

DESIGN OF BEAMS

[Refr. Design of Steel Structures - Dr. S. S. Bhavigatti]  
Beam is structural member with length considerably

larger than cross-section dimensions subject to lateral which gives rise to bending moment shear force in the member. purlins which rest on the trusses and support roof sheet.

Plastic moment carrying capacity of a section

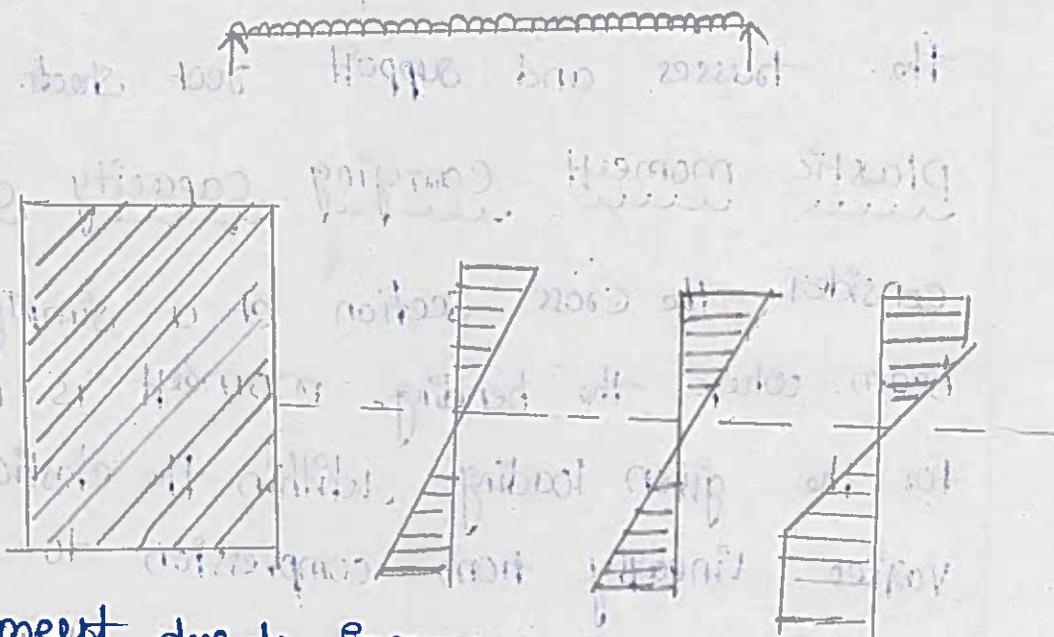
consider the cross section of a simply supported beam where the bending moment is maximum for the given loading. Within the elastic limits varies linearly from compression to tension.

As shown in fig 7.1 As the load is gradually increase stress increase proportionately of analysis

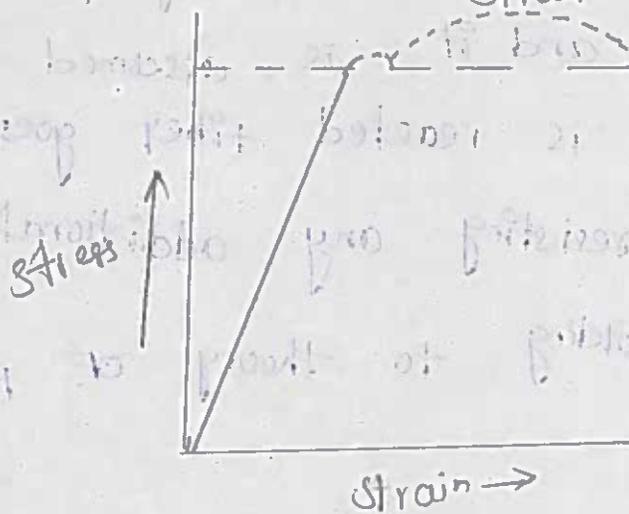
stress strain subjected to yield stress. The extreme fiber yields for simplicity of analysis stress strain for steel is assumed as shown in which strain hardening part of the curve is ignored and it is assumed that after yield point is reached fiber goes on yielding without resisting any additional load

Hence according to theory of plastic fibers

But interior fibers are not yet yielding and hence additional load are resisted by unyielded portion of the section. As the load is gradually increase one by one fiber reaches yield stress and stops resisting additional load. Show partially yielded case.



Moment due to increase in load. This condition when all fibers at a section yield called formations of plastic hinge. After this stage the rotation at sections will be take place without resisting additional moment but the moment corresponding

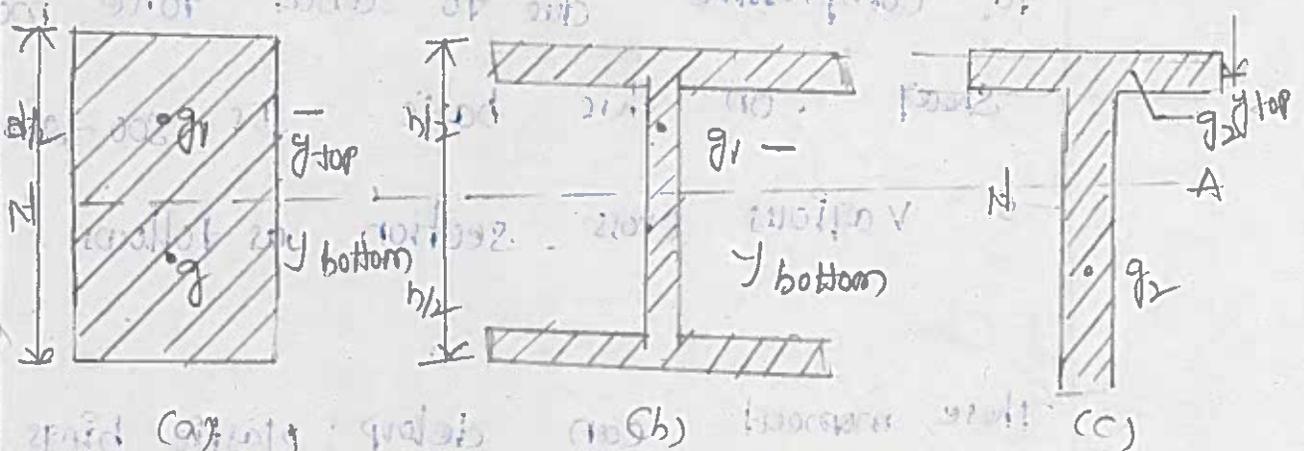


Let the area of the section in compression be  $A_c$  and total area  $A$ . Equating the horizontal forces for the equilibrium condition we get

$$A_c f_y = A_t f_y$$

$$A_c = A_t = A/2$$

Denoting the section where stress changes the sign as plastic neutral axis, we can conclude that plastic neutral axis divides the total area in two equal parts. Obviously such section is at mid depth for symmetric section as shown in fig. For an unsymmetrical section it is to be found from the condition that  $A_c = A_t = A/2$ .



plastic neutral axis

plastic moment capacity may be found by taking moment of horizontal forces about plastic N-A. It may be found by taking moment of resistance is additive of moment resisted by compressive forces and tensile force since the moment have the same sign.

## Classification of Cross-sections:

When plastic analysis is used the members should be capable of forming plastic hinges with sufficient rotation capacity without buckling. Hence it is necessary to see that plate elements of a cross section do not buckle locally due to compressive stress before plastic hinges are found. The local buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross section subjected to compressive due to axial force moment or shear.

on this basis IS 800-2007 classifies various cross section as follows

### 1 Class

These members can develop plastic hinges and have the rotation capacity required for failure of structure by formation of plastic

mechanism. The section having width to thickness ratio of plate element shall be

less than that specified under class

1 as shown in table 7.1 being in the class.

2<sup>nd</sup> part (4) 2) Class 2 (Compact) cross sections-

Such section can develop plastic moment of resistance but have inadequate plastic hinge capacity for formation of plastic mechanism. due to local buckling

Class 3 cross section (semi compact) these are the section in which the extreme fibres in compression can reach yield, but cannot the plastic moment of resistance due to

Class 4 cross sections:- These cross section the element of which buckle locally even than those specified for class

DESIGN PROCEDURE:-

- 1) A trial section is selected assuming it is going to be plastic section
- 2) Check for bending strength
- 3) Check for shear strength
- 4) Check for the deflection

BENDING strength of a Laterally support beam

IF  $\frac{d}{t_w} \leq 67 \sqrt{E}$  IS 800-2007 consider two case one

with design shear strength  $0.6V_d$  than other

with  $V_d$  design shear. when  $\frac{d}{t_w} > 67 \sqrt{E}$  shear buckling or web likely to take

(a) IF  $V \leq 0.6V_d$

$$M_d = \beta_b z_p f_y \times \frac{1}{\gamma_{m0}} \leq 1.2 z_e$$

$$\leq 1.5 z_e \frac{f_y}{\gamma_{m0}} \text{ for cantilever beam}$$

Where

$\beta_b = 1.0$  for plastic and compact sections

(b) If  $v_s < 0.5 v_d$

In such cases

$$M_{dr} = M_d - \beta (M_d + M_{fd}) \leq 1.2 z_e \times f_y \times \frac{1}{\gamma_{m0}}$$

Where

$$\beta = \left( \frac{2v}{v_d} - 1 \right)^2$$

$M_d =$  plastic design

$v =$  factored applied

$v_d =$  design shear strength

$M_{fd} =$  plastic design strength

$$M_{dr} = \frac{z_e f_y}{\gamma_{m0}}$$

A roof of a hall measuring  $8m \times 12m$  consist of  $100mm$  thick r.c slab support one steel I-beam spaced  $3m$  apart as shown in fig

finishing may be taken as  $1.5 \text{ kN/m}^3$  and live

load as  $1.5 \text{ kN/m}^3$

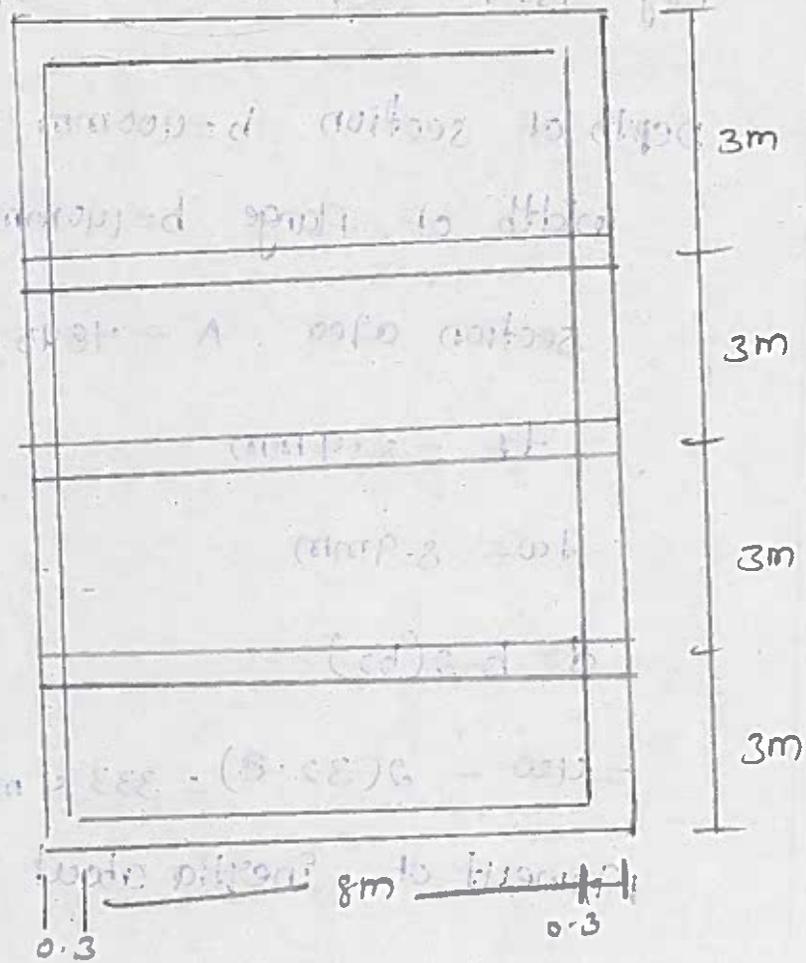
5) 2nd part

Each beam has a clear span of 8m and take of 3m width of slab

weight of R.C. slab =  $0.1 \times 1 \times 1.3 \times 25 = 3.25 \text{ kN}$

finishing load =  $1.5 \times 3 = 4.5 \text{ kN/m}$

self weight (assumed) =  $0.8 \text{ kN/m}$



∴ Total dead load =  $12.8 \text{ kN/m}$

Live load =  $1 \times 3 \times 1.5 = 4.5 \text{ kN/m}$

∴ factored load =  $1.4 \times 4.5 = 6.3 \text{ kN/m}$

Total factored load =  $25.95 \text{ kN/m}$

Design moment  $M = \frac{wl^2}{8}$

$= \frac{25.95 \times 8.3^2}{8} = 233.46 \text{ kN-m}$

Design shear force  $V = \frac{25.95 \times 8.3}{2} = 107.69 \text{ kN}$

$$\therefore \text{section shear force } V = \frac{29.59 \times 8.3}{9}$$

$$\text{section modulus required} = \frac{M}{f_y} \times \gamma_m$$

$$\therefore Z_p = \frac{283.46 \times 10^6 \times 1.1}{250} = 983224 \text{ mm}^3$$

Try IS 40B 400 which has  $Z_p = 1176.163 \times 10^3 \text{ mm}^3$

Depth of section  $h = 400 \text{ mm}$

width of flange  $b = 140 \text{ mm}$

section area  $A = 7845.58 \text{ mm}^2$

$$t_f = 8.9 \text{ mm}$$

$$t_w = 8.9 \text{ mm}$$

$$d = h - 2(b_2)$$

$$= 400 - 2(32.8) = 333.4 \text{ mm}$$

Moment of inertia about  $z-z$  axis

$$I_{z-z} = 20458.4 \times 10^4 \text{ mm}^4$$

$$\text{Elastic modulus } Z_e = 1022.7 \times 10^3 \text{ mm}^3$$

$$r_b = \frac{140}{2} = 70$$

Section Classification

$$C = \left( \frac{250}{f_y} \right)^{1/2} = \left( \frac{250}{250} \right)^{1/2} = 1$$

$$\frac{b}{t_f} = \frac{70}{8.9} = 7.87 < 9.41$$

$$\frac{d}{t_w} = \frac{333.4}{8.9} = 37.57 < 84.1$$

37d  
P15

Plastic section

Weight of section = 0.604 kN/m

Assumed weight = 0.8 kN

Design shear  $V = 107.61 \text{ kN}$

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \text{ shear area}$$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 1000 \times 8.9$$

$$= 467128 \text{ N} = 467.128 \text{ kN} > 107.61 \text{ kN}$$

$$0.6 V_d = 0.6 \times 467.128 = 280.277 > 107.61 \text{ kN}$$

$$\frac{d}{t_w} = 28.2 \text{ which less than } 67.6 \text{ since } e=1$$

$$M_d = B_b = \gamma_{m0} \frac{f_y}{\gamma_{m0}} \leq 1.2 \frac{Z_{eff}}{\gamma_{m0}}$$

$$M_d = 1.0 \times 467.128 \cdot 10^3 \times \frac{250}{1.1} < 1.2 \times 10^6$$

$$\times 10^3 \times \frac{250}{1.1}$$

$$= 267.310 \times 10^6 \leq 278.986 \times 10^6$$

$$M_d = 267.310 \times 10^6 \text{ N-mm}$$

Total working load = 12.8 + 0.5 = 17.3 kN

17.3 kN/m

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

$$\therefore \delta = \frac{5}{384} \times \frac{17.3(8300)^4}{2 \times 10^8 \times 204584 \times 10^4}$$

$$= 28.178 \text{ mm}$$

$$\text{(Def table 7.2)} = \frac{le}{300} = \frac{8300}{300} = 27.67$$

Provide ISMB 400.

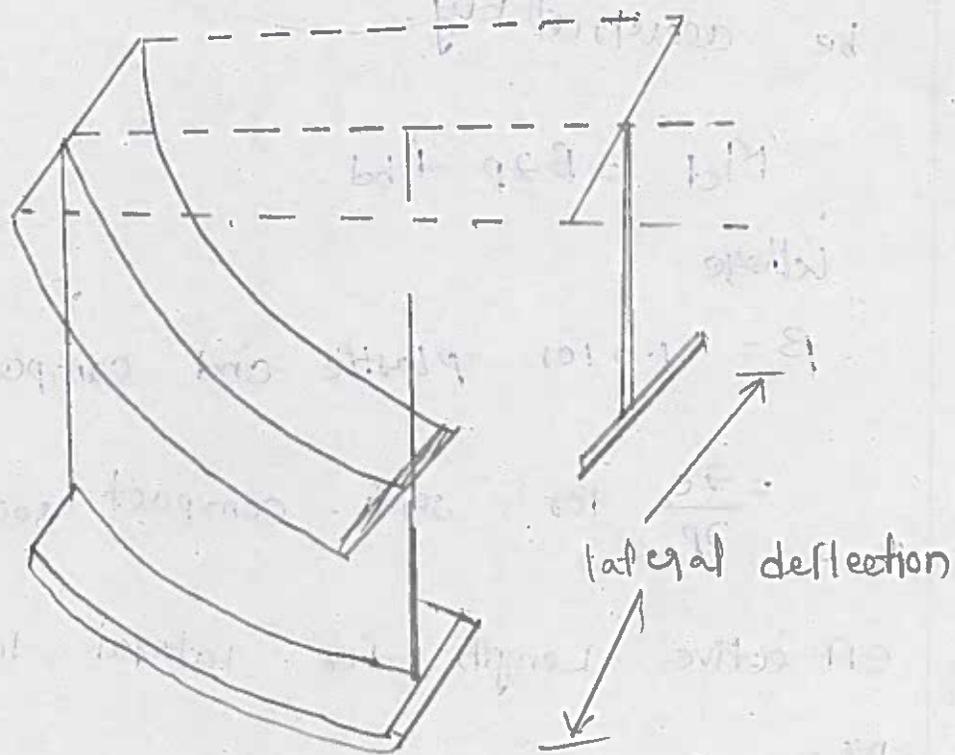
Design strength of Laterally unsupported beam.

Beams are normally used so as to bend about major (z-z) axis to bend about major (y-y) axis since they have higher value support - ted It has tendency to bend the lateral direction with twisting

Bending moment of compression flange with twisting reduced the load carrying capacity of the section.

- (a) Building the into the walls or
- (b) web or flange cleats
- (c) bearing stiffener acting on conjugates with bearing.

Intermediate lateral supports are provided by system or by connecting to an independent rodent post of structure. such restraint should be able to resist per cent of the exam force in the compression flange and should be connected to compression flanges



Is 800 - 2007 specified that the member may be treated as laterally supported following

- (a) Bending is about the minor axis
- (b) section is hollow (rectangular / circular)
- (c) Note dimensional slenderness ratio  $\lambda$  is less than 0.4 where

$$\lambda = \sqrt{\frac{f_y}{f_{cr}}}$$

The elastic buckling stress  $f_{cr}$  is to bound considering torsional and flexural rigidities of the member, support condition and non-symmetrical section all IS 800-2007 for different beams section considering loading support condition and non-symmetrical section shall be accurately.

$$M_d = \beta z_p f_{bd}$$

where

$\beta = 1.0$  for plastic and compact section

$= \frac{z_e}{z_p}$  for semi-compact section

Effective Length for lateral torsional buckling.

for simply support beams and girders of span length of span  $L$  where no lateral resistance of compression flange is provided but end of the beam is restrained against the effective

LT shall be taken in IS 800 (C)

Table 15 IS 800 normal loading means through shear center and destabilizing loading means that the load acting is not through shear centre.

2nd part  
 ③ An ISHE 500 section is used as a beam over a 6m with simply supported ends. Determine the maximum factored uniformly distributed load that the beam can carry if ends are restrained against torsion but compression flange is laterally unsupported.

for IS 198 = 500

overall depth  $h = 500 \text{ mm}$

width of flange  $b = 180 \text{ mm}$

Thickness of flange  $= t_f = 17.2 \text{ mm}$

Thickness web  $= 10.2 \text{ mm}$

Effective length for torsional buckling  $= 6 \text{ m}$

$$\frac{kl}{l} = \frac{6 \times 1000}{35.2} = 170.45 \quad \text{where } kl = \text{effective length}$$

$$\frac{h}{t_f} = \frac{500}{17.2} = 29.06$$

from table

$$\frac{h}{t_f} \rightarrow 25 \quad 29.6 \quad 30$$

$$\frac{kl}{l}$$

↓

$$170 \quad 136.7 \times 121.3$$

$$170.45 \quad - \quad 0$$

$$180 \quad 124.1 \times 112.2$$

To get the value for  $\frac{h}{t_f} = 29.6$  and  $\frac{kl}{l} = 170.45$

first get the value of  $x$  and  $y$  corresponding

$$\text{to } \frac{h}{t} = 29.6$$

to get that the value of  $x \left[ \frac{kt}{l} = 170 \cdot \frac{h}{t} = 29.6 \right]$

$$f_{crb} = 136.7 - \frac{4.6}{5} (136.7 - 121.3) = 122.53$$

to get the value of  $y \left( \frac{kt}{l} = 180 \frac{h}{t} = 29.6 \right)$

$$f_{cb} = 127.1 - \frac{4.6}{5} (127.1 - 112.2) = 117.39 \text{ N/mm}^2$$

to get value of  $z$

$$f_{crb} = 122.53 - \frac{0.45}{10} (122.53 - 113.39)$$

$$= 122.119 \text{ N/mm}^2$$

Referring to table 13(a) in S 800 (7.4(a)) for

$$f_y = 250 \text{ N/mm}^2$$

we find  $f_b \geq 77.3 \text{ N/mm}^2$  for  $f_{crb} = 100$

and  $f_{bd} = 106.8 \text{ N/mm}^2$  for  $f_{cb} = 150$

for  $f_{crb} = 122.12$

$$f_{bd} = 77.3 + \frac{22.12}{50} (106.8 - 77.3) = 90.35 \text{ N/mm}^2$$

section Classification

$$\epsilon = \sqrt{\frac{250}{250}} = 1.0$$

width of web standing leg  $b = 180/2 = 90 \text{ mm}$

$$\frac{b}{t_f} = \frac{90}{17.2} = 5.23 < 9.4$$

$$d = h + 2r_1 = 480 + 2 \times 17 = 514 \text{ mm}$$

$$\therefore \frac{d}{t_w} = \frac{514}{10.2} \leq 8.4$$

Hence it belongs to class I

$$\therefore M_d = B_b \cdot Z_p$$

$$B_b = 1 \cdot Z_p = 2074.7 \times 10^4 \text{ mm}^3$$

$$\therefore M_d = 1 \times 2074.7 \times 10^3 \times 90.35 = 187.449 \times 10^6 \text{ N-mm}$$
$$= 187.009 \text{ kNm}$$

If web  $w$  is  $\text{kN/m}$  then  $\frac{wL^2}{8} = M_d$

$$w \times \frac{6^2}{8} = 187.009$$

$$w = 41.685 \text{ kN/m}$$

$$\text{self weight} = 86.9 \text{ kg/m} = 86.9 \times 9.81 = 852 \text{ N/m}$$

$$= 0.852 \text{ kN/m}$$

$$\text{factored self weight} = 1.5 \times 0.852 = 1.278 \text{ kN/m}$$

super imposed load that beam carry

$$= 41.685 - 1.278 = 40.407 \text{ kN/m}$$

Design is possible only by trial. First find it to be assumed to find required  $Z_p$  of the section. After selecting the trial section, calculation are made to find moment.

Calculations are made to find moment carrying capacity of the section. If section selected is not adequate larger section is to be tried. If the section selected is having capacity.

Purlin is a member which rests between roof trusses and support roof sheeting. I-sections, channel, angles, cold formed, C or Z-section are commonly used as purlins. The purlins are spaced at main rafter or trusses.