24 of the code)

Slenderness

The slenderness, λ of a column is defined as the ratio of effective length to radius of gyration about relevant axes.

$$\lambda = \frac{Effective\ length}{Radius\ of\ gyration\ about\ relevant\ axes} = \frac{L_e}{r}$$

Buckling takes place at the axis with the least radius of gyration $r = \sqrt{\frac{I}{A}}$

The code also states that for members resisting load other than wind load. λ must not exceed 180

Compression Resistance

The compression resistance of a strut is defined as:

$P_c = A_g p_c$ where A_g = gross sectional area

p_c = compressive strength obtained from Table 27 A-D of the code

Design Procedure (A)

1. Determine the effective length of column using Table 24 of the code
2. Determine the minimum possible area

$$A_{min} = \frac{P}{p_y} = \frac{load\ (P)}{design\ strength}$$

3. Select the section
4. Calculate slenderness ratios

$$\lambda_{xx} = \frac{L_{exx}}{r_{xx}}$$

$$\lambda_{yy} = \frac{L_{eyy}}{r_{yy}}$$

5. Refer to Table 25 for appropriate part of Table 27 A-D
6. Read off the values of P_{cx} and P_{cy} (compressive strength)
7. Limiting stress, p_c is the lower of P_{cx} and P_{cy}
8. Compute the compressive resistance [$P_c = A_g p_c$]
9. Check; $\frac{P}{P_C} \leq 1$

<u>Design Procedure (B)</u>

Compression members with axial load P and moment M_x *and* M_y. This type requires primarily two checks to be carried out.

1. Local capacity check
2. Overall buckling check

In each case, 2 procedures are given:

A. Simplified approach
B. More exact approach

A. **<u>Simplified Approach</u>**

1. <u>Local capacity check</u>: The members should be checked at point of greatest bending moment and axial load. This is usually at the end, but it could be within the column height if lateral loads are also applied.

i. Select a section and classify

ii. If section is slender, reduced value of P_y must be used. The stress reduction factor is obtained from Table 8 of BS – 5950.

iii. Calculate the axial capacity $P_y = A_g p_y$

iv. Check; $\frac{P}{P_y} \leq 1$

v. Calculate moment capacities

$$M_{cx} = S_{xx}.p_y \leq 1.2\, Z_{xx}.p_y$$
$$M_{cy} = S_{yy}.p_y \leq 1.2\, Z_{yy}.p_y$$

Where S = Plastic modulus

Z = Elastic modulus

vi. Check; $\frac{M_x}{Mcx} < 1$, $\frac{M_y}{Mcy} < 1$

vii. Check;

$$\frac{P}{P_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

2. <u>Overall buckling check</u>

i. Decide upon effective length using Table 24

ii. Calculate slenderness ratios

$$\lambda_{xx} = \frac{L_{exx}}{r_{xx}}$$
$$\lambda_{yy} = \frac{L_{eyy}}{r_{yy}}$$

iii. Refer to Table 25 for appropriate part of Table 27

iv. Read off the values of P_{cx} and P_{cy} (compressive strength)

v. The compressive stress, p_c is the lower of P_{cx} and P_{cy}

vi. Compressive resistance, $P_c = A_g p_c$

vii. Check; $\frac{P}{A_g p_c} = \frac{P}{P_c} \leq 1$

viii. Calculate $\beta = \frac{M_{xmin}}{M_{xmax}}$

ix. Read off “m” from Table 18 and calculate mM_{xmax}

x. Similarly, about y-y axis calculate $\beta = \frac{M_{ymin}}{M_{ymax}}$, then calculate mM_{ymax}

xi. For selected section, read off the value of “u” and “x” and calculate $\frac{\lambda}{x}$

xii. Read off the value of “v” from Table 14 and the value of “p_b” from Table 11

xiii. Calculate buckling resistance from the equation $M_b = S_{xx}.p_b$

xiv. Check;

$$\frac{P}{P_c} + \frac{mM_x}{M_b} + \frac{mM_y}{p_y.Z_{yy}} \leq 1$$

B. **<u>Exact Approach</u>**

1. <u>Local capacity check</u>

i. Select and classify the section

ii. If the section is neither plastic nor compact, simplified approach should be used

iii. Obtain values of reduced moment capacities $M_{rx}\ and\ M_{ry}$ from published tables in handbook

iv. Choose values of Z_1 and Z_2 from BS – 5950 plus 4.8.2

v. Check;

$$\left(\frac{M_x}{M_{rx}}\right)^{Z_1} + \left(\frac{M_y}{M_{ry}}\right)^{Z_2} \leq 1$$

2. <u>Overall buckling check</u>

i. Decide upon the effective length

ii. Calculate slenderness ratios

$$\lambda_{xx} = \frac{L_{exx}}{r_{xx}}$$

$$\lambda_{yy} = \frac{L_{eyy}}{r_{yy}}$$

iii. Refer to Table 25 for appropriate part of Table 27

iv. Read off the values of P_{cx} and P_{cy}

v. Calculate compressive resistance, $P_{cx} = A_g p_c$

$$P_{cy} = A_g p_y$$

vi. Calculate moment capacities

$$M_{cx} = S_{xx}.p_y \leq 1.2.Z_{xx}.p_y$$

$$M_{cy} = S_{yy}.p_y \leq 1.2.Z_{yy}.p_y$$

vii. Calculate $\beta = \frac{M_{xmin}}{M_{xmax}}$

viii. Read off the value of "m" from Table 18 and calculate mM_{xmax}

ix. Similarly, calculate $\beta = \frac{M_{ymin}}{M_{ymax}}$, then calculate mM_{ymax}

x. Read off the value of "u" and "x" from section handbook and calculate $\frac{\lambda}{x}$

xi. Read off the value of "v" from Table 14

xii. Read off the value of "p_b" from Table 11

xiii. Calculate buckling moment, $M_b = S_{xx}.p_b$

xiv. Also, calculate

$$M_{ax} = M_{cx}.\frac{\left(1 - {P}/{P_{cx}}\right)}{\left(1 + {0.5P}/{P_{cx}}\right)}$$

And, $M_{ax} = M_b.\left(1 - {P}/{P_{cy}}\right)$ (The lesser of the two is M_{ax})

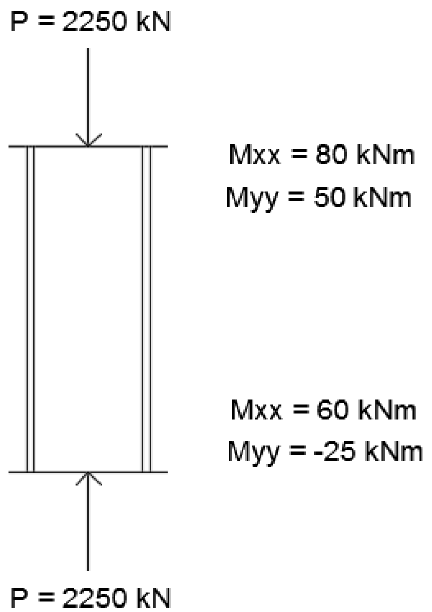
$$M_{ay} = M_{cy}.\frac{\left(1 - {P}/{P_{cy}}\right)}{\left(1 + {0.5P}/{P_{cy}}\right)}$$

xv. Final check;

$$\frac{mM_x}{M_{ax}} + \frac{mM_y}{M_{ay}} \leq 1$$

EXAMPLE 1

A steel column of effective length of 5.2m is subjected to end moments on both axes according to the diagram below. Find a suitable universal column that will be required to carry the load P. Use grade 43 steel and assume thickness is greater than 16mm.



Solution

T > 16mm; $p_y = 265 \ N/mm^2$

$$A_{min} = \frac{P}{p_y} = \frac{2250 * 10^3}{265} = 8490.56 \, mm^2$$

$A_{min} = 84.91 \, cm^2$

$Try \ 356 * 368 * 177 \ UC$

$A = 226 \, cm^2$

$r_{xx} = 15.9 \, cm$

$r_{yy} = 9.54 \, cm$

$Z_{xx} = 3103 \, cm^3$

$Z_{yy} = 1102 \, cm^3$

$S_{xx} = 3455 \, cm^3$

$S_{yy} = 1671 \, cm^3$

$u = 0.844$

$x = 15$

$T = 23.8 \, mm$

$b/T = 7.83$

Classify section

$$\varepsilon = \left(\frac{275}{p_y}\right)^{0.5} = \left(\frac{275}{265}\right)^{0.5} = 1.0187$$

$8.5\varepsilon = 8.5 * 1.0187 = 8.66$

$8.5\varepsilon > \frac{b}{T}$ (section is plastic)

Local capacity check

- Axial capacity, $P_y = A_g p_y = \frac{226*265*10^2}{10^3} = 5989\ kN$

Check 1; $\frac{P}{P_y} \leq 1$

$$\Longrightarrow \frac{2250}{5989} = 0.38 \leq 1 \quad (section\ is\ O.K.)$$

- Moment capacities

$M_{cx} = S_{xx}.p_y \leq 1.2.\ Z_{xx}.p_y$

$M_{cy} = S_{yy}.p_y \leq 1.2.Z_{yy}.p_y$

$$M_{cx} = \frac{3455 * 265 * 10^3}{10^6} \leq \frac{1.2 * 3103 * 265 * 10^3}{10^6}$$

$M_{cx} = 915.58\ kNm \leq 986.75\ kNm$

$\therefore M_{cx} = 915.58\ kNm$

$$M_{cy} = \frac{1671 * 265 * 10^3}{10^6} \leq \frac{1.2 * 1102 * 265 * 10^3}{10^6}$$

$M_{cy} = 442.82\ kNm \leq 350.44\ kNm$

$\therefore M_{cy} = 350.44\ kNm$

Check 2; $\frac{P}{P_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$

$$\Longrightarrow 0.38 + \frac{80}{915.58} + \frac{60}{350.44} = 0.64 < 1 \qquad (section\ is\ O.K.)$$

Overall buckling check

$$l_{exx} = l_{eyy} = 5.2\ m = 520\ m$$

$$\lambda_{xx} = \frac{L_{exx}}{r_{xx}} = \frac{520}{15.9} = 32.7$$

$$\lambda_{yy} = \frac{L_{eyy}}{r_{yy}} = \frac{520}{9.54} = 54.51$$

$$P_{cx} \Longrightarrow 30 - - - - - - - - - - 253$$

$$32.7 - - - - - - - - - x$$

$$35 - - - - - - - - - - 247$$

$$\frac{35 - 30}{32.7 - 30} = \frac{247 - 253}{x - 253}$$

$$\frac{5}{2.7} = \frac{-6}{x - 253}$$

$$5x = -16.2 + 1265$$

$$x = 249.76$$

$$\therefore P_{cx} = 249.76\ \ N/mm^2$$

$$P_{cy} \Longrightarrow 54 - - - - - - - - - - 206$$

$$54.51 - - - - - - - - - x$$

$$56 - - - - - - - - - - 202$$

$$\frac{56 - 54}{54.51 - 54} = \frac{202 - 206}{x - 206}$$

$$\frac{2}{0.51} = \frac{-4}{x - 206}$$

$2x = -2.04 + 412$

$x = 204.98$

$\therefore P_{cy} = 204.98\ N/mm^2$

$\therefore P_c = 204.98\ N/mm^2$

- Compressive resistance, $P_c = A_g p_c = \frac{226*204.98*10^2}{10^3} = 4632.548\ kN$

Check 3; $\frac{P}{P_c} \leq 1$

$\Longrightarrow \dfrac{2250}{4632.548} = 0.49 < 1 \quad (section\ is\ O.K.)$

$\beta = \dfrac{M_{min}}{M_{max}}$

$\beta_x = \dfrac{M_{xmin}}{M_{xmax}} = \dfrac{60}{80} = 0.75$

$m \Longrightarrow 0.7 - - - - - - - - - - 0.85$

$0.75 - - - - - - - - - x$

$0.8 - - - - - - - - - - 0.90$

$\dfrac{0.8 - 0.7}{0.75 - 0.7} = \dfrac{0.9 - 0.85}{x - 0.85}$

$\dfrac{0.1}{0.05} = \dfrac{0.05}{x - 0.85}$

$0.1x = 2.5 * 10^{-3} + 0.085$

$x = 0.875$

$\therefore m = 0.875$

$\Longrightarrow mM_{xmax} = 0.875 * 80 = 70\ kNm$

$\beta_y = \dfrac{M_{ymin}}{M_{ymax}} = \dfrac{-25}{50} = -0.5$

$\therefore m = 0.43$

$\Longrightarrow mM_{ymax} = 0.43 * 50 = 21.5\ kNm$

$$\frac{\lambda}{x} = \frac{54.51}{15} = 3.634$$

$v \Longrightarrow 3.5 - - - - - - - - - - 0.89$

$3.634\ - - - - - - - - - x$

$4.0 - - - - - - - - - -\ 0.86$

$$\frac{4 - 3.5}{3.634 - 3.5} = \frac{0.86 - 0.89}{x - 0.89}$$

$$\frac{0.5}{0.134} = \frac{-0.03}{x - 0.89}$$

$0.5x = -4.02 * 10^{-3} + 0.445$

$x = 0.88196$

$\therefore v = 0.88196$

$\lambda_{LT} = nuv\lambda = 1 * 0.844 * 0.88196 * 54.51 = 40.58$

$p_b \Longrightarrow 40\ - - - - - - - - - - 254$

$40.58\ - - - - - - - - - x$

$45 - - - - - - - - - -\ 242$

$$\frac{45 - 40}{40.58 - 40} = \frac{242 - 254}{x - 254}$$

$$\frac{5}{0.58} = \frac{-12}{x - 254}$$

$5x = -6.96 + 1270$

$x = 252.608$

$\therefore p_b = 252.608\ N/mm^2$

$$\Longrightarrow M_b = S_{xx}.p_b = \frac{3455 * 252.608 * 10^3}{10^6} = 827.76\ kNm$$

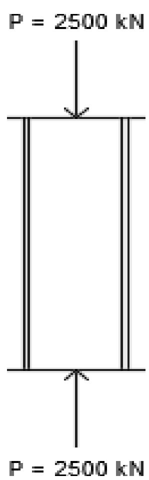
$$\Longrightarrow p_y.Z_{yy} = \frac{265 * 1102 * 10^3}{10^6} = 292.03\ kNm$$

Final check; $\frac{P}{P_c} + \frac{mM_x}{M_b} + \frac{mM_y}{p_y.Z_{yy}} \leq 1$

$$\Longrightarrow 0.49 + \frac{70}{827.76} + \frac{21.5}{292.03} = 0.65 < 1 \quad (section\ is\ O.K.)$$

EXAMPLE 2

A universal column section is required to resist an axial load of $2500\ kN$. The effective length about both axes is 6.2 m. Use grade 50 steel and select a suitable section. Assume T < 16 mm.



<u>Solution</u>

T < 16mm; $p_y = 355\ \ N/mm^2$

$$A_{min} = \frac{P}{p_y} = \frac{2500 * 10^3}{355} = 7042.25\ mm^2$$

$$A_{min} = 70.4\ cm^2$$

$Try\ 254\ *\ 254\ *\ 167\ UC$

$r_{xx} = 11.9\ cm$ $\quad r_{yy} = 6.81\ cm$ $\quad A = 213\ cm^2$

$T = 31.7\ mm$ $\quad \frac{b}{T} = 4.18$

Classify section

$$\varepsilon = \left(\frac{275}{p_y}\right)^{0.5} = \left(\frac{275}{355}\right)^{0.5} = 0.88$$

$8.5\varepsilon = 8.5 * 0.88 = 7.48$

$8.5\varepsilon > \frac{b}{T}$ (section is plastic)

Local capacity check

Axial capacity, $P_y = A_g p_y = \frac{213*355*10^2}{10^3} = 7561.5\ kN$

Check; $\frac{P}{P_y} = \frac{2500}{7561.5} = 0.33 \leq 1$ (section is O.K.)

Overall buckling check

$l_{exx} = l_{eyy} = 6.2\ m$

$$\lambda_{xx} = \frac{L_{exx}}{r_{xx}} = \frac{620}{11.9} = 52.1$$

$$\lambda_{yy} = \frac{L_{eyy}}{r_{yy}} = \frac{620}{6.81} = 91.0$$

$P_{cx} = 293\ N/mm^2\ \ (Interpolation)$

$P_{cy} = 178.5\ N/mm^2\ \ (Interpolation)$

$\therefore p_c = 178.5\ N/mm^2\ \ (lower\ of\ P_{cx}\ and\ P_{cy})$

Compressive resistance, $P_c = A_g p_c = \frac{213*178.5*10^2}{10^3} = 3802.1\ kN$

Check; $\frac{P}{P_c} = \frac{2500}{3802.1} = 0.66 \leq 1$ (section is O.K.)

2.0 BEAMS

Beams span between supports to carry lateral load which are resisted by bending and shear. However, deflections and local stresses are also important. Some beams may have full lateral restraint while others may not.

Beams with full lateral restraint

The design procedures include:

1. Calculate ultimate shear force, F_v and bending moment, M based on factor load
2. Determine the plastic modulus, $S = \frac{M}{p_y}$
3. Select a section
4. Calculate shear capacity, $P_v = 0.6t.D.p_y \nless F_v$
5. Check deflection due to un-factored imposed load (Table 5 of BS – 5950)
6. Where the beam has bend bearing, it may be necessary to check web bearing and buckling for very short span beams with heavy load. With high shear associated with the design moment, e.g. cantilever and short span beams with concentrated load such that $F_v > 0.6P_v$. It is necessary to check the moment capacity, $M_c = p_y(S - S_v.P_i) \nless M$

$$S_v = \frac{t.D^2}{4}$$

$$P_i = \frac{2.5F_v}{P_v} - 1.5$$

Beams without full lateral restraint

Check for lateral torsional buckling and follow the design procedure below:

1. Calculate plastic modulus, $S = \frac{M}{p_y}$ and choose a trial section
2. Determine equivalent uniform moment, $\overline{M}$ when $\overline{M} = mM_a$. Where M_a is the design moment at section considered, and m is the equivalent uniform moment factor from Table 13 of the code
3. Determine effective length, L_e between restraints
4. Determine equivalent slenderness ratio, $\lambda_{LT} = nuv\lambda$.

Where n = slenderness correction factor from Table 13

u = buckling parameters

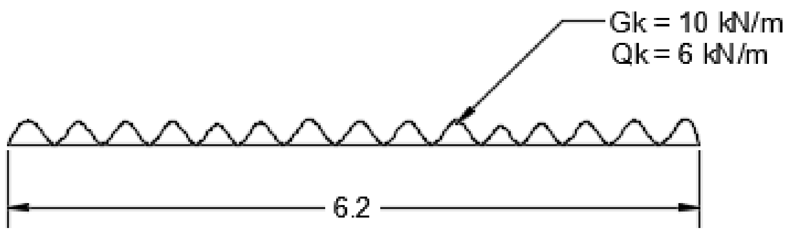
v = slenderness factor from Table 14

λ = slenderness ratio = $\frac{L_e}{radius\ of\ gyration\ (y-axis)}$

5. Determine design stress, p_b from Table 11
6. Check $M_b = S_x.p_b > \bar{M}$
7. Check for shear and deflection

EXAMPLE 1

Consider the steel beam below:



Use grade 43 steel

Solution

T < 16 mm

$p_y = 275\ N/mm^2$

Loading

$DL = 1.4G_k + 1.6Q_k = (1.4 * 10) + (1.6 * 6) = 23.6\ kN/m$

$$Bending\ moment, \quad M = \frac{wl^2}{8} = \frac{23.6 * 6.2^2}{8} = 113.4\ kNm$$

$$Shear\ force, \quad F_v = \frac{wl}{2} = \frac{23.6 * 6.2}{2} = 73.2\ kN$$

$Plastic\ modulus,\qquad S = \frac{M}{p_y} = \frac{113.4 * 10^6}{275 * 10^3} = 412.3\ cm^3$

$Try\ 356 * 127 * 39\ \ kg/m\ \ UB$

$S_x = 659\ cm^3 \qquad D = 353.4\ mm \qquad t = 6.6\ mm \qquad I_{xx} = 10170\ cm^4$

$T = 10.7\ mm \qquad \frac{b}{T} = 5.89$

<u>Classify section</u>

$$\varepsilon = \left(\frac{275}{p_y}\right)^{0.5} = \left(\frac{275}{275}\right)^{0.5} = 1$$

$8.5\varepsilon = 8.5 * 1 = 8.5$

$8.5\varepsilon > \frac{b}{T}$ (section is plastic)

Shear capacity, $P_v = 0.6t.D.p_y = \frac{0.6*6.6*353.4*275}{10^3} = 384.9kN > \ F_v$ (section is O.K.)

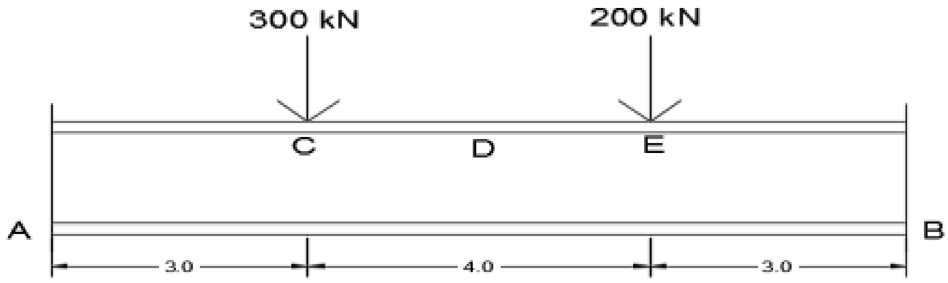
Deflection, $\delta = \frac{5Q_k.L^4}{384EI} = \frac{5*6*6.2^4*10^5}{384*10170*205} = 5.5\ mm$

$Limiting\ deflection = \frac{span}{360} = \frac{6200}{360} = 17.2\ mm > \ \delta$ (section is O.K.)

EXAMPLE 2

Consider the arrangement of the load below and select a suitable section for the UI section.

Use grade 50 steel



<u>Solution</u>

T > 16 mm

$p_y = 340\ N/mm^2$

Assume beam is restrained at loads

$\Sigma M_A = 0$

$-10R_B + (300 * 3) + (200 * 7) = 0$

$R_B = 230\ kN$

$R_A + R_B = 300 + 200$

$R_A = 500 - 230 = 270\ kN$

$\therefore\ F_v = 270\ kN$

$SF_A = 270\ kN$

$SF_C = 270 - 300 = -30\ kN$

$SF_D = -30\ kN$

$SF_E = -30 - 200 = -230\ kN$

$SF_B = -230 + 230 = 0$

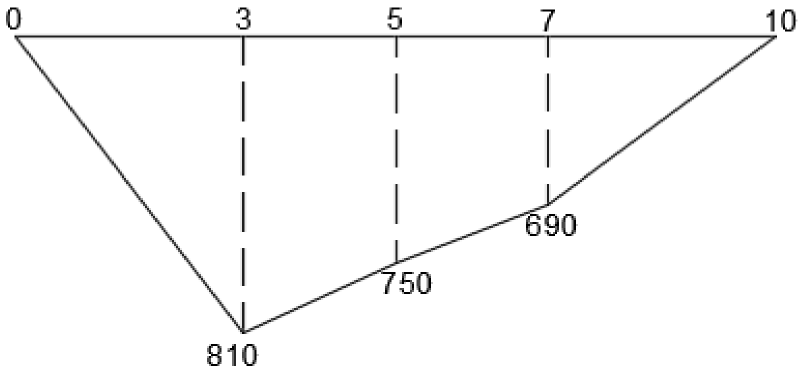
$SF_{equ} = R_A - 300 - 200$

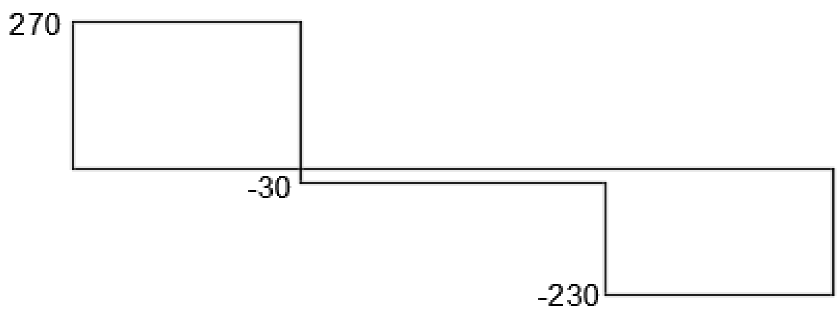
$M_{equ} = 270x - 300(x - 3) - 200(x - 7)$

at x = 3, $M_1 = 270(3) = 810\ kNm$

at x = 5, $M_2 = 270(5) - 300(5 - 3) = 750\ kNm$

at x = 7, $M_3 = 270(7) - 300(7 - 3) = 690\ kNm$





$Plastic\ modulus,\qquad S = \frac{M}{p_y} = \frac{810 * 10^6}{340 * 10^3} = 2382.4\ cm^3$

$Try\ 610\ *\ 229\ *\ 125\ \ kg/m\ \ UB$

$D = 612.2\ mm$

$t = 11.9\ mm$

$T = 19.6\ mm$

$\frac{b}{T} = 5.84$

$r_{yy} = 4.97\ cm$

$z_x = 3221\ cm^3$

$S_x = 3676\ cm^3$

$u = 0.873$

$x = 34.1$

<u>Classify section</u>

$$\varepsilon = \left(\frac{275}{p_y}\right)^{0.5} = \left(\frac{275}{340}\right)^{0.5} = 0.899$$

$8.5\varepsilon = 8.5 * 0.899 = 7.6$

$8.5\varepsilon > \frac{b}{T}$ (section is plastic)

Shear capacity, $P_v = 0.6t.D.p_y = \frac{0.6*11.9*612.2*340}{10^3} = 1486.2\ kN > F_v$ (section is O.K. in shear)

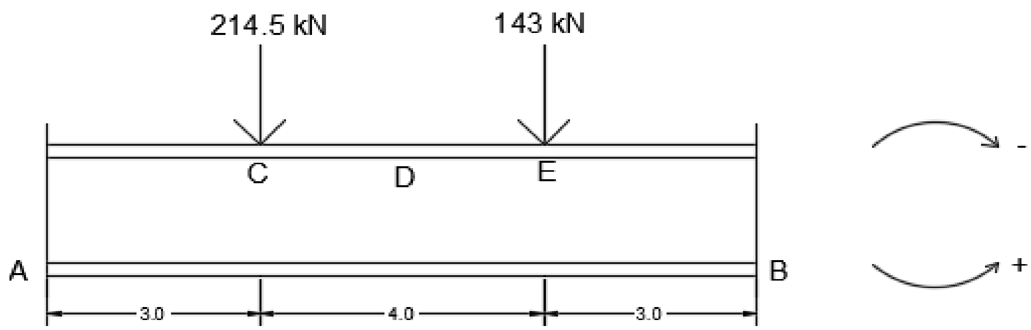
$0.6P_v = 0.6 * 1486.2 = 891.7\ kN > F_v$ (section is O.K.)

Description	Span AC	Span CE	Span EB
$\boldsymbol{\beta = \frac{M_{min}}{M_{max}}}$	0	0.85	0
m (Table 18)	0.57	0.925	0.57
$\boldsymbol{\bar{M} = mM_{max}}$	461.7	749.25	393.3
$\boldsymbol{l_e}$	3	4	3
$\boldsymbol{\lambda = \frac{L_{eyy}}{r_{yy}}}$	60.36	80.48	60.37
$\boldsymbol{\lambda/x}$	1.77	2.36	1.77
v (Table 14)	0.97	0.94	0.97
$\boldsymbol{\lambda_{LT} = nuv\lambda}$	51.11	66.04	51.11
$\boldsymbol{p_b}$**(Table 11)**	279	228.8	279
$\boldsymbol{M_b = S_x.p_b}$	1025.6	841.07	1025.6
$\boldsymbol{M_b - \bar{M}}$	563.9	91.82	632.3

Since $M_b - \bar{M}$, section is adequate

Check for deflection

71.5% of load is used



$\Sigma M_B = 0$

$(R_A * 10) - (214.5 * 7) - (143 * 3) = 0$

$R_A = 193.05\ kN$

$R_A + R_B = 214.5 + 143$

$R_B = 357.5 - 193.05 = 164.45\ kN$

$M_{equ} = 193.05x - 214.5(x-3) - 143(x-7)$

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

$$EI\frac{d^2y}{dx^2} = -193.05x + 214.5(x-3) + 143(x-7)$$

$$EI\frac{dy}{dx} = -\frac{193.05x^2}{2} + \frac{214.5(x-3)^2}{2} + \frac{143(x-7)^2}{2} + A$$

$$EI(y) = -\frac{193.05x^3}{6} + \frac{214.5(x-3)^3}{6} + \frac{143(x-7)^3}{6} + Ax + B$$

at x = l, l = 10m, y = 0, B = 0

$$EI(0) = -\frac{193.05l^3}{6} + \frac{214.5(l-3)^3}{6} + \frac{143(l-7)^3}{6} + Al + 0$$

$$0 = -\frac{193.05(10)^3}{6} + \frac{214.5(10-3)^3}{6} + \frac{143(10-7)^3}{6} + A(10)$$

$$0 = -32175 + 12262.25 + 643.5 + 10A$$

$10A = 19269.25$

$A = 1926.93\ m^2$

at l = 3m

$$EI(y) = -\frac{193.05(3)^3}{6} + 1926.93(3)$$

$$y = \frac{4912.065}{EI} = \frac{4912.065 * 10^9}{205 * 98610 * 10^4} = 24.3\ mm$$

at l = 5m

$$EI(y) = -\frac{193.05(5)^3}{6} + \frac{214.5(5-3)^3}{6} + 1926.93(5)$$

$$y = \frac{5898.775}{EI} = \frac{5898.775 * 10^9}{205 * 98610 * 10^4} = 29.18\ mm$$

at l = 7m

$$EI(y) = -\frac{193.05(7)^3}{6} + \frac{214.5(7-3)^3}{6} + 1926.93(7)$$

$$y = \frac{4740.485}{EI} = \frac{4740.485 * 10^9}{205 * 98610 * 10^4} = 23.45\ mm$$

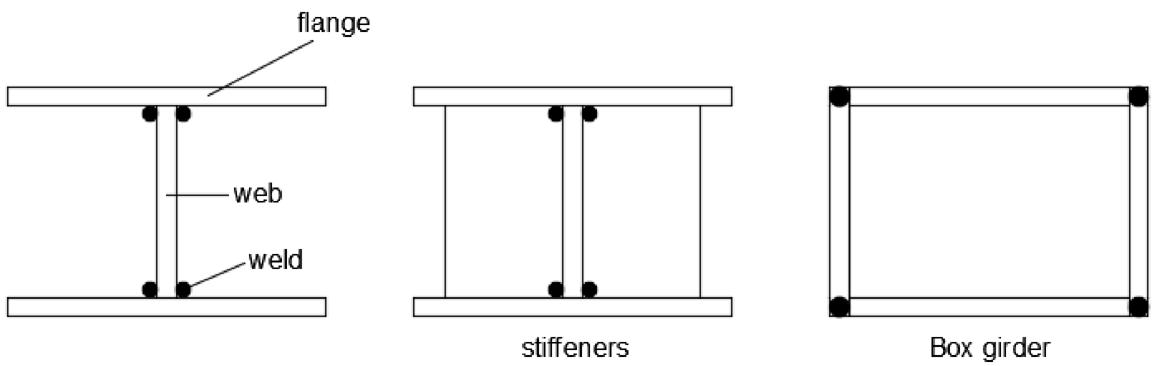
$$\delta_{max} = \frac{span}{360} = \frac{10000}{360} = 27.78\ mm$$

$y < \delta_{max}$ (section is adequate)

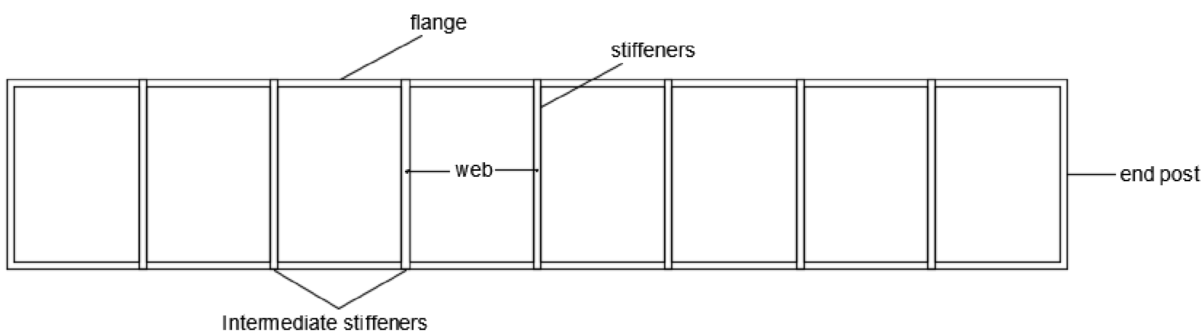
3.0 PLATE GIRDER DESIGN

Plate girder are used to carry larger loads over longer spans than are possible with universal or compound beams. Plate girders are constructed by welding steel plates together to form I-section. A closed section is termed a box girder. The web of a plate girder is relatively thin and stiffeners are required either to prevent buckling due to compression from bending and shear or to promote tension field action depending on the design method used.

Stiffeners are also required at load point and at supports. Depth is usually $1/_{10}\ to\ 1/_{12}$ of span and breadth of flange plate is usually $1/_{3}$ of the depth.



Typical Sections



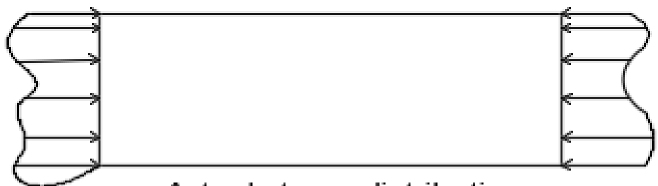
Elevation of plate girder

Post-buckling strength of plates

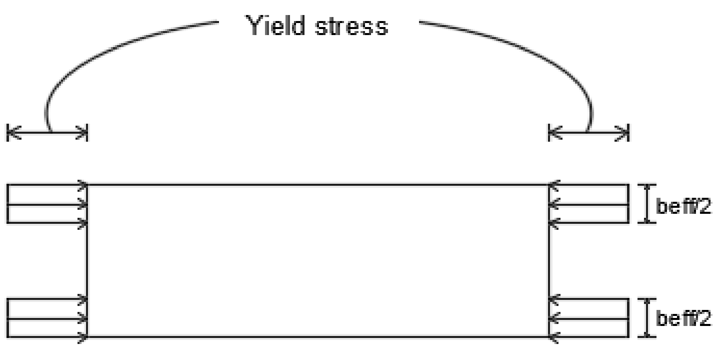
1. Plates in compression: Once a plate in compression start buckling, there is a re-distribution of stress. The outer part of the plate can support more load following buckling of the center portion. The behavior can be approximated by assuming that the load is carried by strips at the edges while the central region carries more load. The load is considered to be carried on an effective width given by;

$$\frac{b_{eff}}{b} = \sqrt{\frac{P_{cx}}{P_y}}\left(1 - 0.22\sqrt{\frac{P_{cr}}{P_y}}\right)$$

Then, $P_y = p_y . b_{eff} . t$

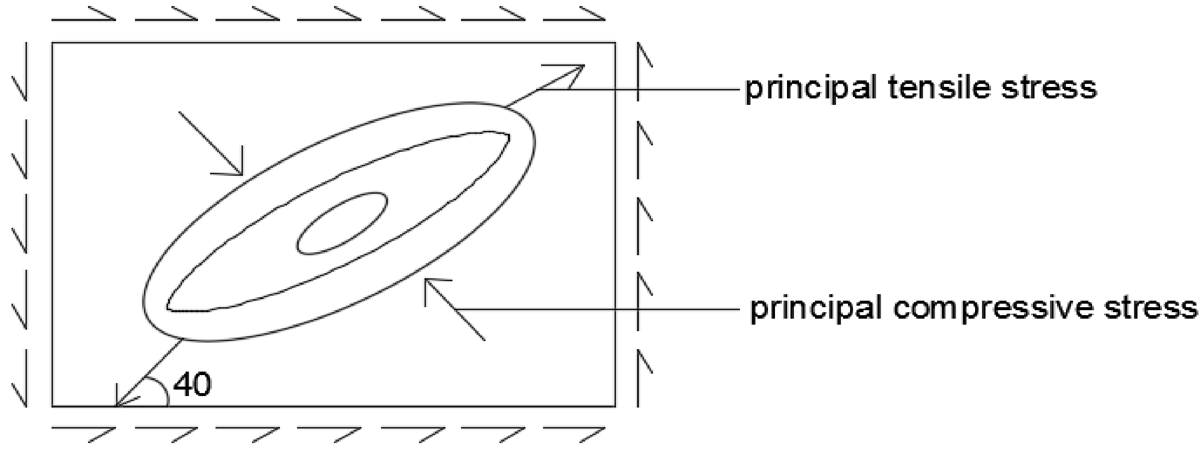


Actual stress distribution

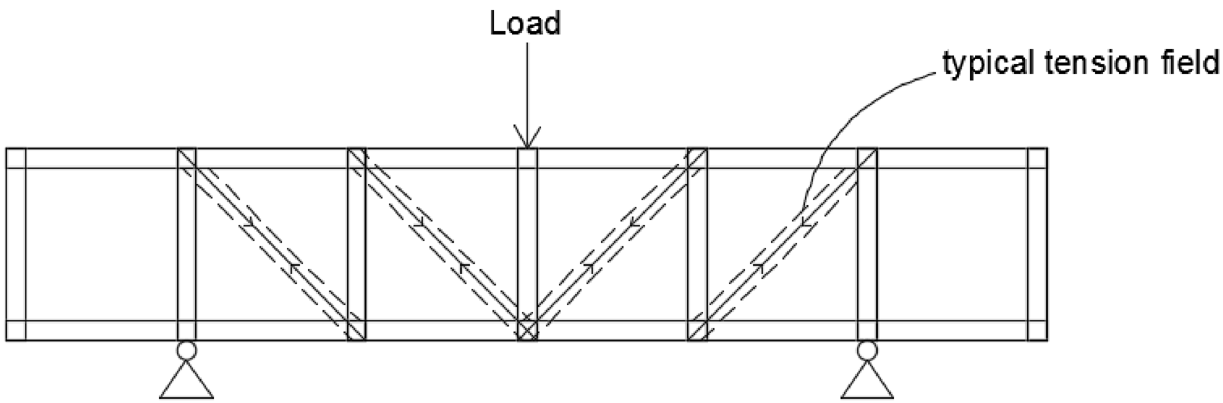


Effective width simplification

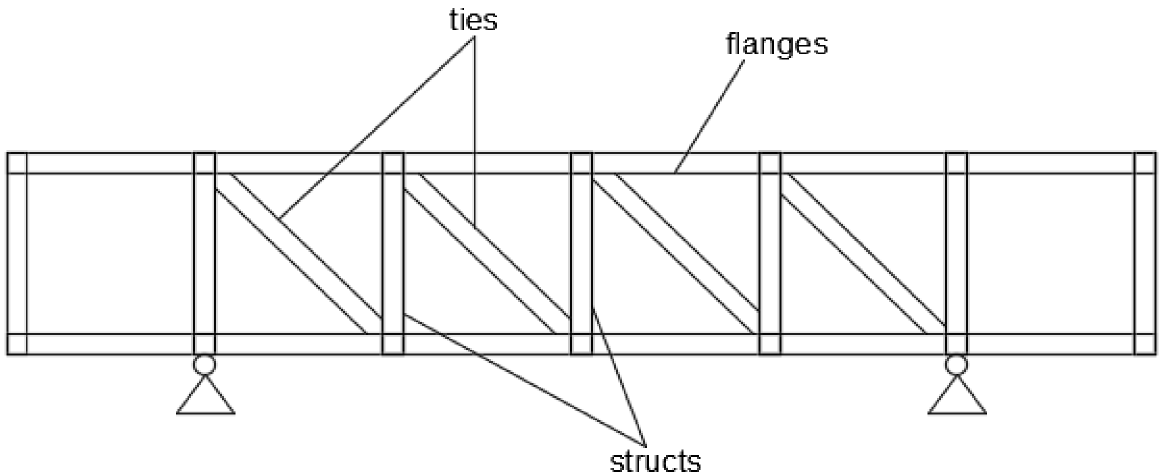
2. Plates in edge bending: These plates sustain load in excess of that causing buckling. Longitudinal stiffeners in the compression region are very effective in increasing the load that can be carried. Such stiffeners are commonly provided on deep plate girders used in bridges. A stiffener increases the load carrying capacity in a section.
3. Plates in shear (tension field action): In the formation of buckling as shown below



Direction of buckling in web



Tension field action as a result of transverse stiffeners in individual sub-panel



Types of stiffeners

1. Intermediate stiffeners: They divide the web into panels and prevent it from buckling. They also resist forces from tension field action if utilized and possible external loads.
2. Load bearing stiffeners: They are required at all points where substantial loads are applied and at supports to prevent local buckling and crushing of the web.
3. End post stiffeners: At the extreme end of the girder are the end posts. A single end post act as load bearing stiffeners and as support elements at the end of a member.
4. Double stiffeners: They are used in carrying vertical reaction from the girder.

Design and procedure of a plate girder

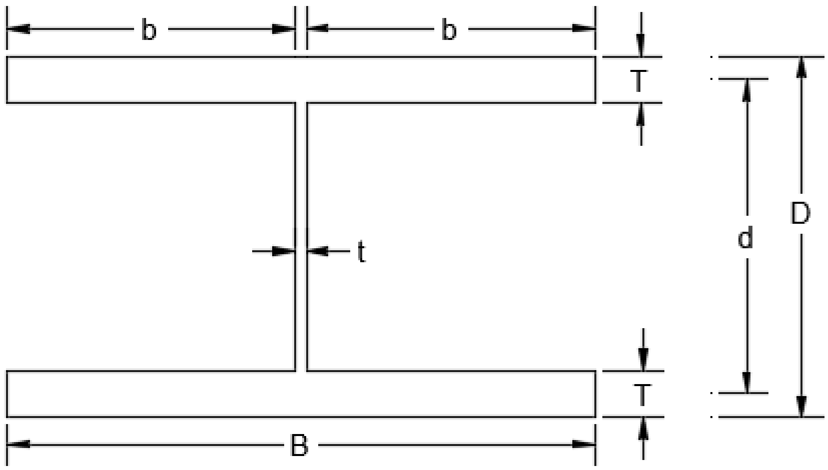
A.

1. Select depth of girder, i.e. $^1/_{10}\ to\ ^1/_{12}$ of span
2. Calculate the breadth of flanges $= \frac{depth}{3}$
3. Calculate the flange area, $A_f = \frac{M_{max}}{Depth * p_y}$

B.

1. Design using optimum chart method: The optimum depth is based on minimum area of cross-section considered which is derived as follows:

$$d_o = d = distance\ between\ centers\ of\ flanges$$



$$A_f = 2BT$$

$$A_w = t(D - 2T)$$

$$d_o = D - 2T$$

$R = \frac{d_o}{t}$ (ratio of depth to the thickness)

$$t = \frac{d_o}{R}$$

$$Plastic\ modulus, \qquad S = A_f . d_o$$

$$A_f = \frac{S}{d_o} ----------------(1)$$

$$A_w = d_o . t --------------(2)$$

$$A_w = \frac{d_o . d_o}{R} = \frac{{d_o}^2}{R} ------------(3)$$

$$A = A_f + A_w$$

$$A = \frac{2S}{d_o} + \frac{{d_o}^2}{R}$$

$$\frac{dA}{dd_0} = -\frac{2S}{{d_o}^2} + \frac{2d_0}{R} = 0$$

$$\frac{2d_o}{R} = \frac{2S}{{d_o}^2}$$

$$2RS = 2{d_o}^3$$

$$d_o = (RS)^{\frac{1}{3}}$$

<u>Moment capacity</u>

The moment capacity of a plate girder is given by:

a. If the depth to thickness ratio, $\frac{d}{t} < 63\varepsilon$, the moment capacity is determined from;

   $M_c = p_y.S_x = 1.2p_yZ_x$

b. If $\frac{d}{t} > 63\varepsilon$, then $M_c = BT(d+T).p_y = S_x.p_y$

<u>Shear capacity</u>

a. To prevent damage in handling, stiffeners spacing is given by $a > d,\ t \geq \frac{d}{250}$; $a < d,\ \ t \geq \frac{d}{250}\left(\frac{a}{d}\right)^{0.5}$. a = spacing of the stiffeners. The closer the stiffeners, the better the bearing capacity.
b. To avoid flange buckling due to web, stiffeners spacing is given by $a \leq 1.5d,\ t \geq \frac{d}{250}\left(\frac{P_{yf}}{455}\right)^{0.5}$. $Where\ p_{yf} = design\ strength\ of\ compression\ flange$

<u>Design of web for strength</u>

a. Design without tension field action. Design of shear buckling resistance is given by;

   $V_{cr} = q_{cr}.d.t$ $\qquad q_{cr} =$ critical shear strength in Table 21 A-D and it depends on the ratio of

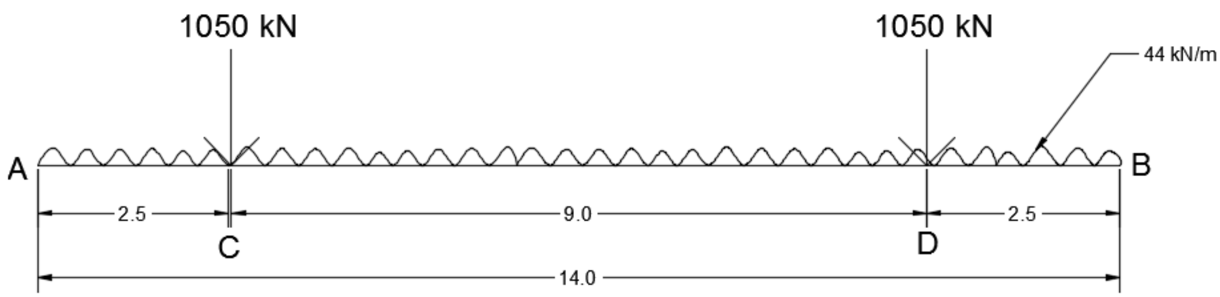
$$a - d \left(\frac{a}{d}\right)$$

b. Design using tension field action. Shear buckling resistance is given by; $V_b = q_b . d . t$

$$q_b = Basic\ shear\ strength\ in\ table\ 27\ A - D$$

EXAMPLE 1

A simply supported plate girder has a span of 14m and carries two concentrated loads on top of the flange at 2.5m from each end and consisting of $350\ kN$ dead load and $350\ kN$ life load. In addition, it carries a UDL of $20\ kN/m$ as dead load which include self-weight and an imposed load of $10\ kN/m$. Assume that the compression flange is fully restrained laterally and the girder is supported on heavy stiffened bracket at each end. Design using grade 43 steel.



<u>Solution</u>

T > 40 mm

$p_y = 245\ N/mm^2$

$DL = 1.4G_k + 1.6Q_k$

$For\ conc. load = (1.4 * 350) + (1.6 * 350) = 1050\ kN$

$For\ UDL = (1.4 * 20) + (1.6 * 10) = 44\ kN/m$

$\Sigma M_B = 0$

$(R_A * 14) - (1050 * 11.5) - (1050 * 2.5) - \left(44 * 14 * {}^{14}/_{2}\right) = 0$

$$R_A = \frac{19012}{14} = 1358\ kN$$

$$R_A + R_B = 1050 + 1050 + (44 * 14)$$

$$R_B = 2716 - 1358 = 1358\ kN$$

$$SF_{equ} = 1358 - 44x - 1050 - 1050$$

$$BM_{equ} = 1358x - \frac{44x^2}{2} - 1050(x - 2.5) - 1050(x - 11.5)$$

$at\ x = 2.5$

$$SF = 1358 - 44(2.5) = 1248\ kN \Longrightarrow 1248 - 1050 = 198\ kN$$

$$BM = 1358(2.5) - \frac{44(2.5)^2}{2} = 3257.5\ kNm$$

$at\ x = 7$

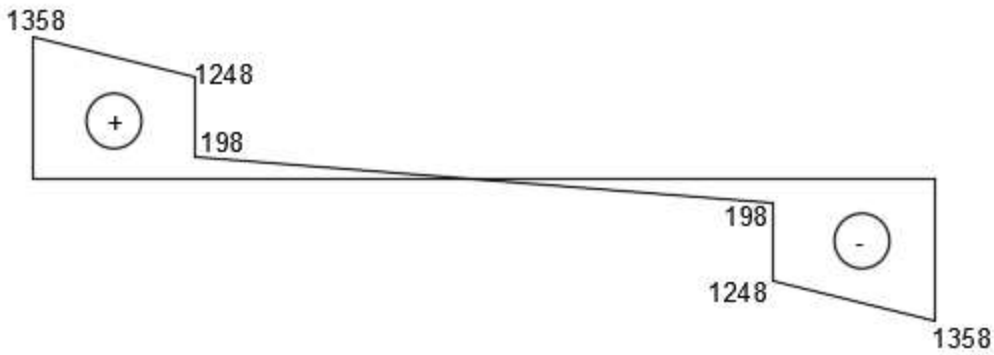
$$SF = 1358 - 44(7) - 1050 = 0$$

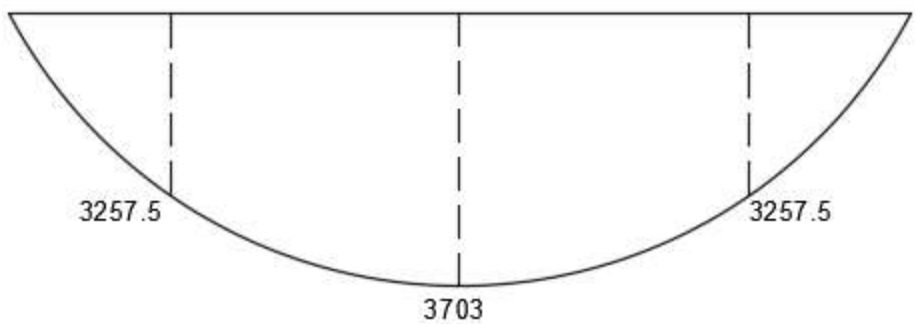
$$BM = 1358(7) - \frac{44(7)^2}{2} - 1050(7 - 2.5) = 3703\ kNm$$

$at\ x = 11.5$

$$SF = 1358 - 44(11.5) - 1050 = -198\ kN \Longrightarrow -198 - 1050 = -1248\ kN$$

$$BM = 1358(11.5) - \frac{44(11.5)^2}{2} - 1050(11.5 - 2.5) = 3257.5\ kNm$$





$M_{max} = 3703\ kNm$

$F_{max} = 1358\ kN$

<u>Girder section</u>

Depth of girder, $D = \frac{span}{10} = \frac{14000}{10} = 1400\ mm$

Breadth of girder, $B = \frac{D}{3} = \frac{1400}{3} = 466.7\ mm \Longrightarrow B \simeq 500\ mm$

Flange area, $A_f = \frac{M}{D * p_y} = \frac{3703*10^6}{1400*245} = 10795.9\ mm^2$

Assume, $A_f = 2BT = 2 * 500 * 40 = 40000\ mm^2$

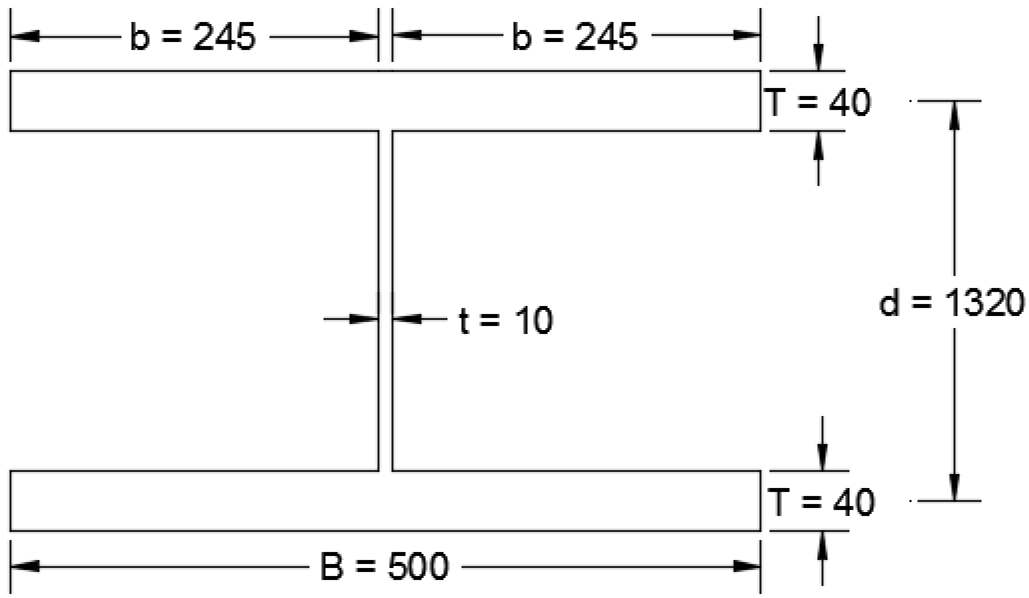
$t = 10\ mm\ (use\ if\ not\ given)$

Web area, $A_w = t(D - 2T) = 10\big(1400 - 2(40)\big) = 13200\ mm^2$

$A = A_f + A_w = 40000 + 13200 = 53200\ mm^2$

$$\frac{b}{T} = \frac{245}{40} = 6.125$$

$7.5\varepsilon = 7.5\left(\frac{275}{245}\right)^{0.5} = 8.45 > \frac{b}{T}$ (section is plastic)



<u>Design using optimum chart</u>

$T < 40$

$p_y = 265\ N/mm^2$

$$S = \frac{M}{p_y} = \frac{3703 * 10^6}{265 * 10^3} = 13973.6\ cm^3 \Rightarrow S \simeq 14 * 10^3 cm^3$$

From chart, $D = 1333\ mm$

Assume, $D = 1400\ mm$

Breadth of girder, $B = \frac{D}{3} = \frac{1400}{3} = 466.7\ mm \Rightarrow B \simeq 500\ mm$

Flange area, $A_f = \frac{M}{D * p_y} = \frac{3703*10^6}{1400*265} = 9981.13\ mm^2$

Provide flanges $500 * 30\ mm^2$

Assume, $A_f = 2BT = 2 * 500 * 30 = 30000\ mm^2$

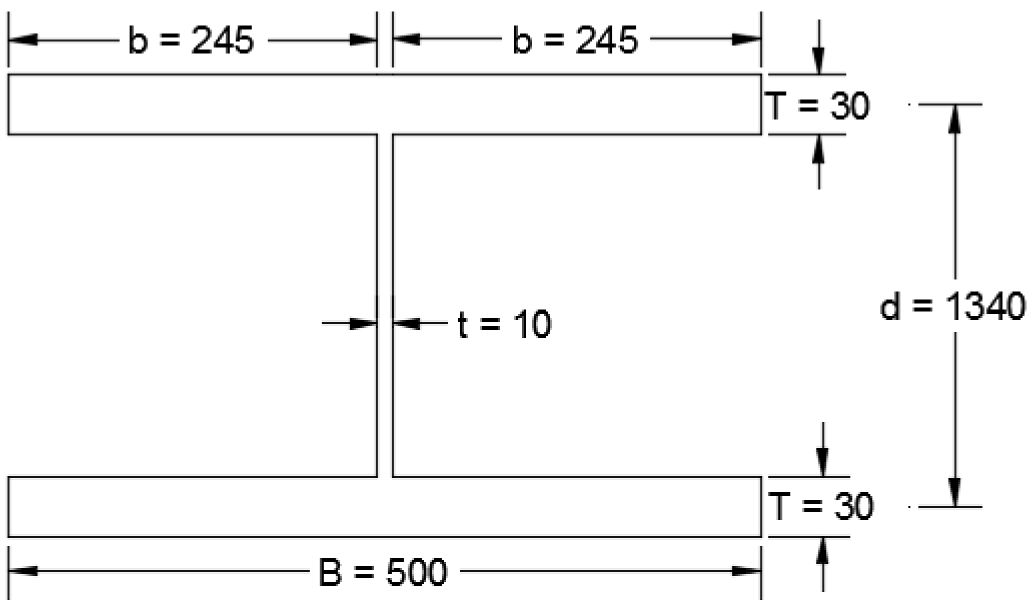
$t = 10\ mm$

Web area, $A_w = t(D - 2T) = 10\big(1400 - 2(30)\big) = 13400\ mm^2$

$A = A_f + A_w = 30000 + 13400 = 43400\ mm^2$

$$\frac{b}{T} = \frac{245}{30} = 8.17$$

$7.5\varepsilon = 7.5\left(\frac{275}{265}\right)^{0.5} = 7.64 < \frac{b}{T}$ (section is compact)



<u>Moment capacity</u>

$d = D - 2T = 1400 - 2(30) = 1340$

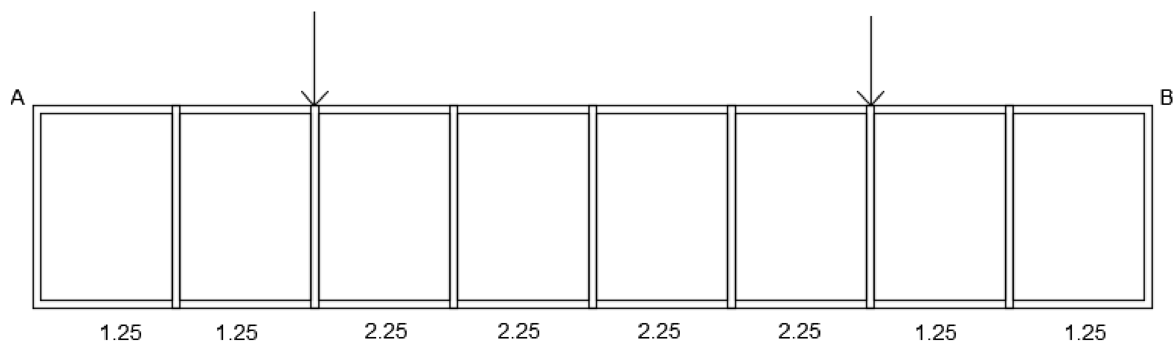
$$\frac{d}{t} = \frac{1340}{10} = 134$$

$$63\varepsilon = 63\left(\frac{275}{265}\right)^{0.5} = 64.17$$

$$Since\ \frac{d}{t} > 63\varepsilon\ \therefore M_c = BT(d+T).p_y$$

$$M_c = 500 * 30(1340 + 30) * 265 = 5445.75\ kNm > M_{max}$$

<u>Shear capacity</u>



a. To prevent damage in handling

$$a < d, \qquad t \geq \frac{d}{250}\left(\frac{a}{d}\right)^{0.5}$$

$$a = stiffener\ spacing = 1.25m = 1250\ mm$$

$$1250 < 1340, \qquad 10 \geq \frac{1340}{250} * \left(\frac{1250}{1340}\right)^{0.5}$$

$$10\ mm \geq 5.17\ mm \text{ (O.K.)}$$

b. To avoid flange buckling

$$a \leq 1.5d, \qquad t \geq \frac{d}{250}\left(\frac{P_{yf}}{455}\right)^{0.5}$$

$$1250 \leq 1.5 * 1340, \qquad 10 \geq \frac{1340}{250} * \left(\frac{265}{455}\right)^{0.5}$$

$$1250 \leq 2010, \qquad 10\ mm \geq 4.1\ mm \text{ (O.K.)}$$

Design strength of web

$V_{cr} = q_{cr}.d.t$ – Shear buckling resistance

$$\frac{d}{t} = 134$$

$$\frac{a}{d} = \frac{1250}{1340} = 0.93$$

From Table 21,

$130\ ------------ 104$

$134\ ------------ x$

$135 ------------ 96$

$$\frac{135-130}{134-130} = \frac{96-104}{x-104}$$

$$\frac{5}{4} = \frac{-8}{x-104}$$

$5(x-104) = -32$

$5x - 502 = -32$

$5x = 488$

$x = 97.6$

$\therefore\ q_{cr} = 97.6\ N/mm^2$

$$V_{cr} = q_{cr}.d.t = \frac{97.6 * 1340 * 10}{10^3} = 1307.8\,kN$$

$V_{cr} < F_{max}$ (not O.K.)

Redesign

$a = 0.83m = 830\ mm$

$$\frac{a}{d} = \frac{830}{1340} = 0.62$$

$q_{cr} = 149.8\ \ N/mm^2$

$$V_{cr} = q_{cr}.d.t = \frac{149.8 * 1340 * 10}{10^3} = 2007.3\ kN$$

$V_{cr} > F_{max}$ (O.K.)

Design of intermediate stiffeners (IS)

Trial size: $2\ No. 60\ mm * 8\ mm\ flats$

$$b_s \leq 19t_s\varepsilon = 19 * 18 * \left(\frac{275}{265}\right)^{0.5} = 154.8$$

$$Bt_s\varepsilon = 13 * 8 * \left(\frac{275}{265}\right)^{0.5} = 105.9$$

Minimum stiffeners

$a < \sqrt{2}d$

$830 < 1.414 * 1340$

$830 < 1894.8$

$$IS = 1.5d^3.\frac{t^3}{a^2} = 1.5 * 1340^3 * \frac{8^3}{830^2} = 2.68 * 10^6\ mm^4$$

$$IS = \frac{th^3}{12} = \frac{8 * 130^3}{12} = 1.4 * 10^6\ mm^4 \qquad (60 + 60 + 10) = 130$$

$Try\ 2\ No. 100 * 8\ mm^2 \quad outstand$

$$IS = \frac{th^3}{12} = \frac{8 * 210^3}{12} = 6.2 * 10^6\ mm^4 \qquad (100 + 100 + 10) = 210$$

Local bearing stiffeners

$Try\ 2\ No. 150 * 15\ mm^2$

Area of bearing, $A_b = 2 * 135 * 15 = 4050\ mm^2$

$p_y = 265\ N/mm^2$

$$A_b = \frac{0.8F_x}{p_y} = \frac{0.8 * 1358 * 10^3}{265} = 4099.6\ mm^2$$

$Try\ 2\ No. 150 * 20\ mm^2$

Area of bearing, $A_b = 2 * 135 * 20 = 5400\ mm^2$ (section is okay)

Buckling check

$A_s = 2b_st_s + 40t^2$

$A_s = 2 * 150 * 20 + 40 * 10^2 = 10000\ mm^2$

Assume compression flange is restrained

$l_e = 0.7l$

$$\lambda = \frac{l_e}{r_x}$$

$$I_x = \frac{20 * 310^3}{12} = 4.97 * 10^7\ mm^4$$

$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{4.97 * 10^7}{10000}} = 70.5\ mm$$

$$\lambda = \frac{0.7 * 1340}{70.5} = 13.3$$

Buckling resistance, $P_x = A_g p_c = \frac{10000*265}{10^3} = 2650\ kN > 1358\ kN$ $(O.K.)$

4.0 CONNECTIONS

Connections are needed to join members in trusses and lattice girders, plates to form built up members, beams to beams, beams to columns in structural frames and column to foundation. Connections are made by either bolting or welding. Bolting are of two kinds, these are:

a. Ordinary bolt in clearance holes
b. Friction grip bolt

Welding is also of two kinds, namely:

a. Fillet welds
b. Butt welds

Bolting is commonly used in fastening members on site. Example of ordinary bolt include:

- Black hexagonal head with nut and washer

BS – 4190 specifies the strength grade of ordinary bolt, these are:

a. Grade 4.6 – mild steel - $F_y = 235\ N/mm^2$
b. Grade 8.8 – high yield steel - $F_y = 627\ N/mm^2$

The main diameters of bolts used are: (16, 20, 22, 24, 27, 30) mm

Procedure for design of ordinary bolt

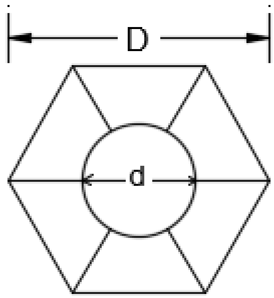
1. Determine shear capacity of bolt, $P_s = A_s p_s$ where p_s is the shear strength in Table 32 of the code

i. Bearing check: Taken as the lesser of:

a. Capacity of bolt, $P_{bb} = d.t.p_{bb}$ where d is the nominal diameter of bolt
   t is the thickness of connected ply
   p_{bb} is the bearing strength of bolt in Table 32
b. Capacity of connected plate, $P_{bs} = d.t.p_{bs} \leq {}^{1}/_{2}\, e.t.p_{bs}$

   Where p_{bs} is the bearing strength of connected parts (Table 33) e is the end distance obtained when $d.t.p_{bs} = {}^1\!/_2\, e.t.p_{bs}$

$\therefore e = 2d$



2. Direct tension joints: Check for tension capacity, $P_t = A_t p_t$

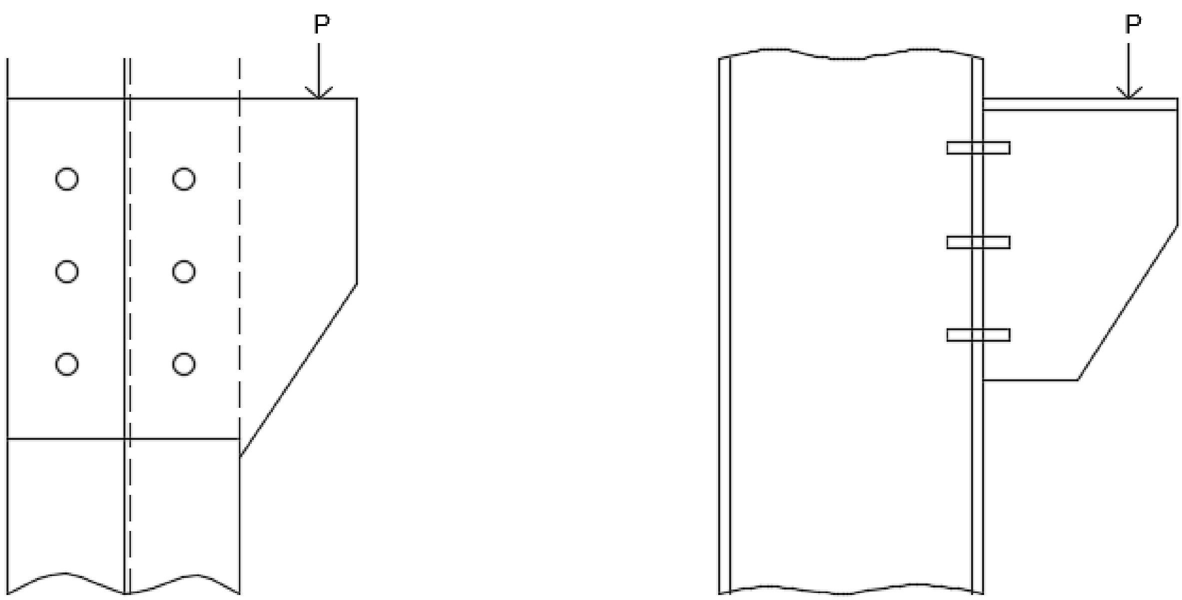
   Where p_t is the tension strength from Table 32

   A_t is the tensile stress area

3. Eccentric connections

a. Bolt group in direct shear & torsion

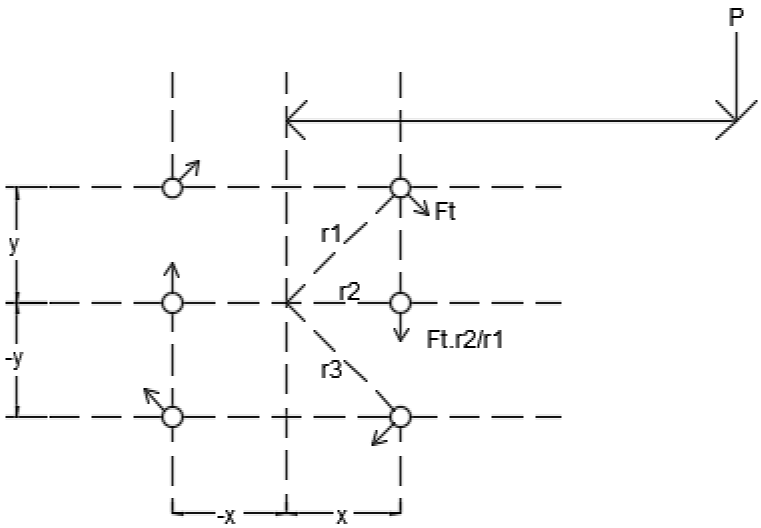
b. Bolt group in direct shear & tension

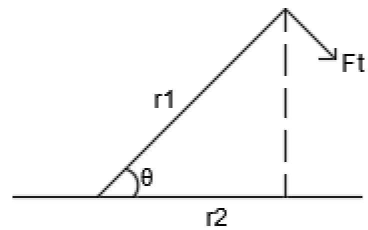


(a) (b)

a. <u>Bolt group in direct shear & torsion</u>

In the connection shown above, the moment is applied in the plane of the connection and the bolt group rotates about its center of gravity as in the diagram below





$$\theta = \frac{r_2}{r_1}$$

$$F_T.\frac{r_2}{r_1}$$

$$M_2 = F_T.\frac{r_2}{r_1}.r_1$$

$$M_2 = F_T.r_1$$

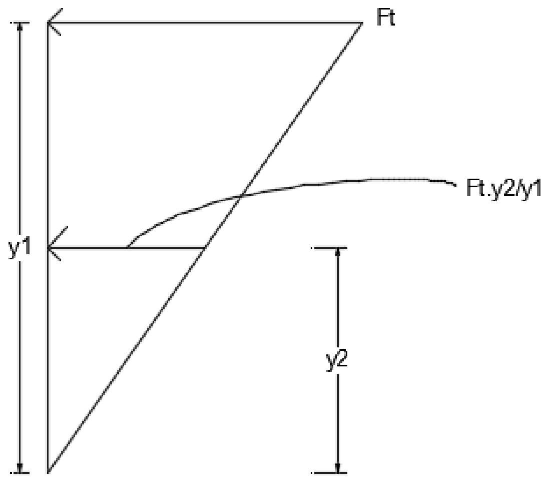
The moment of resistance of bolt group

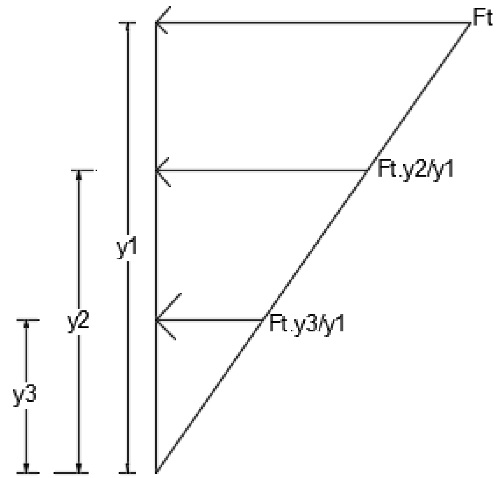
$$M_R = F_T.r_1 + F_T.\frac{r_2}{r_1}.r_2 + \cdots + F_T.\frac{r_n}{r_1}.r_n = P_e$$

$$M_R = \frac{F_T}{r_1}.(r_1{}^2 + r_2{}^2 + \cdots + r_n{}^2) = P_e$$

$$M_R = \frac{F_T}{r_1}.\sum r^2$$

$$\left[M_R = \frac{F_T}{r_1}.\left(\sum x^2 + \sum y^2\right)\right]$$





$$M_R = 2\left(F_T.y_1 + F_T.\frac{{y_2}^2}{y_1} + F_T.\frac{{y_3}^2}{y_1} + \cdots\right)$$

$$M_R = \frac{2F_T}{y_1}.\left({y_1}^2 + {y_2}^2 + \cdots +\right)$$

$$M_R = \frac{2F_T}{y_1}.\sum y^2 = P_e$$

$$\therefore F_T = \frac{P_e.y_1}{2.\sum y^2} \Rightarrow max.bolt\ tension$$

$$F_s = \frac{P}{No.\ of\ bolts}$$

Check for combine shear

$$\frac{F_s}{P_s} + \frac{F_T}{P_T} \leq 1.4$$

Friction grip bolts

They are made from high strength steel so they can be tightened to give a high shank tension. Bolts are manufactured in three types in accordance with BS – 4395. They include:

i. General grade strength similar to grade 8.8 of ordinary bolt and this is generally used
ii. Higher grade parallel shank
iii. Higher grade waisted shank

The bolt must be used with hardened steel washers to prevent damage to the connected parts.

Design procedure for general grade bolt

a. Bolts in shear: The capacity is the lesser of slip resistance and bearing capacity

   Slip resistance, $P_{sl} = 1.1k_s.u.p_0$ where $p_0 = max.shank\ tension$

   $u = slip\ factor\ taken\ as\ 0.45$

   Bearing resistance, $P_{bg} = d.t.p_{bg} \leq {}^{1}/_{3}\, e.t.p_{bg}$ where $d = nominal\ diameter\ of\ bolt$

   $t = thickness\ of\ connected\ ply$

   $e = end\ distance$

   $p_{bg} = bearing\ strength\ of\ parts\ connected\ (Table\ 34)$

For waisted shank, slip resistance is given by: $P_{sl} = 0.9k_s.u.p_o$

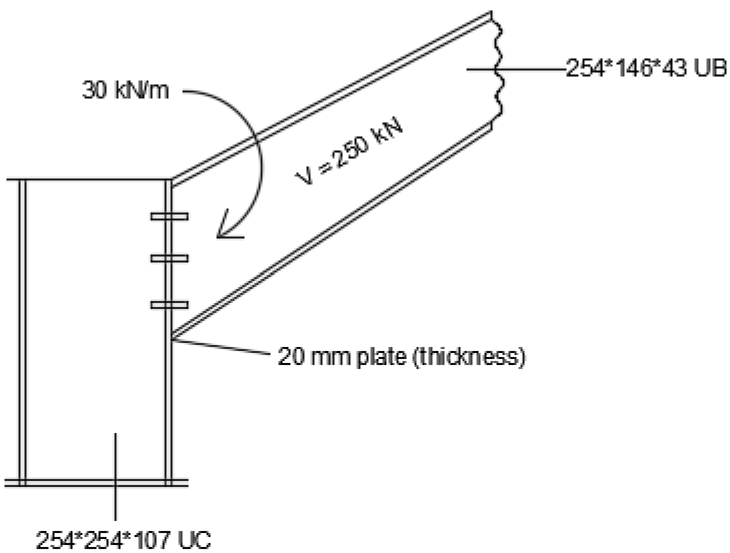
Tension capacity, $P_t = 0.9p_o$

Check;

$$\frac{F_S}{P_{sl}} + \frac{0.8F_T}{P_t} \leq 1$$

EXAMPLE 1

Determine the diameter of the ordinary bolt required for the eaves joint shown below. Use grade 4.6 bolt



Solution

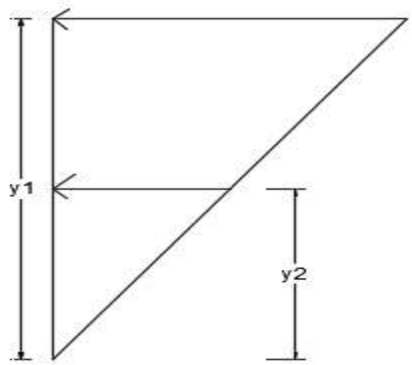
Total length of connection, $T_{total} = 220\ mm$

$\theta = 24\ mm$

$A_t = 353\ mm^2$

$d = 2.5d = 2.5 * 24 = 60\ mm$

$$e = \frac{220 - 2d}{2} = \frac{220 - 2(60)}{2} = 50\ mm$$



$y_1 = 0.12m$

$y_2 = 0.06m$

Max tension, $F_T = \frac{M.y_1}{2.\sum y^2} = \frac{30*0.12}{2(0.12^2+0.06^2)} = 100\ kN$

Vertical shear per bolt, $F_s = \frac{V}{No.\ of\ bolts} = \frac{250}{6} = 41.67\ kN$

Shear capacity, $P_s = A_s p_s = \frac{353*160}{10^3} = 56.48\ kN$

$P_s > F_s$ (O.K.)

Tension capacity, $P_t = A_t p_t = \frac{353*195}{10^3} = 68.84\ kN$

Bearing capacity of bolt, $P_{bb} = d.t.p_{bb} = \frac{24*20*435}{10^3} = 208.8\ kN$

$P_{bb} > F_T$ (O.K.)

Capacity of connected parts, $P_{bs} = d.t.p_{bs} \leq {}^1\!/_2\, e.t.p_{bs}$

$$P_{bs} = \frac{24*20*460}{10^3} \leq \frac{0.5*50*20*460}{10^3}$$

$P_{bs} = 220.8 \leq 230$

Design of eaves joint using general grip friction bolt

Bearing resistance, $P_{bg} = d.t.p_{bg} \leq {}^{1}/_{3}\, e.t.p_{bg}$

$$P_{bg} = \frac{24 * 20 * 825}{10^3} \leq \frac{{}^{1}/_{3} * 50 * 20 * 825}{10^3}$$

$P_{bg} = 396 \leq 275$

Slip resistance, $P_{sl} = 1.1k_s.u.p_0 = 1.1 * 1.0 * 0.45 * 207 = 102.47\ kN$

$P_{sl} > F_s$ (O.K.)

Tension capacity, $P_t = 0.9p_o = 0.9 * 207 = 186.3\ kN$

$P_t > F_T$ (O.K.)

Combine shear

$$\frac{F_S}{P_{sl}} + \frac{0.8F_T}{P_t} \leq 1$$

$$\frac{41.67}{102.47} + \frac{0.8 * 100}{186.3} \leq 1$$

$0.84 < 1$ (O.K.)

<u>Welding</u>

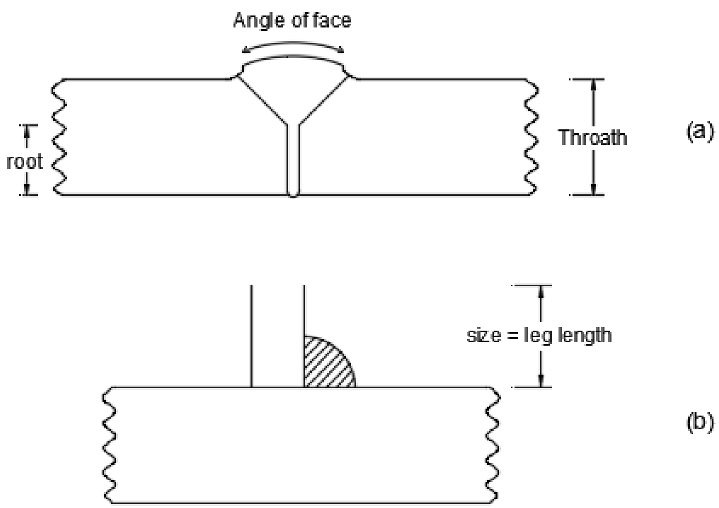
Welding is the process of joining metal parts by fussing them and filling with molten metal from the electrode. Welding produces neat, strong and more efficient joint than are possible with bolting. However, it should be carried out under closed supervision which may only be possible in the fabrication workshop. Though site welding can be done but it is costly and defects are more likely to occur. Electric arc welding is the main system used for structural steel welding. They are of two types:

i. Manual arc welding
ii. Automatic arc welding

Types of welds

i. Butt weld

ii. Fillet weld



5.0 COLUMN BASE DESIGN

Column base transmit axial load, horizontal loads and moment from the steel column to the column foundation. There are three types of bases, these are:

i. Slab base
ii. Gusseted base
iii. Pocket base

Design of slab base

i. Determine minimum plate thickness

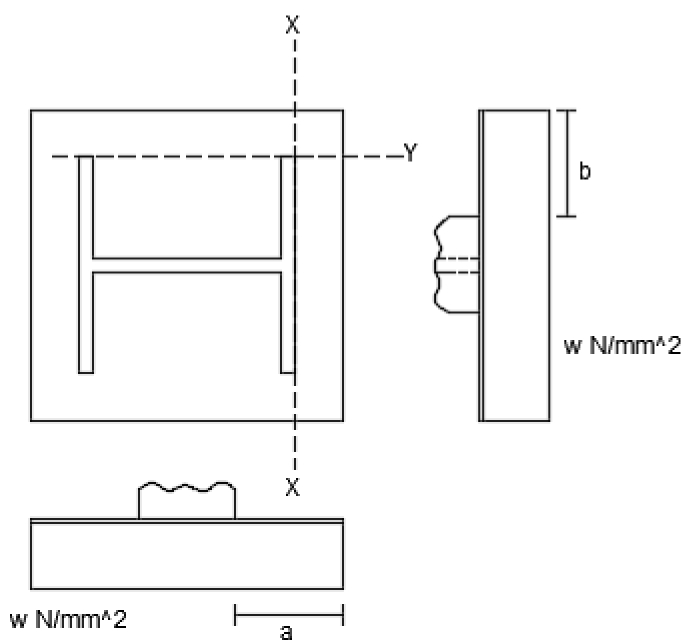
$$t = \left(\frac{2.5}{p_{yp}}.w.(a^2 - 0.3b^2)\right)^{0.5} < flange\ thickness$$

Where $a = greater\ projection\ of\ plate$

$b = lesser\ projection\ of\ plate$

$w = pressure\ under\ base$

$p_{yp} = design\ strength$



$$M_x = \frac{wa^2}{2}$$

$$M_y = \frac{wb^2}{2} \qquad since\ a > b$$

$$M_x = \frac{wa^2}{2} - 0.3.\frac{wb^2}{2} = \frac{w}{2}.(a^2 - 0.3b^2) ------------(I)$$

Moment capacity, $M_c = 1.2p_{yp}.Z = 1.2p_{yp}.\frac{t^2}{6} ----------(II)$

$equating\ (I)$ & (II)

$$1.2p_{yp}.\frac{t}{6} = \frac{w}{2}.(a^2 - 0.3b^2) \Longrightarrow t = \left(\frac{2.5}{p_{yp}}.w.(a^2 - 0.3b^2)\right)^{0.5}$$

EXAMPLE 1

A column consisting of 305*305*198 UC. It carries an axial dead load of $1200\ kN$ and axial life load of $600\ kN$. Adopting a square base plate, determine the size and thickness required. Assume cube strength of $25\ N/mm^2$. Use grade 43 steel and 4. No. of grade 8.8 bolt. Determine also the size of bolt used.

<u>Solution</u>

Design load, $DL = 1.4G_k + 1.6Q_k = 1.4(1200) + 1.6(600) = 2640\ kN$

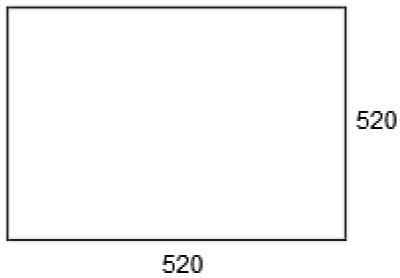
Bearing strength, $b_s = 0.4f_{cu} = 0.4 * 25 = 10\ N/mm^2$

$$A_{min}\ of\ base\ plate = \frac{load}{bearing\ strength} = \frac{2640 * 10^3}{10} = 264000 = 2.64 * 10^5\ mm^2$$

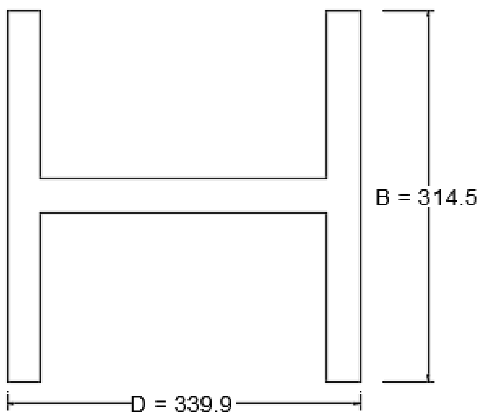
$A = l^2 \quad \therefore l = 513.8 \simeq 514m$

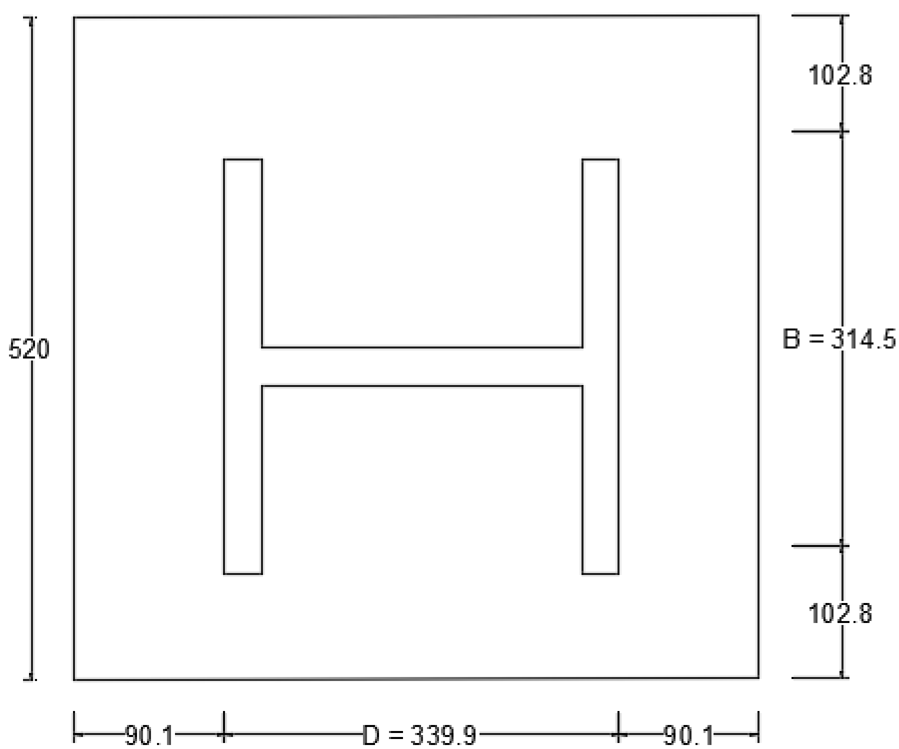
$l = 520m$

$b = 520m$



From table, $B = 314.5\ and\ D = 339.9$





$a = 102.8$

$b = 90.1$

$$t = \left(\frac{2.5}{p_{yp}}.w.(a^2 - 0.3b^2)\right)^{0.5}$$

Pressure at base, $w = \frac{load}{area} = \frac{2640*10^3}{520*520} = 9.76\,N/mm^2$

$$t = \left(\frac{2.5}{265} * 9.76 * (102.8^2 - 0.3 * 90.1^2)\right)^{0.5} = 27.4\,mm$$

Assume $t = 40\,mm$

$$Vertical\ load\ of\ bolt = \frac{load}{No.\ of\ bolts} = \frac{2640}{4} = 660\,kN$$

Bearing capacity, $P_{bb} = d.t.p_{bb}$

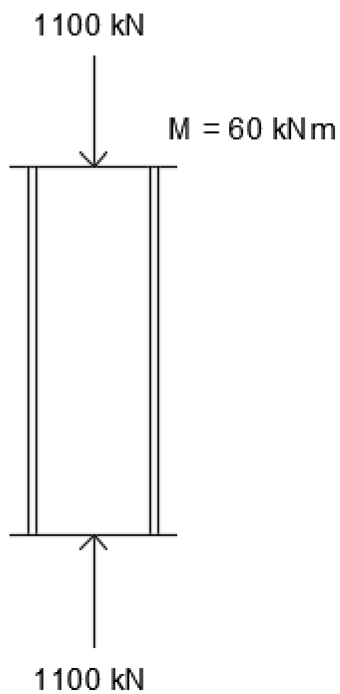
$$d = \frac{P_{bb}}{t.p_{bb}} = \frac{660 * 10^3}{40 * 970} = 17\,mm$$

Assume diameter of bolt, $d = 20\,mm$

EXAMPLE 2

A column base is subjected to a factor moment of $60\,kNm$ and a factored axial load of $1100\,kN$. Assume a column section of $356 * 406 * 235\ kg/m$ UC. Assume a cube strength, f_{cu} of $20\,N/mm^2$. Design the slab base and diameter of bolt to be used using grade 50 steel

Solution



$$e = \frac{M}{N} = \frac{60 * 10^3}{1100} = 54.55\ mm$$

$$L = 6e = 6 * 54.55 = 327.3\ mm$$

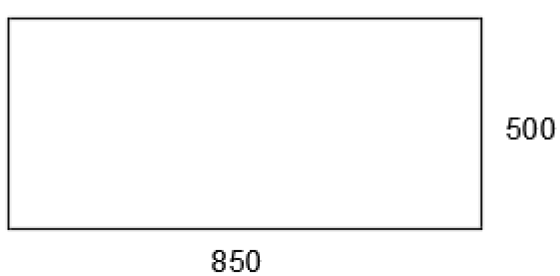
Bearing strength, $b_s = 0.4 f_{cu} = 0.4 * 20 = 8\ N/mm^2$

Area of base, $A_{base} = \frac{2N}{bearing\ strength} = \frac{2*1100*10^3}{8} = 275000\ mm^2$

$$A = Lb \Longrightarrow b = \frac{A}{L} = \frac{275000}{327.3} = 840.2\ mm$$

From table, $B = 394.8\ and\ D = 381.0$

$Try\ 500 * 850\ mm$



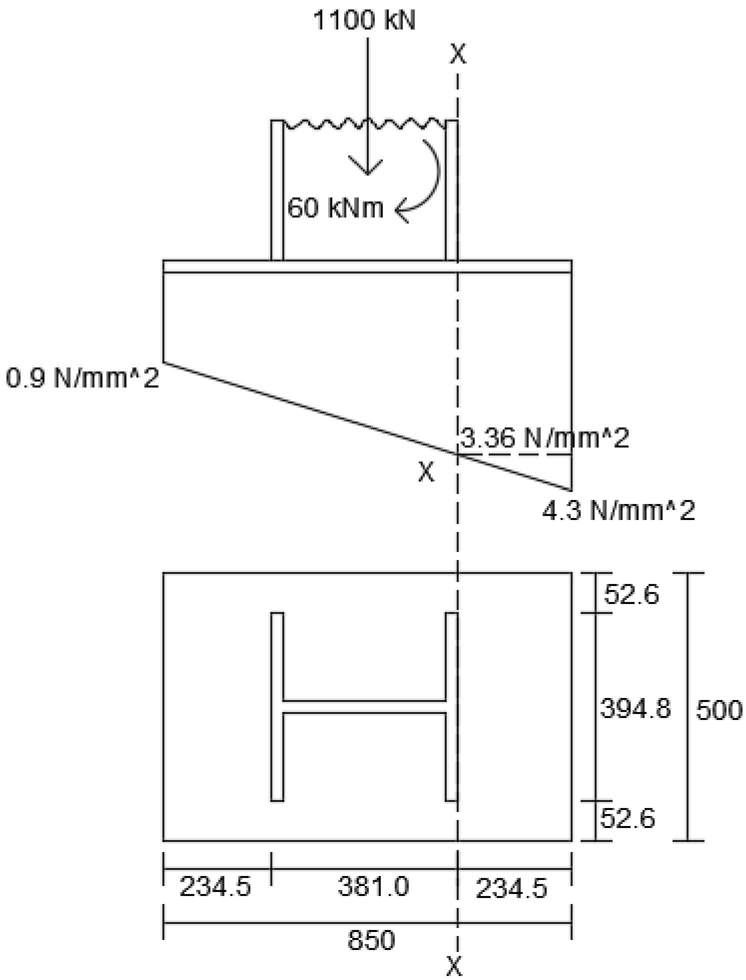
$A = 500 * 850 = 425000\ mm^2$

Section modulus, $Z = \frac{bh^2}{6} = \frac{850*500^2}{6} = 35.4 * 10^6\ mm^3$

Pressure at base, $P_{max} = \frac{N}{A} \pm \frac{M}{Z} = \frac{1100*10^3}{425000} \pm \frac{60*10^6}{35.4*10^6}$

$= 2.6 \pm 1.7$

$\therefore\ P_{max} = 4.3\ N/mm^2\ \ and\ P_{min} = 0.9\ N/mm^2$



Pressure at x-x

$$P_{x-x} = 0.9 + \frac{615.5}{850}.(4.3 - 0.9) = 3.36\ \ N/mm^2$$

Moment at x-x

$$M_{x-x} = \left(3.36 * \frac{234.5^2}{2}\right) + \left(0.94 * \frac{234.5}{2} * \frac{2}{3} * 234.5\right) = 109613.9\ Nmm$$

$$Z = \frac{t^2}{6}$$

$$t \ngtr 40\ mm$$

$$p_y = 355\ N/mm^2$$

$$M = 1.2p_y.z$$

$$M = \frac{1.2p_y.t^2}{6}$$

$$t = \sqrt{\frac{6M}{1.2p_y}}$$

$$\therefore t = \sqrt{\frac{6 * 109613.9}{1.2 * 355}} = 39.3\ mm$$

Assume thickness of plate to be 40 mm

$$F = load\ per\ bolt = \frac{N}{4} + \frac{M}{r} = \frac{1100}{4} + \frac{60}{0.6155} = 372.5\ kN$$

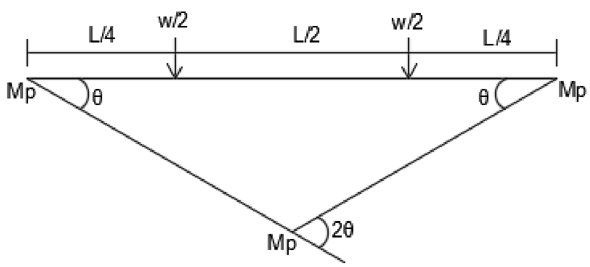
Bearing capacity of bolt, $P_{bb} = d.t.p_{bb}$

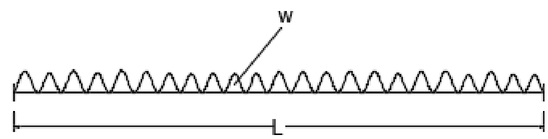
$$d = \frac{P_{bb}}{t.p_{bb}} = \frac{372.5 * 10^3}{40 * 970} = 9.6\ mm$$

Use 12 mm bolt

6.0 PLASTIC DESIGN

When a structure is loaded, the stress increases with increase in moment caused by the load until the structure collapses. Plastic design is based on collapse load of a structure to obtain plastic moment. The collapse load is obtained by multiplying the working load with safety factor.

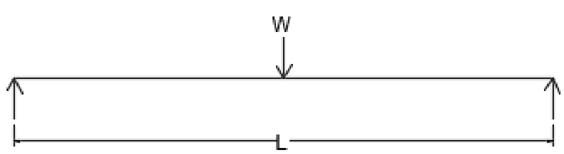


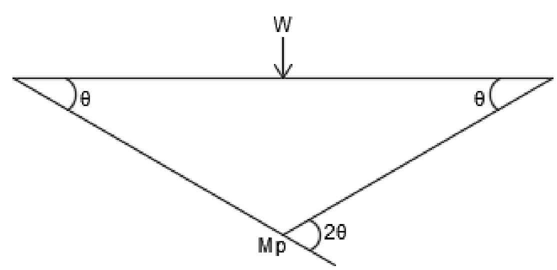


$$IW = M_p\theta + M_p\theta + M_p.2\theta = 4M_p\theta$$

$$EW = \frac{w}{2} * \frac{l}{4} * \theta + \frac{w}{2} * \frac{l}{4} * \theta = \frac{wl\theta}{4}$$

$$4M_p\theta = \frac{wl\theta}{4} \Longrightarrow M_p = \frac{wl}{16}\ (kNm)$$

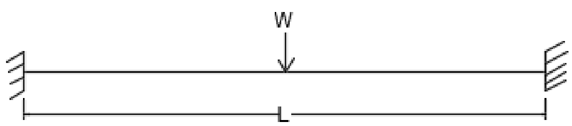


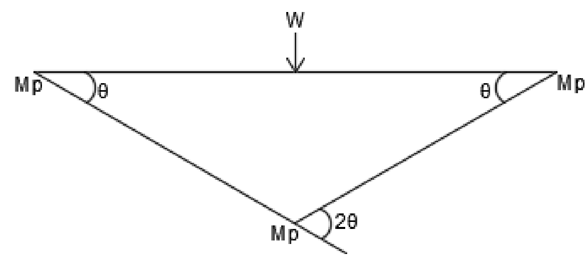


$$IW = 2M_p\theta$$

$$EW = \frac{wl\theta}{2}$$

$$M_p = \frac{wl}{4}$$





$$IW = M_p\theta + M_p\theta + M_p.2\theta = 4M_p\theta$$

$$EW = \frac{wl\theta}{2}$$

$$4M_p\theta = \frac{wl\theta}{2} \Longrightarrow M_p = \frac{wl}{8}\ (kNm)$$

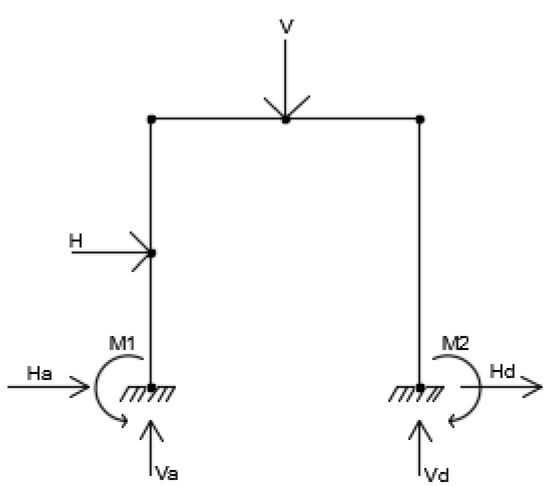
<u>Collapse mechanism of frame</u>

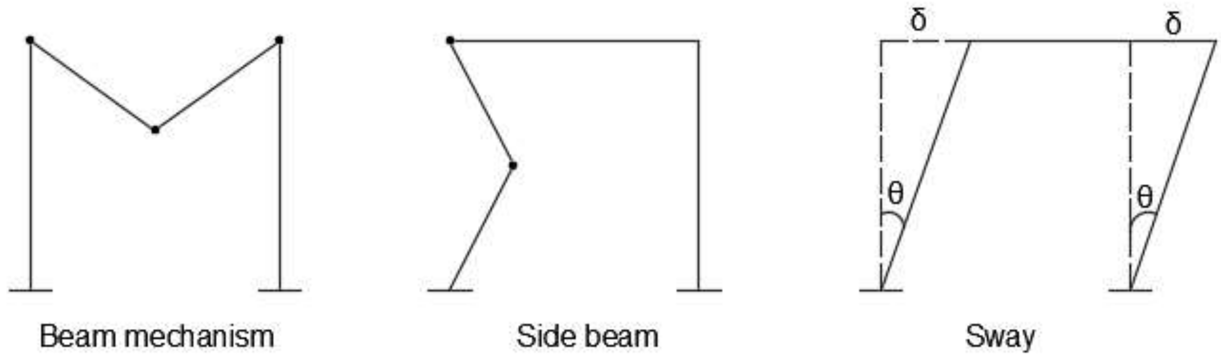
Determine:

i. Possible hinge positions

ii. Degree of redundancy

iii. Nos. of independent mechanism of the frame below





$(H) \Rightarrow Nos.of\ possible\ hinges = 6$

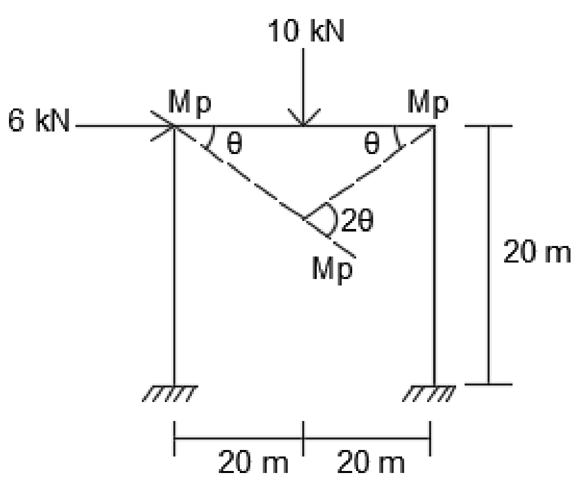
$(T) \Rightarrow Total\ unknown = 6$

$(E) \Rightarrow Equilibrium\ equation = 3$

$Degree\ of\ redundancy\ (DOR) = T - E = 6 - 3 = 3$

$Nos.of\ independent\ mechanism\ (NIM) = H - DOR = 6 - 3 = 3$

EXAMPLE 1



$H = 5$

$T = 6$

$E = 3$

$DOR = T - E = 6 - 3 = 3$

$NIM = H - DOR = 5 - 3 = 2$

Beam mechanism

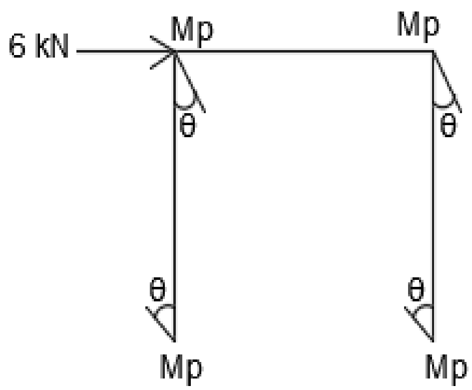
$IWD = 4M_p\theta$

$EWD = 10 * 20 * \theta = 200\theta$

$4M_p\theta = 200\theta$

$M_p = 50\ kNm$

Frame mechanism



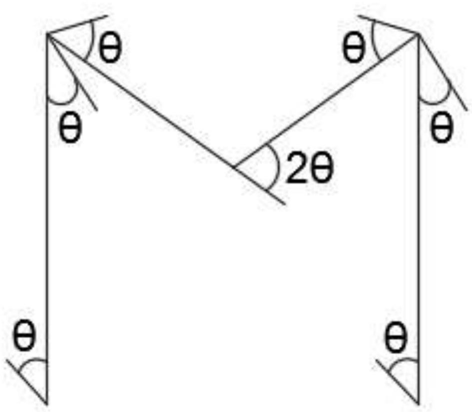
$IWD = 4M_p\theta$

$EWD = 6 * 20 * \theta = 120\theta$

$4M_p\theta = 120\theta$

$M_p = 30\ kNm$

Combined mechanism



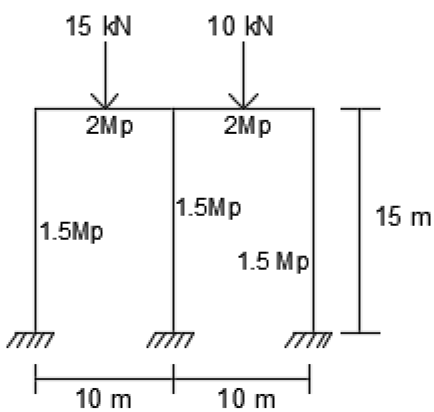
$IWD = 8M_p\theta$

$EWD = 320\theta$

$4M_p\theta = 320\theta$

$M_p = 40\ kNm$

EXAMPLE 2



$H = 10$

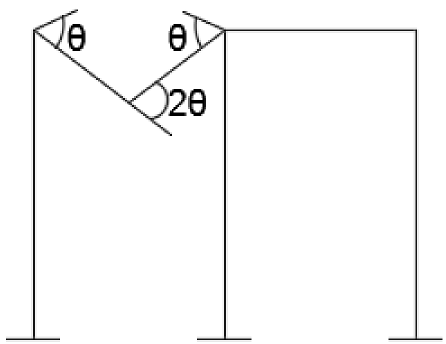
$T = 9$

$E = 3$

$DOR = T - E = 9 - 3 = 6$

$NIM = H - DOR = 10 - 6 = 4$

<u>LHB</u>



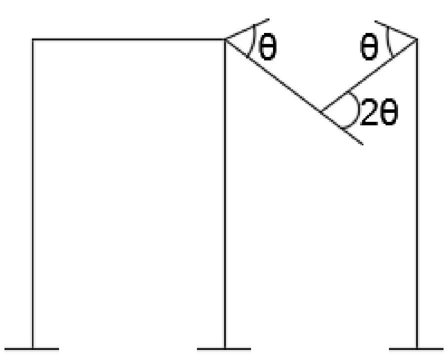
$IWD = 2M_p\theta + 2M_p\theta + 2M_p.2\theta = 8M_p\theta$

$EWD = 15 * 10\theta = 150\theta$

$8M_p\theta = 150\theta$

$M_p = 18.75\ kNm$

<u>RHB</u>



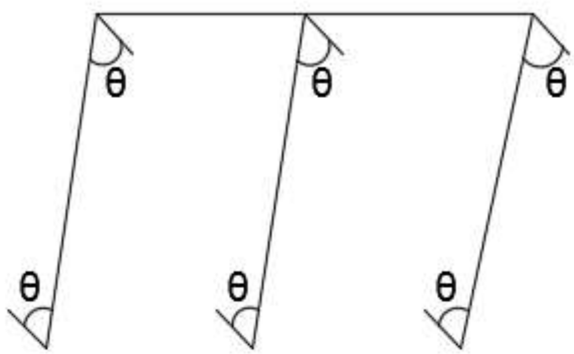
$IWD = 8M_p\theta$

$EWD = 10 * 10\theta = 100\theta$

$8M_p\theta = 100\theta$

$M_p = 12.5\ kNm$

Sway



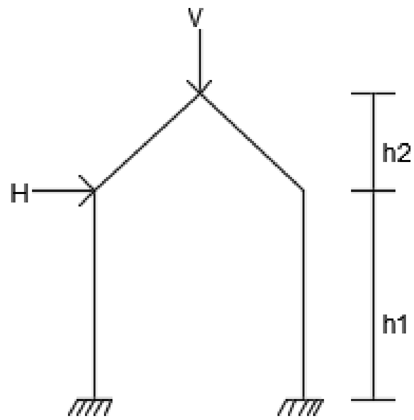
$IWD = 1.5M_p\theta * 6 = 9M_p\theta$

$EWD = 10 * 15\theta = 150\theta$

$9M_p\theta = 150\theta$

$M_p = 16.67\ kNm$

Pitched roof portal frame



$H = 5$

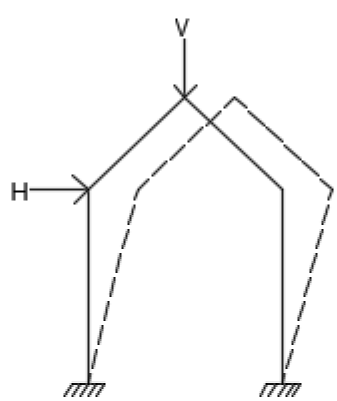
$T = 6$

$E = 3$

$DOR = T - E = 6 - 3 = 3$

$NIM = H - DOR = 5 - 3 = 2$

Sway

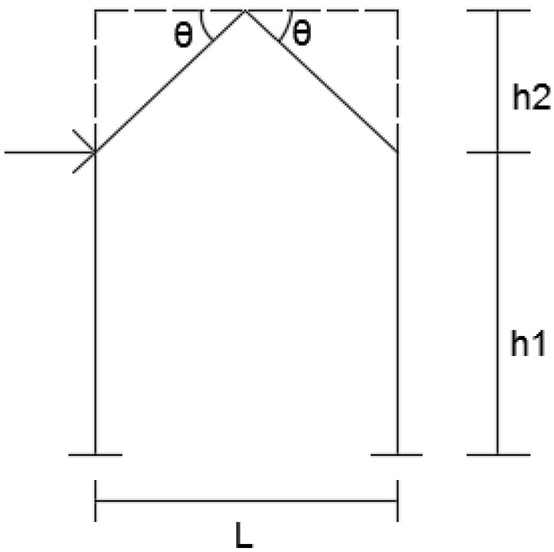


$IWD = 4M_p\theta$

$EWD = Hh_1\theta$

$4M_p\theta = Hh_1\theta$

$M_p = \frac{Hh_1}{4}$



$IWD = M_p\theta + M_p.2\theta + M_p(\theta + \alpha) = 4M_p\theta + M_p.\alpha$

$EWD = Hh_2\theta + \frac{vl\theta}{2}$

$IWD = 4M_p\theta + M_p.\alpha$

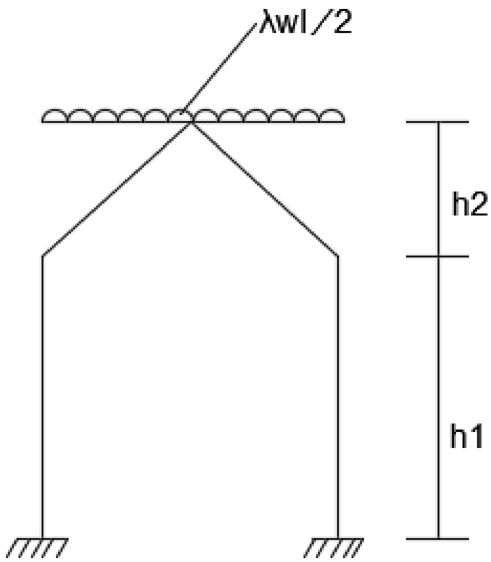
$IWD = 4M_p\theta + 2M_p.\frac{h_2}{h_1}\theta$

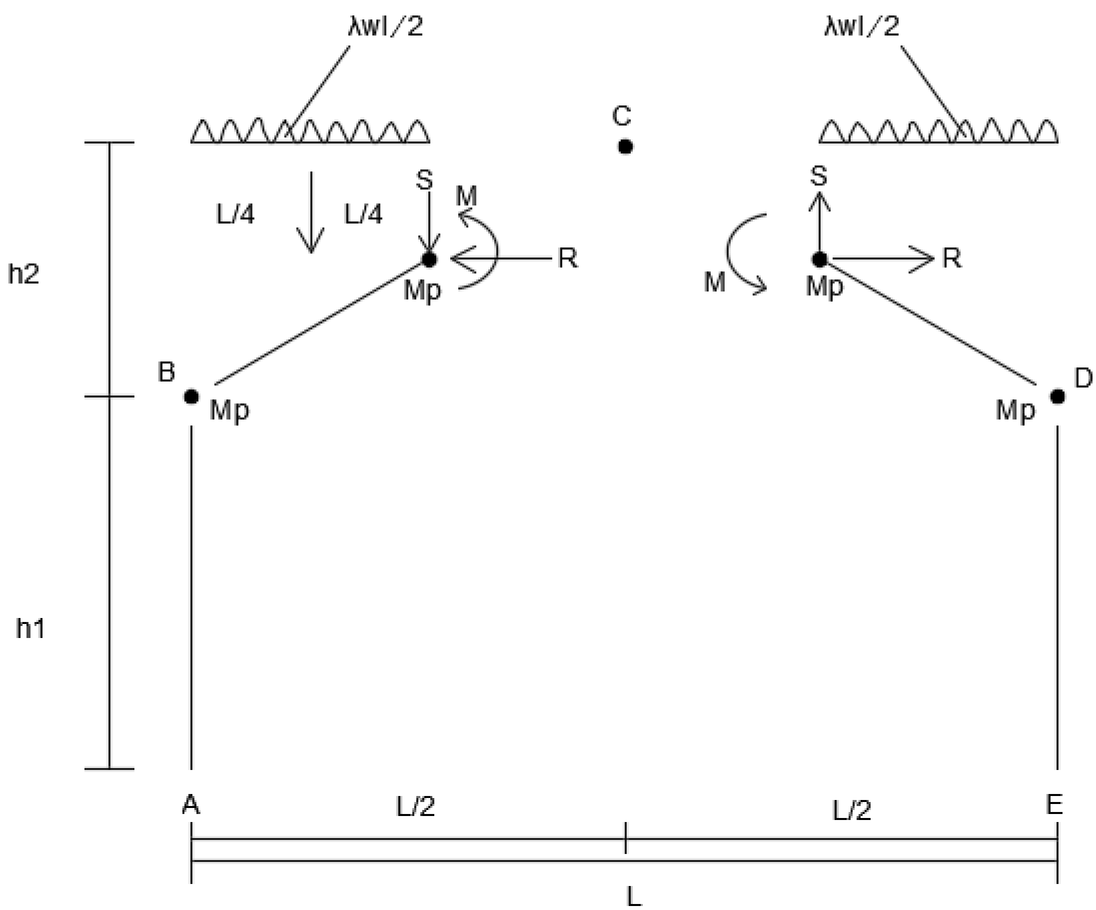
$IWD = M_p\theta.\left(4 + \frac{2h_2}{h_1}\right)$

$\therefore M_p = \frac{\left(2Hh_2 + \frac{vl}{2}\right)}{\left(4 + \frac{2h_2}{h_1}\right)}$

<u>For combine mode</u>

$$M_p = \frac{H(h_1 + 2h_2) + {}^{vl}/_{2}}{\left(6 + {}^{2h_2}/_{h_1}\right)}$$


λwl ⁄ 2
h2
h1


λwl ⁄ 2
λwl ⁄ 2
C
S
M
L/4
L/4
R
Mp
S
M
R
Mp
h2
B
Mp
D
Mp
h1
A
L/2
L/2
E
L

At point C

$$M_p = M \text{ - } (1)$$

At point B

$$M_p = \frac{\lambda wl^2}{8} - M - Rh_2 + \frac{Sl}{2} \text{ - - - - - - - - - - - - - - - } (2)$$

At point D

$$M_p = \frac{\lambda wl^2}{8} - M - \frac{Sl}{2} - Rh_2 \text{ - - - - - - - - - - - - - - - } (3)$$

At point E

$$M_p = -\frac{\lambda wl^2}{8} + M + \frac{Sl}{2} + R(h_2 + h_1) \text{ - - - - - - - - - - - } (4)$$

$$M_p = M$$

$$-2Sl = 0 \Longrightarrow S = 0$$

$$from\ (2), \quad M_p = \frac{\lambda wl^2}{8} - M - Rh_2$$

$$\frac{\lambda wl^2}{8} = M_p + M + Rh_2 \text{ - - - - - - - - - - - - - - - - - } (5)$$

$$substituting\ (5)\ into\ (4)$$

$$M_p = -M_p - M - Rh_2 + M + R(h_2 + h_1)$$

$$2M_p = -Rh_2 + Rh_2 + Rh_1$$

$$M_p = \frac{Rh_1}{2} \Longrightarrow R = \frac{2M_p}{h_1} \text{ - - - - - - - - - - - - - - - - - } (6)$$

$$substituting\ (6)\ into\ (2)$$

$$M_p = \frac{\lambda wl^2}{8} - M - \frac{2M_p.h_2}{h_1} \qquad (M_p = M)$$

$$2M_p = \frac{\lambda w l^2}{8} - \frac{2M_p . h_2}{h_1}$$

$$\frac{\lambda w l^2}{8} = 2M_p + \frac{2M_p . h_2}{h_1}$$

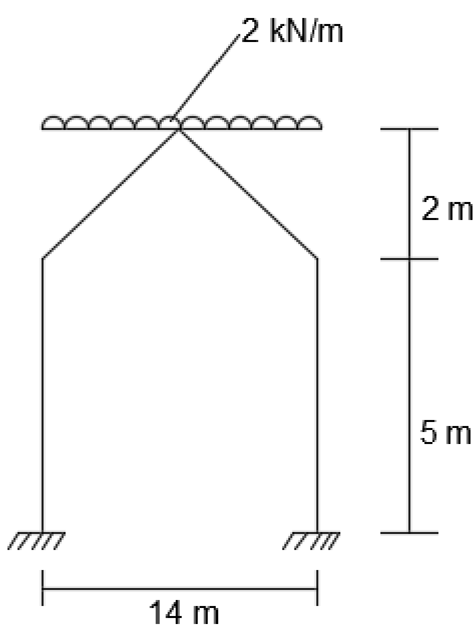
$$\frac{\lambda w l^2}{8} = 2M_p \left(1 + {}^{h_2}/_{h_1}\right)$$

$$2M_p = \frac{\lambda w l^2}{8\left(1 + {}^{h_2}/_{h_1}\right)}$$

$$\therefore M_p = \frac{\lambda w l^2}{16\left(1 + {}^{h_2}/_{h_1}\right)}$$

EXAMPLE 1

Evaluate the plastic moment of the portal frame shown below



$$v = wl = 2 * 14 = 28\ kN$$

$$M_p = \frac{\left(2Hh_2 + {}^{vl}/_{2}\right)}{4\left(1 + {}^{h_2}/_{h_1}\right)}$$

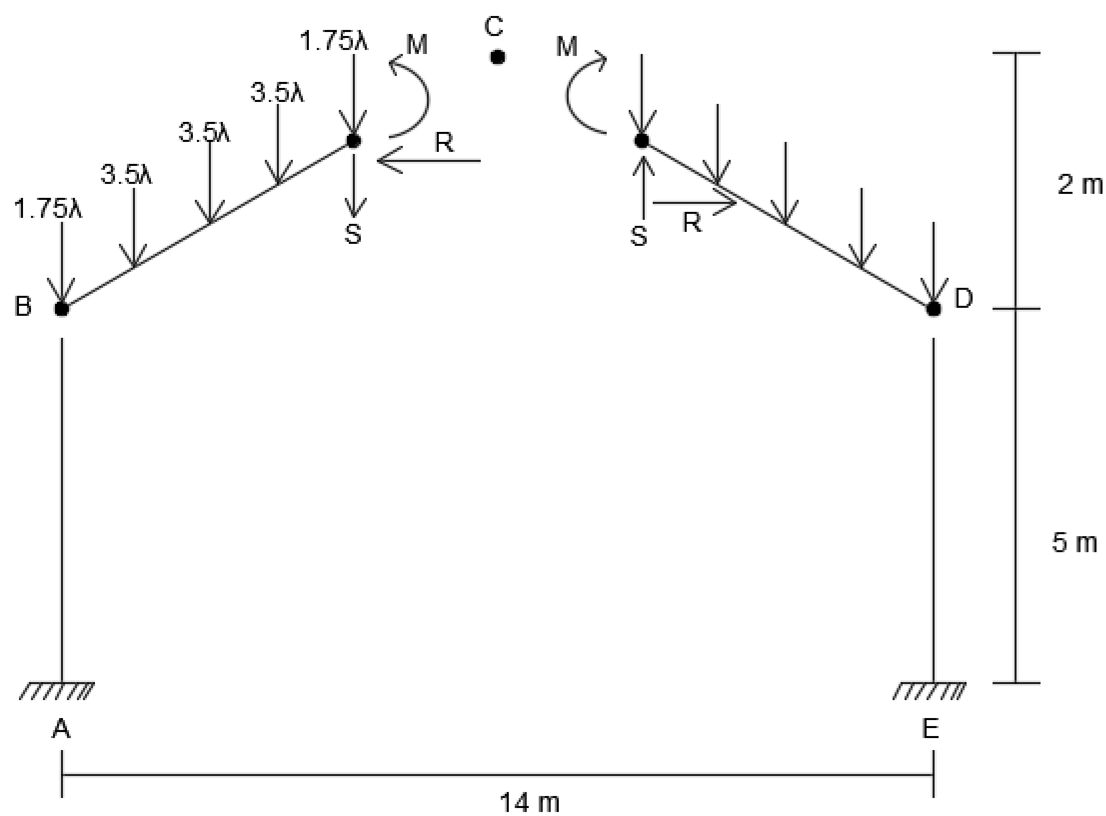
$$M_p = \frac{28 * {}^{14}/_{2}}{4\left(1 + {}^{2}/_{5}\right)}$$

$$M_p = 35\ kNm$$

Assume purlin spaced at 1.75m

$\Longrightarrow 35\lambda\ kNm$

$$Each\ purlin = \frac{35\lambda}{10} = 3.5\lambda$$



$M_p = M$

At point B

$$M_p = 1.75\lambda * 7 + 3.5\lambda(5.25 + 3.5 + 1.75) - M + 7S - 2R$$

$$M_p = 12.25\lambda + 36.75\lambda - M + 7S - 2R$$

$$M_p = 49\lambda - M + 7S - 2R ----------------(I)$$

At point D

$$M_p = 49\lambda - M - 7S - 2R ----------------(II)$$

At point E

$$M_p = -49\lambda + M + 7S + 7R ----------------(III)$$

$$(I) - (II)$$

$$0 = 14S \Longrightarrow S = 0$$

$$substituting\ S = 0\ into\ (I)$$

$$M_p = 49\lambda - M - 2R = 49\lambda - M_p = 2R$$

$$2M_p = 49\lambda - 2R$$

$$49\lambda = 2M_p + 2R ----------------------\ (IV)$$

$$substituting\ (IV)\ into\ (III)$$

$$M_p = -2M_p - 2R + M_p + 7R$$

$$2M_p = 5R$$

$$M_p = \frac{5R}{2} \Longrightarrow R = \frac{2M_p}{5}$$

$$substituting\ R = \frac{2M_p}{5}\ into\ (II)$$

$$M_p = 49\lambda - M_p - 2\left(\frac{2M_p}{5}\right)$$

$$M_p = 49\lambda - M_p - \frac{4M_p}{5}$$

$$2M_p = 49\lambda - \frac{4M_p}{5}$$

$$\frac{2M_p}{1} + \frac{4M_p}{5} = 49\lambda$$

$$\frac{10M_p + 4M_p}{5} = 49\lambda$$

$$\frac{14M_p}{5} = 49\lambda$$

$$M_p = \frac{49\lambda * 5}{14} = 17.5\lambda$$

$$\lambda = 1.75m$$

$$M_p = 17.5 * 1.75 = 30.625\ kNm$$