Design of plate girders :

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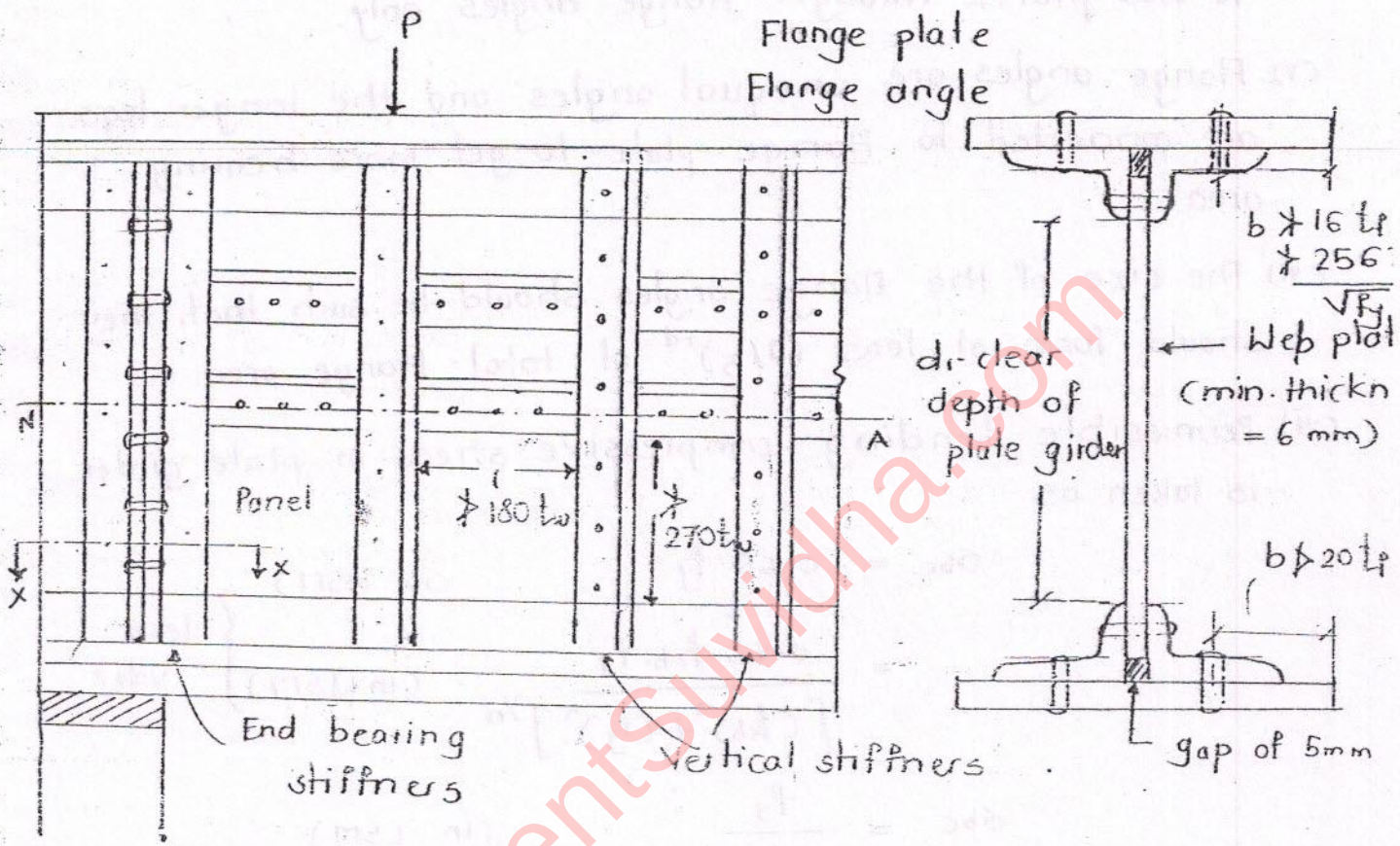
(i) If built up beams cannot withstand applied loads, then plate girders are used. Plate girders consist of flange plates, flange angle, web plates.



(ii) Compression flange consist of flange plate, flange angle and web equivolent. Web equivalent is the web area imbedded between two flange angles. In compression zone, web equivalent is taken as ($\frac{1}{6}$ × Area of web, A_w)

(iii) Tension flange consist of flange plate, flange angle and web equivalent. In tension zone web equivalent is taken as ($\frac{1}{8}$ × Area of web, A_w)

(less area is taken to take care of rivet holes in the web)

(iv) It is assumed that entire shear force is taken by web plate and bending moment is taken by flanges. (To ensure that web takes only shear force, a gap of 5mm

will be maintained between flange plate and web plate. So that direct bearing action between flange plate and web plate is avoided.

The load is transfered from flange plates to web plates through flange angles only.

(v) Flange angles are unequal angles and the longer legs are connected to flange plate to get more bearing area.

(vi) The size of the flange angles should be such that, they should form at least $(1/3)^{rd}$ of total flange area.

(vii) Permissible bending compressive stress in plate girder is taken as

$$\sigma_{bc} = 0.66 f_y \quad \text{(in WSM)}$$

$$= \frac{0.66 f_{cb} \cdot f_y}{[(f_{cb})^n + (f_y)^n]^{1/n}} \quad \text{(in WSM)}$$

} less value

$$\sigma_{bc} = \frac{f_y}{1.1} \quad \text{(in LSM)}$$

(viii) Width of outstand in compression flange.

$$b \not> 16\, t_f$$

$$\not> \frac{256\, t_f}{\sqrt{f_y}}$$

(ix) The width of outstand in tension flange:

$$b \not> 20\, t_f$$

(If it exceeds, $20\, t_f$, outstand may bend due to self weight of overhanged portion)

(x) In plate girders for resisting shear, depth of web is taken as, depth of web plate only (not the overall depth of plate girder because there is disconnection between flange plate and web plate.)

$\tau_{va,cal}$ - calculated avg. shear stress in web plate.

$$\frac{V}{d_w \times t_w} \leq \tau_{va} = 0.4 f_y \quad \text{(in WSM)}$$

$$= \frac{f_y}{\sqrt{3} \times 1.1} \quad \text{(in LSM)}$$

d_w - depth of web plate only.

(xi) Economical depth of web plate (which corresponds to minimum weight but not minimum cost) is given by

$$d_e = 1.1 \sqrt{\frac{M}{\sigma_{bc} \times t_w}}$$

where, t_w - thickness of web (assume, ≤ 6 mm).

(xii) Self wt. of plate girder is assumed as

$$w = \frac{W}{300} \text{ kN/m}$$

where, W - total load on plate girder.

(xiii) Clear depth of plate girder is given as.

d_1 = distance between toes of flange angles.

(xiv) If $\frac{d_1}{t_w} < 85$, web buckling due to shear will not happe so vertical stiffners are not provided.

(xv) If $\frac{d_1}{t_w} > 85$, vertical stiffners are provided to prevent buckling of web due to diagonal compression which is developed due to shear force.

(xvi) Under point load vertical stiffners are provided to prevent load buckling of web or web crippling. These stiffners are called load bearing stiffners.

(xvii) If $\frac{d_1}{t_w} > 200$, the horizontal stiffners are provided above N.A. (they prevent buckling of web due to bending compressive stress)

(xviii) If $\frac{d_1}{t_w} > 250$ then additional horizontal stiffner is provided at N.A. (This stiffner provides buckling of web between vertical stiffness due to shear force)

(xix) If $\frac{d_1}{t_w} > 400$, then section must be re-designed.

(xx) At the supports, to prevent bending of flange plate and buckling of web plate, due to support reaction, end bearing stiffners are provided.

(xxi) After providing all the stiffners, lesser clear dimension of web panel should not exceed $(180 \times t_w)$. and greater clear dimension should not exceed $(270 \times t_w)$

Design of end bearing stiffners: