Problem 1

Design a hand operated overhead crane, which is provided in a shed, whose details are:

- Capacity of crane = 50 kN
- Longitudinal spacing of column = 6m
- Center to center distance of gantry girder = 12m
- Wheel spacing = 3m
- Edge distance = 1m
- Weight of crane girder = 40 kN
- Weight of trolley car = 10 kN

**Design by allowable stress method as per IS: 800 - 1984**

To find wheel load (refer fig.1):
RA = 20 + 60 (11 / 12) = 75 kN
Wheel load = RA / 2 = 37.5 kN

To find maximum BM in gantry girder (refer fig.2):

RA = 46.88 kN
RB = 28.12 kN

Max. BM = 28.12 x 2.25 = 63.27 kN-m
Adding 10% for impact,
M1 = 1.1 x 63.27 = 69.60 kN-m

Max. BM due to self-weight of girder and rail taking total weight as 1.2 kN/m

\[ M_2 = \frac{w l^2}{8} = 5.4 \text{kN-m} \]

Therefore Total BM, M = 75 kN-m

To find maximum shear force (refer fig.3):

SF = RA = 59.85 kN.

To find lateral loads:

This is given by 2.5% of (lateral load / number of wheel = 0.025 x 60 / 2 kN = 0.75 kN

Therefore Max BM due to lateral load by proportion is given by, ML = (63.27 / 37.5) x 0.75 = 1.27 kN-m

Design of section

Approximate section modulus Zc required, (M / σbc) = 75 x 10^6 / 119 = 636 x 10^3 mm^3 [for λ = 120, D / T = 25].

Since, the beam is subjected to lateral loads also, higher section is selected.

For, ISMB 450 @ 710.2 N/m,

 ZX = 1350.7 cm^3, T = 17.3 mm,
\[ t = 9.4 \text{mm}, \quad I_{yy} = 111.2 \text{cm}^3 \]

\[ r_y = 30.1 \text{mm}, \]

\[ b_f = 150 \text{mm} \]

To find allowable stresses,

\[ \frac{T}{t} = \frac{17.4}{9.4} = 1.85 < 2 \]

\[ \frac{D}{T} = \frac{450}{17.4} = 25.86 \sim 26 \]

\[ \frac{L}{r_y} = \frac{6000}{30.1} = 199.3 \sim 200 \]

Therefore allowable bending compression about major axis is, \( s_{bc'}x = 77.6 \) N/mm\(^2\)

Actual stress in compression side, \( s_{b'}x = \frac{M}{Z} = 55.5 \) N/mm\(^2\).

The bending moment about Y-axis is transmitted only to the top of flange and the flange is treated as rectangular section. The allowable stress is \( s_{bc.y} = 165 \) MPa (i.e. 0.66 \( f_y \)).

\[ Z_y \text{ of the flange} = \frac{111.2}{2} = 55.6 \text{cm}^3 \]

Therefore \( s_{by} = \frac{M_y}{Z_y} = \frac{1.27 \times 10^6}{55.6 \times 10^3} \]

\[ = 22.84 \text{ MPa} \]

The admissible design criteria is \( \frac{s_{bx}}{s_{bcx}} + \frac{s_{by}}{s_{bcy}} = \frac{55.5}{77.6} + \frac{22.84}{165} = 0.715 + 0.138 = 0.854 < 1 \). Hence, the design is safe. So, ISMB 450 is suitable.

Check for shear:

Design shear stress, \( \tau_x V / (Dt) = \frac{59.85 \times 10^3}{450 \times 9.4} = 14.15 \) MPa. This is less than \( \tau_a (0.4f_y) \). Hence, the design is o.k.

Check for deflection and longitudinal bending can be done as usual.
Design by limit state method as per IS: 800 draft code

For ISMB 450, properties are given below:

\[ T = 17.4\text{mm}, \ t = 9.4\text{mm}, \ b = 150\text{mm}, \ r_y = 30.1\text{mm}, \ Z_p = 1533.33 \text{ cm}^3, \ Z_c = 1350.7 \text{ cm}^3, \ \text{Shape factor} = 1.15, \ I_{zz} = 30390.8 \text{ cm}^4, \ H^1 = d = 379.2\text{mm} \]

Section classification:

Flange criteria: \[ \frac{b}{T} = \frac{75}{17.4} = 4.31 < 9.4 \]

No local buckling. Therefore OK

Web criteria: \[ \frac{d}{t_w} = \frac{379.2}{9.4} = 40.34 < 83.9 \]

No local buckling. Therefore OK

Section plastic.

Shear capacity:

\[
F_{vd} = (f_{yw} A_y) / (\sqrt{3} \gamma_{m0}) \\
= \left( \frac{250 \times 450 \times 9.4}{\sqrt{3} \times 1.1} \right) \\
= 555043 \text{ N} \approx 555 \text{ kN}
\]

\[ F_v / F_{vd} = (59.85 \times 1.5) / 555 = 0.1634 < 0.6 \]

Check for torsional buckling (Cl.8.2.2):

\[ t_t / t_w <= 17.4 / 9.4 = 1.85 < 2. \]

Therefore \( \beta_{LT} = 1.2 \) for plastic section

\( M_{cr} = \text{Elastic critical moment} \)
\[ M_{\sigma} = \frac{\beta_{LT} \pi^2 h EI_z}{2(KL)^2} \left[ 1 + \frac{1}{20} \left( \frac{KL}{r_p} \right)^2 \right]^{0.5} \]

\[ = \frac{1.2 \pi^2 \times 450 \times 2 \times 10^3 \times 834 \times 10^4}{2 \times 6000^2} \times \left[ 1 + \frac{1}{20} \left( \frac{6000}{30.1} \right) \right]^{0.5} \]

\[ = 246 \times 10^6 \text{ N-m} \]

Now, \( \lambda_{LT} = \sqrt{\frac{\beta_{LT} f_y}{M_{\sigma}}} = 1.248 \)

(\( \beta_b = 1 \) for plastic section)

Therefore \( \phi_{LT} = 0.5 \left[ 1 + \alpha_{LT} (\lambda_{LT} - 0.2) + \lambda_{LT}^2 \right] \)

\[ = 0.5 \left[ 1 + 0.21 (1.248 - 0.2) + 1.248^2 \right] \]

\[ = 1.389 \]

\[ X_{IF} = \frac{1}{\phi_{IF} + (\phi_{IF}^2 - \lambda_{IF}^2)^{0.5}} \]

\[ = 1/1.999 = 0.5 \]

\[ f_m = \frac{X_{IF} f_y}{\gamma_{mf}} = (0.5 \times 250) / 1/1 \]

\[ = 113.7 \text{ MPa} \]

Therefore \( M_d = \beta_b \cdot z_p \cdot f_{bd} \)

\[ = 1 \times 1533.3 \times 10^3 \times 113.7 \]

\[ = 1.7434 \times 10^8 = 174.34 \text{ kN-m} \]

Factored longitudinal moment, \( M_l = 75 \times 1.5 = 112.5 \text{ kN-m} \)

Factored lateral moment, \( M_{il} = 1.27 \times 1.5 = 1.91 \text{ kN-m} \)
Lateral BM capacity = \( M_{dL} \)

\[
\frac{Z_{pL} f_y}{1.10} = \frac{Z_{pL} \text{(Shape Factor)} f_y}{1.10} \\
\left[ \frac{111.2 \times 10^3}{2} \right] 1.15 \times 250 \\
= \frac{14.53 \times 10^6 \text{ N or } 14.53 \text{ kN} \cdot \text{m}}{
}

For Safety,

\[
\frac{M_{f}}{M_{d}} \text{long} + \frac{M_{f}}{M_{d}} \text{brum} \\
= \frac{112.5}{135.9} + \frac{1.91}{14.53} = 0.96 < 1
\]

Hence, it is safe

**Problem 2:**

Design a member (beam - column) of length 5.0\( \text{m} \) subjected to direct load 6.0\( \text{T} \) (DL) and 5.0\( \text{T} \) (LL) and bending moments of \( M_{zz} \{3.6\text{TM (DL)} + 2.5\text{TM (LL)}\} \) and \( M_{yy} \{0.55\text{TM (DL)} + 0.34\text{TM (LL)}\} \) at top and \( M_{zz} \{5.0\text{TM (DL)} + 3.4\text{TM (LL)}\} \) and \( M_{yy} \{0.6\text{TM (DL)} + 0.36\text{TM (LL)}\} \) at bottom.

**LSM {Section 9.0 of draft IS: 800 (LSM version)}**

Factored Load,

\( N = 6.0 \times 1.50 + 5.0 \times 1.5 = 16.5\text{T} \) (Refer Table 5.1 for load factors)

Factored Moments:

At top, \( M_z = 9.15\text{TM}, \ M_y = 1.335\text{TM} \)

At bottom, \( M_z = 12.60\text{TM}, \ M_y = -1.44\text{TM} \)

Section used = MB500.

For MB500:
\[ A = 110.7 \text{ cm}^2; \ r_{yy} = 3.52 \text{ cm}, \]
\[ I_{zz} = 48161.0 \text{ cm}^4; \ Z_{zz} = 1809 \text{ cm}^3 \]
\[ Z_{yy} = 152.2 \text{ cm}^3; \ Z_{pz} = 2074.7 \text{ m}^3; \]
\[ Z_{py} = 302.9 \text{ cm}^3 \]
\[ d = 465.6 \text{ mm}, \ t_w = 10.29 \text{ mm} \]
\[ r_{zz} = 20.21 \text{ cm}, \ b = 180 \text{ mm}; \]
\[ I_{yy} = 1369.8 \text{ cm}^3 \]

(i) Sectional Classifications (Refer Fig: 3.1 and Table - 3.1 of draft code):
\[ \frac{b}{t_f} = \frac{90}{17.2} = 5.23 < 8.92, \text{ therefore the section is plastic.} \]
\[ \frac{d}{t_w} = \frac{465.6}{10.2} = 45.65 < 47, \text{ therefore the section is semi-compact for direct load and plastic for bending moment.} \]

(ii) Check for resistance of the section against material failure due to the combined effects of the loading (Clause-9.3.1):
\[ N_d = \text{Axial strength} = \frac{A \cdot f_y}{\gamma_m} \]
\[ = \frac{110.7 \times 2500}{1.10 \times 10^{-3}} = 251.68^T \]
\[ N = 16.5^T \]
\[ n = \frac{N}{N_d} = \frac{16.5}{251.68} = 0.066 < 0.2 \]
\[ \text{Therefore } M_{ndz} = 1.1M_{dz} (1-n) < M_{dz} \]
\[ \text{Therefore } M_{ndz} = 1.1\{1.0 \times 2074.7 \times 2500 / 1.10 \} (1 - 0.066) \times 10^{-5} < M_{dz} \]
\[ \text{Therefore } M_{ndz} = 48.44^{T-m} > M_{dz} (=47.15^{T-m}) \]
\[ \text{Therefore } M_{ndz} = M_{dz} = 47.15^{T-m} \]
\[ \text{For, } n = 0.2 \ M_{ndy} = M_{dy} \]
\[ M_{ndy} = (1.0 \times 302.9 \times 2500) / (1.10 \times 100000) = 6.88^{T-m} \]
\[ (\beta_b = 1.0 \text{ for calculation of } M_{dz} \text{ and } M_{dy} \text{ as per clause 8.2.1.2}) \]
Therefore \( \left( \frac{M_y}{M_{ndy}} \right)^{\alpha_1} + \left( \frac{M_z}{M_{ndz}} \right)^{\alpha_2} = \left( \frac{1.44}{6.88} \right)^1 + \left( \frac{12.60}{47.15} \right)^2 = 0.281 \leq 1.0 \)

\[ \{ \alpha_1 = 5n \text{ but } \geq 1, \text{ therefore } \alpha_1 = 5 \times 0.066 = 0.33 = 1 \]

\[ \alpha_2 = 2 \text{ (As per Table 9.1)} \}

Alternatively,

\[ \left( \frac{N}{N_d} \right) + \left( \frac{M_z}{M_{dz}} \right) + \left( \frac{M_y}{M_{dy}} \right) = \left\{ \frac{(16.5 \times 10^3 \times 1.10)}{(110.7 \times 2500)} + \frac{(12.60 \times 10^5 \times 1.10)}{(302.9 \times 2500)} \right\} = 0.54 \leq 1.0 \]

(iii) Check for resistance of the member for combined effects of buckling (Clause 9.3.2):

(a) Determination \( P_{dz}, P_{dy} \) and \( P_d \) (Clause 7.12)

\[ K_{Ly} = K_{Lz} = 0.85 \times 500 = 425 \text{mm} \]

\[ \left( \frac{K_{Lz}}{r_z} \right) = \frac{425}{20.21} = 21.03 \]

\[ \left( \frac{K_{Ly}}{r_y} \right) = \frac{425}{3.52} = 120.7 \]

Therefore, non-dimensional slenderness ratios, \( \lambda_z = 0.237 \) and \( \lambda_y = 1.359 \).

For major axis buckling curve 'a' is applicable (Refer Table 7.2).

From Table 7.4a,

\[ f_{cdz} = 225.4 \text{ MPa} \]

\[ P_{dz} = 225.4 \times 10 \times 110.7 \times 10^{-3} = 249.65^T \]

For minor axis buckling curve 'b' is applied (Refer Table 7.2)

From Table 7.4b,

\[ f_{cdy} = 90.80 \text{ MPa} \]

\[ P_{dy} = 90.80 \times 10 \times 110.7 \times 10^{-3} = 100.60^T \]

Therefore, \( P_d = P_{dy} = 100.60^T \)

(b) Determination of \( M_{dz} \) (Clause 9.3.2.2 and Clause 8.2.2).
Elastic critical moment is given by (clause 8.2.21).

\[ M_{cr} = [(\beta_{LT} \pi^2 EI_y h) / \{2(KL)^2\}] \left[ 1 + 1/20 \{(KL / r_y) / (h / t)^2\}\right]^{0.5} \]

\[ = [(1.20 \times \pi^2 \times 2 \times 10^5 \times 1369.8 \times 10^4 \times 500) / (2 \times 4250^2)] \left[ 1 + 1/20 \{(120.70) / (500 / 17.2)\}\right]^{0.5} \]

\[ = 6.129 \times 10^8 \text{ N-mm} \]

\[ \lambda_{lt} = \sqrt{\frac{\beta Z_y f_y}{M_{cr}}} \]

\[ = \sqrt{\frac{(1.0 \times 2074.7 \times 10^3 \times 250 \times 6.129 \times 10^8)}{6.129 \times 10^8}} \]

\[ = 0.92 \]

\[ \phi_{lt} = 0.5 \left[ 1 + \alpha_{lt} \left( \lambda_{lt} - 0.2 \right) + \lambda_{lt}^2 \right] \]

\[ = 0.5 \left[ 1 + 0.21(0.92 - 0.2) + 0.92^2 \right] \]

\[ = 0.999 \]

\[ X_{lt} = 1 / \left[ \phi_{lt} + \left( \phi_{lt}^2 - \lambda_{lt}^2 \right)^{0.5} \right] \]

\[ = 0.72 \leq 1.0 \]

\[ f_{yd} = X_{lt} f_y / \gamma_m = 0.72 \times 250 / 1.1 = 168.3 \text{ MPa} \]

\[ M_{dy} = \beta_s Z_{py} f_{yd} \]

\[ = 1.0 \times 2074.7 \times 1638 \times 10^5 \]

\[ = 33.98^{T-m} \]

\[ \left[ \beta_{yt} = 1.20, \beta_s = 1.0, \alpha_{yt} = 0.21 \right] \]

(c) Determination of \( M_{dy} \) (Clause 9.3.2.2)

\[ M_{dy} = \beta_b Z_{py} f_y / \gamma_m \]

\[ = 1.0 \times 302.90 \times 2500 / 1.1 \times 10^{-5} = 6.88^{T-m} \]

(d) Determination of \( C_z \) (Clause 9.3.2.2) From Table - 9.2,

\[ \psi_s = 9.15 / 12.6 = 0.726 \]

\[ \therefore \beta_{zs} = 1.8 - 0.7 \times 0.726 = 1.292 \]

\[ \therefore \mu_s = \lambda_s (2 \beta_{zs} - 4) + (Z_{zs} - Z_s) / Z_s \leq 0.9 \]
\begin{align*}
\mu_{tr} &= 0.237 (2 \times 1.292 - 4) + 0.1469 \\
&= 0.188
\end{align*}

For torsional buckling,

\begin{align*}
\mu_{tr} &= 0.15 \beta_{tr} \beta_{tr} - 0.15 \leq 0.90 \\
\beta_{tr} &= 0.15 \times 1.292 \times 1.359 - 0.15 = 0.113 \\
\therefore & \text{since } \mu_{tr} \text{ is larger of } \mu_{ta} \text{ and } \mu_{tr}, \mu_{tr} = 0.113 \\
C_{z} &= 1 - \frac{(\mu_{ta} P)}{P_{a}} \\
&= 1 - \frac{0.113 \times 16.5}{249.65} = 0.993 \leq 1.50
\end{align*}

(e) Determination of \(C_{r}\) (Clause 9.3.2.2)

\begin{align*}
\gamma_{r} &= \frac{1.335}{-1.44} = -0.927 \\
\therefore \beta_{dr} &= 1.8 - 0.7 \times (-0.927) = 2.449 \\
\therefore \mu_{r} &= \lambda_{r} (2 \beta_{dr} - 4) + (Z_{pr} - Z_{dr}) / Z_{dr} \leq 0.9 \\
&= 1.359 (2 \times 2.449 - 4) + 0.99 = 2.22 \approx 0.90 \\
\therefore C_{r} &= 1 - \frac{(\mu_{r} P)}{P_{a}} \\
&= 1 - 0.90 \times 16.5 / 100.6 = 0.85 \\
\therefore P / P_{a} + C_{z}M_{x} / M_{a} + C_{r}M_{y} / M_{a} \\
&= 16.50 / 100.60 + 0.993 \times 12.60 / 33.98 + 0.85 \times 1.44 / 6.88 \\
&= 0.164 + 0.368 + 0.178 = 0.71 \leq 1.0
\end{align*}

Therefore, section is safe. Interaction value is less in LSM than WSM.
Design of typical beam

Example-1

The beam (ISMB 400) in Fig.1 is designed considering it is fully restrained laterally.

1. WSM (clause 6.2 of IS:800 - 1984):

   Bending moment, \( M=18.75\, Tm \)
   For ISMB 450, \( Z=1350.7\, \text{cm}^3 \)
   Therefore
   \[
   s_{bc\,(cal)} = \frac{18.75 \times 10^5}{1350.7} \\
   = 1388.17\, \text{kg/cm}^2 < 1650\, \text{Kg/cm}^2
   
   \]
   Therefore percentage strength attained is 0.8413 or 84.13%.

2. LSM (clause 8.2 of draft IS:800):

   Factored load = \( 1.5 \times 2.0 + 1.5 \times 4.0 = 9.0\, T \)
   Factored bending moment = \( 9.0 \times 5.0^2 / 8 \)
   \[
   = 28.125\, Tm
   
   \]
   Factored shear force = \( 22.50\, T = Fv \)

Fig.1
For, ISMB450,

\[
\begin{align*}
D &= 450 \text{ mm} \quad B = 150 \text{ mm} \\
t_w &= 9.4 \text{ mm} \quad T = 17.4 \text{ mm} \\
I_{xx} &= 30390.8 \text{ cm}^4 \quad I_{yy} = 834.0 \text{ cm}^2 \\
r_{yy} &= 3.01 \text{ cm} \quad h_1 = 379.2 \text{ cm} = d
\end{align*}
\]

i) Refering Table 3.1 of the code for section classification, we get:

Flange criterion = \( \frac{B}{2T} = \frac{150}{2 \times 17.4} \)

\[= 4.31 < 9.4.\]

Web criteria = \( \frac{d}{t_w} = \frac{379.2}{9.4} \)

\[= 40.34 < 83.9.\]

Therefore, section is plastic.

ii) Shear capacity (refer clause 8.4 of draft code):

\[
F_{vd} = \frac{f_{yw} \cdot A_v}{g_{mo}} = \frac{f_{yw} \cdot h_{tw}}{\sqrt{3} \cdot g_{mo}}
\]

\[= \frac{(2500 \cdot 45.0 \cdot 0.94)}{(\sqrt{3} \cdot 1.10 \cdot 1000)} \]

\[= 55.50T \]

\[
\frac{F_v}{F_{vd}} = \frac{22.50}{55.5} = 0.405 < 0.6.
\]

Therefore, shear force does not govern permissible moment capacity (refer clause 8.2.1.2 of the draft code).

iii) Since the section is 'plastic',

\[
M_d = \frac{Z_p \cdot f_y}{g_{mo}}
\]

where, \( Z_p = \text{Plastic Modulus} = 1533.36 \text{ cm}^3 \)

(refer Appendix-I of draft code).

Therefore, \( M_d = \frac{(1533.36 \times 2500)}{1.10 \times 10^5} \)

\[= 34.85 T_m \]

Percentage strength attained for LSM is \( \frac{28.125}{34.85} = 0.807 \) or 80.7% which is less than 84.3% in the case of WSM.
iv ) Now , let us check for web buckling :

\[ K_v = 5.35 \] (for transverse stiffeners only at supports as per clause 8.4.2.2 of draft code).

The elastic critical shear stress of the web is given by :

\[ \tau_{cr.e} = \frac{K_v \cdot \pi^2 \cdot E}{12 (1 - \mu^2)(d/t_w)^2} \]

\[ = \frac{(5.35 \cdot \pi^2 \cdot E)}{12 (1 - 0.32)(379.2/9.4)^2} \]

\[ = 594.264 \text{ N/mm}^2 \] .

Now as per clause 8.4.2.2 of draft code ,

\[ \lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3} \tau_{cr.e}}})^{0.5} \]

\[ = \frac{250}{\sqrt{3} \cdot 594.264} \] \(0.5 \) \[ = 0.4928 < 0.8 \] .

where , \( \lambda_w \) is a non-dimensional web slenderness ratio for shear buckling for stress varying from greater than 0.8 to less than 1.25 .

Therefore , \( \tau_b \) (shear stress corresponding to buckling ) is given as :

\[ \tau_b = \frac{f_{yw}}{\sqrt{3}} = 250 / \sqrt{3} = 144.34 \text{ N/mm}^2 \] .

\[ V_d = d \cdot t_w \cdot \tau_b / \gamma_m \]

\[ = 37.92 \cdot 0.94 \cdot 1443.34 \cdot 10^{-3} / 1.10 \]

\[ = 46.77T_s \]

As \( V_D > F_v \) (\( = 22.5T \) ), the section is safe in shear.

For checking of deflection :

\[ s = \frac{(5 \cdot 60 \cdot 500^4)}{(384 \cdot 2.1 \cdot 10^6 \cdot 30390.8)} \]

\[ = 0.76 \text{ cm} = 7.6 \text{ mm} = \text{L} / 658 \]

Hence O.K.
Example - 2

The beam (ISMB 500) shown in Fig2 is carrying point load as shown, is to be designed. The beam is considered to be restrained laterally.

1. In WSM (clause 6.2 of IS : 800 - 1984):

   Bending Moment, \( M = (5 + 20) \times \frac{3}{4} \)
   
   \[ = 26.25\text{Tm} \]

   For, ISMB 500: \( Z = 1808.7\text{ cm}^3 \)

   So, \( \sigma_{bc(cal)} = \frac{26.25 \times 10^5}{1808.7} \text{ Kg/cm}^2 \)
   
   \[ = 1451.318 \text{ Kg/m}^2 . \]

   Therefore, percentage strength attained is 0.88 or 88%.

2. In LSM (clause 8.2 of draft IS : 800):

   Factored load = \( 1.5 \times 15 + 1.5 \times 20 = 52.50\text{T} \)

   Therefore,

   Factored bending moment = \( 52.5 \times 3.0 / 4 \)
   
   \[ = 39.375\text{Tm} \]

   Factored shear force, \( F_v = 52.50 / 2 = 26.25\text{T} \)

   i) For section classification of this ISMB500, (refer table 3.1 of the code), we get:
\[ D = 500 \text{ mm} \quad d = 424.1 \text{ mm} \]
\[ r_1 = 17 \text{ mm} \quad r_y = 3.52 \text{ cm} \]
\[ Z_p = 20.2574 \text{ cm}^3 \quad T = 17.2 \text{ mm} \]
\[ b = 180 \text{ mm} \quad t_w = 10.2 \text{ mm} . \]

Flange criterion, \( \frac{b}{2} / T = \frac{180}{2} / 17.2 \)
\[ = 5.23 < 9.4 \]

Web criterion, \( d / t_w = 424.1 / 10.2 \)
\[ = 41.57 < 83.9 \text{ (O.K.)} \]

Therefore the section is plastic.

ii) Shear capacity (clause 8.4):

\[ F_{vd} = \frac{f_{yw} \cdot A_v}{\gamma_{mo}} \]
\[ = \frac{2500 \cdot 50 \cdot 1.02}{1.10 \cdot 1000} \]
\[ = 66.92T \]

\[ F_v = 26.25T \]

Since \( F_v / F_{vd} < 0.6 \) shear force does not govern Bending Strength

iii) For moment check:

Since the section is plastic,

\[ M_d = \frac{Z_p \cdot f_y}{\gamma_{mo}} \]
\[ = \frac{2025.74 \cdot 2500}{1.10 \cdot 10^5} \]
\[ = 46.04T_{m} \]

Therefore, Percentage strength attained is \( \frac{39.375}{46.04} = 0.855 \) or 85.5% which is less than 88% in case of WSM

iv) Check for web buckling:
K_v = 5.35 (for transverse stiffeners only at supports as per clause 8.4.2.2 of draft code).

\[
\tau_{cr,e} = \left( \frac{K_v \cdot \pi^2 \cdot E}{12 \left(1 - \mu^2\right)(d/t_w)^2} \right)
\]

\[
= \frac{5.35 \cdot \pi^2 \cdot E}{12 \left(1 - 0.3^2\right)(424.1 / 10.2)^2} \]

\[
= 559.40 \text{ N/mm}^2.
\]

Now as per clause 8.4.2.2,

\[
\lambda_w = \sqrt{\frac{f_{yw}}{\tau_{cr,e}}}^{0.5}
\]

\[
= \frac{250}{\sqrt{559.40}}^{0.5} = 0.508 < 0.8 .
\]

Therefore,

\[
\tau_b = \frac{f_{yw}}{\sqrt{3}} = \frac{250}{\sqrt{3}} = 144.34 \text{ N/mm}^2.
\]

\[
V_D = d.t_w \cdot \tau_b / \gamma_{mo}
\]

\[
= 42.41 \cdot 1.02 \cdot 1443.4 \cdot 10^{-3} / 1.10
\]

\[
= 56.76^T
\]

Since \(V_D > F_v (= 26.25^T)\), the section is safe against shear.

For checking of deflection:

\[
\sigma = \frac{(35000 \cdot 300^3)}{(48 \cdot 2.1 \cdot 10^6 \cdot 45218.3)}
\]

\[
= 0.207 \text{ cm} = 2.07 \text{ mm} = L / 1446
\]

Hence O.K.
Example - 3

The beam, ISMB500 as shown in Fig.3 is to be designed considering no restraint along the span against lateral buckling.

i) In WSM (clause 6.2, 6.2.2, 6.2.3 and 6.2.4 and 6.2.4.1 of IS : 800 - 1984):

Bending moment, \( M = 2.1 \times 6^2 / 8 = 9.45 \text{Tm} \)

For, ISMB500:

- \( L = 600 \text{ cm} \)
- \( r_y = 3.52 \text{ cm} \)
- \( T = 17.2 \text{ mm} \)
- \( Z = 1808.7 \text{ cm}^3 \)

\[ Y = \frac{26.5 \times 10^5}{(600/3.52)^2} = 91.2 \]

\[ X = 91.21 \sqrt{1 + \left( \frac{1}{20} \right) \left( \frac{600 \times 1.72}{3.52 \times 50} \right)^2} = 150.40 \]

and \( f_{cb} = k_1 (X + k_2 Y) \frac{C_2}{C_1} = X \)

(since \( C_2 = C_1, k_1 = 1 \) and \( k_2 = 0 \))

Now,

\[ \sigma_{bc(perm)} = \frac{0.66 f_{cb} f_y}{f_{cb}^n + f_y^{m/1/n}} \]

\[ = 746 \text{ Kg/cm}^2. \]
and \( \sigma_{bc(cal)} = \frac{9.45 \times 10^5}{1808.7} \)

\[ = 522.5 \text{ Kg/cm}^2. \]

Therefore, percentage strength attained is \( \frac{522.5}{746} = 0.7 \) or 70%.

ii) In LSM (clause 8.2 of draft IS: 800):

For MB 500:

\[ D = 500 \text{ mm} \quad T = 17.2 \text{ mm} \]
\[ B = 180 \text{ mm} \quad t = 10.2 \text{ mm} \]
\[ Z_p = 2025.74 \text{ cm}^3 \quad r_{yy} = 3.52 \text{ cm} \]
\[ H_1 = d = 424.1 \text{ mm} \]
\[ I_{zz} = 45218.3 \text{ cm}^4, \quad I_{yy} = 1369.8 \text{ cm}^4 \]

iii) Classification of section (ref. Table 3.1 of the code):

\[ \frac{b}{T} = \frac{90}{17.2} = 5.2 < 9.4, \text{ hence O.K.} \]
\[ \frac{d}{t} = \frac{424.1}{10.2} = 41.6 < 83.90 \text{ (O.K)} \]

Therefore, the section is Plastic.

iv) Check for torsional buckling (clause 8.2.2):

\[ \frac{t_f}{t_w} \text{ for ISMB 500} = \frac{17.2}{10.2} = 1.69 < 2.0 \]

Therefore, \( \beta_{LT} = 1.20 \), for plastic and compact sections.

\( M_{cr} \) = Elastic critical moment given as:

\[ M_{cr} = \left\{ \frac{\beta_{LT} \pi^2 EI_y}{(KL)^2} \right\} \left[ 1 + \left( \frac{1}{20} \right) \frac{(KL/r_y)}{(h/t_f)} \right]^0.5 \frac{h}{2} \]

\[ (\text{ref. clause 8.2.2.1}) \]

\[ = (1.20 \pi^2 \times 2 \times 10^6 \times 1369.8 / 600^2) \left[ 1 + \left( \frac{1}{20} \right) \frac{(600/3.52)}{(50/1.72)} \right]^0.5 \frac{50}{2} \]

\[ = 371556.3 \text{ Kg-cm} \]

Now, \( \lambda_{LT} = \sqrt{\left( \beta_b \cdot Z_p \cdot f_y / M_{cr} \right)} = 1.1675 \)

( since \( \beta_b = 1.0 \) for plastic and compact sections)

Therefore,
\[ \phi_{LT} = 0.5 \left[ 1 + \alpha_{LT} \left( \lambda_{LT} - 0.2 \right) + \lambda_{LT}^2 \right] \]
\[ = 0.5 \left[ 1 + 0.2 \left( 1.1675 - 0.2 \right) + 1.1675^2 \right] \]
\[ = 1.283 \left( \alpha_{LT} = 0.21 \text{ for rolled section} \right). \]

Therefore \( X_{LT} = \frac{1}{\phi_{LT} + \left\{ \phi_{LT}^2 - \lambda_{LT}^2 \right\}^{0.5}} \)
\[ = \frac{1}{1.283 + \left\{ 1.283^2 - 1.1675^2 \right\}^{0.5}} \]
\[ = 0.55 \]

\( M_d = 1.0 \times 0.55 \times 2025.74 \times 2500 / 1.10 \times 10^{-5} \]
\[ = 25.32^{\text{Tm}} \]

Now actual moment is obtained as follows:

Factored load = \( 1.0 \times 1.5 + 1.1 \times 1.5 = 3.15^{\text{Tm}} \)

Factored moment = \( 14.175^{\text{Tm}} < M_d \)

Percentage strength attained is \( \frac{14.75}{25.32} = 0.56 \text{ or 56 \% which is less than 70 \% in case of WSM.} \)

Hence, the beam is safe both in LSM and WSM design but percentage strength attained is comparatively less in LSM for the same section

**Problem:**

The girder showed in Fig. E1 is fully restrained against lateral buckling throughout its span. The span is 36 m and carries two concentrated loads as shown in Fig. E1. Design a plate girder.

Yield stress of steel, \( f_y = 250 \text{ N/mm}^2 \)

Material factor for steel, \( \gamma_m = 1.15 \)

Dead Load factor, \( \gamma_{fd} = 1.50 \)

Imposed load factor, \( \gamma_{fl} = 1.50 \)
1.0 Loading

Dead load:

Uniformly distributed load, \( w_d \) = 18 kN/m

Concentrated load, \( W_{1d} \) = 180 KN

Concentrated load, \( W_{2d} \) = 180 KN

Live load:

Uniformly distributed load, \( w \) = 35 kN/m

Concentrated load, \( W_{1l} \) = 400 kN

Concentrated load, \( W_{2l} \) = 400 kN

Factored Loads

\[
w' = w_d \cdot \gamma_{ld} + w_l \cdot \gamma_l = 18 \cdot 1.5 + 35 \cdot 1.5 = 79.5 \text{ kN/m}
\]

\[
W'_1 = W_{1d} \cdot \gamma_{fd} + W_{1l} \cdot \gamma_l = 180 \cdot 1.5 + 400 \cdot 1.5 = 870 \text{ kN}
\]

\[
W'_2 = W_{2d} \cdot \gamma_{fd} + W_{2l} \cdot \gamma_l = 180 \cdot 1.5 + 400 \cdot 1.5 = 870 \text{ kN}
\]
2.0 Bending moment and shear force

<table>
<thead>
<tr>
<th></th>
<th>Bending moment (kN-m)</th>
<th>Shear force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UDL effect</td>
<td>( \frac{wL^2}{8} )</td>
<td>( \frac{wL}{2} )</td>
</tr>
<tr>
<td>Concentrated load effect</td>
<td>( \frac{wL}{4} )</td>
<td>( w = 870 )</td>
</tr>
<tr>
<td>Total</td>
<td>20709</td>
<td>2301</td>
</tr>
</tbody>
</table>

The design shear forces and bending moments are shown in Fig. E2.

3.0 Initial sizing of plate girder

Depth of the plate girder:

The recommended span/depth ratio for simply supported girder varies between 12 for short span and 6 for long span girder. Let us consider depth of the girder as 2600 mm.

\[
\frac{L}{d} = \frac{36000}{2600} = 13.8
\]

Depth of 2600 mm is acceptable.

*For drawing the bending moment and shear force diagrams, factored loads are considered*
**Flange:**

\[ P_y = \frac{250}{1.15} = 217.4 \text{ N/mm}^2 \]

Single flange area,

\[ A_f = \frac{M_{\text{max}}}{d_{P_y}} = \frac{20709 \times 10^4}{2600 \times 217.4} = 36637.5 \text{ mm}^2 \]

By thumb rule, the flange width is assumed as 0.3 times the depth of the section.

Try 780 X 50 mm, giving an area = 39000 mm\(^2\).
**Web:**

Minimum web thickness for plate girder in buildings usually varies between 10 mm to 20 mm. Here, thickness is assumed as 16mm.

Hence, web size is 2600 X 16 mm

---

**4.0 Section classification**

**Flange:**

\[
g = \left( \frac{250}{f_y} \right)^{\frac{1}{3}} = \left( \frac{250}{250} \right)^{\frac{1}{3}} = 1.0
\]

\[
b = \frac{B - t}{2} = \frac{780 - 16}{2} = 382
\]

\[
\frac{b}{T} = \frac{382}{50} = 7.64 < 9g
\]

Hence, Flange is COMPACT SECTION.

**Web:**

\[
\frac{d}{t} = \frac{2600}{250} = 162.5 > 67g
\]

Hence, the web is checked for shear buckling.

---

**5.0 Checks**

**Check for serviceability:**

\[
\frac{d}{250} = \frac{2600}{250} = 10.4 \text{mm} < 1
\]

Since, \( t > \frac{d}{250} \)

Web is adequate for serviceability.
Check for flange buckling in to web:

Assuming stiffener spacing, \( a > 1.5 \ d \)

\[
t \geq \frac{d}{294} \left( \frac{P_y}{250} \right)^{\frac{1}{2}} = \frac{2600}{294} \times \left( \frac{217.4}{250} \right)^{\frac{1}{2}} = 8.2 \text{ mm}
\]

Since, \( t (=16 \text{ mm}) > 8.2 \text{ mm} \), the web is adequate to avoid flange buckling into the web.

Check for moment carrying capacity of the flanges:

The moment is assumed to be resisted by flanges alone and the web resists shear only.

Distance between centroid of flanges, \( h_s = d + T = 2600 + 50 = 2650 \text{ mm} \)

\( A_f = B \times T = 780 \times 50 = 39000 \text{ mm}^2 \)

\( M_c = P_{yr} \times A_f \times h_s = 217.4 \times 39000 \times 2650 \times 10^{-6} = 222468.3 \text{ kN-m} \)

\( > 20709 \text{ kN-m} \)

Hence, the section is adequate for carrying moment and web is designed for shear.

6.0 Web design

The stiffeners are spaced as shown in Fig.E5. Three different spacing values 2500, 3250 and 3600 mm are adopted for trial as shown in Fig. E5.

End panel (AB) design:

d = 2600 mm

t = 16 mm

Maximum shear stress in the panel is
Calculation of critical stress,

\[ q_e = \left(0.75 + \frac{1}{(a/d)^2}\right) \left[\frac{1000}{(d/t)}\right]^2 = 69.5 \text{ N/mm}^2 \]

Slenderness parameter,

\[ \lambda_w = \left[0.6\left(\frac{f_y}{f_o}\right)/q_e\right]^{1/3} = \left[0.6\left(\frac{250}{1.15}\right)/69.5\right]^{1/3} = 1.37 > 1.25 \]

Hence, Critical shear strength \((q_{cr} = q_e) = 69.5 \text{ N/mm}^2\)

Since, \(f_v < q_{cr}\)  \((55.3 < 69.5)\)

**Tension field action need not be utilised for design.**

**Checks for the end panel AB:**

End panel AB should also be checked as a beam (Spanning between the flanges of the girder) capable of resisting a shear force \(R_{tf}\) and a moment \(M_{tf}\) due to anchorage forces.

(In the following calculations boundary stiffeners are omitted for simplicity)
Check for shear capacity of the end panel:

\[ H_q = 0.75d t \gamma \left[ 1 - \frac{q_{s,t}}{0.6\gamma} \right]^{\frac{1}{3}} \]

\[ q_{s,t} = 69.5 \text{ N/mm}^2 \]

\[ H_q = 0.75 \times 2600 \times 16 \times 250 / 1.15 \left[ 1 - \frac{69.5}{0.6 \times (250 / 1.15)} \right]^{\frac{1}{3}} = 4636 \text{ kN} \]

\[ R_{\gamma} = \frac{H_q}{2} = \frac{4636}{2} = 2318 \text{ kN} \]

\[ A_{\gamma} = t \cdot a = 16 \times 2500 = 4000 \text{ mm}^2 \]

\[ P_{\gamma} = 0.6 \gamma A_{\gamma} = 0.6 \times (250 / 1.15) \times 40000 / 1000 = 5217 \text{ kN} \]

Since, \( R_{\gamma} < P_{\gamma} \), the end panel can carry the shear force.

Check for moment capacity of end panel AB:

\[ M_q = \frac{H_q d}{10} = \frac{4636 \times 2600 \times 10^{-3}}{10} = 1205 \text{ kN-m} \]

\[ y = \frac{a}{2} = \frac{2500}{2} = 1250 \]

\[ I = \frac{1}{12} t a^3 = \frac{1}{12} \times 16 \times 2500^3 = 2083 \times 10^7 \text{ mm}^4 \]

\[ M_q = \frac{1}{y} y = \frac{2083 \times 10^7}{1250} \times (250 / 1.15) \times 10^{-6} = 3623 \text{ kN-m} \]

Since, \( M_q < M_q \) \hspace{1cm} (1205 < 3623)

The end panel can carry the bending moment.

Design of panel BC:

Panel BC will be designed using tension field action

\[ d = 2600 \text{ mm} \hspace{1cm} t = 16 \text{ mm} \]
Calculation of basic shear strength, $q_b$:

**Elastic critical stress, $q_e$ (when $a/d > 1$):**

$$q_e = \left[ 1.0 + 0.75 \left( \frac{a}{d} \right)^2 \right] \left[ \frac{1000}{(d/t)^2} \right]$$

$$= \left[ 1.0 + 0.75 \left( \frac{3250}{2600} \right)^2 \right] \left[ \frac{1000}{(2600/16)^2} \right]$$

$$= 56.0 \text{ N/mm}^2$$

**Slenderness parameter, $\lambda_h$:**

$$\lambda_h = \left[ 0.6 \left( \frac{f_{ye}}{f_{re}} \right) / q_e \right]^{\frac{1}{2}}$$

$$= \left[ 0.6 \left( \frac{250/1.15}{56.0} \right) / 56.0 \right]^{\frac{1}{2}}$$

$$= 2.33 > 1.25$$

**Hence, Critical shear strength**

$$q_a = q_e = 56.0 \text{ N/mm}^2$$

$$\phi_1 = \frac{1.5q_a}{\sqrt{1 + \left( \frac{a}{d} \right)^2}} = \frac{1.5 \times 56.0}{\sqrt{1 + (1.25)^2}} = 52.5$$

$$y_b = \left( p_{ye}^2 - 3q_a^2 - \phi_1^2 \right)^{\frac{1}{2}} - \phi_1 = \left( 217.4^2 - 3 \times 56.0^2 + 52.5^2 \right)^{\frac{1}{2}} - 52.5 = 149$$

$$q_b = q_a + \frac{y_b}{2 \sqrt{1 + \left( \frac{a}{d} \right)^2}} = 56.0 + \frac{149}{2 \sqrt{1 + (1.25)^2}} = 82.1 \text{ N/mm}^2$$

Since, $q_b > f_y$ (82.1 > 50.5)

Panel BC is safe against shear buckling.
7.0 Design of stiffeners

**Load bearing stiffener at A:**

Design should be made for compression force due to bearing and moment.

Design force due to bearing, \( F_b = 2301 \text{ kN} \)

Force \((F_m)\) due to moment \( M_{tf} \), is

\[
F_m = \frac{M_{tf}}{a} = \frac{1205 \times 10^3}{2500} = 482 \text{ kN}
\]

Total compression = \( F_c = F_b + F_m = 2301 + 482 = 2783 \text{ kN} \)

**Area of Stiffener in contact with the flange, A:**

Area \((A)\) should be greater than

\[
A = \frac{0.8 F_c}{P_{ps}} = \frac{0.8 \times 2783 \times 10^3}{217.4} = 10241 \text{ mm}^2
\]

Try stiffener of 2 flats of size 270 x 25 mm thick

Allow 15 mm to cope for web/flange weld

\( A = 255 \times 25 \times 2 = 12750 \text{ mm}^2 > 10241 \text{ mm}^2 \)

Bearing check is ok.

**Check for outstand:**

Outstand from face of web should not be greater than

\[
s = \left(\frac{250}{f_y}\right)^{\frac{1}{3}} = \left(\frac{250}{250}\right)^{\frac{1}{3}} = 1.0
\]

Outstand \( d = 250 \text{ mm} < 20 t_e s (=20 \times 25 \times 1.0 = 500) \)

\( b_e = 250 \text{ mm} < 13.7 t_e s (= 13.7 \times 25 \times 1.0 = 342.5) \)
Hence, outstanding criteria is satisfied.

**Check stiffener for buckling:**

The effective stiffener section is shown in Fig. E3

![Fig. E3 End bearing stiffener](image)

The buckling resistance due to web is neglected here for the sake of simplicity.

\[
I_x = \frac{25 \times 556^2}{12} - \frac{1}{12} \times 25 \times 16^3 = 35807 \times 10^4 \text{ mm}^4
\]

\[
A_x = \text{Effective area} = 270 \times 25 \times 2 = 13500 \text{ mm}^2
\]

\[
r_x = \left[ \frac{I_x}{A_x} \right]^{\frac{1}{3}} - \left[ \frac{35807 \times 10^4}{13500} \right]^{\frac{1}{3}} = 162.8 \text{ mm}
\]

Flange is restrained against rotation in the plane of stiffener, then

\[
l_e = 0.71 = 0.7 \times 2600 = 1820
\]
Buckling resistance of stiffener is

\[ P_c = \sigma_c A_y / \gamma_m = 250 * 1350 / 1.15 = 2935 \text{ kN} \]

Since \( F_e < P_c \) (2783 < 2935)

Therefore, stiffener provided is safe against buckling.

**Check stiffener A as a bearing stiffener:**

Local capacity of the web:

Assume, stiff bearing length \( b_1 = 0 \)

\[ n_2 = 2.5 \times 50 \times 2 = 250 \]

BS 5950 : Part -1, Clause 4.5.3

\[ P_{crip} = (b_1 + n_2) tP_{yw} \]

\[ = (0 + 250) \times 16 \times (250/1.15) \times 10^{-3} = 870 \text{ kN} \]

Bearing stiffener is designed for \( F_A \)

\[ F_A = F_x = P_{crip} = 2783 - 870 = 1931 \text{ kN} \]

Bearing capacity of stiffener alone

\[ P_A = P_{ys} \times A = (50/1.15) \times 13500/1000 = 2935 \text{ kN} \]

Since, \( F_A < P_A \) (1931 < 2935)

The designed stiffener is OK in bearing.

**Stiffener A** - Adopt 2 flats 270 mm X 25 mm thick
Design of intermediate stiffener at B:

Stiffener at B is the most critical one and will be chosen for the design.

Minimum Stiffness

\[ I_y \geq 0.75d^3 \quad \text{for} \quad a \geq d\sqrt{2} \]
\[ I_y \geq \frac{0.75d^3}{a^3} \quad \text{for} \quad a < d\sqrt{2} \]

\[ d\sqrt{2} = \sqrt{2 \times 2600} = 3677 \text{ mm} \]
\[ a < d\sqrt{2} \quad (3250 < 3677) \]

Conservatively ' \( t \) ' is taken as actual web thickness and minimum ' \( a \) ' is used.

\[ \frac{1.5d^3t^3}{a^3} = \frac{1.5 \times 2600^3 \times 16^3}{3250^3} = 1022 \times 10^4 \text{ mm}^4 \]

Try intermediate stiffener of 2 flats 120 mm X 14 mm

\[ \left( I_y \right)_{\text{required}} = \frac{14 \times 256^3}{12} - \frac{14 \times 16^3}{12} = 1957 \times 10^4 \text{ mm}^4 \]

\[ I_y \geq \frac{1.5d^3t^3}{a^3}, \text{ the section satisfied minimum stiffness requirement} \]

Check for outstand:

Finally, the stiffener:

Outstand is less than 13.7 \( t_s \) s

13.7\( t_s \) s = 13.7 \times 14 \times 1.0 = 192 \text{ mm} \]

Outstand = 120 mm \( (120 < 192) \)

Hence, outstand criteria is satisfied.

Buckling check:

\[ F_q = V - V_s \]

Where \( V \) = Total shear force

\[ V_s = V_{cr} \text{ of the web} \]

\[ a / d = 3600 / 2600 = 1.38 \]
\[ \frac{d}{t} = \frac{2600}{16} = 162.5 \]

**Elastic critical stress, \( q_e \) (when \( \frac{a}{d} > 1 \))**

\[
= [1.0 + 0.75 \left( \frac{a}{d} \right)^2] \left[ \frac{1000 \left( \frac{d}{t} \right)}{162.5} \right]^2
\]

\[
= [1.0 + 0.75 \left( \frac{1.38}{1.38} \right)^2] \left[ \frac{1000 \left( \frac{162.5}{162.5} \right)}{162.5} \right]^2
\]

\[
= 52.8 \text{ N/mm}^2
\]

**Slenderness parameter, \( \lambda_w \)**

\[
= \left[ \frac{6 (f_{yw} / f_m)}{q_e} \right]^{\frac{1}{2}}
\]

\[
= \left[ 0.6 \left( \frac{250}{1.15} \right) / 52.8 \right]^{\frac{1}{2}}
\]

\[
= 2.47 > 1.25
\]

Hence, Critical shear strength \( q_c = q_e \)

\[ V_{cr} = q_c dt = 52.8 \times 2600 \times 16 \times 10^{-3} = 2196 \text{ kN} \]

**Buckling resistance of intermediate stiffener at B:**

![Diagram](image-url)
Design of Steel Structures  Prof. S.R.Satish Kumar and Prof. A.R.Santha Kumar

Shear force at B, \( V_B = 2301 - \left( \frac{2301 - 1585.5}{2500} \right) \times 9000 \) = 2102 kN

Buckling resistance = \( \left( \frac{213.2}{1.15} \right) \times 13600 \times 10^{-3} = 2521 \) kN

From table3 of chapter on axially compressed columns,

\[ \sigma_c = 213.2 \ \text{kN/mm}^2 \]

Buckling resistance = \((213.2/1.15) \times 13600 \times 10^{-3} = 2521 \) kN

Shear force at B, \( V_B = 2301 - \{(2301 - 1585.5) \times (2500/9000)\} = 2102 \) kN

Stiffener force, \( F_q = [21201 - 2196] < 0 \)

and

\[ F_q < \text{Buckling resistance} \]

Hence, intermediate stiffener is adequate

**Intermediate stiffener at B** - Adopt 2 flats 120 mm X 14 mm

**Intermediate stiffener at E** (Stiffener subjected to external load):

Minimum stiffness calculation:

\[ a = 3600 \]
\[ d\sqrt{2} = 3677 \]
\[ a < d\sqrt{2} \quad (3600 < 3677) \]

\[ I_x \geq \frac{1.5d^3t^3}{a^2} = \frac{1.5 \times 2600^3 \times 16^3}{3600^2} = 833 \times 10^4 \]

Try intermediate stiffener 2 flats 100 mm X 12 mm thick
\[ (I_x)_{\text{Provided}} = 1007 \times 10^4 \text{ mm}^4 \]
\[ (I_x)_{\text{Provided}} > I_x \quad \left[ 1007 \times 10^4 > 833 \times 10^4 \right] \]

Hence, OK

**Buckling Check:**

\[ \frac{F_q - F_x}{P_x} + \frac{F_x}{P_x} + \frac{M_y}{M_y} \leq 1 \]
\[ F_q = V - V_s \quad V = 1585.5 \text{ kN} \]
\[ V_s = V_s = \sigma_s \cdot d = 52.8 \times 2600 \times 16 \times 10^{-3} = 2196 \text{ kN} \]
\[ F_q \text{ is negative and } F_q - F_x = 0 \]
\[ M_y = 0 \]
\[ F_x = 870 \text{ kN} \]

**Buckling resistance of load carrying stiffener at D:**

*(Calculation is similar to stiffener at B)*

\[ 20 t_w = 20 \times 16 = 320 \text{ mm} \]
\[ L_s = \frac{1}{12} \times 12 \times 216^3 + \frac{640 \times 16^3}{12} - \frac{12 \times 16^3}{12} = 1029 \times 10^6 \text{ mm}^4 \]
\[ A = 200 \times 12 + 640 \times 16 = 12,640 \text{ mm}^2 \]
\[ r_s = \left[ \frac{1029 \times 10^6}{12640} \right]^{\frac{1}{2}} = 28.5 \]
\[ I_s = 0.7 \times 2600 = 1820 \]
\[ \lambda = \frac{l_s}{r_s} = \frac{1820}{28.5} = 63.9 \]
\[ F_{so} = 250 \text{ N/mm}^2 \text{ and } \lambda = 63.9 \]

From table 3 of chapter on axially compressed columns,

\[ \sigma_s = 180 \text{ N/mm}^2 \]

Buckling resistance, \( P_x (180/1.15) \times 2640 \times 10^{-3} = 1978 \text{ kN} \).

\[ \frac{F_x}{P_x} = \frac{870}{1978} = 0.44 < 1.0 \]
Hence, Stiffener at D is OK against buckling

**Stiffener at D** - Adopt flats 100 mm X 12 mm thick

**Web check between stiffeners:**

\[ f_{ed} \leq P_{ed} \]

\[ f_{ed} = \frac{w_1}{t} = \frac{79.5}{16} = 4.97 \text{ N/mm}^2 \]

When compression flange is restrained against rotation relative to the web

\[ P_{ed} = \left[ 2.75 + \frac{2}{(a/d)^2} \right] \frac{E}{(d/t)^2} = \left[ 2.75 + \frac{2}{(3600/2600)^2} \right] \frac{200000}{2600^2} \]

\[ = \frac{3.79 \times 20000}{26406} = 28.7 \text{ N/mm}^2 \]

Since,

\[ f_{ed} \leq P_{ed} \]

[4.97 <28.7], the web is OK for all panels.
8.0 FINAL GIRDER

(All dimensions are in mm)

(a) Longitudinal section of plate girder

(b) Flange and web proportions

Fig. E5 Final girder