UNIT I INTRODUCTION AND ALLOWABLE STRESS DESIGN

Design of Laterally supported beam Procedure

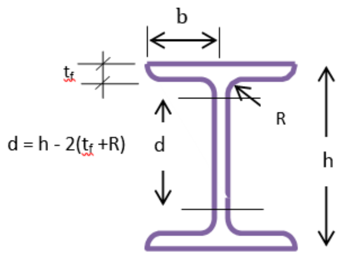
PROCEDURE:

- Calculate the B.M
  Calculate the section modulus requried $(Z_{PZ,req})$
- $$Z_{PZ,req} = \frac{M}{f_y / \gamma_{mo}} = \frac{M}{f_y} \times \gamma_{mo}$$
- Select a suitable section from IS 808 code for the $Z_{PZ,\ req}$

Check for design shear strength (P- 59, Cl 8.4)

$$V \le V_d$$

$$V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v f_{yw}}{\sqrt{3} \times \gamma_{mo}} = \frac{h \times t_w \times f_{yw}}{\sqrt{3} \times \gamma_{mo}}$$



Section classification: P – 18, Table 2.

$$\text{Yield stress ratio } \varepsilon = \sqrt{\frac{250}{f_y}}$$

If $\frac{b}{t_f} \le 9.4\varepsilon$ and $\frac{d}{t_w} \le 84\varepsilon$	the section is classified as " Plastic class of section"
If $\frac{b}{t_f} > 9.4\varepsilon$ and $\le 10.5\varepsilon$, and $\frac{d}{t_w} \le 105\varepsilon$.	the section is classified as " Compact class of section"
If $\frac{b}{t_f} > 10.5\varepsilon$ and $\le 15.7\varepsilon$, and $\frac{}{t_w} \le 126\varepsilon$.	the section is classified as " Semi - Compact class of section"
If $\frac{b}{t_f} > 15.7\varepsilon$	the section is a slender section. It has to be Redesigned

Check for design shear strength (P- 59, Cl 8.4)

$$V \le V_d$$

$$V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v f_{yw}}{\sqrt{3} \times \gamma_{mo}} = \frac{h \times t_w \times f_{yw}}{\sqrt{3} \times \gamma_{mo}}$$

Check for low / High shear (Cl 8.2.1.3)

if) $V < 0.6\, V_d$, This is a case is of low shear

Design Moment capacity (P-53, Cl 8.2.1.1)

$M \leq M_d$

$if\ \frac{d}{t_w} \leq 67\varepsilon$ and $V < 0.6\, V_d$

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$M_d = \frac{\beta_b Z_P fy}{\gamma_{mo}} \leq$ 1) $\frac{1.2 Z_e fy}{\gamma_{mo}}$ for simply supported beam

2) $\frac{1.5 Z_e fy}{\gamma_{mo}}$ for cantilever beams

$\beta_b = 1$, for Plastic and compact section

$\beta_b = \frac{Z_e}{Z_P}$ for semi compact section

if) $V < 0.6\, V_d$, This is a case is of low shear

If $\frac{d}{t_w} \leq 67\varepsilon$ and $V > 0.6\, V_d$, This is a case of "High shear"

$M_d = M_{dv}$

Where,

M_{dv} = Design bending strength under high shear as defined in 9.2.2 , P - 70

a) Plastic or compact section

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \le \frac{1.2 Z_e f_y}{\gamma_{mo}}$$

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2$$

$$M_d = \beta_b \frac{Z_p f_y}{\gamma_{mo}}, \qquad \text{here } \beta_b = 1$$

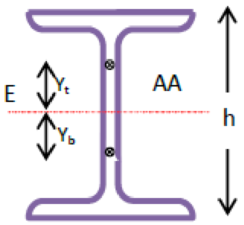
M_{fd} = Plastic design strength of the area of the cross - section excluding the shear area, considering partial safety factor γ_{mo} and

Z_e = Elastic section modulus of the whole section.

Plastic design strength of the area of the cross section excluding shear area.

$$M_{fd} = \frac{Z_{fd} f_y}{\gamma_{mo}}$$

$$Z_{fd} = Z_{pz} - \frac{A}{2} \times (y_t + y_b) = Z_{pz} - \frac{h \times t_w}{2} \times \left(\frac{h}{4} + \frac{h}{4}\right)$$



b) Semi - Compact section

$$M_{dv} = \frac{Z_e f_y}{\gamma_{mo}}$$

Check for Deflection

$$\delta = \frac{5wl^4}{384EI}$$

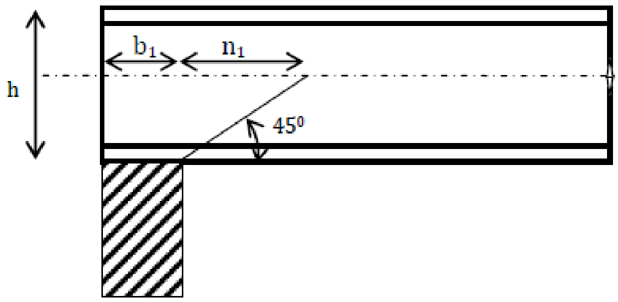
Check for web buckling at support: P-67, Cl 8.7.3.1

$$F_{cdw} = (b_1 + n_1) \times t_w \times f_c \nless V$$

$n_1 = h/2 \quad b_1 =$ Stiffner Bearing

f_c is obtained from $S.R(\lambda) = \frac{KL}{r_y}$

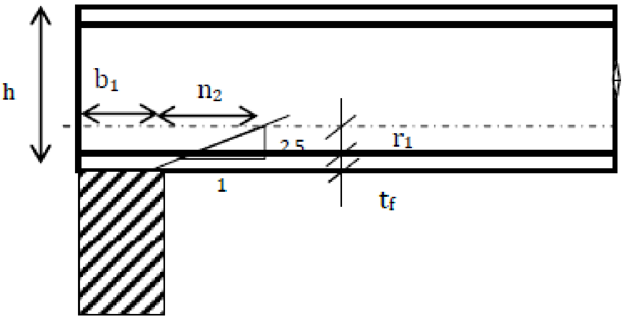
$$\lambda = \frac{0.7d\sqrt{12}}{t_w} \quad \text{P-42, table 9(c)}$$



Check for web crippling at support: P-67, Cl 8.7.4

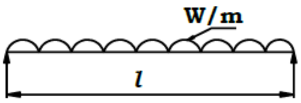
$$F_w = \frac{(b_1 + n_2)t_w f_{yw}}{\gamma_{mo}} \not< V$$

$$n_2 = 2.5\,(t_f + R)$$



Problem1: Design a simply supported beam of span 5m carrying a reinforced concrete floor capable of providing lateral restraint to the top compression flange. The uniformly distributed load is made up of 20 KN/m imposed load and 20 KN/m dead load (section is stiff against bearing). Assume Fe 410 grade steel.

Solution:



Calculation of factored loads:

Dead load = 1.5 x 20 = 30 KN/m
Live load = 1.5 x 20 = 30 KN/m
Total factored load on the beam = 60 KN/m

Effective length of the beam = 5 m

Calculation of Bending moment and Shear force:

$$\text{Design BM} = \frac{w_u l^2}{8} = \frac{60 \times 5^2}{8} = 187.5 \text{ KN - m.}$$

$$\text{Design shear force } (V) = \frac{W_u l}{2} = \frac{60 \times 5}{2} = 150 \text{ KN}$$

$$\text{Section modulus } Z_{pz} = \frac{M}{f_y / \gamma_{mo}} = \frac{M}{f_y} \times \gamma_{mo} = \frac{187 \times 10^6}{250} \times 1.1 = 825 \times 10^3 \text{ mm}^3 = 825\text{cm}^3.$$

Try ISLB 350 @ 486 N / m (0.486 KN / m)

$h = 350$ mm, $b_f = 82.5$mm, $t_w = 7.4$ mm. $t_f = 11.4$mm, $R = r_1 = 16$ mm

$I_{zz} = 13200 \text{ cm}^4 = 13200 \times 10^4 \text{ mm}^4. Z_e = Z_z = 751.9 \text{ cm}^3, Z_{pz} = 851.11 \text{ cm}^3,$

Section classification: P – 18, Table 2.

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\frac{b}{t_f} = \frac{41.25}{11.4} = 3.62 < 9.4\varepsilon$$

$$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.9 < 84\varepsilon$$

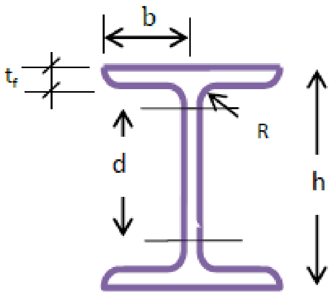
$b = b_f / 2 = 82.5 / 2$

$= 41.25$ mm

$d = h - 2(t_f + R)$

$d = 350 - 2(11.4 + 16)$

$= 295.2$ mm



Hence the section is classified as "**Plastic Section**".

Check for Shear Strength : P 59, Cl8.4

$V = 150$ KN

$V \le V_d$

$$V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v f_{yw}}{\sqrt{3} \times \gamma_{mo}} = \frac{h \times t_w \times f_{yw}}{\sqrt{3} \times \gamma_{mo}}$$

$$V_d = \frac{350 \times 7.4 \times 250}{\sqrt{3} \times 1.1 \times 1000} = 340KN > 150 \ KN$$

Check for low/ high shear:

$0.6 V_d = 0.6 \times 340 = 204KN > V(150KN)$

$\because 0.6V_d > V$, It is a case of low shear

Check for DesignMoment Capacity : P 53, Cl 8.2.1.1

$M \le M_d$

$$\because \frac{d}{t_w} = \frac{295.2}{7.4} = 39.9 \le 67\varepsilon \text{ and } V < 0.6V_d$$

$$M_d = \frac{\beta_b Z_p fy}{\gamma_{mo}} \le \frac{1.2 Z_e f_y}{\gamma_{mo}}$$

$\beta_b = 1$, for Plastic and compact section

$$M_d = \frac{1 \times 851.11 \times 10^3 \times 250}{1.1 \times 10^6} = 193.43KN\text{-}m > M(187.5\ KN\text{-}m)$$

$$\text{and} \quad < \frac{1.2 \times 751.9 \times 10^3 \times 250}{1.1 \times 10^6} = 205.10KN$$

Safe

Check for deflection:

Max deflection is calculated corresponding to Imposed working live load

W = 20 KN/m = 20 N/mm

$$\delta_{cal} = \frac{5wl^4}{384EI_{xx}} \not> \delta_{allowable} = \frac{l}{300} = \frac{5000}{300} = 16.67 \text{ mm}$$

$$\delta_{cal} = \frac{5\times 20\times 5000^4}{384\times 2\times 10^5\times 13200\times 10^4} = 6.165 \text{ mm.} < 16.67 \text{ mm}$$

The deflection is less than the allowable maximum deflection.

Safe,

Check for web Crippling: P 67, *Cl 8.7.4*

$$F_w = \frac{(b_1 + n_2)t_w f_{yw}}{\gamma_{mo}} \not< V$$

$$n_2 = 2.5(t_f + R) = 2.5(11.4 + 16) = 68.5 \text{ mm}$$

$$\frac{(100 + 68.5)\times 7.4\times 250}{1.1\times 1000} = 283.38 \text{ KN} > 150 \text{ KN} \qquad \textbf{Safe}$$

∴ **Provide ISLB 350@486N/m** (0.486 KN/m)

Problem: Design a simply supported beam to carry a uniformly distributed load of 44 kN/m. The effective span of beam is 8 meters. The effective length of compression flange of the beam is also 8 m. The ends of beam are not free to rotate at the bearings.

Step 1: Load supported, bending moment and shear force

Uniformly distributed load = 44 kN/m

Assume self-weight of beam = 1.0 kN/m

Total uniformly distributed load w= 45 kN/m

The maximum bending moment, M occurs at the center

$$M = \frac{Wl^2}{8}$$

$$= \frac{45\times 8^2}{8} = 360 Kn.m$$

The maximum Shear Force V

$$V = \frac{Wl}{2}$$

$$= \frac{45\times 8}{2} = 180\ KN$$

Step 2: Permissible bending stress

It is assumed that the value of yield stress, f_y for the structural steel is 250 N/mm^2 (MPa).

The ratios (T/t_w) and (d_1/t_w) are less than 2.0 and 85 respectively.

The maximum permissible stress in compression or tension may be assumed as

$\sigma_{bc} = \sigma_{bt}$ = (0.66 x 250) = 165 N/mm^2

$$Section Modulus required \quad Z = \frac{M}{\sigma_{bc}}$$

$$Z = \frac{360 \times 10^6}{165}$$

$$= 2181.82 \times 10^3 \ mm^3$$

The steel beam section shall have (D/T) and (l/r_y) ratios more than 8 and 40 respectively. The trial section of beam selected may have modulus of section, Z more than that needed (about 25 to 50 per cent more).

Step 3: Trial section modulus

1.50 x 2181.82 x 10^3 mm^3=3272.73 x 10^3 mm^3

From steel section tables, try WB 600, @1.337 kN/m

Section modulus, Z_{xx}=3540.0 x 10^3 mm^3

Moment of inertia, I_{xx}=106198.5 x 10^4 mm^4

Thickness of web, t_w=11.2 mm

Thickness of flange, T=t_f=21.3 mm

Depth of section, h=600 mm

Step 4: Check for section modulus

$$\frac{D}{T} = \frac{600}{21.3} = 28.169$$

$$\frac{T}{t_w} = \frac{21.3}{11.2} = 1.901 < 2, also \left(\frac{d_1}{t_w} < 85\right)$$

The effective length of compression flange of beam is 8 m.

$$\frac{l}{r_y} = \left(\frac{0.7 \times 8 \times 10}{52.5}\right) = 106.66$$

From IS: 800-1984, the maximum permissible bending stress σ_{bc}=118.68 N/mm^2(MPa)
Section modulus required

$$\left(\frac{360 \times 1000 \times 1000}{118.68}\right) = 3033.34 \times 10^3 mm^3 < 3540 \times 10^3 mm^3 provided$$

Further trial may give more economical section.

Step 5: Check for shear force

Average shear stress,

$$\tau_{v,cal} = \left(\frac{F}{h \times t_w}\right) = \left(\frac{180 \times 1000}{600 \times 11.2}\right) = 26.78\, N/mm^2$$

Permissible average shear stress

0.4 x f_y = (0.4 x 250) = 100 N/mm^2 > Actual average shear stress

Step 6: Check for deflection

Maximum deflection of the beam

$$y_{max} = \left(\frac{5}{384} \times \frac{wl^4}{EI}\right)$$

$$= \frac{5}{384} \times \left(\frac{45 \times 8^4 \times 1000^4}{2.047 \times 10^5 \times 106198.5 \times 10^4}\right) = 24.53\ mm$$

Allowable deflection

$$\delta = \frac{Span}{325}$$

$$= \frac{8000}{325} = 24.60\, mm$$

The maximum deflection is less than allowable deflection, hence the beam is safe. Provide WB 600, @1.337 kN/m