INTRODUCTION

When the need for a new structure arises, an individual or agency has to arrange the funds required for its construction. The individual or agency henceforth referred to as the owner then approaches an architect. The architect plans the layout so as to satisfy the functional requirements and also ensures that the structure is aesthetically pleasing and economically feasible. In this process, the architect often decides the material and type of construction as well. The plan is then given to a structural engineer who is expected to do locate the structural elements so as to cause least interference to the function and aesthetics of the structure. He then makes the strength calculations to ensure safety and serviceability of the structure. This process is known as structural design. Finally, the structural elements are fabricated and erected by the contractor. If all the people work as a team then a safe, useful, aesthetic and economical structure is conceived. However in practice, many structures fulfill the requirements only partially because of inadequate coordination between the people involved and their lack of knowledge of the capabilities and limitations of their own and that of others. Since a structural engineer is central to this team, it is necessary for him to have adequate knowledge of the architects and contractors work. It is his responsibility to advise both the architect and the contractor about the possibilities of achieving good structures with economy. Ever since steel began to be used in the construction of structures, it has made possible some of the grandest structures both in the past and also in the present day (Fig. 1.1). In the following paragraph, some of the aspects of steel structures, which every structural engineer should know, are briefly discussed.

Steel is by far the most useful material for building structures with strength of approximately ten times that of concrete, steel is the ideal material for modern construction. Due to its large strength to weight ratio, steel structures tend to be more economical than concrete structures for tall buildings and large span buildings and bridges. Steel structures can be constructed very fast and this enables the structure to be used early thereby leading to overall economy. Steel structures are ductile and robust and can withstand severe loadings such as earthquakes. Steel structures can be easily repaired and retrofitted to carry higher loads. Steel is also a very eco-friendly material and steel structures can be easily dismantled and sold as scrap. Thus the lifecycle cost of steel structures, which includes the cost of construction, maintenance, repair and dismantling, can be less than that for concrete structures. Since steel is produced in the factory under better quality control, steel structures have higher reliability and safety. To get the most benefit out of steel, steel structures should be designed and protected to resist corrosion and fire. They should be designed and detailed for easy fabrication and erection.
Good quality control is essential to ensure proper fitting of the various structural elements. The effects of temperature should be considered in design. To prevent development of cracks under fatigue and earthquake loads the connections and in particular the welds should be designed and detailed properly. Special steels and protective measures for corrosion and fire are available and the designer should be familiar with the options available.

A structural member subjected to transverse loads (Loads perpendicular to its longitudinal axis) is called a beam. See Figure ABOVE

Beams are most critical members in any structure. Their design should therefore not only be economical but also safe. The main considerations in the design of beams are:

1. They should be proportioned for strength in bending keeping in view the lateral and local stability of the compression flange and the capacity of the selected shape to develop the necessary strength in shear and local bearing.
2. They should be proportioned for stiffness, keeping in mind their deflections and deformations under service conditions.

3. They should be proportioned for economy, paying attention to the size and grade of steel to yield the most economical design.

Beam design consists merely of the provision of adequate bending and shear resistance. For optimum bending resistance, as much of the beams material as possible should be displaced as far as practicable from the neutral axis. The web area should be sufficient to resist shear.

Maximum moment and maximum shear usually occur at different sections. Though simple in design, the lateral buckling of beam as a whole, or of its compression flange or its web pose complications. Another problem is of proper depth – an increase in depth may be desirable for moment resistance, it may at the same impair resistance to lateral or web buckling (Figure).
Lateral-Torsional Buckling

- Instability due to buckling of compression half of the beam (perpendicular to the plane of loading).

Figure 2. Response of a slender cantilever beam to vertical loading: lateral-torsional buckling

Local (Plate) buckling of Web

Figure 2. Vertical web buckling.
Web buckling

Types of Beams:

Beams are generally classified according to their geometry and the manner in which they are supported. They may be straight or curved.
Figure: Curved Beam

**Girders** usually the most important beams which are frequently at wide spacing.

**Joists** usually less important beams which are closely spaced, frequently with truss type webs.

**Stringers** - Longitudinal beams spanning between floor beams.

**Purlins** - Roof beams spanning between trusses

**Girts** - Horizontal wall beams serving principally to resist bending due to wind on the side of an industrial building.

**Lintels** - Members supporting a wall over window or door openings.
Sections Used As Beams

- Figure 4. W Section as a Beam

Bending Stresses

- Bending moment produces bending strains on a beam, and consequently compressive and tensile stresses.

- Under positive moment (as normally the case), compressive stresses are produced in the top of the beam and tensile stresses are produced in the bottom.

- Bending members must resist both compressive and tensile stresses.
Stresses in Beams

- For introduction to bending stress the rectangular beam and stress diagrams of Fig. 5 are considered.
- If the beam is subjected to some bending moment that stress at any point may be computed with the usual flexure formula:

$$f_b = \frac{Mc}{I}$$

Figure 5. Variation in Bending Stresses
It is important to remember that the expression given by Eq. 1 is only applicable when the maximum computed stress in the beam is below the elastic limit.

The formula of Eq. 1 is based on the assumption that the stress is proportional to the strain, and a plane section before bending remains plane after bending.

The value of \( I/c \) is a constant for a particular section and is known as the section modulus \( S \).

The flexure formula may then be written as follows:

\[
\sigma = \frac{M}{S}
\]

(2)
• Plastic Moment

– In reference to Fig. 5:

  • Stress varies linearly from the neutral axis to extreme fibers, as shown in Fig. 5b.
  • When the moment increases, there will also be a linear relationship between the moment and the stress until the stress reaches the yield stress $F_y$, as shown in Fig. 5c.
  • In Fig. 5d, when the moment increases beyond the yield moment, the outermost fibers that had previously stressed to their yield point will continue to have the same but will yield.

• The process will continue with more and more parts of the beam cross section stressed to the yield point as shown by the stress diagrams of parts (d) and (e) of Fig. 5., until finally a full plastic distribution is approached as shown in SKYUP'S MEDIA.
Plastic Moment

- Definition

“The plastic moment can be defined as the moment that will produce full plasticity in a member cross section and create a plastic hinge”.

Shape Factor

- Definition

“The shape factor of a member cross section can be defined as the ratio of the plastic moment $M_p$ to yield moment $M_y$”.

- The shape factor equals 1.50 for rectangular cross sections and varies from about 1.10 to 1.20 for standard rolled-beam sections.
Plastic Hinges

The Concept of Plastic Hinge

- The plastic hinge concept is illustrated as shown in the simple beam of Fig. 6.
- The load shown in the figure is applied to the beam and increased in magnitude until the yield moment is reached and the outermost fiber is stressed to the yield stress.
- The magnitude of the load is further increased with the result that the outer fibers begin to yield.

Figure 6. Plastic Hinge
– The yielding spreads out to other fibers away from the section of maximum moment as indicated in Fig. 6.

– The length in which this yielding occurs away from the section in question is dependent on the loading conditions and the member cross section.

– For a concentrated load $P_c$ applied at the center line of a simply-supported beam with a rectangular cross section, yielding in extreme fibers at the time the plastic hinge is formed will extend for one-third of the span.

– For a W section in similar circumstances, yielding will extend for approximately one-eighth of the span.

– During the same period, the interior fibers at the section of maximum moment yield gradually until nearly all of them have yielded and a plastic hinge is formed.

– The effect of the plastic hinge is assumed to be concentrated at one section for analysis purposes.

– However, it should be noted that this effect may extend for some distance along the beam.

– For the calculation of deflection and for the design of bracing, the length over which yielding extends is very important.
Design of eccentric connection, framed, stiffened and seat connection.

1. Determine the safe load $P$ that can be carried by the joint shown in Figure. The bolts used are 20 mm diameter of grade 4.6. The thickness of the Flange of I-section is 9.1 mm and that of bracket plate 10 mm. [5 Marks]

**Solution:**

For Fe 410 grade of steel: $f_u = 410$ MPa

For bolts of grade 4.6: $f_{ub} = 400$ MPa

Partial safety factor for the material of bolt: $\gamma_{mb} = 1.25$

$A_{nb} =$ stress area of 20 mm diameter bolt $= 0.78 \times \pi \times 20^2/4 = 245$ mm$^2$

**Given:** diameter of bolt, $d = 20$ mm; pitch, $p = 80$ mm; edge distance, $e = 40$ mm (2 x 20 mm), $d_0 = 20 + 2 = 22$ mm.

Strength of bolt in single shear, $V_{dsb} = A_{nb} \frac{f_{ub}}{1.732 \times 1.25}$
Strength of bolt in bearing, \( V_{dpb} = 2.5 k_b \ dt \ \frac{f_u}{\gamma_{mb}} \)

\( K_b \) is least of \( \frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.606; \ \frac{p}{3d_0} - 0.25 = \frac{80}{3 \times 22} - 0.25 = 0.96 \)

\( \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975; \) and 1.0

Hence \( K_b = 0.606 \)

\( V_{dpb} = 2.5 k_b \ dt \ \frac{f_u}{\gamma_{mb}} = 2.5 \times 0.606 \times 20 \times 9.1 \times \frac{410}{1.25} \times 10^{-3} = 90.44 \text{kN} \)

Hence strength of bolt is \( V_{sd} = 45.26 \text{kN} \)

Let, \( P_1 \) be the factored load.

Service load, \( P = \frac{P_1}{\text{load factor}} = \frac{P_1}{1.50} \)

The bolt which is stressed maximum at A (see Figure)

Total number of bolts in the joint, \( n = 10 \)

The force direct force, \( F_1 = \frac{P_1}{n} = \frac{P_1}{10} \)

\[ The \ force \ in \ the \ bolt \ due \ to \ torque, \ F_2 = \frac{P_0r_n}{\Sigma r^2} \]

\[ r_n = \sqrt{(80 + 80)^2 + \left(\frac{120}{2}\right)^2} = 170.88 \text{mm} \]

\[ \Sigma r^2 = 4 \times [(160^2 + 60^2) + (80^2 + 60^2)] + 2 \times 60^2 = 164,000 \text{mm}^2 \]

\[ F_2 = (P_1 \times 200 \times 170.88)/(164,000) = 0.20839P_1 \]
\[ \cos \theta = \frac{60}{\sqrt{60^2 + 160^2}} = 0.3511 \]

The resultant force on the bolt should be less than or equal to the strength of bolt.

\[ 45.26 \leq \sqrt{\left(\frac{P_1}{10}\right)^2 + 0.20839P_1} + 2 \times \frac{P_1}{10} \times 0.20839P_1 \times 0.3511 \]

Implies \( P_1 = 173.49 \text{ kN} \)

The service load, \( P = \frac{P_1}{load \ factor} = \frac{173.49}{1.5} = 115.65 \text{ kN} \)

2. Design a bracket connection to transfer an end reaction of 225 kN due to factored loads as in Figure below. The end reaction from the girder acts at an eccentricity of 300 mm from the face of the column flange. Design bolted joint connecting the Tee-flange with the column flange. Steel is of grade Fe 410 and bolts of grade 4.6. [5 Marks] [Page No. 721, S.K. Duggal 2nd edition]
**Solution:** For Fe 410 grade of steel: $f_u = 410$ Mpa

For bolts of grade 4.6: $f_{ub} = 400$ MPa

Partial safety factor for the material bolt: $\gamma_{mb} = 1.25$

The bolts along section AA are subjected to

(i) Shear due to the load, $P = 225\text{kN}$ passing through the c.g. of the joint
(ii) Tension due to bending moment, $M = 225 \times 300 = 67,500\text{kNmm}$

Let us provide 24 mm diameter bolts for making the connection.

For 24 mm diameter bolts

Stress area, $A_{nb} = 353\text{ mm}^2 \left[\left(\pi \times 24^2 \times 0.78\right)/4\right]$

Minimum pitch, $p = 2.5 \times 24 = 60\text{ mm} \approx 65\text{ mm}$

Edge distance = $1.5 \times (24+2) = 39\text{ mm} \approx 40\text{ mm}$

Strength of the bolt in single shear,

$$V_{dsb} = V_{sd} = A_{nb} \frac{f_{ub}}{1.732 \times \gamma_{mb}} = 353 \times \frac{400}{1.732 \times 1.25} \times 10^{-3}$$

$$= 65.22\text{ kN}$$

Strength of bolts in tension $T_{db} = T_{nb}/\gamma_{mb}$

$$T_{nb} = 0.9 \times f_{ub} \times A_{nb} = 0.9 \times 400 \times 353 \times 10^{-3} = 127.08\text{ kN}$$

$$f_{yb} \frac{\gamma_{mb}}{\gamma_{mo}} A_{sb} = 250 \times \frac{125}{1.10} \times 452 \times 10^{-3} = 128.40\text{ kN}$$

Hence, $T_{db} = T_{nb}/\gamma_{mb} = 127.08 / 1.25 = 101.66\text{ kN}$

The bolts will be provided in two vertical rows, one on each side of the web of the Tee section, connecting the flanges of the two sections.
Number of bolts required in one row, \( n = \frac{6M}{\sqrt{pn'V_{sd}}} = \sqrt{\frac{6 \times 67,500}{2 \times 65 \times 65.22}} = 6.91 \approx 7 \)

Hence provide 7 bolts in each row at a pitch of 65 mm and edge distance of 40 mm.

Total depth of the bracket plate = \( 6 \times 65 + 2 \times 40 = 470 \) mm

\( h = 470 - 40 = 430 \) mm

The neutral axis is assumed to lie at \( h/7 \) from the bottom of the bracket, i.e., at \( 430/7 = 61.42 \) mm

\[
\Sigma y_i = 2 \times [(65 + 40 - 61.42) + (130 + 40 + 61.42) + (195 + 40 - 61.42) + (260 + 40 - 61.42) + 325 + 40 - 61.42] + (390 + 40 - 61.42)\]

\[
= 2472.96 \text{ mm}
\]

\[
\Sigma y_i^2 = 2 \times [43.58^2 + 108.58^2 + 173.58^2 + 238.58^2 + 303.58^2 + 368.58^2] = 657,502.6 \text{ mm}^2
\]

\[
M' = \frac{M}{1 + \frac{2h \Sigma y_i}{21 \Sigma y_i^2}} = \frac{67.5 \times 10^3}{2 \times 430 \times \frac{2472.96}{657502.6}} = 58.49 \times 10^3 \text{ kNm}
\]

Tensile force in the critical bolt, \( T_b = \frac{M' \times y_n}{\Sigma y_i^2} = \frac{58.49 \times 10^3 \times 368.58}{657502.6} = 32.79 \text{ kN} \) (\( y_n = 368.58 \text{ mm} \))

Shear force in the critical bolt, \[
V_{sb} = \frac{p}{\text{number of bolts}} = \frac{225}{2 \times 7} = 16.07 \text{ kN}
\]

Check
\[ \left( \frac{V_{sb}}{V_{dsb}} \right)^2 + \left( \frac{T_b}{T_{db}} \right)^2 \leq 1.0 \]

\[ \left( \frac{16.07}{65.22} \right)^2 + \left( \frac{32.79}{101.66} \right)^2 = 0.1647 \leq 1.0 \]

Which is as it should be.

3. An ISLB 300 @ 369.8 N/m transmits an end reaction of 385 kN, under factored loads, to the web of ISMB 450 @ 710.2 N/m. Design a bolted framed connection. Steel is of grade Fe410 and bolts are of grade 4.6. [5 Marks] (L.Date 25/03/2015)

**Solution:** For Fe 410 grade of steel: \( f_u = 410 \) MPa, \( f_y = 250 \) MPa

For bolts of grade 4.6: \( f_{ub} = 400 \) MPa

Partial safety factor for material of bolt, \( \gamma_{mb} = 1.25 \)

Partial safety factor for material, \( \gamma_{m0} = 1.10 \)

The relevant properties of the sections from Steel Tables are:

<table>
<thead>
<tr>
<th>Property</th>
<th>ISLB 300</th>
<th>ISMB 450</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of section, ( h )</td>
<td>300 mm</td>
<td>450 mm</td>
</tr>
<tr>
<td>Width of flange, ( b_f )</td>
<td>150 mm</td>
<td>150 mm</td>
</tr>
<tr>
<td>Thickness of flange, ( t_f )</td>
<td>9.4 mm</td>
<td>17.4 mm</td>
</tr>
<tr>
<td>Thickness of web, ( t_w )</td>
<td>6.7 mm</td>
<td>9.4 mm</td>
</tr>
<tr>
<td>Gauge, ( g )</td>
<td>90 mm</td>
<td>90 mm</td>
</tr>
</tbody>
</table>

Note down this Figure.
Connection of web of ISLB 300 with framing angle leg

Let us provide 24 mm diameter bolts. The bolts will be in double shear. Assuming the aggregate thickness of the angle legs to be more than the thickness of web, the bolts will bear on the web of ISLB 300.

For 24 mm diameter bolt,

Stress area, $A_{nb} = 353 \text{ mm}^2$

Minimum pitch, $p = 2.5 \times 24 = 60 \text{ mm}$ == 65 mm

Edge distance, $e = 39 \text{ mm}$ == 40 mm

Diameter of bolt hole, $d_0 = 24 + 2 = 26 \text{ mm}$

Strength of bolt in double shear,

\[ V_{dsb} = 2 \times A_{nb} \frac{f_{ub}}{\sqrt{3} \gamma_{mb}} \]
\[ = 2 \times 353 \times \frac{400}{1.732 \times 1.25} \times 10^{-3} = 130.43 \text{ kN} \]

Strength of bolt in bearing,

\[ V_{dpb} = 2.5 K_b d t \frac{f_u}{\gamma_{mb}} \]

$K_b$ is least of
\[ \frac{e}{3d_0} = \frac{40}{3 \times 26} = 0.513; \quad \frac{p}{3d_0} = \frac{65}{3 \times 26} = 0.25 \]

\[ = 0.58 \]
\[
\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975; \text{ and } 1.0 \\
\]
Hence \(K_b = 0.513\)

\[
V_{dpb} = 2.5 \times 0.513 \times 24 \times 6.7 \times (410/1.25) \times 10^{-3} = 67.64 \text{ kN} \\
\]
Hence strength of the bolt = 67.64 kN

Number of bolts required for making the connection, \(n = 385 / 67.64 = 5.69 \approx 6\)

Two framing angles, one on each side of the web will be provided. Provide the bolts in two vertical rows.

Minimum size of angle leg = 2 x 40 + 65 = 145 mm \(\approx 150\) mm

\textit{Connection of web of ISMB 450 with framing angle leg}

Let us provide 24 mm diameter bolts. The bolts will be in single shear. Assuming the thickness of angle leg to be more than the thickness of web of ISMB450, the bolts will bear on web of ISMB 450.

Strength of bolt in single shear,

\[
V_{dsb} = A_{nb} \frac{f_{ub}}{1.732 \times \gamma_{mb}} = 353 \times \frac{400}{1.732 \times 1.25} \times 10^{-3} = 65.22 \text{ kN} \\
\]

Strength of bolt in bearing,

\[
V_{dpb} = 2.5 \ K_b \ dt \frac{f_u}{\gamma_{mb}} = 2.5 \times 0.513 \times 24 \times 9.4 \times (410/1.25) \times 10^{-3} = 94.90 \text{ kN} \\
\]
Hence, strength of the bolt = 65.22 kN

Number of bolts for making the connection, \(n = 385 / 65.22 = 5.90 \approx 6\)
Provide 3 bolts each on the legs of the two framing angles as shown in the above Figure.

Minimum size of angle leg = 2 x 40 = 80 mm ≈ 115 mm

Minimum depth of framing angle leg, h = 2 x 40 + 2x65 = 210 mm

The thickness of the angle section can be determined by equating the shear force (end reaction) to the shear capacity of the angle leg.

\[ V = V_d = \frac{f_y}{1.732\gamma_m} h(2t_w) \] (Since there are two angles)

\[ 385 \times 10^3 = \frac{250}{1.732\times1.10} \times 210 \times (2 \times t_w) \]

\[ \Rightarrow t_w = 6.98 \text{ mm} \approx 10 \text{ mm} \]

Hence, provide 2 framing angles 150 x 115 x 10 mm in size.

4. Design a stiffened seat connection for an ISMB 350 @ 514 N/m transmitting an end reaction of 320 kN (due to factored loads) to a column section ISHB 300 @ 576.8 N/m. The steel is of grade Fe 410 and bolts of grade 4.6. [5 Marks]

Solution:
For Fe 410 grade of steel: \( f_u = 410 \text{ Mpa}, f_{yw} = 250 \text{ Mpa} \)
For bolts of grade 4.6: \( f_{ub} = 400 \text{ Mpa} \)
Partial safety factor for material of bolt: \( \gamma_{mb} = 1.25 \)
Partial safety factor for material: \( \gamma_{m0} = 1.10 \)

Yield stress ratio, \( \varepsilon = \sqrt{\frac{250}{250}} = \sqrt{1} = 1.0 \)

The relevant properties of the sections to be connected from steel tables are:
<table>
<thead>
<tr>
<th>Property</th>
<th>ISMB 350</th>
<th>ISHB 300</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of flange, $b_f$</td>
<td>140 mm</td>
<td>250 mm</td>
</tr>
<tr>
<td>Thickness of flange, $t_f$</td>
<td>14.2 mm</td>
<td>10.6 mm</td>
</tr>
<tr>
<td>Thickness of web, $t_w$</td>
<td>8.1 mm</td>
<td>7.6 mm</td>
</tr>
<tr>
<td>Gauge, $g$</td>
<td>80 mm</td>
<td></td>
</tr>
<tr>
<td>Radius at the root, $R_1$</td>
<td>14 mm</td>
<td></td>
</tr>
</tbody>
</table>

The length of seat angle, $B = \text{width of beam flange} = 140 \text{ mm}$, ($b_f = 140 \text{ mm}$)

Bearing length of seat leg, $b = \frac{R}{t_w} \times \frac{\gamma_m}{f_{yw}} = \frac{320 \times 10^3}{8.1} \times \frac{1.10}{250} = 173.82 \text{ mm}$.

Provide a clearance $c$ of 5 mm between the beam and the column flange.

Required length of outstanding leg $= 173.82 + 5 = 178.82 \text{ mm} \approx 200 \text{ mm}$

Let us provide seat angle 200 x 150 x 10 mm with seat leg of 200 mm connected to the flange of beam with 2, 24 mm diameter bolts of grade 4.6

Radius at root of angle, $Ra = 13.5 \text{ mm}$ (From steel Tables)
**Stiffener angles**

Bearing area required by stiffener angles,

\[ A = R \frac{\gamma_{m0}}{f_y} = 320 \times 10^3 \times (1.1/250) = 1408 \text{ mm}^2 \]

Let us provide two angles ISA 90 x 60 x 8 mm

Area provided by the stiffening legs of the angles = 2 x (90 x 8) = 1440 mm\(^2\)

Length of outstanding leg = 90 – 8 = 82 mm

Thickness of the angle, \( t_a = 8 \text{ mm} \) should be more than \( t_w \) i.e. (8.1 mm)

Since the thickness is almost same and stiffener angle section may be used.

Length of outstand of stiffener \( \geq 14 \, t_a \, \varepsilon \), i.e., 14 x 8 x 1 = 112 mm (\( \varepsilon = 1 \))

which is as it should be.

Distance of end reaction from column flange, \( e_x = 200/2 = 100 \text{ mm} \)

Stiffener angles provide some rigidity to the seat angle and the reaction is assumed to act at the middle of the seat leg. Thus, the eccentricity is increased.

**Design of connections**

Let us provide 24 mm diameter bolts of grade 4.6, at a pitch of 60 mm. The bolts connecting the legs of stiffener angles with column flange will be in single shear and bearing.

For 24 mm diameter bolt, \( A_{nb} = 353 \text{ mm}^2 \)

Minimum pitch, \( p = 2.5 \times 24 = 60 \text{ mm} \)

Edge distance, \( e = 39 \text{ mm} \approx 40 \text{ mm} \)

Diameter of bolt hole, \( d_0 = 24 + 2 = 26 \text{ mm} \)

Strength of the bolt in single shear,
\[ V_{dsb} = A_{nb} \frac{f_{ub}}{1.732 \times \gamma_{mb}} = 353 \times \frac{400}{1.732 \times 1.25} \times 10^{-3} = 65.22 \text{ kN} \]

Strength of bolt in bearing,
\[ V_{dpb} = 2.5 \text{ kb dt} \frac{f_{u}}{\gamma_{mb}} \]

\[ K_b \text{ is least of } \frac{e}{3d_o} = \frac{40}{3 \times 26} = 0.513; \quad \frac{p}{3d_o} = 0.25 = \frac{60}{3 \times 26} - 0.25 = 0.519 \]

\[ \frac{f_{ub}}{f_{u}} = \frac{400}{410} = 0.975; \text{ and } 1.0 \]

Hence \( K_b = 0.513 \)
\[ V_{dpb} = 2.5 \times 0.513 \times 24 \times 8 \times (410/1.25) \times 10^{-3} = 80.76 \text{ kN} \]

Hence strength of the bolt = 65.22 kN

There will be two vertical rows of bolts connecting legs of the two stiffener angles with the column flange.

Number of bolts in one row,
\[ n = \sqrt{\frac{6M}{p \times n \times V_{sd}}} = \sqrt{\frac{6 \times 320 \times 10^3 \times 100}{60 \times 2 \times 65.22 \times 10^3}} = 4.95 \approx 5 \]

The depth of stiffener angle = 4 x 60 + 2 x 40 = 320 mm

\[ H = 320 - 40 = 280 \text{ mm} \]

\[ h/7 = 280/7 = 40 \text{ mm (refer Figure above)} \]

The critical bolt will be A.

\[ \Sigma y_i = 2 \times [0 + 60 + 120 + 180 + 240] = 1200 \text{ mm} \]

\[ \Sigma y_i^2 = 2 \times [0 + 60^2 + 120^2 + 180^2 + 240^2] = 216000 \text{ mm}^2 \]
Moment shared by the critical bolt,

\[ M' = \frac{M}{1 + \frac{2h}{21} \sum \gamma_i^2} = 27.87 \times 10^3 \text{ kNmm} \]

Tensile force in the critical bolt,

\[ T_b = \frac{M' y_n}{\sum \gamma_i^2} = 30.96 \text{ kN} \quad (y_n = 240 \text{ mm}) \]

Shear force in the critical bolt,

\[ V_{sb} = \frac{P}{\text{number of bolts}} = 32 \text{ kN} \]

**Check**

\[ \left( \frac{V_{sb}}{V_{dsb}} \right)^2 + \left( \frac{T_b}{T_{db}} \right)^2 \leq 1.0 \]

Strength of bolt in tension, \( T_{db} = T_{nb}/\gamma_{mb} \)

\[ T_{nb} = 0.9 f_{ub} A_{nb} = 127.08 \text{ kN} \]

\[ \Rightarrow f_{yb} \frac{\gamma_{mb}}{\gamma_{m0}} A_{sb} = 128.40 \text{ kN} \]

Hence, \( T_{nb} = 127.08 \text{ kN} \)

and, \( T_{db} = T_{nb}/\gamma_{mb} = 101.66 \text{ kN} \)

\[ \left( \frac{32}{65.22} \right)^2 + \left( \frac{30.96}{101.66} \right)^2 = 0.333 \leq 1.0 \]

Which is it should be.
25. When the seated beam connections are preferred and name the types?
**Answer:** When a beam is connected to the flange (or the web) of a steel stanchion, the width of the flange (or the depth of the web) may be insufficient to accommodate the connecting angles, in such cases framed connection is not suitable and seated connection is preferred.

27. What is stiffened seat connection?
**Answer:** In addition to the seat angle, a web cleat is provided when the beam is connected to a beam and a flange cleat is used when the beam is connected to a stanchion. The angle cleats are essential because they keep the beam stable in a vertical position and prevent it from lateral buckling.

In the stiffened seat connection, a T-section built-up of two plates is used. (Pag698)

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**Design of Plate Girders**

1. Design a welded plate girder 24 m in span and laterally restrained throughout. It has to support a uniform load of 100 kN/m throughout the span exclusive of self-weight. Design the girder without intermediate transverse stiffeners. The steel for the flange and web plates is of grade Fe 410. Yield stress of steel may be assumed to be 250 MPa
irrespective of the thickness of plates used. Design the cross section, the end load bearing stiffener and connections.

Solution:

For Fe 410 grade of steel: \( f_u = 410 \text{ MPa}, f_y = f_{yp} = f_{yw} = 250 \text{ MPa} \)

\[ \mu = 0.3 \]

\[ E = 2 \times 10^5 \text{ MPa} \]

Partial safety factors, \( \gamma_{mw} = 1.50 \) (for site welding)

\[ = 1.25 \) (For shop welding)

\[ \varepsilon = \varepsilon_w = \varepsilon_f = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0 \]

Design Forces

Total superimposed load = 100 kN/m

Factored superimposed load = 1.5 x 100 = 150 kN/m

Let, self-weight of plate girder = \( \frac{WL}{400} = \frac{(100 \times 24) \times 24}{400} \)

= 144 kN

Self-weight of plate-girder per meter length = \( \frac{144}{24} \) = 6 kN/m

Factored self weight = 1.5 x 6 = 9 kN/m

Total uniform factored load = 150 + 9 = 159 kN/m

Maximum bending moment = \( \frac{159 \times 24^2}{8} \) = 11,448 kNm

Maximum shear force = \( \frac{159 \times 24}{2} \) = 1908 kN

Design of web
Optimum depth of plate girder, \( d = \left( \frac{M_z k}{f_y} \right)^{0.33} \)

When intermediate transverse stiffeners are not to be provided;

\( d/t_w \leq 200 \varepsilon \) i.e., 200 (from serviceability criteria)

\( \leq 345 \varepsilon_f^2 \) i.e., 345 (from flange buckling criteria)

and

Let us assume \( k = d/t_w = 180 \)

Optimum depth of plate girder, \( d = \left( \frac{M_z k}{f_y} \right)^{0.33} = \left( \frac{11448 \times 10^6 \times 180}{250} \right)^{0.33} \)

= 1871.9 mm \( \approx \) 1800 mm

Optimum web thickness, \( t_w = \left( \frac{M_z}{f_y k^2} \right)^{0.33} = \left( \frac{11448 \times 10^6}{250 \times 180^2} \right) = 10.95 \) mm \( \approx \) 12 mm

(Thickness provided is more since intermediate transverse stiffeners are not to be provided)

Let us try web plate 1800 x 12 mm in size.

**Design of Flanges**
Let us assume that bending moment will be resisted by the flanges and shear by the web.

Required area of Flange, \( A_f = \frac{M_z Y_m}{f_y d} = \frac{11448 \times 10^6 \times 1.10}{250 \times 1800} = 27984 \text{ mm}^2 \)

Assuming width of flange equal to 0.3 times depth of girder, \( b_f = 0.3 \times 1800 = 540 \text{ mm} \approx 560 \text{ mm} \)

Thickness of flange, \( t_f = \frac{27984}{560} = 49.97 \approx 50 \text{ mm} \)

**Classification of flanges**

For the flanges to be classifiable as plastic \( b/t_f \leq 8.4\varepsilon \) (\( \varepsilon \) is yield stress ratio)

The outstand of flange, \( b = \frac{b_f - t_w}{2} = \frac{560 - 12}{2} = 274 \text{ mm} \)

\[
\frac{b}{t_f} = \frac{274}{50} = 5.48
\]

\( < 8.4 \) (\( 8.4\varepsilon = 8.4 \times 1 = 8.4 \))

Hence, the flanges are plastic. (\( \beta_b = 1.0 \))

**Check for bending strength**

The trial section of the plate girder is shown in Figure 1. The plastic section modulus of the section,
\[ Z_{pz} = 2 \, b_f \, t_f \, \frac{(D-t_f)}{2} = 2 \times 560 \times 50 \times \frac{1900-50}{2} = 51.80 \times 10^6 \text{mm}^3 \]

Moment Capacity,
\[ M_d = \beta_b Z_{pz} \frac{f_y}{\gamma_m} = 1.0 \times 51.80 \times 10^6 \times \frac{250}{1.10} \times 10^{-6} = 11772.7 \text{kNm} \]
\[ > 11448 \text{kNm} \]
which is safe.

Shear capacity of web
Let us use simple post-critical method.
\[ \frac{d}{t_w} = \frac{1800}{12} = 150 \]
\[ < 200 \quad (200\varepsilon = 200 \times 1 = 200) \]
and also <345
\[ (345\varepsilon^2 = 345 \times 1 = 345) \]
which is all right.

Elastic critical shear stress,
\[ \tau_{cr,e} = \frac{k_v\pi^2 E}{12(1-\mu^2)(\frac{d}{t_w})^2} \]
Transverse Stiffeners will be provided at supports only. Hence, $K_v = 5.35$

$$\tau_{cr,e} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)(150)^2} = 42.98 \text{ N/mm}^2$$

The non-dimensional web slenderness ratio for shear buckling stress,

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3} \times \tau_{cr,e}}} = \sqrt{\frac{250}{\sqrt{3} \times 42.98}} = 1.83 \approx 1.80$$

$>1.20$

Shear stress corresponding to buckling (For $\lambda_w > 1.20$),

$$\tau_b = \frac{f_{yw}}{\sqrt{3} \times \lambda_w^2} = \frac{250}{\sqrt{3} \times 1.80^2} = 44.55 \text{ N/mm}^2$$

Shear force corresponding to web buckling,

$$V_{cr} = dt_w \tau_b = 1800 \times 12 \times 44.55 \times 10^{-3} = 962.28 \text{ kN} < 1908 \text{ kN}$$

Which is unsafe.

Let us revise the web thickness from 12 mm to 16 mm.

New values of $\tau_{cr,e}$, $\lambda_w$, $\tau_b$, and $V_{cr}$ will be as follows.

$$d \quad t_w = \frac{1800}{16} = 112.5$$

$$\tau_{cr,e} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)(1)^2} = 76.41 \text{ N/mm}^2$$
\[
\lambda_w = \frac{f_{yw}}{\sqrt{3} \times \tau_{cr,e}} = \frac{250}{\sqrt{3} \times 76.41} = 1.374 = 1.37 > 1.2
\]

\[
\tau_b = \frac{f_{yw}}{\sqrt{3} \times \lambda_w^2} = \frac{250}{\sqrt{3} \times 1.37^2} = 76.90 \text{ N/mm}^2
\]

\[V_{cr} = dt_w \tau_b = 1800 \times 12 \times 44.55 \times 10^{-3} = 2214.7 \text{ kN} > 1908 \text{kN}\]

Which is safe.

**Check for lateral-torsional buckling**

Since the compression flange of the girder is laterally restrained throughout, the possibility of lateral-torsional buckling is not there and this check is not required.

**Flange to web connection**

There will be two weld lengths along the span for each flange to web connection [Figure 1]

\[
I_z = \frac{b_fD^3}{12} - \frac{(b_f - t_w)d^3}{12}
\]

\[
= \frac{560 \times 1900^3}{12} - \frac{(560-16) \times 1800^3}{12}
\]

\[= 55702.6 \times 10^6 \text{ mm}^4\]
Let us provide weld of size, \( S = 6 \text{ mm} \)

\( K_S = 0.7 \times 6 = 4.2 \text{ mm} \)

Strength of shop weld per unit length,

\[
F_{wd} = \frac{4.2 \times 250 \times 10^{-3}}{\sqrt{3} \times 1.25} = 0.485 \text{ kN/mm} > 0.4436 \text{ kN/mm}
\]

Which is all right.

**End bearing stiffener**

Local capacity of the web,

\[
F_w = (b_1 + n_2) \frac{t_w}{\gamma_m} f_{yw}
\]

\( B_1 = 125 \text{ mm} \)

\( N_2 = 50 \times 2.5 = 125 \text{ mm} \)

\[
F_w = (125 + 125) \times 16 \times \frac{250}{1.10} \times 10^{-3} = 909.09 \text{ kN}
\]

\(< 1908 \text{ kN}\)

Hence, stiffener will be required.

Maximum reaction = 1908 kN
Let us try two flat sections, as stiffener, one on each side of web. 
Maximum width of flat that can be accommodated = \( \frac{560-16}{2} = 272 \) mm

Let us provide 16 mm thick flat section.

Maximum permissible outstand = \( 2 \times t_q \varepsilon = 20 \times 16 \times 1 = 224 \) mm

Let us try flat section 224 x 16 mm in size [Figure 2]

![Figure 2]

**Check for buckling of the stiffener**

Effective area of stiffener = \( 2 \times 224 \times 16 + (2 \times 20 \times 16) \times 16 = 17408 \) mm\(^2\).

Moment of Inertia of the stiffener,

\[
I_x = 2 \times \left[ \frac{16 \times 224^3}{12} + 16 \times 224 \times \left( \frac{224}{2} + \frac{16}{2} \right)^2 \right] = 13319.1 \times 10^4 \text{ mm}^4
\]
Radius of Gyration, \( r = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{13319.1 \times 10^4}{17408}} = 87.47 \text{ mm} \)

Slenderness ratio, \( \lambda = \frac{0.7 \times 1800}{87.47} = 14.41 \)

For \( \lambda = 14.41 \), \( f_y = 250 \text{ N/mm}^2 \), and buckling curve \( c \), the design compressive stress from Table 8.7,

\( f_{cd} = 225.67 \text{ N/mm}^2 \)

Buckling resistance, \( P_d = A_e f_{cd} = 17408 \times 225.67 \times 10^{-3} = 3928.46 \text{ kN} > 1908 \text{ kN} \)

*Which is safe. Hence, stiffener is safe in compression*

**Check for bearing capacity of the stiffener**

Since the stiffener will be coped to accommodate the fillet weld of flange plate to the web, the available effective width of stiffener flat for bearing will be lesser than the actual width. Let the stiffener plate be copied by 15 mm [Figure 3]

Width available for bearing = 224 – 15 = 209 mm

Bearing strength of the stiffener,

\[
F_{psd} = \frac{A_q f_{yp}}{0.8 \gamma_{m0}} \geq F_c - F_w
\]

**Area of stiffener in contact with flange,**

\( A_q = 2 \times 209 \times 16 = 6688 \text{ mm}^2 \)

\( F_c - F_w = 1908 - 909.09 = 998.91 \text{ kN} \)
\[ F_{psd} = \frac{6688 \times 250 \times 10^{-3}}{0.8 \times 1.10} = 1900 \text{kN} > 998.91 \text{kN} \]

Which is safe.

**Check for torsional resistance provided by end bearing stiffener**

The ends of the plate girder must have sufficient torsional resistance from transportation and erection viewpoint.

The moment of inertia of the end bearing stiffener at support,

\[ I_s \geq 0.34 \alpha_s D^3 T_{cf} \]

\[ I_y = \frac{2 t_f b_f^3}{12} + \frac{d t_w^3}{12} = \frac{2 \times 50 \times 560^3}{12} + \frac{1800 \times 16^3}{12} = 1464.08 \times 10^6 \text{mm}^4 \]

\[ A = 2 \times 560 \times 50 + 1800 \times 16 = 84,800 \text{ mm}^2 \]

\[ r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{1464.08 \times 10^6}{84,800}} = 131.40 \text{ mm} \]

Slenderness ratio, \( \lambda = \frac{L_{LT}}{r_y \sqrt{\frac{24 \times 10^3}{r_y}}} = \frac{24 \times 10^3}{131.40} = 182.65 > 100 \) (See section 12.17 in S.K. Duggal, 2nd edition)

For \( L_{LT} > 100 \), \( \alpha_s = \frac{30}{\lambda^2} = \frac{30}{182.65^2} = 8.99 \times 10^{-4} \)

\[ I_{s, \text{provided}} \geq 0.34 \times 8.99 \times 10^{-4} \times (1800 + 2 \times 50)^3 \times 50 = 104.82 \times 10^6 \text{ mm}^4 \]

\[ I_{s, \text{provided}} = \frac{16 \times (2 \times 224)^3}{12} = 119.89 \times 10^6 \text{ mm}^4 > 104.82 \times 10^6 \text{ mm}^4 \]
Which is safe.

**End-stiffener connection**

There will two weld lengths along the depth of web on each side of stiffener plates.

$b_s = 224 - 15 = 209$ mm

Tension capacity of one flat,

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times (209 \times 16) \times 410}{1.25} \times 10^{-3} = 987.15 \text{ kN}$$

Shear per unit length $q_1 = \frac{987.15}{2 \times (1800 - 2 \times 15)} = 0.278 \text{ kN/mm}$

Let us provide weld of size, $S = 5$ mm

$KS = 0.7 \times 5 = 3.5$ mm

Strength of shop weld per unit length, $f_{wd1} = \frac{3.5 \times 250}{1.732 \times 1.25} \times 10^{-3} = 0.404 \frac{\text{kN}}{\text{mm}} > 0.278 \text{ kN/mm}$

Hence provide 5 mm fillet weld to connect the end bearing stiffener with the web plate (Figure 4).
Design of Gantry Girder

Figure: Typical arrangement of gantry girder on a stepped column

1. What is Gantry Girder and what are the forces that are acting on it?
Answer: A Gantry girder having no lateral support in its length, has to withstand vertical loads from the weight of crane, hook load and impact and horizontal loads from crane surge. [Meaning of Surge: To move quickly and forcefully in particular direction].

2Q. Where the gantry girders are used?
Answer: Gantry girders or crane girders carry hand operated or electric over head cranes in industrial buildings such as factories, workshops, steel works, etc., to lift heavy materials,
equipment etc., to carry them from one location to the other, within the building.

3Q. What is drag force?
**Answer:** This is caused due to the starting and stopping of the crane bridge moving over the crane rails as the crane moves longitudinally, i.e., in the direction of gantry girders.

4Q. What is the permissible deflection where the electrically overhead cranes operated over 5000 kN.
**Answer:** The maximum vertical deflection for crane girder, under dead and imposed loads shall not exceed L/1000, where L is the span of the crane runway girder.

5Q. Mention some of the requirements of a good joint.
**Answer:** 1. The line of thrust should pass through the C.G. of the rivet group and the rivets should be symmetrically arranged about this line.

2. For a member, the rivets should be so arranged that the area of the member joined is not reduced more than necessary.

3. The number and diameter of rivets should be sufficient to develop the maximum stresses induced in all the members at the connection.

4. Members should be straight and bolts used to draw them together before the rivets are driven.
2. In what sense the design of plate girders by elastic method and limit state method is different? [2 marks]
3. What is tension field action in plate girders? [2 Marks]
4. How does a plate girder derive post-buckling strength? [2 marks]
5. Give the expression for the optimum depth of plate girder. [1 Mark]

6. Design a gantry girder to be used in an industrial building carrying a manually operated overhead travelling crane, for the following data:
   
   Crane capacity = 200 kN
   Self weight of the crane girder excluding trolley = 200 kN
   Self weight of the trolley, electric motor, hook, etc. = 40 kN
   Approximate minimum approach of the crane hook to the gantry girder = 1.20 m
   Wheel base = 3.5 m
   c/c distance between gantry rails = 16 m
   c/c distance between columns (span of gantry girder) = 8 m
   Self weight of rail section = 300 N/m
   Diameter of crane wheels = 150 mm

   Steel is of grade Fe 410. Design also the field welded connection if required. The support bracket connection need not be designed. [5 marks]
DESIGN OF TRUSSES

1. June 2014: Design a channel section purlin for a trussed roof from the following data.
   - Span of roof = 12 m
   - Spacing of purlin along slope = 2 m
   - Spacing of truss = 4 m
   - Slope of roof truss = 1 vertical, 2 horizontal
   - Wind load on roof = 800 N/m²
   - Vertical loads from roof sheets = 150 N/m²

June 2013: Design I-section purlin with and without sag bars for a trussed roof from the following data
   - Span of roof = 10 m
   - Spacing of purlin along slope or truss = 25 m
   - Spacing of truss = 4 m
   - Slope of roof truss = 1 vertical, 2 horizontal
   - Wind load on roof = 1100 N/m²
   - Vertical loads from roof sheets = 150 N/m².

May 2012: Compute the loads on a steel roof truss to suit the following data,
   - Span of the truss = 12 meters
   - Type of truss = Fan type
   - Roof cover = Galvanised corrugated G.C. sheeting
   - Spacing of roof truss = 4.5 meters
   - Wind pressure = 1.2 kN/m²

May 2011(SET-1)- Design I-section purlin with and without sag bars for a trussed roof from the following data,
   - Span of roof = 10 m
   - Spacing of purlin along slope or truss = 2.5 m
   - Spacing of truss = 4 m
Slope of roof truss = 1 vertical, 2 horizontal  
Wind load on roof = 1100 N/m²  
Vertical loads from roof sheets = 150 N/m².

**May 2011 (SET-2)** Design a channel section purlin with and without sag bars for a trussed roof from the following data,  
Span of roof = 12 m  
Spacing of purlin along slope or truss = 2 m  
Spacing of truss = 4 m  
Slope of roof truss = 1 vertical, 2 horizontal  
Wind load on roof = 1100 N/m²  
Vertical loads from roof sheets = 150 N/m².

**May 2011 (SET-3)** Design I-section purlin with and without sag bars for a trussed roof from the following data,  
Span of roof = 15 m  
Spacing of purlin along slope or truss = 3 m  
Spacing of truss = 4 m  
Slope of roof truss = 1 vertical, 2 horizontal  
Wind load on roof = 1200 N/m²  
Vertical loads from roof sheets = 160 N/m².

**May 2011 (SET-4)** Design a channel section purlin with and without sag bars for a trussed roof from the following data,  
Span of roof = 12 m  
Spacing of purlin along slope or truss = 2 m  
Spacing of truss = 4 m  
Slope of roof truss = 1 vertical, 2 horizontal  
Wind load on roof = 1200 N/m²  
Vertical loads from roof sheets = 160 N/m².
June 2013: Design I-section purlin with and without sag bars for a trussed roof from the following data,

- Span of roof = 10 m
- Spacing of purlin along slope or truss = 2.5 m
- Spacing of truss = 4 m
- Slope of roof truss = 1 vertical, 2 horizontal
- Wind load on roof surface normal to roof = 1100 N/m$^2$
- Vertical loads from roof sheets = 150 N/m$^2$

**Solution:**

**Given data,**

- Span of roof = 10 m
- Spacing of purlin along slope or truss = 2.5 m
- Spacing of truss = 4 m
- Slope of roof truss = 1 vertical, 2 horizontal

Slope $\theta = \frac{1}{2}$

$\tan \theta = \frac{1}{2}$

$\theta = \tan^{-1}\left(\frac{1}{2}\right)$

$\theta = 26.565$ degrees

$\sin \theta = 0.447$
Cosθ = 0.894

Tan θ = 0.5

Wind load on roof surface normal to roof = 1100 N/m²
Vertical load from roof sheets = 150 N/m²

Calculating the Dead Load (D.L.)
Load from roof sheeting = 150 x Spacing of purlin
                      = 150 x 2.5 = 375 N/m²
Assume self weight = 120 N/m²
Total dead weight (W_{DL}) = 495 N/m²

Calculation of Wind Load
Given, Wind Load on roof surface = 1100 N/m²
Total wind load = (W_{w.l.}) = 1100 x spacing of purlin =
1100 x 2.5 2750 N/m²

(i) Design of I-Section Purlin Without Sag Bars
It is assumed that the load combination of (Dead Load + Wind Load) creates greater effect on purlin than that of load combination of (Dead Load + Live Load)
Consider the load combination (Dead load + Wind load) for I-section purlin.
Dead load + Wind load

\( W_{D,Wx} = \) Load normal to the slope

\[ W_{w,L.} + W_{D,L.} \cos \theta = 2750 + 495 \cos(26.565) \]

\[ = 3192.742 \text{N} \]

\( W_{D,Wy} = \) Load parallel to the slope

\[ = W_{D,L.} \sin(26.565) = 495 \sin(26.565) \]

\[ = 221.37 \text{N} \]

\[ M_{xx} = \frac{(W_{D,Wx})^2 L^2}{10} = \frac{(3192.742)^2 \times 4^2}{10} = 5108.387 \times 10^3 \text{Nmm} \]

\[ M_{yy} = \frac{(W_{D,Wy})^2 L^2}{10} = \frac{(221.37)^2 \times 4^2}{10} = 354.192 \times 10^3 \text{Nmm} \]

Assume \( \frac{Z_{xx}}{Z_{yy}} = 6 \) and \( \sigma_{bt} = 0.66 f_y = 0.66 \times 250 = 165 \text{N/mm}^2 \)

\( E = 2 \times 10^5 \text{N/mm}^2 \)

**Finding the Required Sectional Modulus**
\[ Z_{xx,req} = \frac{M_{xx} + \frac{Z_{xx}}{Z_{yy}} \times M_{yy}}{\sigma_{bt}} = \frac{5108.387 \times 10^3 + [6 \times 354.192 \times 10^3]}{165} = 43.84 \times 10^3 \text{ mm}^3 \]

Select ISMB 100 @ 11.5 kg/m

\[ Z_{xx} = 51.5 \times 10^3 \text{ mm}^3 \]

\[ Z_{yy} = 10.9 \times 10^3 \text{ mm}^3 \]

**Check for Permissible Stress**

\[ \sigma_{bt} = \frac{M_{xx}}{Z_{xx}} + \frac{M_{yy}}{Z_{yy}} \]

\[ = \frac{5108.387 \times 10^3}{51.5 \times 10^3} + \frac{354.192 \times 10^3}{10.9 \times 10^3} = 131.687 \text{ N/mm}^2 < 165 \text{ N/mm}^2 \]

Hence safe.

(ii) **Design of I-Section with Sag Bar**

Dead load + Wind load

\[ W_{D,wx} = \text{Load normal to the slope} = W_{w.L.} + W_{D,L.} \cos \theta = 2750 + 495 \cos(26.565) \]

\[ = 3192.742 \text{N} \]

\[ W_{D,wy} = \text{Load parallel to slope} \]
= W_{D,L} \sin(26.565) = 495 \sin(26.565)

= 221.37 \text{ N}

**Bending Moment**

\[ M_{xx} = \frac{(W_{D,wx})^2 L^2}{10} = \frac{(3192.742 \times 4^2)}{10} = 5108.387 \times 10^3 \text{ Nmm} \]

\[ M_{yy} = \frac{(W_{D,wy})^2 (\frac{L}{2})^2}{10} = \frac{(221.37 \times (\frac{4}{2})^2)}{10} = 88.548 \times 10^3 \text{ N-mm} \]

**Finding the Required Sectional Modulus**

\[ Z_{xx_{req}} = \frac{M_{xx} + Z_{xx} \times M_{yy}}{\sigma_{bt}} = \frac{5108.387 \times 10^3 + [6 \times 88.548 \times 10^3]}{165} = 34.180 \times 10^3 \text{ mm}^3 \]

Select ISJB 150 @ 7.1 kg/m from steel Tables

\[ Z_{xx} = 42.9 \times 10^3 \text{ mm}^3 \]

\[ Z_{yy} = 3.7 \times 10^3 \text{ mm}^3 \]

**Check for Permissible Stress**

\[ \sigma_{bt} = \frac{M_{xx}}{Z_{xx}} + \frac{M_{yy}}{Z_{yy}} \]
Compute the loads on a steel roof truss to suit the following data,
Span of the truss = 12 meters
Type of truss = Fan type
Roof cover = Galvanised corrugated G.C. sheeting
Spacing of roof truss = 4.5 meters
Wind pressure = 1.2 kN/m²

Solution:

Given that,

Span of the truss, \( l = 12 \) m,
Spacing of roof truss, \( S = 4.5 \) m

Wind pressure = 1.2 kN/m²

Pitch of roof truss, \( P = \frac{1}{4} \) (assumed)

Let, Slope of roof truss be ‘\( \theta \)’

Therefore \( \tan \theta = 2p \)

\( \tan \theta = 2 \times \left( \frac{1}{4} \right) \)

\( \tan \theta = \frac{1}{2} \)

\( \theta = \tan^{-1}(1/2) \)
\( \theta = 26.565 \)

Rise of roof truss, \( R = \frac{1}{4} \times l = \frac{1}{4} \times 12 = 3 \text{ m} \).

Length along the sloping roof, \( L = \left( \frac{12}{2} \right)^2 + (3^2) \right)^{1/2} \)

\( L = 6.708 \text{ m} \)

Length/panel = \( \frac{6.708}{4} = 1.677 \text{ m} \)

(i) Load at Each Panel

(a) Dead Load

Assuming, weight of galvanized corrugated iron sheets,
\( W_{\text{GI}} = 0.133 \text{ kN/m}^2 \)

Weight of Purlins, \( W_p = 0.150 \text{ kN/m}^2 \)

Weight of bracing, \( W_b = 0.015 \text{ kN/m}^2 \)

Self weight of roof truss,
\( W_s = \frac{1}{100} \left( \frac{l}{3} + 5 \right) = \frac{1}{100} \left( \frac{12}{3} + 5 \right) = 0.09 \text{ kN/m}^2 \)

Total dead load, \( W_{\text{D.L.}} = W_{\text{GI}} + W_p + W_b + W_s \)

\( = 0.133 + 0.150 + 0.015 + 0.09 = 0.388 \text{ kN/m}^2 \)

Length of panel in plan,
\( L_p = 1.677 \cos\theta \)

\( L_p = 1.677 \cos(26.565) \)

\( = 1.499 \approx 1.5 \text{ m} \)

Load acting on each intermediate panel,
\[ W_1 = 0.388 \times 4.5 \times 1.5 = 2619 \text{ kN} \]

Load acting at end panel,
\[ W_2 = \frac{W_1}{2} = \frac{2.619}{2} = 1.310 \text{ kN} \]

1. Give briefly the design steps to be followed in the design of a roof truss.
2. Design a steel roof truss to suit the following data:
   - Span of the truss = 10 m
   - Type of truss = Fan-type
   - Roof cover = Galvanised corrugated (GC) sheeting
   - Materials: Rolled steel angles
   - Spacing of roof trusses = 4.5 m
   - Wind pressure \( P_d \) = 1.0 kN/m\(^2\)

3. Draw the elevation of the roof truss and the details of joints