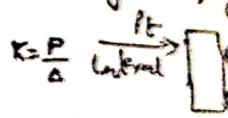
24/2/14

Unit - II

BEAMS

⇒ Introduction :-

A Structural Member Subjected to Transverse load (Loads perpendicular to the longitudinal axis) is called a Beam. When a beam is provided in buildings they are called with specific terminology known as "Joists". When a Large beam supporting a no. of Joists is called a "Girder". Beams which carry roof loads in trusses are called "Purlins". and the beam which supports from the masonary is called "Lintels". Beams are more critical members in any structure. Their design must be economical & safe. The main guidelines for the design of beams are as follows:



1. Beams should be proportioned for strength in Bending, keeping in view the Lateral and Local stability of the compression Flange.

2. Beams should be proportioned for stiffness (K) keeping in mind their deflections and deformations under certain service conditions

3. They should be proportioned for economy paying attention to the size & grades of steel to produce most economical design.

It is difficult task for the designer to select a beam size for a given span and load which will satisfy all the above

⇒ <u>Rolled steel beam sections</u> :-

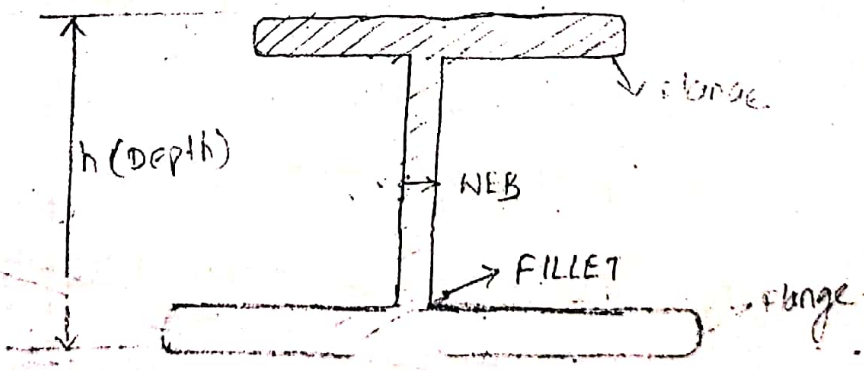
The Rolled steel beam sections are classified into 4 types

1. Indian standard Junior Beams (ISJB)
2. Indian standard Light Beams (ISLB)
3. Indian standard Medium weight Beams (ISMB)
4. Indian standard wide flange Beams (ISWB)

sometimes rolled steel columns are heavy weight which may be classified into two types :-

1. Indian standard heavy weight Beams (ISHB)
2. Indian standard column sections (ISCS)

The c/s of the Rolled steel beam has been shown in following figure.



The Beam section consist of a web and two Flanges both at top and bottom portion. The Junction between the web & Flange is known as "Fillet." The Rolled steel beams are mainly employed to resist bending. The Rolled steel beams are

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⇒ Principles of Limit state Design :-

The Aim of Limit state Design is to see that the structure which is constructed (or) built must safe. A structure may become unfit when it collapse so the requirements of deflection, shear, Bending moment, Bearing stress etc., must be satisfied in the design. Anyway the design life of permanent structures should not exceed the Limiting condition. ∴ The design is based on the Load applied and the strength of materials.

⇒ Types of Beams :-

Based on steel Design :-

As per code IS : 800 – 2007 is classified into three categories.

1. Laterally supported Beams (or) Laterally Restrained beam
2. Laterally unsupported Beams
3. Built - up - Beams

1. Laterally supported Beams (or) Laterally Restrained Beam

The beam which is provided with lateral support on one end (or) other end is known as laterally supported beam. Generally Bending is produced due to the Load acting on the beam in which tensions & compressions are developed at the flanges. The compression takes place at top portion of the flange and tension takes place at the bottom portion of the flange. These compression flanges acts as a 'column' which tends to buckle (bending) in the beam to avoid such buckling, we provide lateral supports.

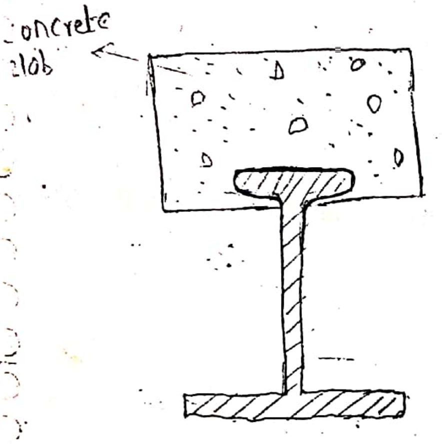
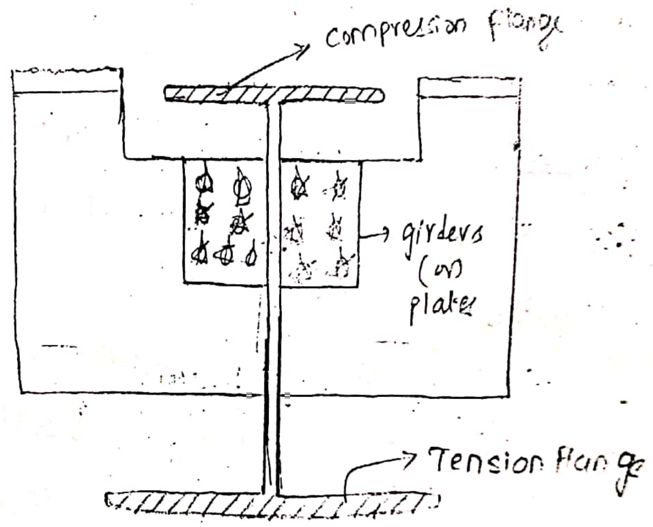


fig (1)



LATERALLY SUPPORTED BEAM

→ Design steps for Laterally supported Beam :-

Step 1 :- calculation of B.M

If the load is acting in the form of UDL then B.M $M = \frac{Wl^2}{8}$

If the load is acting in the form of concentrated load then B.M is $M = \frac{Wl}{4}$

If necessary assume self weight of the beam

Step 2 :- calculation of shear force (S.F)

If the load is U.D.L max S.F V (or) $F = \frac{Wl}{2}$

Step 3 :- Calculation of section modulus (Z) ~~required~~. for

Section Modulus (Z) required for the beam is

$$Z_{req} = \frac{M}{\sigma_{bc}}$$

where σ_b = permissible bending stress

$= 0.66 f_y$

$= 165\ kN/mm^2$

$\frac{I}{y} = Z$

Step 4 :- Selection of suitable beam section

∴ We have to select a beam section from steel tables.

From Beam sections like ISJB, ISLB, ISMB, ISWB etc., from those sections determine (or) list out all the properties like depth of the web, thickness of web (t_w), thickness flange (t_f) etc.,

Step 5 :- check for section Modulus

(i) for checking the section Modulus we have to satisfy the following conditions with reference to properties of steel section

(a) $\frac{D}{t_f}$

(b) $\frac{t_f}{t_w} < 2.0$

(c) $\frac{d_1}{t_w} < 85$

where,

t_f = thickness of flange

t_w = thickness of web

h (or) D = depth of the section beam

$d_1 = h_1 - 2h_2$

$$d_1 = h_0 - 2h_2$$

step 6 :- Determination of slenderness Ratio (λ) (or) K

$$\lambda = \frac{l_{eff}}{r_{yy}} \text{ (radius of gyration)} \qquad l_{eff} = \frac{0.7l}{r}$$

step 7 :- check for shear ~~force~~ stress

Due to shear force there will be forming shear stresses

V = shear force

$\therefore$ shear stress required = $\tau = \frac{F \text{ (or) } V}{h \times t_w}$

shear stress calculated should be less than permissible shear stress

$\tau_E = 0.4 f_y$

$= 100 \text{ N/mm}^2$

step 8 :- Check for Deflection

Deflection is based on type of Loading when

(a) A simply supported Beam (S.S.B) is subjected to U.D.L

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

(b) If the (s.s) beam is subjected to point load

$$y_{max} = \frac{wl^3}{48EI}$$

maximum deflection calculated must be less than allowable deflection.

Scanned with CamScanner

Allowable deflection $Y_{Allow} = \frac{\text{span length}}{325}$

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Problems on Laterally supported beams (or) Laterally Restrained (or) The beam is not free to rotate at the bearing

1) Design a simply supported beam to carry an U.D.L of 44 KN/m. The effective span of the beam is 8m. The Effective length of the compression flange is also 8m. The beam is laterally supported. The ends of the beam are not free to rotate at the bearings.

Sol :- **Step 1 :- calculation of B.M :**

Given U.D.L acting on the beam, W = 44 KN/m

Assume self weight = 1.0 KN/m to the beam

∴ Total load = 44 + 1 = 45 KN/m

The max. B.M for U.D.L $M = \frac{Wl^2}{8}$

$= \frac{45 \times (8)^2}{8}$

$M = 360$ KN-m

$M = 360 \times 10^6$ N-mm

Step 2 :- calculation of S.F :

The max. S.F (or) F $V = \frac{Wl}{2}$

∵ $R_A + R_B = Wl$

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

$R_B = Wl/_2$

$V = \frac{45 \times 8}{2}$

F (or) V = 180 KN

Step 3 :- Section Modulus Required (Z_{req} required)

Section modulus required $Z_{req} = \frac{M}{\sigma_{bc}}$ (To take section)

Permissible compressive stress $\sigma_{bc} = 0.66\, f_y$

Assume $f_y = 250\ N/mm^2$

$\therefore \sigma_{bc} = 0.66 \times 250$

$\sigma_{bc} = 165\ N/mm^2$

$\therefore Z_{req} = \frac{360 \times 10^6}{165} \frac{N.mm}{N/mm^2}$ (mm^3)

$Z_{req} = 2.181 \times 10^6\ mm^3$

Trial Section Modulus = 1.5 (factor) × Z

$\therefore$ (Calculated) $Z_{req} = 1.5 \times 2.181 \times 10^6$

$Z_{req} = 3.272 \times 10^6\ mm^3$

$Z_{req} = 3272 \times 10^3\ mm^3$

step 4:- Selection of suitable steel sections.

The steel section is selected based on calculated Section Modulus (Z_{req}) which should be more than Z (Z_{req}) from steel tables

$Z_{table} > (Z_{req}$ calculated)

This means Z_{xx} from steel tables must be in the range of $3.00 \times 10^6\ mm^3$ to $3.900 \times 10^6\ mm^3$

$\therefore$ Select a section of ISWB 600 (depth of section) @ 133.7 kg (self weight)

⇒ properties of section :-

Thickness of web $t_w = 11.2\ mm$

Thickness of flangle $t_f = 21.3\ mm$

Depth of the section D (or) h = 600 mm

Moment of Inertia $I_{xx} = 106198.5\ cm^4$

$I_{xx} = 106198.5 \times 10^4\ mm^4$

Scanned with CamScan

section Modulus Z_{xx} = 83940.0 x10

h_2 = 42.90 mm

Step 5 :- check for section Modulus

In order to check section Modulus, it may be classified into 3 categories

(a) $\frac{D}{t_f}$ (or) $\frac{h}{t_f} = \frac{600}{21.3} = 28.169$

(b) $\frac{t_f}{t_w} < 2.0$

$\Rightarrow \frac{21.3}{11.2} = 1.902 < 2.0$

(c) $\frac{d_1}{t_w} < 85$ (or) $\frac{h_1}{t_w} < 85$

As $d_1 = h - 2h_2$

$= 600 - 2(42.9)$

$d_1 = 514.2$

$\therefore \frac{d_1}{t_w} = \frac{514.2}{11.2} = 45.91 < 85$

We have satified above three requirements with the properties of steel section

section Modulus to be checked

$Z_{check} = \frac{M}{\sigma_{bc}}$

Here σ_{bc} should be calculated from code book

IS : 800 - 2007

from IS : 800 - 2007 which should be < 165 N/mm

Slenderness ratio K (or) $\lambda = \frac{l_{eff}}{r_{yy}}$ (r_{yy} from properties of steel tables).

$$K = \frac{8000}{52.5}$$

$$K = 152.38$$

~~$f_y = 250\ N/mm^2$~~

The 'K' value is in b/w 150 & 160 (From code book page NO: 40)

	f_y
150 →	68.9
160 →	61.4

$\therefore \sigma_{bc}$ value is obtained by Interpolation Technique

$$\sigma_{bc} = 68.9 - \frac{(68.9 - 61.4)}{160 - 150} \times (152.38 - 150)$$

$$\sigma_{bc} = 67.11\ N/mm^2 < 165\ N/mm^2$$

$$Z_{check} = \frac{M}{\sigma_{bc}} = \frac{360 \times 10^6}{67.11}$$

$$Z_{check} = 5.363 \times 10^6\ mm^3$$ ✗

where "l_{eff}" is the Effective length of the beam based on support.

When the beam is laterally supported (or) fully restrained then Effective length $l_{eff} = 0.7\,l$

$$l_{eff} = 0.7 \times 8000$$

$$\therefore l_{eff} = 5600$$

$$K = \frac{5600}{52.5}$$

$$K = 106.67$$

Here σ_{bc} value is calculated based on the values of slenderness ratio 'k' and yield stress

(From code book page No: 40)

'k' lies (around) b/w 100 & 110

k → 100 → 132

k → 110 → 115

∴ σ_{bc} Value is obtained by Interpolation Technique

$$\sigma_{bc} = 132 - \frac{(132-115)}{110-100} \times (106.67 - 100)$$

$$\therefore \sigma_{bc} = 120.67 \text{ N/mm}^2 < 165 \text{ N/mm}^2$$

$$Z_{check} = \frac{M}{\sigma_{bc}} = \frac{360 \times 10^6}{120.67}$$ [To check whether it lies in the limit (or) not]

$$Z_{check} = 2.983 \times 10^6 \text{ mm}^3$$

$$\therefore Z_{check} = 2983 \times 10^3 \text{ mm}^3$$

∴ The section which we are selected is safe

∴ $Z_{check} < Z_{xx}$ from (Tables)

$2981 \times 10^3 < 3540 \times 10^3 \text{ mm}^3$

step 7 :- check for shear stress

Max. shear stress $\tau_v = \frac{V}{h \times t_w}$

$$= \frac{180 \times 10^3}{600 \times 11.2}$$

$$\therefore \tau_v = 26.78 \text{ N/mm}^2$$

The max. shear stress calculated must be less than permissible shear stress permissible shear stress = $0.4 f_y$

$= 0.4 \times 250$

$= 100\ N/mm^2$

$\therefore 26.78\ N/mm^2 < 100\ N/mm^2$

step 8 :- check for deflection (y_{max})

Max. deflection for a simply supported beam due to U.D.L is $y_{max} = \frac{5}{384} \frac{wl^4}{EI}$

Take 'E' for steel is. i.e $E = 2 \times 10^5\ N/mm^2$

Here 'I' value is nothing but I_{xx} value from properties of the section

$$y_{max} = \frac{5}{384} \times \frac{45 \times (8000)^4}{2 \times 10^5 \times 106198.5 \times 10^4}$$

$$y_{max} = 11.29\ mm$$

The max. deflection must be less than allowable deflection

Allowable deflection $y_{allow} = \frac{\text{spanlength}}{325}$

$= \frac{8000}{325}$

$= 24.61\ mm$

$\therefore y_{max} < y_{allow}$

Note :-

From steel Tables the moment of Inertia about x-x axis (I_{xx}) value is taken, because max. bending takes place about the x-axis and also higher value is

— taken (i.e I_{xx} value is more than I_{yy})

2. In the case of Radius of gyration (r_{yy}). least value is taken

∴ r_{yy} is prefered than r_{xx}

Prd Beam & U.D.L

Q) A simply supported beam having a effective span of 8 m which carries an U.D.L of 40 KN/m. Take $f_y = 250\ N/mm^2$ and $E = 2\times10^5\ N/mm^2$. Design the beam if it is laterally supported. Draw the details of the beam.

Sol :- Step 1 :- Calculation of B.M (M) :-

Given U.D.L $W = 40$ KN/m

Assume self weight $= 1.0$ KN/m if necessary

∴ Total U.D.L $W = 40+1 = 41$ KN/m

The max. B.M for U.D.L $M = \frac{Wl^2}{8}$

$= \frac{41\times(8)^2}{8}$

$= 328$ KN-m

∴ $M = 328\times10^6$ N-mm

Step 2 :- Calculation of S.F (V) :-

The max. S.f for a U.D.L $V = \frac{Wl}{2}$

$= \frac{41\times8}{2}$

$V = 164$ KN

∴ $V = 164\times10^3$ N

'eps:- Section Modulus required (Z_{req})

Section Modulus required $Z_{req} = \frac{M}{\sigma_{bc}}$

Here σ_{bc} is permissible Bending stress

$\sigma_{bc} = 0.66\ f_y$ (Assume $f_y = 250\ N/mm^2$)

$= 0.66 \times 250$

$\therefore \sigma_{bc} = 165\ N/mm^2$

$\therefore Z_{req} = \frac{328 \times 10^6}{165}$

$= 1.987 \times 10^6\ mm^3$

$\therefore Z_{req} = 1987 \times 10^3\ mm^3$

Trial section Modulus $= 1.5 \times Z_{req}$

$\therefore$ Finally required section modulus $\therefore Z_{req}$ $= 1.5 \times 1987 \times 10^3$

$= 2981 \times 10^3\ mm^3$

Step 4 :- Selection of suitable steel section.

Here the beam section is selected which is based on Z_{req} value which should be more than Z_{req}.

$\therefore$ Select ISMB 600 @ 122.6 kg for the complete design.

$\therefore$ properties of the beam section are as follows :

Thickness of the web $t_w = 12.0$ mm

Thickness of the flange $t_f = 20.8$ mm

Depth of the section $h = 600$ mm

Moment of [illegible]

Radius of gyration r_{yy} = 4.12 cm
= 41.2 mm

section modulus $Z_{xx} = 3060.4 \times 10^3\ mm^3$

h_1 = 509.7 mm

h_2 = 45.15 mm

Step 5 :- check for section Modulus (Z_{check}) :

As we know that $Z_{check} = \frac{M}{\sigma_{bc}}$

In order to check the section Modulus we have to satisfy the following categories

(a) $\frac{h}{t_f} = \frac{600}{20.8} = 28.84$

(b) $\frac{t_f}{t_w} < 2.0$

$\Rightarrow \frac{20.8}{12.0} = 1.73\ mm < 2.0$

(c) $\frac{h_1}{t_w} < 85$

$\frac{509.7}{12.0} = 42.475 < 85$

From above σ_{bc} value is calculated by Interpol technique done b/w slender slenderness ratio k Vs

The σ_{bc} value obtained must be less than permissible stress which is nothing but 165 N/mm².

(From code book Pg No : 40)

Slenderness ratio $k = \frac{l_{eff}}{r_{yy}}$

when the support is fully restrained then

$l_{eff} = 0.7 \times l$

$= 0.7 \times 8000$

$= 5,600$

$\therefore k = \frac{5,600}{41.2} = 135.9$

$\therefore k = 135.9$

k lies b/w 130 & 140

for $f_y = 250\ N/mm^2$

~~100~~ k 130 → 88.3

140 → 77.8

$\therefore \sigma_{bc} = 88.3 - \frac{(88.3 - 77.8)}{140 - 130} \times (135.9 - 130)$

$\sigma_{bc} = 82.10 < 165\ N/mm^2$

$\therefore Z_{check} = \frac{M}{\sigma_{bc}} = \frac{328 \times 10^6}{82.10}$

$Z_{check} = 3.994 \times 10^6$

$Z_{check} = 3994 \times 10^3\ mm^3$

$\therefore Z_{check} > Z$ from Tables

$3994 \times 10^3 > 3060.4 \times 10^3$

$\therefore$ The section is not safe then again redesign with another section

Try with another section Take ISWB 600 @ 133.7 kg

∴ Properties of the Beam section are as follows

Thickness of web $t_w = 11.2$ mm

$t_f = 21.3$ mm

$Z_{xx} = 3540.0 \times 10^3 \, mm^3$

$r_{yy} = 5.25 \times 10$ mm

∴ $r_{yy} = 52.5$ mm

$h_1 = 514.2$ mm

$h_2 = 42.9$ mm

step 6:- check for section Modulus (Z_{class}):

In order to check the section modulus we have satisfy the following categories.

(a) $\frac{h}{t_f} = \frac{600}{21.3} = 28.16$

(b) $\frac{t_f}{t_w} < 2.0$

$\Rightarrow \frac{21.3}{11.2} = 1.90 < 2.0$

(c) $\frac{h_1}{t_w} < 85$

$\frac{514.2}{11.2} = 45.91 < 85$

From the above eq i.e $Z_{class} = \frac{M}{\sigma_{bc}}$, σ_{bc} value calculated by Interpolation technique done b/w slenderness ratio & V_s f_y

The σ_{bc} value obtained must be less than 165 N/mm² which is nothing but permissible stress (From code book pg no : 41) slenderness ratio $K = \frac{l_{eff}}{r_{yy}}$

when the support is fully restrained then

$l_{eff} = 0.7 \times l$

$= 0.7 \times 8000$

$l_{eff} = 5,600$

$\therefore K = \frac{5600}{52.5}$

$\therefore K = 106.67$

K lies b/w 100 & 110

for $f_y = 250$ N/mm² (from code book)

K 100 ⟶ 132

110 ⟶ 115

$\therefore \sigma_{bc} = 132 - \frac{(132-115)}{110-100} \times (106.67 - 100)$

$\sigma_{bc} = 120.66$ N/mm² < 165 N/mm²

$\therefore Z_{check} = \frac{M}{\sigma_{bc}} = \frac{328 \times 10^6}{120.66}$

$\therefore Z_{check} = 2.718 \times 10^6$ mm³

$\therefore Z_{check} = 2718 \times 10^3$ mm³

$Z_{check} < Z_{Tables}$

2718×10^3 mm³ < 3540.0×10^3 mm³

$\therefore$

Step 7 :- check for shear stress (τ_v) :

max. shear stress $\tau_v = \frac{V}{h \times t_w}$

$= \frac{164 \times 10^3}{600 \times 11.2}$

$\therefore \tau_v = 24.40\ N/mm^2$

The max. shear stress calculated must be less the permissible shear stress

permissible shear stress $= 0.4 f_y$

$= 100\ N/mm^2$

$\therefore \tau_v < 100\ N/mm^2$

$24.40 < 100\ N/mm^2$

Hence safe

Step 8 :- check for deflection (y_{max}) :-

max. deflection for a SSB due to U.D.L is

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

$\because E = 2\times10^5\ N/mm^2$

Here 'I' value is nothing but I_{xx} value from properties of the section

$$y_{max} = \frac{5}{384} \times \frac{41 \times (8000)^4}{2\times10^5 \times 106198.5\times10^4}$$

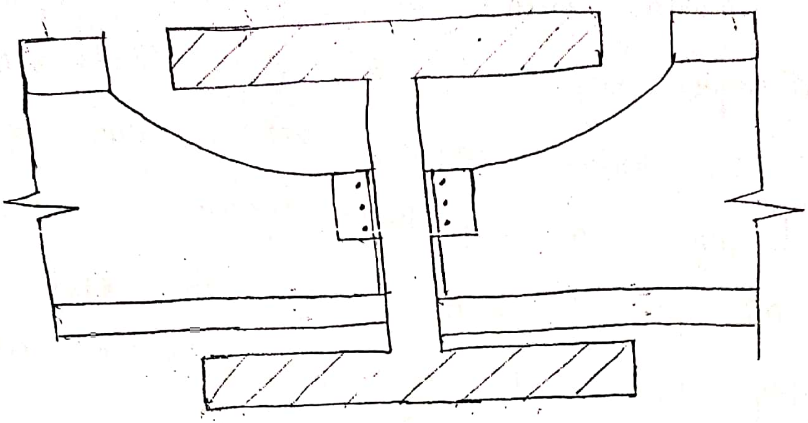
$y_{max} = 10.29$ mm

$\therefore$ The max def should be less than allowable deflection

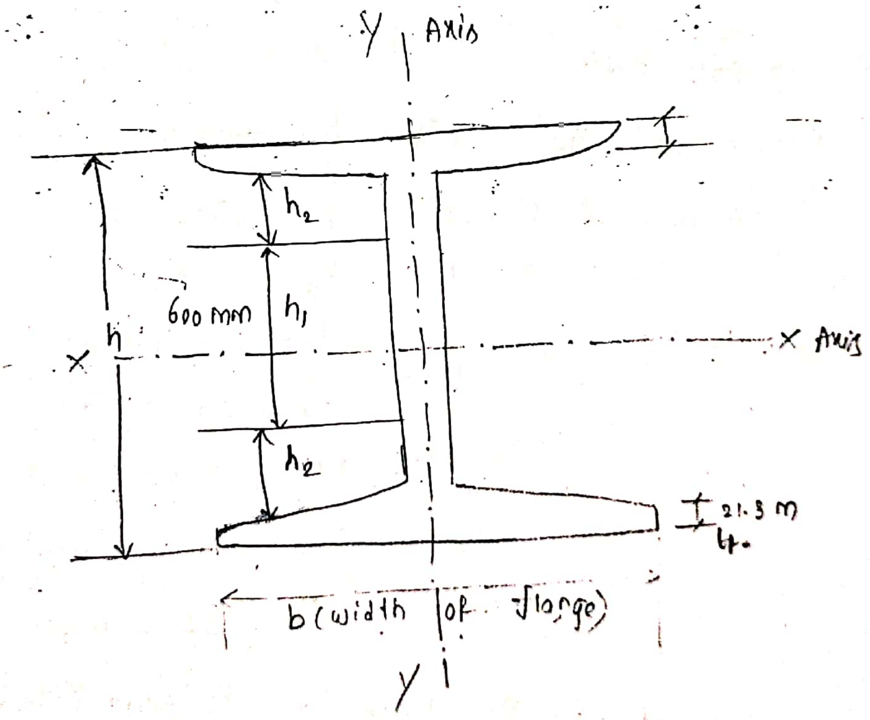
Allowable deflection $= \frac{\text{span length}}{325} = \frac{8000}{325} = 24.61$

$\therefore 10.29 < 24.61$ mm

$\therefore$ We provide a section of ISWB 600 for complete design because it satifies all the requirements of deflection, shear stress, section Modulus etc.,



LATERALLY SUPPORTED BEAMS ISWB 600



5) The Effective length of compression flange of a supported beam with beam section of ISMB 500 0.869 (self weight) KN/m. Is of 8 m. Determine the safe U.D.L per meter length which can be placed over beam having an effective span of 8 m. Adopt per stresses as per IS : 800 – 2007. The end of the beam is fully restrained against rotation at the beam

Sol :- step 1 :- properties of the beam section

In above problem the given section is ISMB 500 @ 0.869 KN/m

Depth of the section $h = 500$ mm

Thickness of flange $t_f = 17.2$ mm

$t_w = 10.2$ mm

$h_1 = 424.1$ mm

$h_2 = 37.95$ mm

$I_{xx} = 45218.3 \times 10^4$ mm^4

$Z_{xx} = 1808.7 \times 10^3$ mm^3

$r_{yy} = 35.2$ mm

step 2 :- Moment of Resistance of the beam (M_R)

As we know that $Z = \frac{M_R}{\sigma_{bc}} \Rightarrow$

$\therefore$ $M_R = Z \times \sigma_{bc}$

In above equation 'Z' is nothing but Z_{xx} value steel tables

Here σ_{bc} value is calculated by Interpolation

Technique based on the values of slenderness ratio and yield stress

$$K = \frac{l_{eff}}{r_{yy}}$$

$$K = \frac{0.7 \times 8000}{35.2}$$

$$K = 159.09 \simeq 160$$

For $f_y = 250\ N/mm^2$ and for $K = 160$

(From code book pg. no: 40) $\sigma_{bc} = 61.4\ N/mm^2$

Substitute the value of σ_{bc} in above eq, we get

Moment of resistance $M_R = Z \times \sigma_{bc}$

$= 1808.7 \times 10^3 \times 61.4$

$\therefore M_R = 111.054 \times 10^6$ N-mm

Step 3 :- Load supported over the beam (W)

Let 'W' be the U.D.L per meter Length

$\therefore$ max. B.M $(M) = \frac{Wl^2}{8}$

As $M = M_R$

$$111.054 \times 10^6 = \frac{W \times (8000)^2}{8}$$

$W = 13.88$ kN/mm

$\therefore W = 13.88$ KN/m

$\therefore$ safe U.D.L per meter length = W − self weight of the beam

self weight of the beam = 0.869 KN/m

$= 13.88 - 0.869$

⇒ Note :-

$1\ Kg = 9.81\ N$ (or) $10\ N$

Kg → KN conversion

86.9 Kg/m

= 86.9 × 10 N/m

= 869 N/m

= 0.869 KN/m

1kg = 10 N. 1 N = 10^{-3} kN

4) A laterally supported beam having an effective span of 6m consist of ISMB 550 @ 103.7 kg/m with cover plate of 250 × 16 mm connected to each of the beam flange. Determine the safe U.D.L in addition to its over weight. Take f_y = 250 N/mm²

5) A Laterally supported beam having a span of 6 m. which carries an U.D.L of 20 kN/m. Take f_y = 250 N/mm². E = 2×10^5 N/mm². The beam is free to rotate at the bearings.

4A) Step 1:- properties of the Beam section

In the above problem the given section is ISMB 550 @ 103.7 Kg/m i.e @ 1.037 KN/m

103.7 × 10 N
1037 N
1.037 KN

properties of the Beam section :-

$h = 550$ mm

$t_f = 19.3$ mm

$t_w = 11.2$ mm

$h_1 = 467.5$ mm

$h_2 = 41.25$ mm

$I_{xx} = 64893.6 \times 10^4\ mm^4$

$Z_{xx} = 2359.8 \times 10^3\ mm^3$

$\therefore r_{yy} = 3.73 \times 10$ mm

$\therefore r_{yy} = 37.3$ mm

step 2 :- Moment of Resistance of the beam (M_R)

As we know that $Z = \frac{M_R}{\sigma_{bc}}$

$$\therefore \boxed{M_R = Z \times \sigma_{bc}}$$

In the above equation 'z' is nothing but Z_{xx} value from steel tables

Here σ_{bc} value is calculated by Interpolation Technique based on the values of slenderness ratio and yield stress

$$K = \frac{l_{eff}}{r_{yy}}$$

$$\therefore K = \frac{0.7 \times 6000}{37.3}$$

~~$\therefore K = 180.13 \approx 180$~~ 112.6

for $f_y = 250$ N/mm² and for ~~K = 180~~ K = 112.6

(From code page NO : 40)

$$\sigma_{bc} = 68.9 \text{ N/mm}^2$$

Moment of resistance $M_R = Z \times \sigma_{bc}$

$= 2359.8 \times 10^3 \times 68.9$

$M_R = 162.59 \times 10^6$ N-mm

step 3 :- Load supported over the beam (W)

Let 'W' be the U.D.L per meter length

$\therefore$ max. B.M $M = \frac{Wl^2}{8}$

$$K = \frac{0.7 \times 6000}{37.3}$$

$\therefore K = 112.60$

K lies b/w 110 & 120

for $f_y = 250\ N/mm^2$ (from code book)

K		
110	$\longrightarrow$	115
120	$\longrightarrow$	101

$$\therefore \sigma_{bc} = 115 - \frac{(115-101)}{120-110} \times (112.60 - 110)$$

$\sigma_{bc} = 111.36\ N/mm^2$

Substitute the value of σ_{bc} in eq we get

Moment of Resistance $M_R = Z \times \sigma_{bc}$

$= 2359.8 \times 10^3 \times 111.36$

$\therefore M_R = 262.787 \times 10^6$ N-mm

Step 3 :- Load supported over the beam (W)

let 'w' be the U.D.L per meter length

$\therefore$ max. B.M $M = \frac{wl^2}{8}$

As $M = M_R$

$$262.787 \times 10^6 = \frac{W \times (6000)^2}{8}$$

$\therefore W = 58.39$ KN/mm

$\therefore W = 58.39$ KN/m.

$\therefore$ Safe U.D.L per meter length = W - self weight of the beam

self weight of the beam = 1.037 KN/m

$= 58.39 - 1.037$

Safe U.D.L $= 57.353$ KN/m

5A) **Step 1 :-** Calculation of B.M

Given U.D.L acting on the beam $W = 20$ KN/m

Assume self weight $= 1.0$ KN/m to the beam

$\therefore$ Total Load $= 20 + 1 = 21$ KN/m

The max. B.M for U.D.L $M = \frac{wl^2}{8}$

$$= \frac{21 \times (6)^2}{8}$$

$$\therefore M = 94.5 \text{ KN-m}$$

$$M = 94.5 \times 10^6 \text{ N-mm}$$

Step 2 :- Calculation of S.F :-

The max. S.F $V = \frac{Wl}{2}$

$$= \frac{21 \times 6}{2}$$

$$V = 63 \text{ KN}$$

Step 3 :- Section Modulus Required (Z_{req}) :

Section modulus required $Z_{req} = \frac{M}{\sigma_{bc}}$

'σ_{bc}' is permissible Bending stress

$$\sigma_{bc} = 0.66 f_y$$

$$= 0.66 \times 250$$

$$\sigma_{bc} = 165 \text{ N/mm}^2$$

$$Z_{req} = \frac{94.5 \times 10^6}{165}$$

$$\therefore Z_{req} = 572.727 \times 10^3 \text{ mm}^3$$

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Trial section Modulus = 1.5 x Z_{req}

∴ finally required section modulus = $1.5 \times 572 \times 10^3$

∴ Z_{req} = 858×10^3 mm^3

Laterally unsupported beam :-

When the compression flange of the beam Laterally unsupported, the compression flange may buckle sideway, leading to failure at the elastic limit.

Laterally unsupported beam are also known as laterally unrestrain beams that means the end of the beam is free to rotate at the bearings. The symmetrical beams even if loaded in the plain contain principle axis may bend out of plain of load under certain conditions the shape of the beams are made economical in the manner. Such shapes have less resistance to bending and torsion in the direction of xy axis. The bending of beams is accomidated by twisting this phenomenon is lateral buckling (or) later torsion buckling. The buckling tendency of compression flange increases as the stress increases on the tendency Maximum. when the strain attains a max. critical stress inorder to avoid lateral buckling due to a load the plane. The beam should have enough torsional stiffness to resist torsional forces and the beam should be laterally supported in the direction perpendicular to the weaker principle axis

⇒ **Note :-**

The Laterally unsupported means laterally the beam is free to rotate at the bearing sometimes the beam is Laterally restrained against torsion.

Design steps of laterally unsupported beams :—

step 1 : calculation of B.M (M)

step 2 : calculation of S.F (V)

step 3 : section modulus required (Z_{req})

step 4 : selection of suitable beam section

step 5 : check for section modulus (Z_{check})

Before calculating section Modulus we have to satisfy the following categories

(a) b/t_f

(b) $\frac{t_f}{t_w} < 2.0$

(c) $\frac{h_1}{t_w} < 85$

$$Z_{check} = \frac{M}{\sigma_{bc}}$$

From above equation σ_{bc} is calculated by making interpolation b/w elastic critical stress (f_{cb}) Vs f_y

From code (Pg : 55)

calculation of critical stress f_{cb}

Generally f_{cb} is calculated by double interpolation b/w slenderness ratio (K) Vs (h/t_f) from code book Pg : 57

step 6 : check for shear stress

step 7 : check for deflection.

1. Design a rolled steel I- section for a simply suppor beam with a clear span of 6m. It carries an u of 50 KN/m exclusive of self wt. of the grider beam is Laterally unsupported.

Sol :- calculation of B.M

Given clear span = 6m

Assume 0.3 m eff. cover at both ends

∴ eff. span length = $6 + \frac{0.3}{2} + \frac{0.3}{2}$ = 6.3 m

Given U.D.L = 50 KN/m

Assume self weight = 1.0 KN/m if necessary

∴ Total U.D.L = 50+1 = 51 KN/m

∴ max. B.M for U.D.L $= \frac{wl^2}{8} = \frac{51 \times 6.3^2}{8}$

= 253.023 KN-m

= 252.02 $\times 10^6$ N-mm

step 2:- calculation of S.F (V)

max. S.F for UDL $V = \frac{wl}{2}$

$= \frac{51 \times 6}{2}$

∴ V = 160.65 $\times 10^3$ N

step 3 :- $Z_{req} = \frac{M}{\sigma_{bc}}$

$\sigma_{bc} = 0.66 f_y$ (f_y = 250 N/mm²)

$\sigma_{bc} = 0.66 \times 250$

∴ σ_{bc} = 165 N/mm²

∴ $Z_{req} = \frac{M}{\sigma_{bc}}$

$$= \frac{253.02 \times 10^6}{165}$$

$$= 1.533 \times 10^6$$

$$Z_{req} = 1553.45 \times 10^3 \ mm^3$$

Trial section Modulus $= 1.5 \times Z$

$$= 1.5 \times 1533.45 \times 10^3$$

$$= 2300.75 \times 10^3 \ mm^2$$

∴ select ISMB 550 @ 103.7 Kg/m

step 4 :- properties of section

$h = 550$ mm

$t_f = 19.3$ mm

$t_w = 11.2$ mm

$r_{yy} = 3.73 \text{ cm} = 37.3$ mm

$I_{xx} = 64893.6 \times 10^4 \ mm^2$

$Z_{xx} = 2359.8 \times 10^3 \ mm^2$

$h_1 = 467.5$ mm

$h_2 = 41.25$ mm

step 5 :- check for section Modulus

$$Z_{check} = \frac{M}{\sigma_{bc}}$$

(a) $\frac{h}{t_f} = \frac{550}{19.3} = 28.49$

(b) $\frac{t_f}{t_w} = \frac{19.3}{11.2} = 1.72 < 2.0$

(c) $\frac{h_1}{t_w} = \frac{467.5}{11.2} = 41.75 < 85$

From above eq. σ_{bc} is calculated by making interpolation b/w critical stress (f_{cb}) & f_y

calculation of critical stress :-

To calculate f_{cb} interpolation is done of

(h/t_f) Vs (K)

$$K = \frac{l_{eff}}{r_{yy}}$$

when support is laterally unrestrain (or) laterally unsupported

$$l_{eff} = \text{span length} = 6.3$$

$$= \frac{6300}{37.3}$$

$$K = 168.9 \text{ mm}$$

K lies b/w 160 & 170, h/t_f lies b/w 25 & 30

K → 160 for 25 = 148.2

K → 160 for 30 = 132.0

$$X = 148.2 - \frac{(148.2 - 132)}{(30-25)} \times (28.49 - 25)$$

$$X = 136.89$$

K → 170 → K

K	25	30
	136.7	121.3

170 K → 25 = 136.7

30 = 121.3

$$Y = 136.7 - \frac{(136.7 - 121.3)}{30-25} \times (28.49 - 25)$$

$$Y = 125.95$$

Again interpolating the values x & y we get critical stress 'f_{cb}'

$x = 136.89$

$y = 125.95$

$$f_{cb} = 136.89 - \frac{(136.89 - 125.95)}{170 - 160} \times (168.9 - 160)$$

$\therefore f_{cb} = 127.153 \, N/mm^2$

f_{cb} should be increased by 20% for safe design.

f_{cb} increases $= 127.15 + 127.15 \times \frac{20}{100}$

$= 152.58 \, N/mm^2$

$\therefore f_{cb} = 152.58 \, N/mm^2$

Now interpolation done b/w f_{cb} Vs f_y

f_{cb} lies b/w 150 & 200 pg.55 (from code book)

f_{cb} → 150 f_y = 106.8

f_{cb} → 200 = 134.1

$$\sigma_{bc} = 134.1 - \frac{(134.1 - 106.8)}{200 - 150} \times (152.58 - 150)$$

$= 132.6 < 165 \, N/mm^2$

$$Z_{check} = \frac{M}{\sigma_{bc}}$$

$$= \frac{258.02 \times 10^6}{132.6}$$

$\therefore Z_{check} = 1908.14 \times 10^3 \, mm^3$

$Z_{check} < Z_{tables}$

$1908.14 \times 10^3 < 2359 \times 10^3 \, mm^3$

step 6:- check for shear stress

max. shear stress $\tau_b = \frac{V}{h \times t_w}$

$= \frac{160.65 \times 10^3}{550 \times 11.2}$

$= 26.07 \, N/mm^2$

check for deflection :-

Max. deflection for ss beam $Y_{max} = \frac{5}{384} \frac{wl^4}{EI}$

$E = 2 \times 10^5 \, N/mm^2$

I value is I_{xx} from properties of section

$I_{xx} = 64893.6 \times 10^4 \, mm^4$

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

$= \frac{5}{384} \times \frac{51 \times (6000)^4}{2 \times 10^5 \times 64893.6 \times 10^4}$

$Y_{max} = 6.631 \, mm$

$Y_{allow} = \frac{\text{span length}}{325} = \frac{6000}{325}$

$Y_{allow} = 18.48 \, mm$

$Y_{max} < Y_{allow}$

$6.631 < 18.48$

10)

Design a rolled steel I-section for a simply supported beam with clear span of 8m it carries a U.D.L of 45 kN/m exclusive of self weight. The beam is free to rotate at the bearings. Draw the

exclusive - we have to add, inclusive - self-weight is added in the load

Step 1 :- calculation of B.M

Given : clear span = 8 m.

Assume 0.3 m effective cover at both ends of the beam

$\therefore$ Effective span $= 8 + 0.3 \left(\frac{0.3}{2} + \frac{0.3}{2}\right)$

$= 8.3$ m

Given U.D.L W = 45 kN/m , Assume self weight = 1.0 kN/m

$\therefore$ max. B.M $M = \frac{Wl^2}{8} = \frac{46 \times (8.3)^2}{8}$

$\therefore M = 396.11 \times 10^6$ N-mm

Step 2 :- calculation of S.F

max. S.F for U.D.L $V = \frac{wl}{2}$

$= \frac{46 \times 8.3}{2}$

$\therefore V = 190.97 \times 10^3$ N

Step 3 :- Section Modulus required

section Modulus required $= Z_{req} = \frac{M}{\sigma_{bc}}$

$\sigma_{bc} = 0.66 f_y$

$\sigma_{bc} = 165$ N/mm²

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$\therefore Z_{req} = 2400.667 \times 10^3\ mm^3$

Trial section Modulus $= 1.5 \times Z_{req}$

$= 1.5 \times 2400.667 \times 10^3$

$\therefore Z_{req} = 3600.99 \times 10^3\ mm^3$

Step 4 :- Selection of suitable steel section

Select ISWB 600 @ 145.1 Kg/m³

Properties of the section :-

$h = 600\ mm$

Thickness of flange $t_f = 23.6\ mm$

$t_w = 11.8\ mm$

$I_{xx} = 115626.6 \times 10^4\ mm^4$

$r_{yy} = 53.5\ mm$

$Z_{xx} = 3854.3 \times 10^3\ mm^3$

$h_1 = 507.9\ mm$

$h_2 = 46.05\ mm$

Check for section Modulus :-

Before checking the section Modulus we have to satisfy the following requirements.

(a) $\frac{h}{t_f} = \frac{600}{23.6} = 25.42$

(b) $\frac{t_f}{t_w} \leq 2.0 \Rightarrow \frac{23.6}{11.8} = 2$

(c) $\frac{h_1}{t_w} < 85 \Rightarrow \frac{507.9}{11.8} = 43.04$

$Z_{check} = \frac{M}{\sigma_{bc}}$

Here the bending stress σ_{bc} may be calculated

by doing : Interpolation b/w F_{cb} [elastic critical stress)
V_s f_y }

Inorder to calculate F_{cb} (critical stress) we have to make double Interpolation b/w slenderness ratio 'K'

$$K \text{ Vs } \frac{h}{t_f}$$

From code book page NO : 57

slenderness Ratio $K = \frac{l_{effective\ length}}{r_{yy}}$

In the case of laterally unsupported (or) Laterally unrestrained $l_{eff} = 8.3$ m (span-length)

$$\therefore K = \frac{8.3 \times 1000}{53.5}$$

$K = 155.14$ and $\frac{h}{t_f}$ lies b/w 25 & 30

K lies b/w 150 & 160

calculation of critical stress (F_{cb})

[code book page NO :- 57]

		h/t_f 25	30
K →	150 →	161.5	144.8

Interpolation (Variable) → $X = 161.5 - \frac{(161.5 - 144.8)}{30 - 25} \times (25.42 - 25)$

$\therefore X = 160.16$

		h/t_f 25	30
K →	160 →	148.2	132.0

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$$\bar{Y} = 148.2 - \frac{(148.2 - 132)}{30 - 25} \times (25.42 - 25).$$

$$Y = 146.90$$

Now, again Interpolate the values of X & Y directly we get F_{cb}

$X = 160.16 \rightarrow 150$
$Y = 146.90 \rightarrow 160$

$$f_{cb} = 160.16 - \frac{(160.16 - 146.90)}{160 - 150} \times (155.14 - 150)$$

$$f_{cb} = 153.30 \ N/mm^2$$

f_{cb} should be increased by 20 % for safe design

$$f_{cb} = 153.30 + 153.30 \times \frac{20}{100}$$

$$\therefore \boxed{f_{cb} = 183.96 \ N/mm^2}$$

From code book pg NO :- 55 f_{cb} should be around 150 & 200

By interpolating f_{cb} Vs f_y we get directly σ_{bc}

$\sigma_{bc} < 165 \ N/mm^2$

$f_{cb} \rightarrow 150 \rightarrow 106.8$
$200 \rightarrow 134.1$

$$\sigma_{bc} = 134.1 - \frac{(134.1 - 106.8)}{200 - 150} \times (183.96 - 150)$$

$$\therefore \sigma_{bc} = 115.5 < 165 \ N/mm^2$$

$Z_{check} = \frac{M}{\sigma_{bc}} = \frac{390.11 \times 10^6}{113.5}$

$Z_{check} = 3429.52 \times 10^3$

$Z_{check} < Z_{tables}$

$3429.18 \times 10^3 < 3854.1 \times 10^3 \; mm^3$

∴ provide ISWB 600 @ 145.1 for the given beam

<u>check for shear stress (τ_v):</u>

max. S.F. $= V$ $\quad \tau_v = \frac{V}{h \times t_w} = \frac{190.9 \times 10^3}{600 \times 11.8}$

$= 26.96 \; N/mm^2$

permissible shear stress $= 0.4 f_y$

$= 100 \; N/mm^2.$

$\tau_v < 100 \; N/mm^2$

<u>check for Deflection:</u>

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

$= \frac{5}{384} \times \frac{46 \times (8300)^4}{2 \times 10^5 \times 115626.6 \times 10^4}$

$\therefore y_{max} = 12.29 \; mm$

$y_{allow} = \frac{\text{spanlength}}{325} = \frac{8300}{325}$

$y_{allow} = 25.5 \; mm$

$\therefore y_{max} < y_{allow}$

3) A beam section having ISMB : 550 @ 1.037 KN/m Th a simply supported beam with span length of 4m. ends of the beam are restrained against Torsio. The beam is laterally unsupported. The ends of the beam is free to rotate at the bearings. Deter the safe U.D.L. per meter length which the beam carry

sol :- step 1 :- Properties of the section :

∴ Given section ISMB : 500 @ 1.037 KN/m

$h = 550$ mm

$t_f = 19.3$ mm

$t_w = 11.2$ mm

$I_{xx} = 64893.6 \times 10^4 \text{ mm}^4$

$Z_{xx} = 2359.8 \times 10^3 \text{ mm}^3$

$r_{yy} = 37.3$ mm

$h_1 = 467.5$ mm

$h_2 = 41.25$ mm

Step 2 :- Moment of Resistance of the beam (M_R)

We have know that $Z = \frac{M}{\sigma_{bc}}$

$$\therefore \boxed{M_R = Z_{xx} \times \sigma_{bc}}$$

From above eq 'σ_{bc}' is calculated by making Interpolation b/w f_{cb} Vs f_y

calculation of critical stress (f_{cb})

To calculate f_{cb} make double interpolation b/w —

K Vs h/t_f

$\therefore \; K = \dfrac{l_{eff}}{r_{yy}} = \dfrac{4000}{37.3}$

$K = 107.23$

$h/t_f = \dfrac{550}{19.3} = 28.49$

K lies b/w 100 & 110 [Pg:-57]

h/t_f lies b/w 25 & 30

Interpolation

	h/t_f	
	25	30
K → 100 →	291.4	270.9

$X = 291.4 - \dfrac{(291.4 - 270.9)}{30 - 25} \times (28.49 - 25)$

$\boxed{X = 277.09}$

Interpolation

	h/t_f	
	25	30
K → 110 →	251.8	232.1

$y = 251.8 - \dfrac{(251.8 - 232.1)}{30 - 25} \times (28.49 - 25)$

$\therefore \; \boxed{y = 238.06}$

Now Interpolating X & Y we get f_{cb} —

$$f_{cb} = 277.09 - \frac{(277.09 - 238.06)}{110 - 100} \times (107.23 - 100)$$

$$\therefore \boxed{f_{cb} = 248.87 \ N/mm^2}$$

* f_{cb} should be increased by 20 % for safe design

$$f_{cb} = 248.87 + 248.87 \times \frac{20}{100}$$

Actual $\boxed{f_{cb} = 298.64 \ N/mm^2}$

From pg No : 55. f_{cb} should be around 250 & 300

$\therefore$ By interpolating f_{cb} Vs f_y we get directly σ_{bc}

$$\sigma_{bc} < 165 \ N/mm^2$$

	f_y
f_{cb} → 250	→ 152.3
300	→ 163.6

$$\sigma_{bc} = 163.6 \frac{(163.6 - 152.3)}{300 - 250} \times (298.64 - 250)$$

$$\therefore \sigma_{bc} = 152.60 < 165 \ N/mm^2$$

$$M_R = 2359.8 \times 10^3 \times 152.60$$

$$\therefore \boxed{M_R = 360.10 \times 10^6 \ N\text{-}mm}$$

Step 3 :- Load supported for the beam

Let 'W' be the U.D.L acting

$\therefore$ max B.M $M = \frac{Wl^2}{8}$ (As $M = M_R$)

$$\therefore M_R = \frac{Wl^2}{8}$$

$$360.133 \times 10^6 = \frac{W \times (4000)^2}{8}$$

$$\therefore W = 180.06 \ N/mm$$

$$W = 180.06 \ kN/m$$

Safe U.D.L $= W - \text{self weight}$

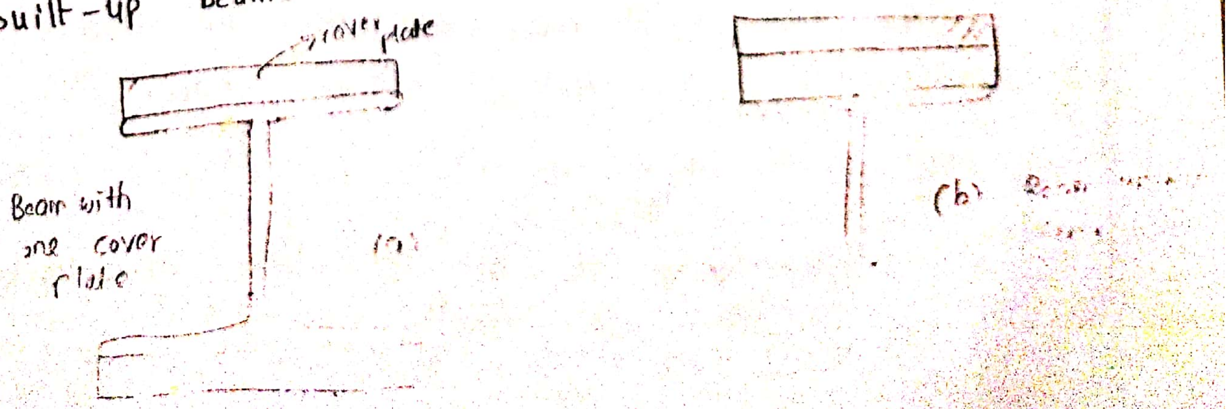
$= 180.06 - 1.037$

$$\boxed{W = 179.02 \ kN/m}$$

⟹ Built-up-Beams (or) compound Beams :-

When span lengths are more, a very large B.M's are generated due to the applications of high loads. Also the depth of the beam may be limited (or) restricted. Rolled steel beams of Largest size cannot resist. For that purpose built-up-beams are used.

The built-up-beams are also termed as compound girders. The built-up compound beams (or) compound beams are used when the rolled steel beam section are in adequate. In the case of the strength of the beam section. We have to attach coverplates both at top & bottom flanges of the beam. The following are the built-up beams shown diagramatically as follows.



⇒ Design steps for built-up beams :-

step 1 :- calculation of B.M

step 2 :- calculation of S.F

step 3 :- section Modulus required

step 4 :- Selection of suitable steel section :-

Here in the case of built-up-beams, beam depth is limited. so, we have to take the section based on the depth of the beam. sim... section Modulus from tables will be less than Z_{req}. Here plates are attached both at top & bottom flanges

Step 5 :- Dimensions of the cover plate

Here Area of the cover plate must be calculated

∴ Area of cover plate $A_p = \frac{Z_{req} - Z_{xx}}{h}$

Width of the plate $= b = \frac{(Area)\ A_p}{\text{thickness of cover plate}}$

Here thickness of cover plate may be selected by trial & error Method which ranges from 10 mm, 12mm, 14, 16, 18, 20, 22, 25, 30 ....

step 6 :- calculation of Bending stress in compression & Tension

Here Bending stress in compression —

$$\sigma_{b\,comp} = \frac{\text{Max. B.M}}{\text{Gross M.I}} \times y$$

Gross Moment of Inertia $I_{zz} = I_{xx(table)} + 2A_p\left(\frac{h'}{2}\right)^2$ (Area of plate)

y = depth of Neutral axis including thickness of plates provided both at top and bottom.

h' = distance b/w centroid at top & bottom flange

$$\therefore \frac{h'}{2} = \frac{h}{2} + \frac{\text{thickness of one cover plate}}{2}$$

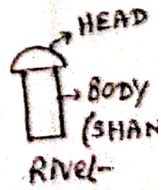
Step 7 :- Bending stress in Tension — σ_{bt}

$$\sigma_{bt} = \sigma_{b\,comp} \times \frac{\text{Gross area of flange}}{\text{Net area of Flange}}$$

Gross area of flange $= (b_f \times t_f) \pm A_p$ (b_f = width of flange)

Net area of Flange = Gross area $- 23.5 \times (t_f + t_w)$

where 23.5 = Gross diameter of the Rivet



The calculated Bending stress both in Compression and Tension must be $< 165\ N/mm^2$

Step 8 :- check for shear stress $\tau = \frac{V}{h \times t_w}$

step 9 :- check for deflection

$$y_{max} = \frac{5}{384}\,\frac{wl^4}{EI_{zz}}$$

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1) Design a simply supported beam which carries an UDL of 50 kN/m inclusive. The effective span of the beam is 10 m. Depth of the beam is limited to 500 mm. The beam is laterally restrained at both ends.

step 1 :- calculation of B.M :-

Given U.D.L $w = 50$ kN/m

Effective span $= 10$ m

max. B.M $M = \frac{wl^2}{8} = \frac{50 \times (10)^2}{8}$

$\therefore M = 625 \times 10^6$ N-mm

step 2 :- calculation of S.F (V)

max. S.F $V = \frac{wl}{2} = \frac{50 \times 10}{2}$

$\therefore V = 250 \times 10^3$ N

step 3 :- section Modulus required (Z_{req})

$Z_{req} = \frac{M}{\sigma_{bc}}$

$\sigma_{bc} = 0.66 f_y$

$\therefore \sigma_{bc} = 165$ N/mm²

$\therefore Z_{req} = \frac{625 \times 10^6}{165} = 3787.87 \times 10^3$ mm³

$\therefore Z_{req} = 3787.87 \times 10^3$ mm³

Trial section Modulus $= 1.5 \times Z$

$= 1.5 \times 3787.87 \times 10^3$

$Z_{req} = 5681.81 \times 10^3$ mm³

$Z_{req} = 5681.81 \times 10^3\ mm^3$

Step 3 :- Selection of suitable steel section :-

Here steel section is selected based on beam depth, Here depth of beam is limited to 500.

Select ISWB : 450 @ 79.4

properties of section :-

$h = 450\ mm$

$t_f = 15.4\ mm$

$t_w = 9.2\ mm$

$I_{xx} = 35057.6 \times 10^4\ mm^4$

$r_{yy} = 41.1\ mm$

$Z_{xx} = 1558.1 \times 10^3\ mm^3$

Dimensions of cover plate -:

The dimensions of cover plate nothing but area of plate

$$A_p = \frac{Z_{req} - Z_{xx}}{h}$$

$$\Rightarrow \frac{5681.81 \times 10^3 - 1558.1 \times 10^3}{450}$$

$$A_p = 9163.81\ mm^2$$

$$\text{width of the plate} = \frac{A_p}{\text{thickness of cover plate}}$$

let us Assume thickness of plate = 25 mm

$= \frac{9163.8}{25}$

$= 366.55$ mm

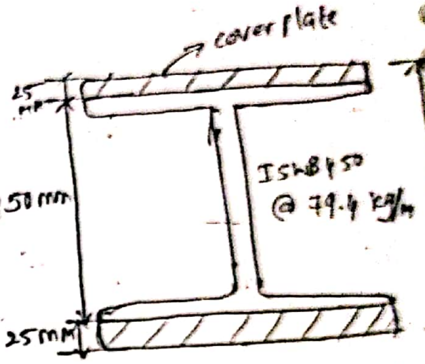
$\simeq 366$ mm

∴ provide the dimensions of cover plate is 366 x 25 mm

step :- calculation of Bending stress in compression and Tension

Bending stress in compression $\sigma_{b_{comp}} = \frac{\text{Max B.M}}{\text{Gross M.I}} \times y$

$y = \frac{500}{2} = 250$ mm
(Including plates)



$\text{GROSS M.I} = I_{xx\,(tables)} + 2A_p\left(\frac{h'}{2}\right)^2$

$\frac{h'}{2} = \frac{450}{2} + \frac{25}{2} = 237.5$ mm

$\text{Gross M.I} = 35057.6 \times 10^4 + 2 \times 9163.81 \times (237.5)^2$

$\text{Gross M.I} = 1.384 \times 10^9 \text{ mm}^4$

$\sigma_{b_{comp}} = \frac{\text{max. B.M}}{\text{Gross M.I}} \times y$

$= \frac{625 \times 10^6}{1.384 \times 10^9} \times 250$

$\sigma_{b_{comp}} = 112.86 \text{ N/mm}^2 < 165 \text{ N/mm}^2$

Bending stress in Tension $\sigma_{b_t} = \sigma_{b_c} \times \frac{\text{Gross area}}{\text{Net area}}$

$\text{Gross area} = (b_f \times t_f) + A_p$

$= (200 \times 15.4) + 9163.81$

$\text{Gross AREA} = 12243.81 \text{ mm}^2$

Scanned with CamScanner

Net area = Gross area − 23.5 (tf + tw)

$= 12243.81 - 23.5\,(15.4 + 9.2)$

Net area = 11665.71 mm²

$\sigma_{bt} = 112.86 \times \frac{12243.81}{11665.71}$

$\therefore \sigma_{bt} = 118.456\ N/mm^2 < 165\ N/mm^2$

check for shear

$\tau_v = \frac{V}{h \times t_w}$

$= \frac{250 \times 10^3}{450 \times 9.2}$

$\therefore \tau_v = 60.38\ N/mm^2$

$< 100\ N/mm^2$

$y_{max} = \frac{5}{384}\ \frac{wl^4}{E\, I_{zz}\,(I_{max})}$

$= \frac{5}{384}\ \frac{50 \times (10000)^4}{2 \times 10^5 \times 1.384 \times 10^9}$

$= 23.52\ mm$

$y_{allow} = \frac{\text{span length}}{325} = \frac{10000}{325} = 30.76\ mm$

$y_{max} < y_{allow}$

Q:- Design a beam with 6.5 m effective span carrying a U.D.L 40 KN/m inclusive of self weight over the entire span. The overall depth of the beam is re-stricted to 350 mm. The compression flange of the beam is laterally supported. Take $f_y = 250\ N/mm^2$

Step 1 :- Calculation of B.M

Given Load U.D.L $W = 40\ KN/m$

$l = 6.5\ m$

$M = \frac{40 \times (6.5)^2}{8} = 211.25 \times 10^6\ N\text{-}mm$

Step 2 :- Calculation of S.F :

max. S F $V = \frac{wl}{2} = \frac{40 \times 6.5}{2} = 130 \times 10^3\ N$

$\therefore V = 130 \times 10^3 N$

step 3 :- Section Modulus required (Z_{req}):

$$Z_{req} = \frac{M}{\sigma_{bc}}$$

$$= \frac{211.25 \times 10^6}{165}$$

$$= 1.280 \times 10^5 \ mm^3$$

Trial = 1.5 x Z

$$Z_{req} = 1920 \times 10^3 \ mm^3$$

step 4 :- selection of suitable steel section :

As the given beam depth is limited to 350 mm. Therefore the section is selected based on depth of the beam.

select ISMB 300 @ 44.2 kg/m

properties of the section :- $h = 300 \ mm$, $b_f = 140 \ mm$

$t_f = 12.4 \ mm$

$t_w = 7.5 \ mm$

$I_{xx} = 8603.6 \times 10^4 \ mm^4$

$Z_{xx} = 573.6 \times 10^3 \ mm^3$

step 5 :- Dimensions of the cover plate

In order to increase the strength of the section we have to attach cover plates both at top & bottom using rivets

Area of the plate $A_p = \frac{Z_{req} - Z_{xx}}{h}$

$= \frac{1920 \times 10^3 - 573.6 \times 10^3}{300}$

$\therefore A_p = 4489.5 \text{ mm}^2$

width of the plate $= \frac{A_p}{\text{thickness of cover plate}}$

Let us assume 25 mm thickness of cover plate

$\therefore$ width $= \frac{4484.5}{25} = 179.58$

$\simeq 180$ mm

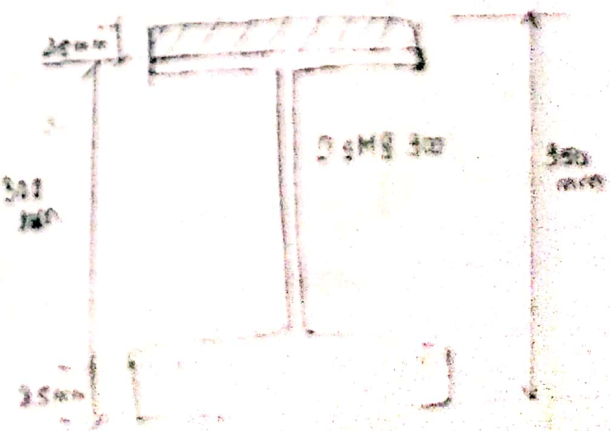
$\therefore$ provide dimensions of cover plate 180 mm × 25 mm

Step 6 :- Calculation of Bending stress in compression and Tension.

Bending stress in compression $\sigma_{b_{comp}} = \frac{\text{max. B.M}}{\text{Gross M.I}} \times y$

$y = \frac{350}{2} = 175$ mm

(Including plates)



Gross M.I $(I_{zz}) = I_{xx\,(table)} + 2 A_p \left(\frac{h'}{2}\right)^2$

$\frac{h'}{2} = \frac{300}{2} + \frac{25}{2} = 162.5$ mm

$h'/2 = 162.5$ mm

$\therefore I_{zz} = 8603.6 \times 10^4 + 2 \times 4489.5 \times (162.5)^2$

$I_{zz} = 323.16 \times 10^6 \text{ mm}^4$

$$\sigma_{b\,comp} = \frac{211.25 \times 10^6}{323.16 \times 10^6} \times 175$$

$$\sigma_{b\,comp} = 114.40 \ N/mm^2 < 165 \ N/mm^2$$

Bending stress in Tension $\sigma_{bt} = \sigma_{b\,comp} \times \frac{\text{Gross area}}{\text{Net area}}$

Gross area $= (b_f \times t_f) + A_f$

$= (140 \times 12.4) + 4489.5$

$= 6225.5 \ mm^2$

Net area $=$ Gross area $- 23.5\ (t_f + t_w)$

$= 6225.5 - 23.5\ (12.4 + 7.5)$

$= 5757.85 \ mm^2$

$\therefore \sigma_{bt} = 114.40 \times \frac{6225.5}{5757.85}$

$\sigma_{bt} = 123.69 \ N/mm^2 < 165 \ N/mm^2$

step 7 :-

check for shear stress

$$\tau_v = \frac{V}{h \times t_w} = \frac{130 \times 10^3}{300 \times 7.5}$$

$= 57.78 \ N/mm^2 < 100 \ N/mm^2$

step 8 :-

check for deflection :

$$y_{max} = \frac{5}{384} \frac{wl^4}{EI_{zz}}$$

$$= \frac{5}{384} \times \frac{40 \times (6500)^4}{2 \times 10^5 \times 323.16 \times 10^6}$$

$y_{max} = 14.384 \ mm$

$y_{allow} = \frac{\text{span length}}{325} = \frac{6500}{325} = 20$

$y_{max} < y_{allow}$