

## UNIT-5

### DESIGN OF GANTRY GIRDERS

[Ref: Design of steel structures - Dr. S.S. Bhavikatti]

Travelling overhead crans are commonly used in factories and work shop to lift and move heavy materials and assembled part from one point to

other. The crane system consist of a bridge over which a Crab (trolley) hoist and cabin one point which houses the control and operator

move. Ref fig 11.1) The crane bridge (girder) it self is provided with wheels to move over the rail ways provided over gantry girder. Thus gantry girder support crane girder.

The gantry is supported on the columns with bracket figure.

The size of crab wheel spacing etc. depend on the capacity of the crane. this details are standard is used the manufacturer supplying them. The cranes may be supported them.

#### Loads:

The following imposed loads should be considered them.

1) vertical loads from Crane

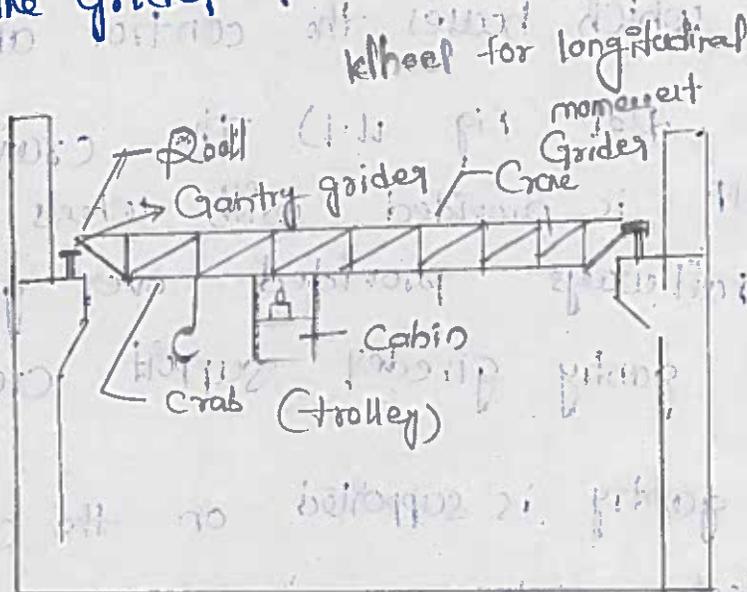
2) Impact loads on frame Crane

3) Longitudinal horizontal force along the

Crane rail

4) Lateral thrust (sweyer) across the Crane rail

In calculating the above design should be positioned such that it give maximum forces in the girder



### Additional impact load

Type of load	Impact Allowance
1) vertical load	25% of maximum static wheel loads
a) for electric operated Cranes	10% of maximum wheel loads
b) for hand operated cranes	
2) Horizontal force transverse to Rail ways	10% of weight of trolley
(a) for electrical operated	15% of weight of trolley
(b) for hand operated cranes.	5% of the static wheel loads

## DESIGN procedure:-

the design it is assumed that

- (a) entire section resists vertical loads
- (b) compression flange with channel resists the horizontal forces.

the following steps may be followed in design.

① with suitable positioning of crane determine maximum moment and shear force on gantry

girders. Add impact load contribution to it

through the position for maximum moment on

due to wheel load is slightly away from the

centre of the girder (under the wheel), it

is just added to maximum moment due to cell

② Calculate horizontal bending moment due to surge load

③ Calculate shear forces due to vertical and

horizontal forces

④ selection of trial section. The design is by method of trials. A trial section is to be selected

(a) The economical depth of about  $\frac{1}{12}$  the span

(b) compression flange width may be kept  $\frac{1}{25}$  th span

(c) compression flange width may be

kept  $\frac{1}{25}$  th span.

5 Calculate  $I_{zz}$ ,  $I_{yy}$  and  $Z_p$  of the I rail section selected

6 Check for buckling resistance as per clause 8.22 of IS Code

7 Check for web biaxial bending

8 Check for shear capacity

9 Check for web buckling and web bending

10 Check for deflection

12 Design the weld

Example:-

Design a simply supported gantry to carry over head travelling crane. Given

Span of gantry girder = 6.5 m

Span of crane girder = 1.6 m

Self weight of crane girder excluding trolley = 200 kN

Self weight of trolley = 50 kN

Minimum hook approach = 1.0 m

Distance between wheels = 3.5 m

Self weight of rails = 0.3 kN/m

Solution:-

1) Moments:-

Load of Maximum moment:-

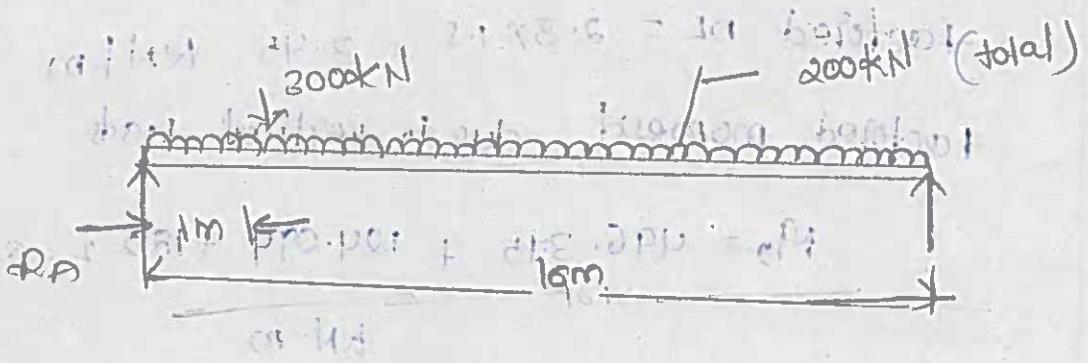
Weight of trolley + lifted load = 250 + 50 = 300 kN

Self weight of crane girder = 200 kN/m

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for maximum reaction gantry the moving loads should be as close to gantry as possible show the load position



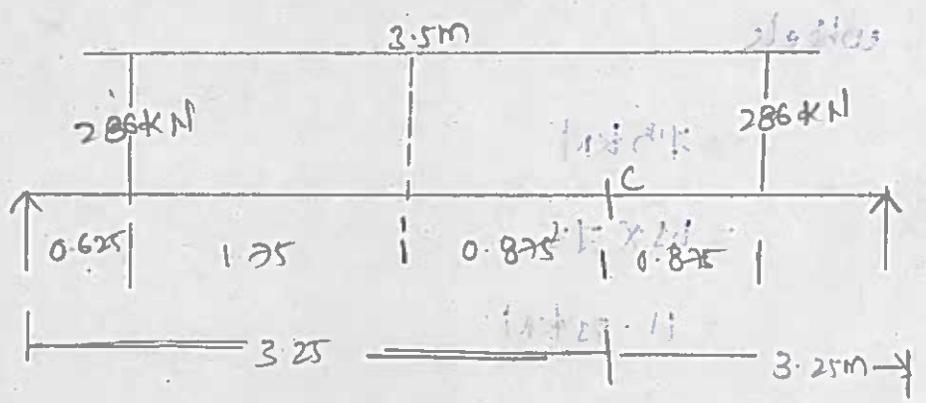
$$R_A = \frac{300 \times 15 + 200 \times 16}{16} = 381.25 \text{ kN}$$

This load is transferred to gantry girders through two wheels

load on gantry from which wheels

$$\text{factor wheel load} = 199.63 \times 1.5 = 299.45 \text{ kN}$$

Maximum



$$R_B = \frac{286 \times 0.625 + 286 \times (3.25 + 0.875)}{6.5} = 209 \text{ kN}$$

$$\text{Max-moment} = 209 \times 23.75 = 496.375 \text{ kN-m}$$

$$\text{moment due to impact} = 0.25 \times 496.375 = 124.094 \text{ kN-m}$$

Assume self weight of girder =  $2 \text{ kN/m}$

$$\therefore \text{Dead load due to self weight + rails} = 2 + 0.3$$

$$\text{factored DL} = 2.3 \times 1.5 = 3.45 \text{ kN/m}$$

factored moment due to vertical loads

$$M_z = 496.375 + 124.094 + 18.22 = 63.689 \text{ kN-m}$$

Maximum moment due to horizontal force (sugar)

Horizontal force

Horizontal force transverse to rail = 10% of weight of trolley plus load fitted

$$= \frac{10}{100} (250 + 50) = 30 \text{ kN}$$

Assuming moment flared wheels this is distributed  
4 wheels

$$= 7.5 \text{ kN}$$

$$= 1.5 \times 7.5$$

$$= 11.25 \text{ kN}$$

for maximum moment in gantry the position of loads is same as shown 11.7 except that it is horizontal.

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$$M_y = \frac{11.25}{286} \times 496.275 = 19.225 \text{ kN-m}$$

shear force

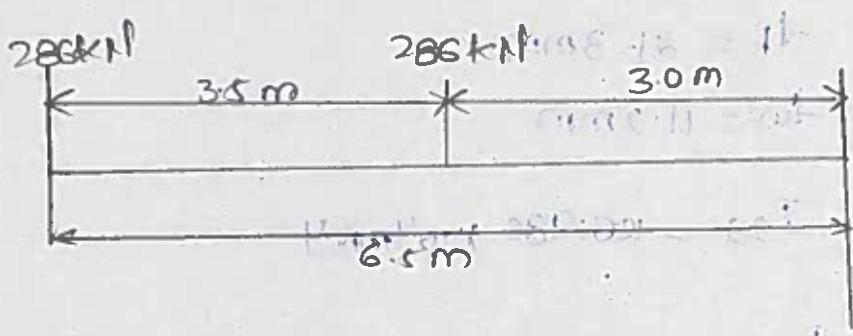
for the maximum shear force gantry girder, the trailing wheel should be just on the girder

$$\text{vertical shear due to wheel loads} = 286 + \frac{286 \times 30}{6.5}$$

$$= 418 \text{ kN}$$

$$\text{vertical shear to Impact} = 0.25 \times 418$$

$$= 104.5 \text{ kN}$$



vertical shear due to self weight =

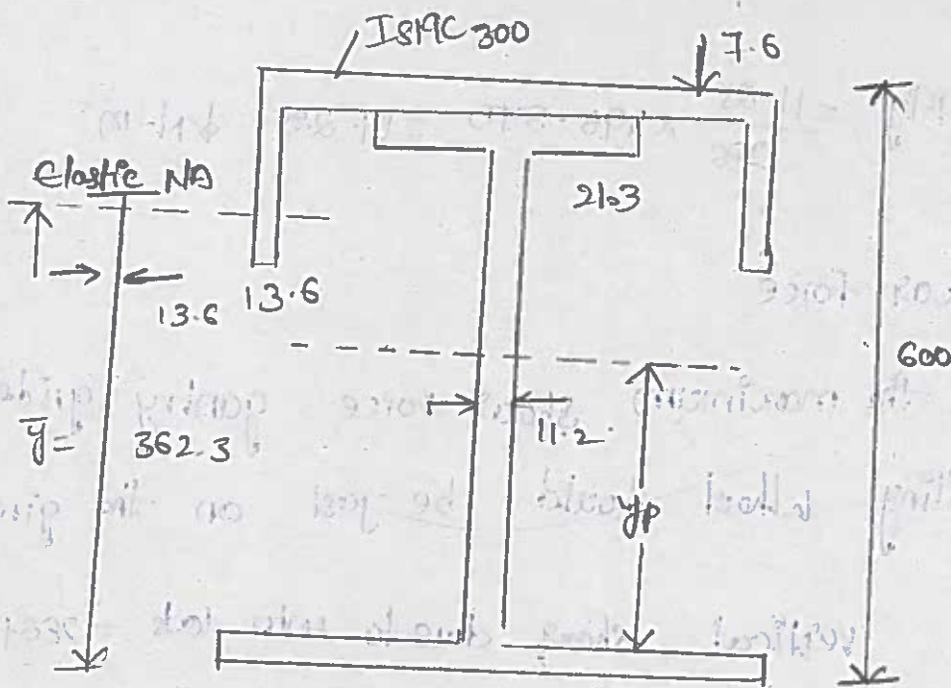
$$3.45 \times \frac{6.5}{2} = 11.1 \text{ kN}$$

$$\therefore \text{Total vertical shear} = 418 + 104.5 + 11.21 = 533.71 \text{ kN}$$

Preliminary section

$$\frac{L}{12} = \frac{6500}{12} = 541.2$$

$$\frac{L}{25} = \frac{6500}{25} = 260 \text{ mm}$$



Properties of ISMB 600 @ 1.312 kN/m

$$A = 11038 \text{ mm}^2$$

$$b = 250 \text{ mm}$$

$$t_f = 21.3 \text{ mm}$$

$$t_w = 11.2 \text{ mm}$$

$$I_{xx} = 1061986 \times 10^4 \text{ mm}^4$$

$$I_{yy} = 47025 \times 10^4 \text{ mm}^4$$

Properties of ISMC 300

$$A = 4654 \text{ mm}^2$$

$$b = 90 \text{ mm}$$

$$t_w = 7.6 \text{ mm}$$

$$I_{xx} = 6362.6 \times 10^4 \text{ mm}^4$$

$$I_{yy} = 3108 \times 10^4 \text{ mm}^4$$

$$C_{yy} = 33.6 \text{ mm}$$

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let distance of N-A from the

$$\bar{y} = \frac{17088 \times 200 + 4654 \times (600 + 7.6 - 23.6)}{1708 + 4564}$$

$$= 360.0 \text{ mm}$$

$$I_{zz} = 106198.5 \times 10^4 + 17038 (360 - 360)^2 + 310.8 \times 10^4$$

$$= 1127.452 \times 10^6 \text{ mm}^4$$

$$Z_e = \frac{I_{zz}}{y_{max}} = \frac{21.3 \times 250^3 + 6362.6 \times 10^4}{9136.04 \times 10^6 \text{ mm}^4}$$

$Z_e$  for compression flange about y-axis

$$I = \frac{1}{12} \times 21.3 \times 250^3 + 6362.6 \times 10^4 = 9136.04 \times 10^4 \text{ mm}^4$$

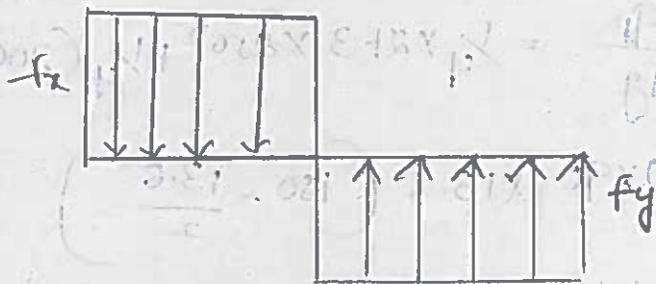
$$Z_{ey} \text{ for compression flange} = \frac{9136.04 \times 10^4}{150} = 609.069 \times 10^3 \text{ mm}^3$$

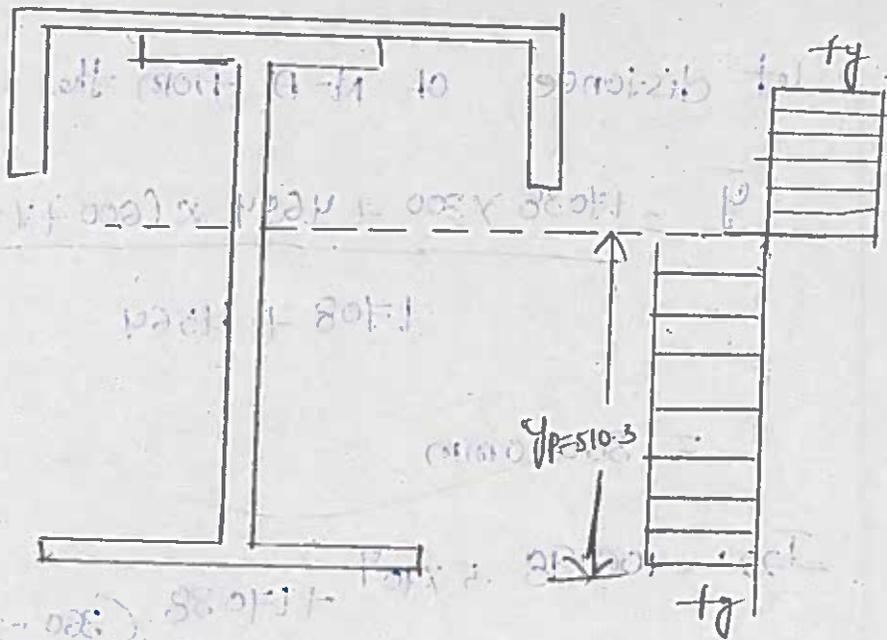
plastic modulus of section =  $17038 + 4564 = 21602 \text{ mm}^2$

let plastic N-A be at distance  $Y_p$  from tension flange then

$$(Y_p - 213) \times 11.2 + 250 + 213 = \frac{21602}{2}$$

$$\therefore Y_p = 510.2 \text{ mm}$$





$\therefore M_p = \sum \text{moment of forces at yield about plastic}$

$$= 21.3 \times 250 \left( 510.2 - \frac{21.3}{2} \right) f_y + \left( \frac{510.2 - 21.3}{2} \right)^2 \times 11.2 f_y$$

$$+ \frac{(600 - 21.3 - 510.2)^2}{2} \times 11.2 f_y + 21.3 \times 250 \left( \frac{600 - 21.3}{2} - 510.2 \right) f_y$$

$$+ 4564 (600 + 13.6 - 25.6 - 510.2) f_y$$

$$= 4686450 f_y$$

$$\therefore Z_p = \frac{M_p}{f_y} = 4686450 \text{ mm}^3$$

for top flange

$$Z_{pf} = \frac{M_p}{f_y} = \frac{1}{4} \times 21.3 \times 250^2 + \frac{1}{4} (300 - 2 \times 13.6)^2$$

$$\times 7.6 + 2 \times 90 \times 13.6 \left( 150 - \frac{13.6}{2} \right)$$

$$= 824.784 \times 10^3 \text{ mm}^3$$

Check for moment Capacity

$$\frac{b}{t} \text{ of flang of IS KLB } 600 = \frac{250 - 11.2}{2 \times 21.3} = 5.6284$$

$$d/t \text{ of klob of IS KLB } 600 = \frac{600 - 2 \times 21.3}{11.2} = 49.78284$$

$$\text{and } b/t \text{ of flange of channel} = \frac{90 - 7.6}{13.6} = 6.06284$$

Hence it is plastic section

Local moment Capacity for bending in vertical plane

$$M_{dy} = \frac{f_y Z_p}{1.1} = \frac{250}{1.1} \times 486450 = 10651 \times 10^3 \text{ N-mm}$$
$$= 1065.1 \text{ kNm-m}$$

$$\frac{1.2 Z_e f_y}{1.1} = \frac{1.2 \times 313.18 \times 10^4 \times 250}{1.1} = 854.127 \text{ kNm-m}$$

$$= 854.127 \text{ kNm-m}$$

for top flang

$$M_{dy} = \frac{f_y Z_p}{1.1} = \frac{250}{1.1} \times 824.784 \times 10^3$$

$$= 187.446 \times 10^6 \text{ N-mm}$$

$$= 187.446 \text{ kNm-m}$$

$$\frac{1.2 Z_e f_y}{1.1} = \frac{1.2 \times 609.069 \times 10^3 \times 250}{1.1} = 166.11 \times 10^6$$

N-mm.

∴ for the flange  $M_{dy} = 166.11 \text{ kN-m}$

Check for Combined Local Capacity:

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1$$

$$\frac{638.689}{854.127} + \frac{19.855}{166.11} = 0.86521$$

Check for Buckling Resistance [Clause 8.22-1]

$$M_d = \beta_b \gamma_p f_{bd}$$

for plastic section  $\beta_b = 1.0$

$$M_d = \gamma_p f_{bd}$$

$$f_{cr} = \frac{1.1 \pi^2 E}{(L_{LT}/r_y)^2} \left[ \frac{I_y}{I_x} \left( \frac{L_{LT}/r_y}{h_f/t_f} \right)^2 \right]^{0.5}$$

$$L_{LT} = 6500 \text{ mm} \quad E = 2 \times 10^5 \text{ N/mm}^2 \quad h_f = 600 + 7.6 = 607.6 \text{ mm}$$

$$I_y = 4702.5 \times 10^4 + 6382.6 \times 10^4 = 11085.1 \times 10^4 \text{ mm}^4$$

$$A = 17038 + 4564 = 21602 \text{ mm}^2$$

$$\therefore r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{11605 \times 10^4}{21602}} = 71.57 \text{ mm}$$

$$\therefore f_{crb} = \frac{1.1 \times \pi^2 \times 2 \times 10^5}{(6500/71.57)^2} \left[ \frac{1}{20} \left( \frac{6500 \times 1.56}{607.6/21.3} \right)^2 \right]^{0.5}$$

$$= 323.06 \text{ N/mm}^2$$

Table 14 of IS 800 also may be used to find  $f_{bd}$  from Table 13(a)

$$f_{bd} = 167.8 \text{ N/mm}^2$$

$$M_{dz} = 1.0 \times 167.8 \times 4686450 = 786.39 \times 10^6 \text{ N-mm}$$

$$= 786.39 \text{ kN-m} > 638.689 \text{ kN-m}$$

Hence the section is adequate

$$M_{dy} = \frac{f_y z_y}{1.1}$$

$$z_y = \frac{I_y}{150} = \frac{11065.1 \times 10^4}{150} = 737.67 \times 10^3 \text{ mm}^2$$

$$\therefore M_d = \frac{220}{1.1} = 737.67 \times 10^3 = 167.65 \times 10^6 \text{ N-mm}$$

$$\therefore \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{638.689}{786.54} + \frac{19.505}{167.65}$$

$$= 0.982 < 1$$

Hence adequate

Check for shear

$$V_z = 533.71 \text{ kN}$$

$$\text{Shear capacity} = \frac{A_v f_y}{\sqrt{3} \times 1.1} = \frac{600 \times 11.6 \times 250}{\sqrt{3} \times 1.1}$$

$$= 913 \times 10^3 \text{ N} = 913 \text{ kN} > 533.71 \text{ kN}$$

$$0.6 \times 913 = 547.8 \text{ slightly less than } v$$

weld stress :-

$$\text{Stress shear} = q = \frac{V}{bI} (\bar{a}\bar{y})$$

$$\therefore \text{shear per unit length} = \frac{V}{I} (\bar{a}\bar{y})$$

$$V = 549.4 \text{ kN}$$

$$a = \text{area of channel} = 5646 \text{ mm}^2$$

$$\bar{I} = I_2 = 1207.28 \times 10^6 \text{ mm}^4$$

$$\bar{y} = \text{distance of L.G of channel from N.A} = 600 + 70 \\ - 29.6 - 362.3 = 224.3 \text{ mm}$$

$$\therefore \text{shear force per unit length } q = \frac{533.71 \times 10^3}{1207.452 \times 10^6} \\ (4564 \times 224.3)$$

$$= 484.6 \text{ N/mm}$$

$$= 22 \times 0.7 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.05} = 265.125 \text{ N/mm}$$

Equating it so shear force we get

$$265.125 = 484.62$$

$$S = 1.83 \text{ N/mm}$$

Hence provide 5mm intermitten fillet weld  
which is minimum on both sides of welding

$$\frac{1.83}{5} \times 100 = 36.6 \text{ provided}$$

# Design web bulking:-

$$d = 600 - 2(21.3 + 17) = 520.4 \text{ mm} \quad t = 11.2 \text{ mm}$$

$$\therefore d/t = \frac{520.4}{11.2} = 46.4 > 67 \text{ Hence need to check}$$

Check for deflection

At working load, deflection is to be limited to  $\frac{L}{750}$

for maximum deflection wheel load is shown

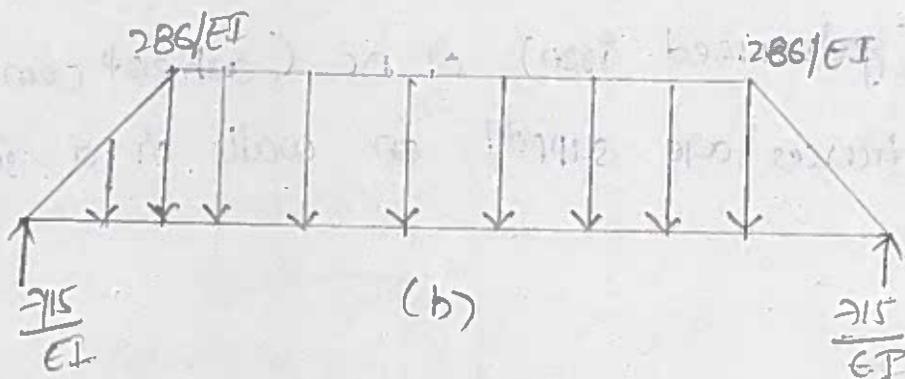
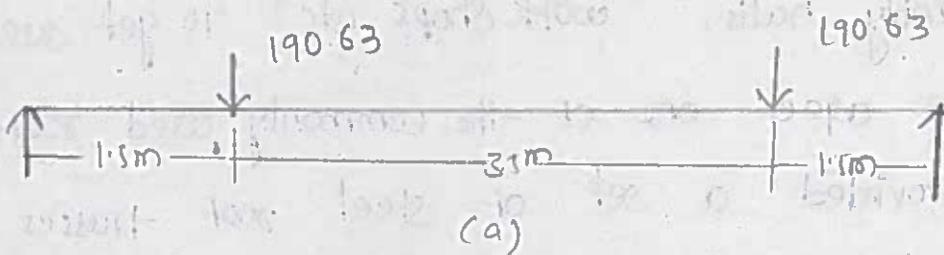
Shows the conjugate beam with  $\frac{M}{EI}$  diagram

Reaction in conjugate beam

$$R_1 = \frac{1}{2} \text{ total } \frac{M}{EI} \text{ diagram}$$

$$= \frac{1}{2} \times 1.5 \times \frac{286}{EI} + \frac{286}{EI} \times 1.75 = \frac{715}{EI}$$

Maximum deflection occurs at mid span = moment of  $\frac{M}{EI}$  load in conjugate beam



$$EI \Delta = 751 \times \frac{6.5}{2} - \frac{1}{2} \times 286 \times 1.5 \times 2.75 \times 286 \times \frac{1.75}{2}$$

$$\Delta = 1295.9$$

Taking EI in kN-m<sup>2</sup> unit

$$EI = 2 \times 10^5 \times 1207.8 \times 10^6 \times \frac{1}{10^9} = 200 \times 1207.8 \text{ kN-m}^2$$

$$\therefore \Delta = \frac{1295.9}{200 \times 1207.8} = 5.75 \times 10^{-3} \text{ m} = 5.75 \text{ mm}$$

Permissible  $\Delta = \frac{L}{750} = \frac{6500}{750} = 8.60 \text{ mm}$

deflection requirement is satisfied

Hence use ICB 600 with ISMC 300 on

Compression flange as shown in fig

## DESIGN OF ROOF TRUSSES

Large column-free areas are required for auditorium assembly halls, work shops etc. To get such column

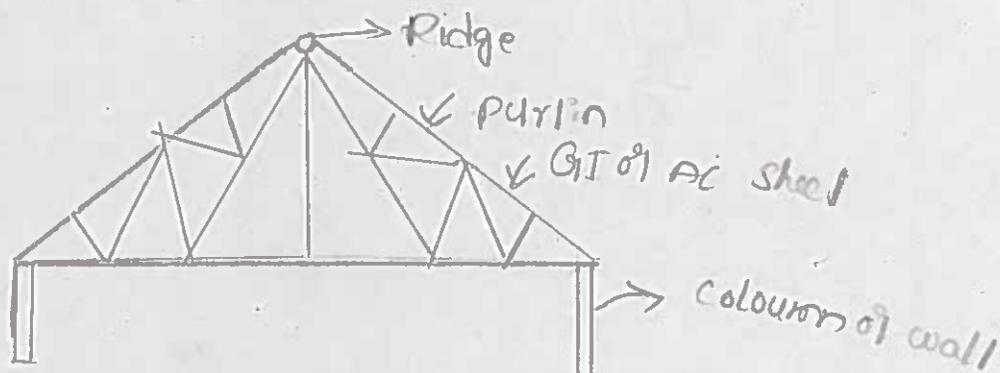
-free area one of the commonly used roofing system

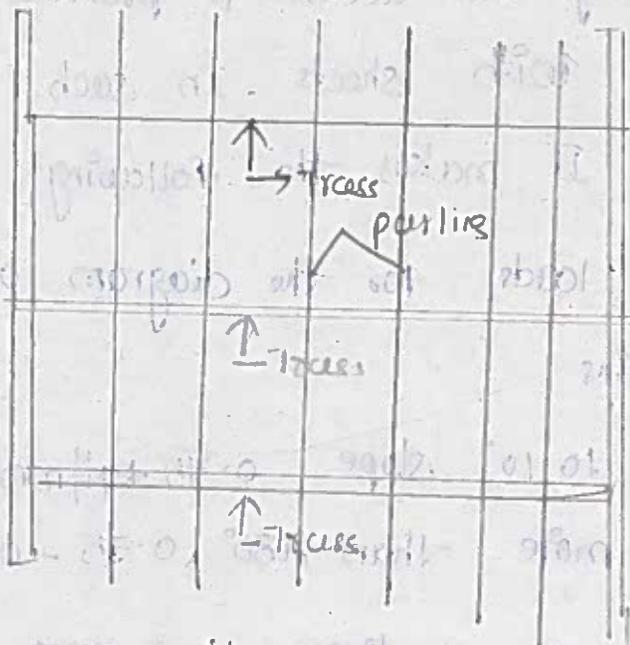
is provided a set of steel roof trusses inter

connected with purlin which in turn support GI

(galvanized iron) or AC (asbestos cement) sheets the roof

trusses are support on walls of a series of column





## Loads:

The main loads on trusses are:-

- (i) Dead loads
- (ii) Imposed loads
- (iii) Wind loads
- (iv) Other loads:

Dead loads:-

It include the weights of sheeting, purlings, bracings, self weights and other load supports from trusses

The units weights of various material are given

In 875 - part I. the following values may be noted

- (i) GI sheets :  $85 \text{ N/mm}^2$
- (ii) A.C sheets :  $130 \text{ N/mm}^2$

## Imposed load (Live load)

Normally no access is provided for sloping roofs with sheets. In such cases IS 875 part II makes the following provision for live loads for the diagram of sheets and purlins

up to  $10^\circ$  slope  $0.75 \text{ kN/m}^2$

for more than  $180^\circ$   $0.75 - 0.02 (0 - 10)$

slope of sheetings However a minimum of  $0.4 \text{ kN/m}^2$  live load should be considered in any case

## Wind loads:-

IS 875 part 3 gives guidelines to determine wind forces on different components of buildings. It consists of the following steps

- (a) determine basic wind speed
- (b) obtained design wind speed
- (c) calculate design wind pressure
- (d) calculate wind pressure on roof
- (e) Basic wind speed:-

for finding basic wind pressure in any place in India IS 875 (part 3) divides the country into six zones, 3 seconds duration over 9 provided

(b) Design Wind Speed:-

The design wind speed for any site may be obtained as

$$V_d = k_1 k_2 k_3 V_b$$

where  $k_1$  = risk coefficient

$k_2$  = Terrain height and structure size factor

$k_3$  = topography factor

(i) Risk coefficient ( $k_1$ )

depending on important of the building and basic wind speed IS 875 has developed an equation to determine risk coefficient  $k_1$  for different types of buildings

Terrain height and structure size

The co-efficient depends on the terrain of the building site height of building and the class the value in the tabular form as shown in

table in which specific structure stands shall be assessed as belongs to the following categories.

categories 1:- Exposed open terrain with a few or no obstructions and which average height of any object surrounding the structure is less than.

A roof truss shed is to be built in Lucknow for an industry. The size of shed is  $24\text{ m} \times 40\text{ m}$  the height of buildings is  $12\text{ m}$  at eaves. determine the basic wind

from wind zone Map of country (IS 875 part 3). the basic wind speed in Lucknow is

$$V_b = 47\text{ m/sec}$$

Risk coefficient  $k_1$  from table 12.1 for all general building

$$V_b = 47\text{ m/sec}$$

$$k_1 = 1.0$$

$$k_2 = 0.88 \text{ if } h = 10\text{ m}$$

$$= 0.94 \text{ if } h = 15\text{ m}$$

$$\therefore \text{for } h = 12\text{ m}$$

$$k_2 = 0.88 + (0.94 - 0.88) \times \frac{2}{5} = 0.904$$

$$\therefore k_3 = 1 + C_s$$

$$\text{where } C_s = z/L = 0$$

$$\therefore k_3 = 1.0$$

design with speed

$$V_d = k_1 k_2 k_3 V_b$$

$$= 1.0 \times 0.904 \times 1.0 \times 47$$

$$= 42.488\text{ m/sec}$$

$$P_d = 0.6 V_d^2 = 0.6 \times 42.488^2$$

$$= 1083\text{ N/m}^2$$

$$= 1083\text{ N/m}^2$$

$$P_d = 1.083\text{ kN/m}^2 \text{ Answer}$$

unit - 5, Pg no - 20/20