PAPER-AS-4210

DEPARTMENT OF CIVIL ENGINEERING, INSTITUTE OF TECHNOLOGY, GGV, BILASPUR

BTECH VIIth SEMESTER, CIVIL ENGINEERING

SUBJECT: STRUCTURAL ENGINEERING DESIGN-III

CLASS- **BTECH VIIth SEMESTER** SUBJECT CODE:**CE41T01** BRANCH-**CIVIL ENGINEERING**

TIME: **3 HOURS** MAX. MARKS: **60**

NOTE: 1) All questions of section-A is compulsory
2) Answer any one part from each unit of section-B

SECTION-A (10x2=20 Marks)

A-I	What are the functions of a web in a plate girder?	**02**
A-II	Why transverse stiffeners are provided in a plate girder.	**02**
A-III	What is a flexible connection?	**02**
A-IV	Detail how a bracket is connected perpendicular to the flange of a column using butt weld	**02**
A-V	What is metal-active gas welding?	**02**
A-VI	Show how cusping distortion occur due to a single V butt weld?	**02**
A-VII	Why impact loads are to be accounted in design of a gantry girder.	**02**
A-VIII	What is a howe roof truss? When it is used?	**02**
A-IX	Specify the situations of suitability for a deck type plate girder bridge?	**02**
A-X	Comment on wind load consideration in bridge design?	**02**

SECTION-B (5x8=40 Marks)

UNIT-I

B-I What is meant by design of a plate girder? Describe the steps in the design of a plate girder? **08**

or

A welded plate girder of span 22m is without intermediate transverse stiffeners and is also laterally restrained throughout. A uniform load of 170 kN/m is acting on the girder including its self weight. The steel for the flanges and web plates is of grade Fe 410. Design the cross section along with the flange to web connections. Assume any missing data suitably.

UNIT-II

B-II Design a seat connection for a factored beam end reaction of 120kN. The beam section is ISMB 250@365.9 N/m connected to the flange of column section ISHB 200@365.9 N/m using bolted connections. Steel is of grade Fe410 and bolts are of grade 4.6. **08**

or

A Bending moment of 200 kNm under factored loads is to be transmitted by the flange ISMB 500 @ 852.5 N/m to a column ISHB 300 @ 576.8 N/m. in addition to an end reaction of 300 kN. Design a welded connection. Assume any missing data suitably.

UNIT-III

B-III Explain with neat sketches, some of the common defects in welds **08**

or

Describe in detail the following activities i) straightening, ii) bending, iii) rolling, iv) fitting and v) reaming in fabrication process

UNIT-IV

B-IV Discuss how the design actions of forces are obtained on a gantry girder **08**

or

Obtain the design the section of a gantry girder, for a crane capacity of 350kN. Self weight of crane and trolley 200 kN. Minimum approach to of the crane hook to the gantry is given as 2.5 m. wheel base is 3.2m. Distance between gantry rails is 17m. Distance b/w columns are 10m. Self wt of rail is 210 kN/m. Diameter of crane wheels is 150mm. Steel Fe410. Fu= 410 N/mm^2, $f_y=f_{yw}=f_{yf}=250$ Mpa. Assume any missing data suitably.

UNIT-V

B-V Write about the economic proportions of a truss bridge for i) No of panels, ii) inclination of diagonals. **08**

or

(b) Show the arrangement of components of deck type truss girder bridge. Give the design steps

X

KEY TO PAPER-AS-4210

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BTECH VII[th] SEMESTER, CIVIL ENGINEERING

SUBJECT: STRUCTURAL ENGINEERING DESIGN-III

SECTION-A (10x2=20 Marks)

Q1 (i) What are the functions of a web in a plate girder? 02

Ans. *Web provides most of the shear strength in a plate girder and indirectly assists to keep the flange plates as far as possible away from the neutral axis catering the flexural strength*

Q1(ii) Why transverse stiffeners are provided in a plate girder? 02

Ans. *Intermediate transverse stiffeners are provided in a plate girder to increase the buckling resistance of the web caused by shear*

Q1(iii) What is a flexible connection? 02

Ans. *The beam to column connections expected to resist and transfer end reactions only are termed as shear connections or flexible connections. These permit free rotation of the beam end and do not have any moment restraint*

Q1(iv) Detail how a bracket is connected perpendicular to the flange of a column using butt weld 02

Ans.

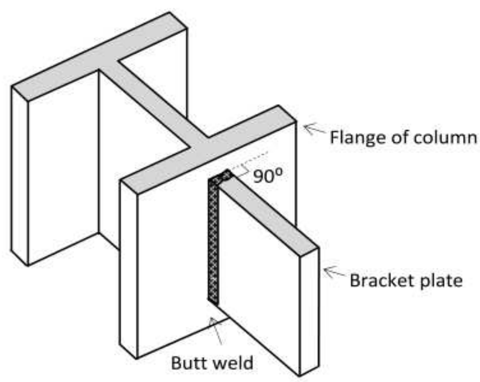


Figure: Detail of a bracket plate connection perpendicular to column flange using Butt weld

Q1(v) What is metal-active gas welding? 02

Ans. *A welding with a gas that does not react with molten steel shields the arc and the weld pool is also called as Metal-inert Gas (MIG) welding.*

Q1(vi) Show how cusping distortion occur due to a single V butt weld? 02

Ans. *Cusping distortion is as shown in figure below*

Q1(vii) Why impact loads are to be accounted in design of a gantry girder. 02

Ans. *The stresses produced in gantry girders due to vertical, lateral and longitudinal loads are more than those caused by gradually applied loads. This is due to the forces set up by the sudden application of brakes to the rapidly moving loaded cranes, acceleration,* *retardation, vibration, possible slip of slings, etc.*

Q1(viii) What is a howe roof truss? When it is used? 02

Ans.

A Howe truss is as shown in figure

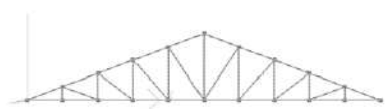


Figure: Howe Truss

Howe Trusses are generally used for spans in the range of 6m to 24m

Q1(ix) Specify the situations of suitability for a deck type plate girder bridge? 02

Ans. *Normally a deck type plate girder bridge is preferred where sufficient head room can be provided In railway bridges. Plate girder bridges are economically used for railway bridges for spans upto 20m and for highway bridges for spans upto 30m*

Q1(x) Comment on wind load consideration in bridge design? 02

Ans. *In a plate girder bridge the wind load acting on the bridge has two effects i) Overturning effect (wind load acting through the C.G. of the exposed area will cause overturning effect) and ii) Horizontal truss effects (bending of the truss in the horizontal plane cause stresses in the wind ward and leeward girders)*

SECTION-B (5x8=40 Marks)

UNIT-I

Q2/i What is meant by design of a plate girder? Describe the steps in the design of a plate girder? 08

Ans. *Design of a plate girder means determining the most efficient and economical size of the web and flanges. By Using slender webs economical design of plate girders can be achieved. With a high strength to weight ratio can be designed by incorporating the post-buckling strength of the web in the design method. Provisions of stiffeners are also made like intermediate, bearing and torsional stiffeners. Finally, the connections between the various elements are designed (welded or bolted).*

The design steps are as follows

i. *First from the given situations/data it is to be ascertained that the girder is laterally restrained throughout or not. In case if the girder is laterally restrained the situation of lateral torsional buckling does not arise and may result economical section compared to a laterally unrestrained section. In case if the girder is laterally not restrained the lateral torsional buckling of the section is required to be checked for its safety.*
ii. *Secondly, with an assumed self weight of the plate girder and estimated imposed loads the critical bending moment and shear force in the girder are computed and partial safety factors are applied to the obtained actual design actions.*
iii. *The optimum thickness of the plate girder is determined by using the eqn.*

And the depth of the plate girder is determined by using the eqn.

$$d = \left(\frac{M_z k}{f_y}\right)^{0.33}$$

Where k is assumed initially.

iv. *The flange area is computed by the equation*

$$A_f = \left(\frac{M_z \gamma_{m0}}{f_y d}\right)$$

Keeping view on the out stand of the flange plate, flange plate is provided from serviceability and buckling criterion. Once the width of the flange is fixed, the flange thickness is calculated by dividing the flange area with the provided width.

v. *The flange is classified. Plastic flanges are preferred. For the flanges to be classified as plastic , the ratio of the outstand and the flange thickness should be* $\frac{b}{t_w} < 8.4\,\epsilon$ *where b=outstand of flange =* $\frac{b-tw}{2}$

vi. *The trial section is checked for bending. The moment capacity of the beam is determined and should be greater than the applied moment.*

vii. *The flanges are curtailed in length to achieve the most economic design.*

viii. *The girder is checked for the maximum shear. The shear strength is a function of the web slenderness ratio* d/t_w *and aspect ratio c/d, where c is the spacing of the intermediate transverse stiffeners, d is the depth of web and tw its thickness. It is ascertained whether intermediate transverse web stiffeners will be required or not. The web panel is then checked by using either post-critical method or tension field action method. The shear capacity of the web comprises of its strength before buckling and the post-buckling strength. The post buckling strength relies on tension field action which is made possible by the presence of intermediate stiffeners.*

ix. *Stiffeners are designed as per the requirements*

x. *Design of web to flange connections and that for stiffeners is carried.*

xi. *All the web and flange splices are made using full strength butt weld or non-slip bolted connections*

or

Q2/ii A welded plate girder of span 22m is without intermediate transverse stiffeners and is also laterally restrained throughout. A uniform load of 170 kN/m is acting on the girder including its self weight. The steel for the flanges and web plates is of grade Fe 410. Design the cross section along with the flange to web connections. Assume any missing data suitably. 08

Ans.

Sol <u>Given data</u>

For Fe 410 grade of steel:

f_u =	410	Mpa	Imposed load	=	170	kN/m
f_y =	250	Mpa	Span (l)	=	22	m
μ =	0.3		Self Wt of plate girder=			included in IL
E =	200000	Mpa				

<u>Partial safety factor</u>

γ_{mw} = 1.5 (for site welding)
1.25 (for shop welding)

γ_{m0} = 1.1

$\varepsilon=\varepsilon_w=\varepsilon_f=\sqrt{(250/f_y)}$=SQRT($\frac{250}{250}$) = 1.0

<u>Design Forces</u>

Total superimposed load = 170 kN/m
Factored superimposed load = 255 kN/m
Total uniform factored load =w= 255 kN/m
Maximum bending moment , $M_z = wl^2/8$ = 15427.50 kNm
Maximum shear force = wl/2 = 2805.00 kN

<u>Design of Web</u>

Optimum depth of plate girder,
Assume, k = 180

d = 2065.65 mm

= 2100.00 mm

$d = \left(\frac{M_z k}{f_y}\right)^{\text{[illegible]}}$

when intermediate transverse stiffeners are not to be provided:

$\frac{d}{t_w} \le 200\varepsilon$= 200 (From serviceabilty criteria)

$\frac{d}{t_w} \le 345\varepsilon^2$= 345 (From flange buckling criteria)

Optimum depth of plate girder,

t_w = 12.09 mm

$t_w = \left(\frac{M_z}{f_y k^2}\right)^{\text{[illegible]}}$

But $\frac{d}{t_w}$ = 173.73

Seviceability criteria OK
Flange Buckling Criteria OK

Therefore provide a web plate of
depth (rounded) = 2100.00 mm
Thickness (rounded) = 14 mm

<u>Design of Flanges</u>

It is assumed that the bending moment will be resisted by the flanges and shear by the web

Therefore required area of flange, A_f = 32324.29 mm^2

$A_f = \left(\frac{M_z \gamma_{m0}}{f_y d}\right)$

Assuming width of flange equal to 0.3 times depth of girder

Width of flange b_f = 630 mm rounded to 630 mm
Thickness of flange t_f = 51.31 mm rounded to 52.00 mm

Therefore provide a flange plate of
Width (rounded) = 630.0 mm
Thickness (rounded) = 52.0 mm

<u>Classification of flanges</u>

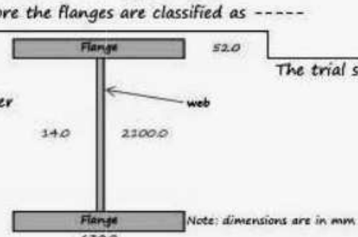
For the flanges to be classifiable as plastic

where, the outstand of the flange,b=(bf-tw)/2

$\frac{b}{t_f} \le 8.4\varepsilon$= $\frac{308}{52.0}$ = 5.9 < 8.4 -Ok

Therefore the flanges are classified as ----- $\boxed{\text{Plastic}}$ Therefore β_b= $\boxed{1}$

Plate Girder



The trial section of the plate girder has been as given in figure

D 2204 mm

Check for bending strength

Plastic section modulus of the trial section of the plate girder is

$Z_{pz} = 2 b_f t_f \frac{(D - t_f)}{2} =$ 70499520 mm^3 | $M_d = \beta_b Z_{pz} \frac{f_y}{\gamma_{m0}}$

Moment capacity, M_d = 16022.62 kNm

$\boxed{\text{Moment Capacity > Mz ---OK}}$

Shear capacity of web

Using the simple post-critical method

$\frac{d}{t_w} = \frac{2100.0}{14.0} = 150$ | Seviceability criteria OK | Flange Buckling Criteria OK

Therefore

Elastic critical shear-stress, $\tau_{cr,e} = \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$ | $\tau_{cr,e}$ = 42.9812 N/mm^2

Kv = 5.35 since the transverse stiffeners are provided at the supports only

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

$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\tau_{cr,e}}}$ | λ_w = 1.83

Shear stress corresponding to buckling τ_b 42.98 N/mm^2

Shear force corresponding to web buckling Vcr = d t_w τ_b

Vcr = 1263.65 kN

$\boxed{\text{Shear NOT OK}}$

Let the web thickness be revised from 14.0 mm to 20 mm

Shear capacity of web

Using the simple post-critical method

$\frac{d}{t_w} = \frac{2100.0}{20.0} = 105$ | Seviceability criteria OK | Flange Buckling Criteria OK

Therefore

Elastic critical shear-stress, $\tau_{cr,e} = \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$ | $\tau_{cr,e}$ = 87.7167 N/mm^2

Kv = 5.35 since the transverse stiffeners are provided at the supports only

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

$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\tau_{cr,e}}}$ | λ_w = 1.28

Shear stress corresponding to buckling τ_b 87.72 N/mm^2

Shear force corresponding to web buckling Vcr = d t_w τ_b

Vcr = 3684.10 kN

$\boxed{\text{Shear OK}}$

Check for lateral torsional buckling

Since the girder is laterally restrained, check for lateral torsional buckling is not required

Flange to web connection

Providing two weld lengths along the span for each flange to web connection

$q_w = \frac{VA\bar{y}}{2I_z}$ $\boxed{I_Z = \frac{b_f D^3}{12} - \frac{(b_f - t_w)d^3}{12}}$

Iz 86676747360 mm^4

Q_w 0.58 kN/mm

Providing a weld size of s mm s = 9 mm

K S = 6.3 mm

Strength of weld per unit length = 0.606 kN/mm

Weld size ok

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UNIT-II

B-II Design a seat connection for a factored beam end reaction of 120kN. The beam section is ISMB 250@365.9 N/m connected to the flange of column section ISHB 200@365.9 N/m using bolted connections. Steel is of grade Fe410 and bolts are of grade 4.6. 08

Sol: Fe 410 grade of steel; $f_u = 410$ Mpa, $f_{yo} = 250$ 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; $\gamma_{mo} = 1.10$

the relevant properties of the sections to be connected are.

Property	ISLB 250	ISHB 200
width of flange, b_f	125 mm	200 mm
Thickness of flange, t_f	12.5 mm	9.0 mm
Thickness of web, t_w	6.9 mm	6.1 mm
Gauge, g	65 mm	55 mm
Radius at the root, R_1	13 mm	

The length of seat angle, B = width of beam flange = 125 mm, ($b_f = 125$ mm)

Bearing length of the seat leg, $b = \dfrac{R.\gamma_{mo}}{t_w f_y} = \dfrac{120 \times 10^3}{6.9} \times \dfrac{1.10}{250}$

$= 76.52$ mm.

Let us provide an angle section with leg length of 80 mm

Provide a clearance of 3 mm between the beam and the column flange.

Required length of outstanding leg $= 76.52 + 3 = 79.52$ mm < 80 mm.

which is all right.

Length of bearing on seat, $b_1 = b - (t_f + R_1)$

$b_1 = 76.52 - (12.5 + 13) = 51.02$ mm.

The reaction of 120 kN may be assumed to be distributed uniformly over the bearing length of 51.02 mm.

Let us try a seat angle 150 x 115 x 15 mm.

Radius at the root of the seat angle, $R_a = 11$ mm.

the distance of end of bearing on seat to root angle

$b_2 = b_1 + c - (t_a + R_a)$

$= 51.02 + 3 - (15 + 11) = 28$ mm.

Moment of the critical section

$M = R \times \dfrac{b_2}{b_1} \times \dfrac{b_2}{2} = 120 \times \dfrac{28.02}{51.02} \times \dfrac{28.02}{2} = 921.99$ kN mm.

Moment Capacity of the angle leg,

$M_d = 1.2\, Z_e \dfrac{f_y}{\gamma_{mo}} = 1.2 \times \left(\dfrac{125 \times 15^2}{6}\right) \times \dfrac{250}{1.10} \times 10^{-3} = 1278.40$ kNmm. > 921.99 kN mm.

Hence ok.

Therefore Provide a seat angle $150 \times 115 \times 15$ mm.

shear Capacity of the outstanding leg (seating leg),

$V_{dp} = B t_a \dfrac{f_y}{\sqrt{3}\, \gamma_{mo}} = 125 \times 15 \times \dfrac{250}{\sqrt{3} \times 1.1} = 246029$ N $= 246.03$ kN > 120 kN. OK.

Connection of seat angle leg with the column flange

Let us provide 20 mm diameter 4.6 grade bolts.

The bolts will be in single shear and bearing.

$A_{nb} = 245\ mm^2$.

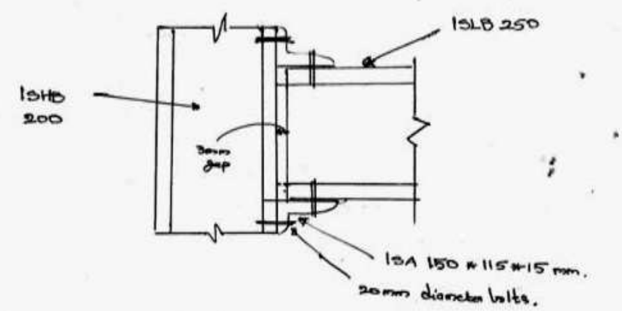
Strength of the bolt in single shear,

$$V_{sb} = A_{nb} \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} = 245 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3} = 45.26\ kN.$$

No of bolts required, $n = \frac{120}{45.26} = 2.65 \simeq 4$

Provide 20 mm diameter bolts of grade 4.6, 4 in numbers in two rows at a pitch of 60 mm.

Provide 20 mm diameter bolts of grade 4.6, 2 in numbers to connect seat leg with the beam flange.



Provide a nominal cleat angle ISA 100 * 75 * 8 mm over the top of the beam flange. Connect the legs of this angle with the flanges of beam and column with two 20 mm diameter bolts each.

— x —

or

A Bending moment of 200 kNm under factored loads is to be transmitted by the flange ISMB 500 @ 852.5 N/m to a column ISHB 300 @ 576.8 N/m. in addition to an end reaction of 300 kN. Design a welded connection. Assume any missing data suitably.

Solution :-

For Fe 410 grade of steel: $f_u = 410$ MPa, $f_y = 250$ MPa.

Partial Safety factor for weld; $\gamma_{mw} = 1.50$

The properties of ISMB 500 @ 852.5 N/m and ISHB 300 @ 576.8 N/m are

Property	ISMB 500	ISHB 300
Depth of Section, D	500 mm	300 mm
Width of flange, b_f	180 mm	250 mm
Thickness of flange, t_f	17.2 mm	10.6 mm
Thickness of web, t_w	10.2 mm	7.6 mm

Provide a plate at the top of beam flange.

Force in the top plate, $P_1 = \frac{M}{D} = \frac{200 \times 10^3}{500} = 400$ kN.

End reaction, P = 300 kN.

Cross-sectional area of the plate $= \frac{P_1 \gamma_{mo}}{f_y} = \frac{400 \times 10^3 \times 1.1}{250}$

$= 1760\ mm^2$.

Let us provide the top plate 180mm wide, tapered to 160 mm at the end.

Thickness of the plate $= \frac{1760}{160} = 11 \simeq 12$ mm.

Let us provide weld size of 8 mm.

Strength of weld per mm. $= 0.7 \times 8 \times \frac{410}{\sqrt{3} \times 1.5} = 883.73$ N/mm.

Length of weld reqd. $= \frac{400 \times 10^3}{883.73} = 452.62 \simeq 460$ mm.

Provide side fillet weld $= \frac{460 - 160}{2} = 150mm \simeq 160$ mm

Total weld length that can be provided $= 2 \times 160 + 160 = 480$ mm.

The length of unwelded portion of the plate $\times$ width of the plate = 180 mm.

$\therefore$ length of top plate $= 180 + 160 = 340$ mm.

The top plate will be connected to the column flange by complete penetration groove weld.

Let us provide a similar plate and connection at bottom flange of the beam.

Let us provide angles on the beam web to transfer the end shear.

Let us provide 6 mm weld for connection.

Strength of weld per mm run = 662.79 N/mm.

Length of weld required for connecting angle leg with beam web $= \frac{300 \times 10^3}{662.79} = 452.6$ mm.

There will be two weld lengths.

Length of weld on each angle leg $= \frac{452.6}{2}$

$= 226.3$ mm $\simeq 240$ mm.

Hence, length of the web angle = 240 mm.

Let us provide 2, ISA : 75 × 75 × 8 mm in size and 240 mm in length.

Stiffeners :-

Force in the stiffeners = $\frac{200\times10^3}{500} = 400$ kN.

Pull in the stiffeners should equal the strength of the weld.

Let us provide 8mm weld size.

The length of the weld required will be = $\frac{400\times10^3}{\left(0.7\times8\times\frac{410}{\sqrt{3}\times1.1}\right)} = 452.6$ mm.

Let us provide stiffener plate for half the depth of column section. The places available to weld the stiffener plate with the column flange will be four in number.

Max. length of column flange available for making connection.

$$= \frac{250-(7.6+2\times5.3)}{2} = 115.9 \text{ mm.}$$

Let us Provide stiffener plates 100mm wide and 12mm thick.

Total length of weld that can be accommodated to connect stiffener with column flange

$$= 4\times(100-5.3) = 378.8 \text{ mm}$$
$$< 452.6 \text{ mm.}$$

The remaining length of the weld can be accomodated on the stiffener plate connecting the column web.

Let us provide 40 mm weld on each side of stiffener plate connecting web of the column.

Total length of weld provided $= 378.8 \text{ mm} + 4\times40$
$= 538.8$ mm
> 452.6 mm

Hence ok.

UNIT-III

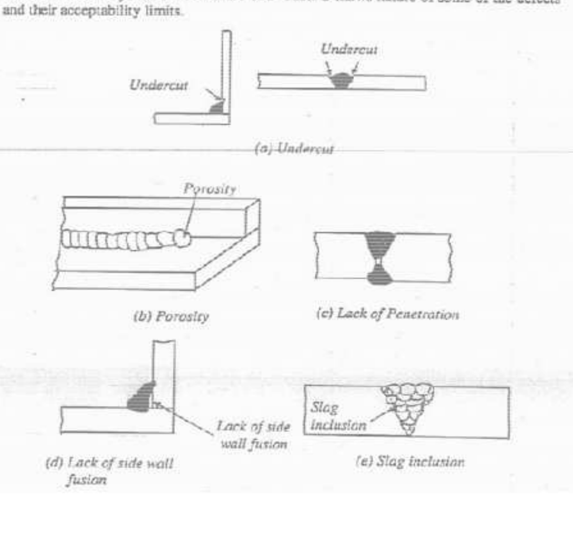
B-III Explain with neat sketches, some of the common defects in welds 08

(i) **Undercut**

(ii) Porosity
(iii) Incomplete Penetration
(iv) Lack of side wall fusion
(v) Slag inclusions
(vi) cracks

All these weld defects are discussed in the chapter on 'Weld – Static and Fatigue strength – I'. It should be emphasised that a 'theoretical 100% error free' weld is not achievable in practice. While good quality welds are the priority of welders and weld inspectors, minor defects do normally creep in. Hence these defects are assessed during a weld inspection.

If the defects are within acceptable limits, they are accepted. If not, alternative measures of rectification may have to be carried out. Table 2 shows nature of some of the defects and their acceptability limits.



(a) Undercut

(b) Porosity

(c) Lack of Penetration

(d) Lack of side wall fusion

(e) Slag inclusion

or

Describe in detail the following activities i) straightening, ii) bending, iii) rolling, iv) fitting and v) reaming in fabrication process

2.4 Straightening, Bending and Rolling

Rolled steel may get distorted after rolling due to cooling process. Further during transportation and handling operations, materials may bend or may even undergo distortion. This may also occur during punching operation. Therefore before attempting further fabrication the material should be straightened. In current practice, either rolls or gag presses are used to straighten structural shapes.

Gag press is generally used for straightening beams, channels, angles, and heavy bars. This machine has a horizontal plunger or ram that applies pressure at points along the bend to bring it into alignment. Long plates, which are cambered out of alignment longitudinally, are frequently straightened by rollers. They are passed through a series of rollers that bend them back and forth with progressively diminishing deformation.

Misalignments in structural shapes are sometimes corrected by spot or pattern heating. When heat is applied to a small area of steel, the larger unheated portion of the surrounding material prevents expansion. Upon cooling, the subsequent shrinkage produces a shortening of the member, thus pulling it back into alignment. This method is commonly employed to remove buckles in girder webs between stiffeners and to straighten members. It is frequently used to produce camber in rolled beams. A press brake is used to form angular bends in wide sheets and plates to produce cold formed steel members.

2.5 Fitting and Reaming

Before final assembly, the component parts of a member are fitted-up temporarily with rivets, bolts or small amount of welds. The fitting-up operation includes attachment of previously omitted splice plates and other fittings and the correction of minor defects found by the inspector.

In riveted or bolted work, especially when done manually, some holes in the connecting material may not always be in perfect alignment and small amount of reaming may be required to permit insertion of fasteners. In this operation, the holes are punched, *4 to 6 mm* smaller than final size, then after the pieces are assembled, the holes are reamed by electric or pneumatic reamers to the correct diameter, to produce well matched holes.

UNIT-IV

B-IV Discuss how the design actions of forces are obtained on a gantry girder **08**

The design actions are as per the solved example below without specific sizes but with proper illustrations in terms of figures with standard nomenclature

or

Obtain the design the section of a gantry girder, for a crane capacity of 350kN. Self weight of crane and trolley 200 kN. Minimum approach to of the crane hook to the gantry is given as 2.5 m. wheel base is 3.2m. Distance between gantry rails is 17m. Distance b/w columns are 10m. Self wt of rail is 210 N/m. Diameter of crane wheels is 150mm. Steel Fe410. Fu= 410 N/mm², $f_y=f_{yw}=f_{yf}$=250 Mpa. Assume any missing data suitably.

Sol:- For Fe410 grade of steel: $f_u = 410$ MPa, $f_y = f_{yw} = f_{yf} = 250$ MPa

For hand-operated OT crane

Lateral loads = 5% of Max. static wheel load.

Longitudinal loads = 5% of weight of crab and weight lifted

Max. Permissible deflection = $\frac{L}{500}$

Partial safety factor — $\gamma_{m0} = 1.10$

$\gamma_{mw} = 1.50$ (for site welds)

Load factor —— $\gamma_{m1} = 1.50$

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

<u>Design force</u>:-

Maximum wheel load:

Maximum concentrated load on crane = 350 kN + 40 kN

(Assume weight of trolley, motor etc, 40 kN) = 390 kN

Max. factored load on crane = 1.5 * 390 = 585 kN.

∴ self wt of crane = 200 − 40 = 160 kN

the crane girder will carry the (its) self wt as a uniformly distributed load.

$= \frac{160}{17} = 9.41$ kN/m.

Factored Uniform load $= 1.5 * 9.41 = 14.115$ kN/m.

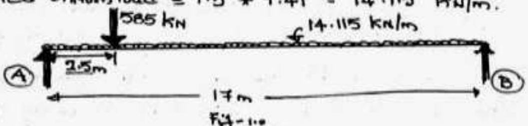


Fig-1.0

For maximum reaction on the gantry girder the loads are placed on the crane girder as shown in figure 1.0

. Taking moment about B,

$R_A * 17 = 585(17-2.5) + 14.115 * 17 * (\frac{17}{2})$

$\therefore R_A = 618.948$ kN ≈ 619 kN

$R_B * 17 = 585 * 2.5 + 14.115 * 17 * (\frac{17}{2})$

$R_B = 206.00$ kN.

the reaction from the crane girder is distributed equally on the two wheels at the end of the crane girder.

$\therefore$ Max. wheel load on each wheel of crane $= \frac{619}{2} = 309.5$ kN. ≈ 310 kN.

Maximum bending moment:

Maximum moments are caused by the moving wheel loads on the gantry girder and self weight of the gantry girder.

For maximum bending moment, the wheel loads shall be placed as shown in Fig. 2.0

the calculation of maximum bending moments due to wheel loads and self weight of gantry girder has been done separately because calculation of impact load and bending moment due to it involve live load only.

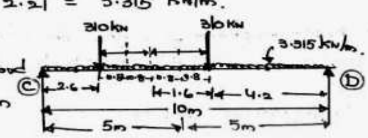
Assume self-weight of gantry girder as 2 kN/m.

Given self-weight of rail section — 210 N/m. = 0.21 kN/m.

Total dead load, $w = 2$ kN/m $+ 0.21$ kN/m $= 2.21$ kN/m.

Factored dead load $= 1.5 * 2.21 = 3.315$ kN/m.

the position of one wheel load from the mid-point of span



$= \frac{\text{wheel base}}{4}$

$= \frac{3.2}{4} = 0.8$ m.

Bending moment due to live load only :

Taking moment about D,

$R_C \times 10 = 310(10-2.6) + 310(4.2)$

$\therefore R_C = 359.6$ kN.

Taking moment about C,

$R_D \times 10 = 310(5.8) + 310 \times (2.6)$

$\therefore R_D = 260.40$ kN.

Max. Bending moment due to live load $= 260.40 * 4.2$

$= 1093.68$ kNm.

Bending moment due to impact $= 0.10 * 1093.68 = 109.36$ kNm.

Total BM. due to live and Impact load $= 1093.68 + 109.36$

$= 1203.048$ kNm

Bending moment due to dead load $= \frac{wl^2}{8} = \frac{3.315 * 10^2}{8}$

$= 41.43$ kNm.

$\therefore$ Max. bending moment $= 1203.048 + 41.43 = \underline{1244.47}$ kNm.

Maximum shear force

It consists of maximum shear due to moving wheel loads and self weight of the girder.

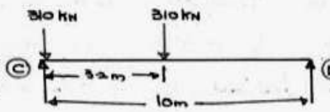


Fig. D

For maximum shear force the wheel loads shall be placed as shown in fig D. i.e one of the wheel loads should be at the support.

Taking moment about D,

$R_C * 10 = 310 * 10 + 310 * (10 - 3.2)$

$R_C = 520.8$ kN.

Hence max. shear force due to wheel loads $= \boxed{520.80}$ kN.

Lateral forces :-

Lateral forces transverse to the rails = 5% of weight of Crab and Trolley weight lifted.

$= 0.05 * (350 + 40) =$

$= 19.5$ kN.

Factored lateral force $= 1.5 * 19.5 = 29.25$ kN.

Lateral force on each wheel $= \frac{29.25}{2} = 14.625$ kN

Max. Horizontal reaction due to the lateral force by proportion at C

$= \frac{\text{lateral force} * \text{reaction at C due to Vertical load}}{\text{Max. wheel load due to Vertical load}} = \frac{14.625 * 359.6}{310}$

$= 16.965$ kN

Horizontal reaction due to lateral force at D $= 29.25 - 16.965$

$= 12.285$ kN.

Maximum bending moment due to lateral load by propotion

$$= \frac{14.625}{310} * 1093.68 = 51.59 \text{ kNm}$$

Max shear force due to lateral load by propotion

$$= \frac{520.84}{310} * 14.625 = 24.57 \text{ kN}.$$

Preliminary trial Section.

Approximate depth of Section $= \frac{L}{12} = \frac{10*10^3}{12} = 833.33$ mm ≈ 850 mm.

Approximate width of flange $= \frac{L}{30} = \frac{10*10^3}{30} = 333.33$ mm ≈ 350 mm.

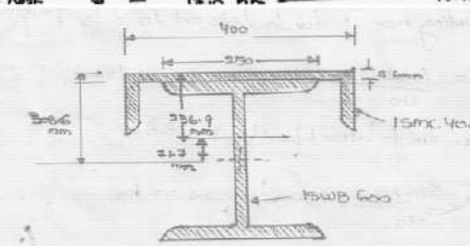
Approximate section modulus required,

$$Z_{pz} = 1.4 \frac{M_z}{f_y} = \frac{1.4 * 1244.47 * 10^6}{250} = 6969.032 * 10^3 \text{ mm}^3.$$

Let us try ISWB 600 @ 1423.43 N/m with ISMC 400 @ 484.61 N/m on its top flange as shown in figure.

The relevant properties of the Sections are.

Property		I-Section ISWB 600 @ 1423 N/m	channel Section ISMC 400 @ 484 N/m
Area	A	186.86 cm^2	62.93 cm^2
thickness of flange	t_f	23.6 mm	15.3 mm
Thickness of web	t_w	11.8 mm	8.6 mm
Width of flange	b_f	250 mm	100 mm
Moment of inertia	I_z	106198.5 cm^4	15083.8 cm^4
	I_y	4702.5 cm^4	504.8 cm^4.
Depth of Section	h	600 mm	400 mm
Radius at root		17.0 mm	15 mm , Cyy-



Moment of inertia of gantry girder

the distance of NA of built-up section from the extreme fibre of Compression flange

$$\bar{y} = \frac{\Sigma AY}{\Sigma A} = \frac{18686(300+8.6)+(6293)*24}{(18686+6293)}$$

$$= 236.90 \text{ mm}.$$

Gross moment of inertia of the built-up section.

$$I_z \text{ gross} = I_z \text{ beam} + I_z \text{ channel}.$$

$$= \left[106198.5 * 10^4 + 18686 \times (308.6 - 236.9)^2\right] +$$

Moment of inertia of gantry girder

The distance of NA of built-up section from the extreme fibre of compression flange

$$\bar{y} = \frac{\Sigma AY}{\Sigma A} = \frac{18686(300+8.6)+(6293)*24}{(18686+6293)}$$

$$= 236.90 \text{ mm.}$$

Gross moment of inertia of the built-up section.

$$I_z \text{ gross} = I_z \text{ beam} + I_z \text{ channel.}$$

$$= \left[106198.5*10^4 + 18686 \times (308.6-236.9)^2\right] + \left[504.8*10^4 + 6293 \times (236.9-24)^2\right]$$

$$= 1158047671 + 290287098.1$$

$$= 1448334769 \text{ mm}^4.$$

$$I_y \text{ gross} = I_y \text{ beam} + I_y \text{ channel}$$

$$= 4702.5*10^4 + 15083.8*10^4$$

$$= 197863000 \text{ mm}^4$$

$$Z_{ez} = \frac{I_z}{y} = \frac{(1448334769)}{(600+8.6-236.9)} = 3896515.386 \text{ mm}^3.$$

Plastic modulus of Section

Equal area axis

$$6293 + \cancel{(250*23.6)} + \bar{y}_1 \times 11.8 = \cancel{(250\times 23.6)} + (600-2\times 23.6-\bar{y}_1)\times 11.8$$

$$\Rightarrow 6293 + 11.8\bar{y}_1 = (552.8-\bar{y}_1)11.8$$

$$6293 + 11.8\bar{y}_1 = 6523.04 - 11.8\bar{y}_1$$

$$\therefore 2*11.8\bar{y}_1 = 6523.04 - 6293$$

$$\bar{y}_1 = \frac{230.04}{2\times 11.8} = 9.747 \text{ mm.}$$

Plastic section modulus of the section above equal area axis,

$$Z_{pz1} = 400\times 8.6\times\left(9.747+23.6+\frac{8.6}{2}\right) + \left[2\times(100-8.6)\times 15.3\times\left(9.747+23.6-\frac{100-8.6}{2}\right)\right]$$
$$+ 250\times 23.6\times\left(9.747+\frac{23.6}{2}\right) + 9.747*11.8*\frac{9.747}{2}$$

$$= 129505.68 - 34549.36 \quad \cancel{+(34549.36)}$$
$$+ 127127.3 + 560.52$$

$$= 222644.14 \text{ mm}^3.$$

Plastic section modulus of the section below equal area axis,

$$Z_{pz2} = 250*23.6*\left(600-21.3-9.747-\frac{23.6}{2}\right) + \frac{(600-2\times 23.6-9.747)^2 * 11.8}{2}$$

$$= 3287202.7 + 1739948.709$$

$$= 5027151.409 \text{ mm}^3.$$

$$\therefore Z_{pz} = 222644.14 + 5027151.409$$
$$= 5249795.549 \text{ mm}^3.$$

Plastic section modulus of compression flange about yy-axis,

$$Z_{pfy} = 250 \times 23.6 \times \frac{250}{4} + \frac{2 \times (400 - 15.3 \times 2)^2 \times 8.6}{8} + 2 \times \left(15.3 \times 100 \times \frac{400-15.3}{2}\right)$$

$= 368750 + 293381.17 + 588591$

$= 1250722 \text{ mm}^3$.

$= 1250.722 \text{ mm}^3$.

Classification of section :

Outstand of flange of I-section, $b = \frac{b_f}{2} = \frac{250}{2} = 125 \text{ mm}$.

$\frac{b}{t_f}$ of flange of I-section $= \frac{125}{23.6} = 5.29 < 8.4$ $(8.4\varepsilon = 8.4 \times 1 = 8.4)$

Outstand of flange of channel section,

$b = b_f - t_w = 100 - 8.6 = 91.4 \text{ mm}$.

b/t_f of flange of channel section $= \frac{91.4}{15.3} = 5.97 < 8.4$ $(8.4\varepsilon = 8.4 \times 1 = 8.4)$

$\frac{d}{t_w}$ of web of I-section $= \frac{h - 2t_f}{t_w}$

$= \frac{600 - 2 \times 23.6}{11.8} = 46.847 < 84$ $(84\varepsilon = 84 \times 1 = 84)$

Hence, the entire section is plastic. $(\beta_b = 1.0)$.

Check for moment Capacity :

Local moment Capacity: $M_{dz} = \beta_b Z_{pz} \frac{f_y}{\gamma_{mo}} \le 1.2 Z_e \frac{f_y}{\gamma_{mo}}$

$M_{dz} = 1.0 * 5249795.55 * \frac{250}{1.10} = 1193.135 \text{ kNm}$.

$\le 1.2 \times \cancel{197863000}\ 3896515 * \frac{250}{1.10} = 1062.68 \text{ kNm}$.

Hence, moment Capacity of the section, $M_{dz} = 1062.68 \text{ kNm}$.
$< 1244.47 \text{ kNm}$

Hence revise the section.

Therefore provide additional flange plates at top and bottom flanges to cater for a moment of $(1244.47 - 1062.68) = 181.314 \text{ kNm}$.
$\simeq 190 \text{ kNm}$

$\therefore$ the area of flange plates reqd.

$$A = \frac{M}{d f_y} = \frac{190 * 10^6}{600 * 250} = 1266.67 \text{ mm}^2.$$

Hence, moment capacity of the section, $M_{dz} = 1063.68$ kNm.
< 1244.47 kNm

Hence revise the section.

therefore provide additional flange plates at top and bottom flanges to cater for a moment of $(1244.47 - 1063.68) = 181.314$ kNm.
$\simeq 190$ kNm

$\therefore$ the area of flange plates reqd.

$$A = \frac{M}{d f_y} = \frac{190 \times 10^6}{600 \times 250} = 1266.67 \text{ mm}^2.$$

Providing 250 wide plates, thickness of plate reqd $= \frac{1266.67 \text{ mm}^2}{250}$
$= 5.06$ mm.
$\simeq 10$ mm.

$\therefore$ Provide a plate of 10 mm thick of 250 mm wide above the channel at top and below the bottom flange.

therefore the section of the Gantry will be as follows. after the check for combined check for local moment capacity.

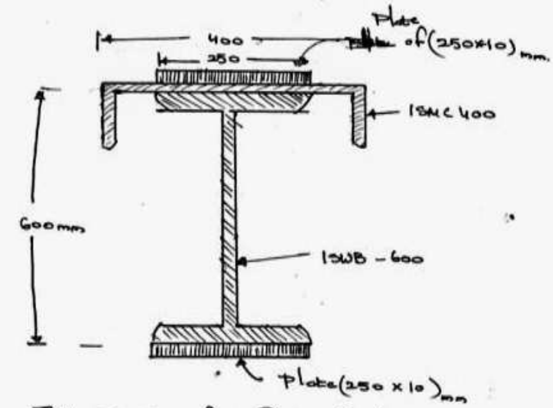


Fig: Section of a Gantry Girder.

UNIT-V

B-V Write about the economic proportions of a truss bridge for i) No of panels, ii) inclination of diagonals. **08**

30.4. ECONOMIC PROPORTIONS OF TRUSS BRIDGE

The cost of a truss bridge depends upon several factors such as panel length, number of panels, height of truss, cost of floor system, fabrication cost etc. The permissible stress in compression members is not constant, but depends upon the l/r ratio of the member. Hence it is difficult to formulate mathematically the most *economical configuration* of a truss bridge. It should be noted that a truss configuration giving minimum weight may not be the most economical because the cost of the flooring in such a configuration may be higher. Similarly, a minimum weight configuration may not give rise to minimum cost of fabrication which in turn, depends upon the number and types of joints.

(i) Number of panels

If it is assumed . at permissible stresses in tension and compression are *constant*, then a *two panel truss* with diagonals inclined at 45° (Fig 30.5 *a*) will theoretically yield minimum

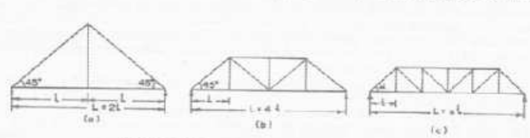


FIG. 30.5. NUMBER OF PANELS FOR A GIVEN SPAN

weight for a given span and loading. However, the permissible compressive stress depends on the length of the member, and for a two panel truss, the value of this stress will be quite small, giving rise to heavy sections. Also, the cost of flooring system will be large. Hence for overall economy, the number of panels should be between 6 to 8 thus reducing the length of the members and also reducing the cost of flooring.

For a bridge of given span, the length of panel will depend upon the number of panels. Generally, a panel length between 6 to 9 m is considered satisfactory. For example, for a span of 48 m, one may choose 6 panels each of 8 m, while for a bridge of 72 m one has to choose 8 panels of 9 m each because 6 panels in such a case will give rise to 12 m panel length which will be highly uneconomical. Again, as the span increases, the number of panels also increase so as keep panel length within 9 m. *But large number of panels make the truss uneconomical.* Hence for large spans, one has to use truss girders with subdivided panels.

(ii) Inclination of diagonals

Let us find an expression for the inclination α of the diagonals so as to give minimum volume of diagonals. From Fig. 30.6, it is quite evident that the total length of all the diagonals is equal to AC.

But $\quad \frac{AB}{AC} = \cos \alpha$

$$\therefore \quad AC = \frac{AB}{\cos\alpha} = \frac{L}{\cos\alpha} \qquad ...(30.1)$$

Hence length of all the diagonals $= L/\cos\alpha$

Let F be the S.F. in a panel, assumed constant for a given span and loading. Hence average force in a diagonal $= F/\sin\alpha$. $\therefore$ Average area of cross-section of a diagonal $\propto \dfrac{F}{\sin\alpha}$

$\therefore$ Volume of diagonals

$$\propto \frac{F}{\sin\alpha}\cdot\frac{L}{\cos\alpha} \propto \frac{1}{\sin\alpha\,.\cos\alpha} \qquad ...(30.2)$$

This gives the value of $\alpha = 45°$, for the volume of diagonals to be minimum. Thus, for minimum weight of diagonals, inclination α should be kept equal to 45°. However, *overall* economy is achieved when this inclination is kept slightly higher, say between 45° to 60° or more closely between 50° to 55°.

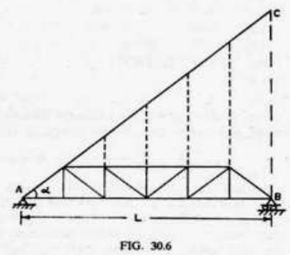


FIG. 30.6

or

(b) Show the arrangement of components of deck type truss girder bridge. Give the design steps

30.2. GENERAL ARRANGEMENT OF COMPONENTS OF TRUSS GIRDER BRIDGE

(a) Through type bridge

A through type truss bridge consists of following components (Fig. 30.1)

(i) Main vertical truss girders (two Nos.)
(ii) Floor system
(iii) Bottom lateral bracing
(iv) Top lateral bracing
(v) Portal bracing
(vi) Sway bracing

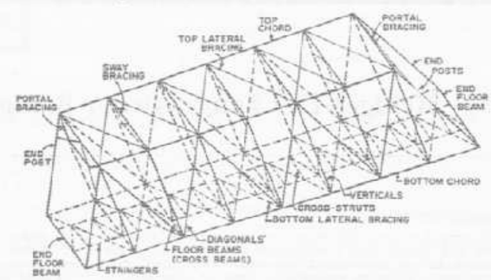


FIG. 30.1. DIAGRAMMATIC VIEW OF A THROUGH TYPE TRUSS GIRDER BRIDGE

Fig. 30.1 shows the diagrammatic view of these components. Fig. 30.2 shows plan, elevation and cross-section of a through type truss girder bridge. A bridge consists of two vertical truss girders, arranged at some distance apart. The corresponding lower panel points of the two truss girders are joined by girders, known as *cross-girders* or *floor beams*. These floor beams receive loads from *stringers* or *longitudinal beams* which run parallel to the length of the two trusses.

The stringers support the sleepers in the case of railway bridges. Sleepers support the rails which are laid parallel to stringers. If open floor is not used, then the ballast is laid on steel or concrete decking which in turn is supported by the stringers. Thus, the vertical moving loads on rails are transferred to the stringers through sleepers, from stringers to the cross-beams and from cross-beams to the lower panel points of the two truss girders.

In addition to the vertical loads, truss girders are also subjected to lateral forces due to wind forces, seismic forces and racking forces. In order to transfer these lateral forces to the bearings, lateral bracing systems are used both at the level of top chords as well as bottom chords. A horizontal truss is formed joining the bottom chords of the two trusses with laterals (Fig. 30.2 *c*), so as to transfer the lateral loads acting on it to the bearings. Similarly horizontal truss is formed joining the top chords of the two trusses with laterals (Fig 30.2 *b*) at the panel points, so as to transfer the lateral loads acting on it to the top of the end posts. The end diagonal panel is inclined and hence diagonal members cannot be provided in this panel. Hence the load transferred to the top of the end posts by the top lateral bracing is, in turn, transferred to the bearings by *portal action* by providing a *portal bracing* in the plane of end posts (Fig 30.2 *d*). In order to keep the rectangular shape of bridge cross-section intact, sway bracings are provided in vertical plane at each panel point (Fig. 30.2 *e*). Both the portal bracings as well as sway bracings should have maximum depth

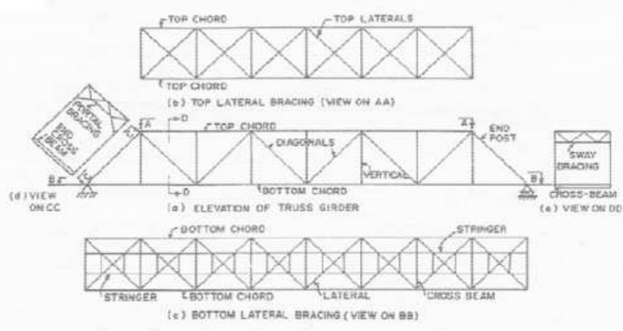


FIG. 30.2. COMPONENTS OF THROUGH TYPE TRUSS GIRDER BRIDGE

permissible with the required head room corresponding to the clearances requirements of the rolling stock. The minimum clearance required for the rolling stock of broad gauge is shown in Fig. 30.3.

Apart from horizontal truss effect, the wind has also the overturning effect. The spacing between the centres of the two truss girders should be sufficient to resist the overturning effect. However, this spacing should neither be less than 1/20 of the effective span of the truss girder, nor less than 1/3 of the height of the truss girder.

(b) **Deck type bridge**

The arrangement for a deck type truss girder bridge is similar to the arrangement for a deck type plate girder bridge, except that the two plate girders are replaced by two truss girders. The sleepers rest directly on the top chord members of the two trusses. These sleepers carry the rails and serve as spring to reduce the impact. The portal and sway bracings(used in through type bridges) are reduced by cross-frames.

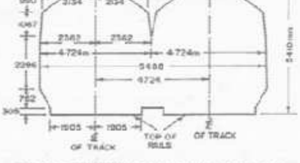


FIG. 30.3 MINIMUM CLEARANCE DIAGRAM

X