Notes by-

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[BEAM]

Defn :- Beam is a structural member subjected to transverse load.

Joist : closely spaced beams supporting floors, roofs of building.

Girder : A large beam supporting number of small beams (Joist)

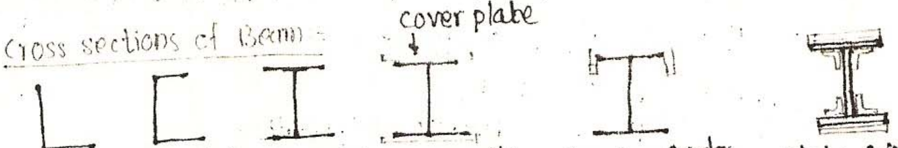
Purlin : A beam which carry roof load, used in truss.

Lintel : A beam over openings viz, window, door that supports wall above it.

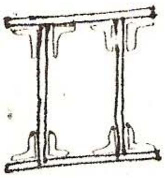
Rafter : A roof beam supporting purlin [Beam-col^m]

Girt : A horizontal beam spanning in wall columns or industrial building used to support wall covering.

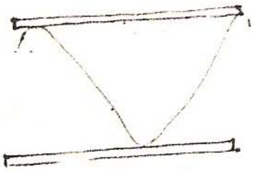
* Cross sections of Beam :-



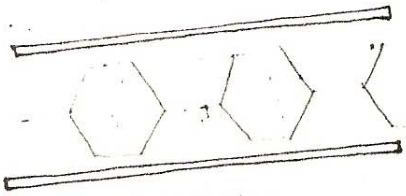
Angle channel I-Joist Composite Gandry Girder Plate Girder



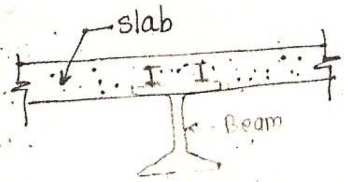
Box Girder

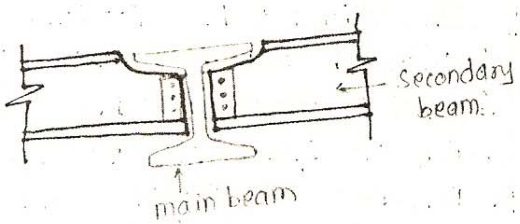


Bar Joist



Castellated Beam.
[Used when box Girder is not sufficient to carry load]

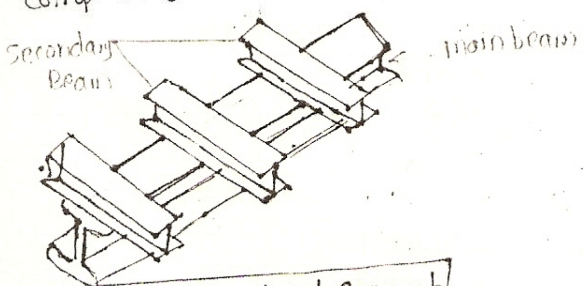




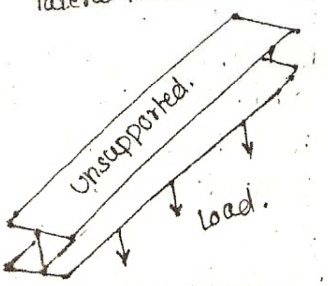
Compression flange has no chance to leave vertical plane.

[Full Laterally Supported Beam.]

compression flange restrained against lateral buckling. A beam is deemed effectively "restrained laterally" if the frictional or positive connection of slab to beam is ~~cab~~ capable of resisting v 2.5% max. force in the lateral force of comp. flange of beam.



[Intermediate lateral Support]



[No lateral support]

comp. flange is not restrained laterally against buckling, such beam is laterally unsupported.

* Modes of failure or Beam is to be designed for fo
  1. Bending
  2. shear
  3. Deflection
  4. web cripling
  5. web buckling.

① Bending stresses:-

a) for laterally supported beam : σ_{ac} & $\sigma_{bt} = 0.66 f_y$

b) for laterally unsupported beam:- [Cl. 6.2 / Pg. 55]

i) for beams & channel with equal flanges:

use table in IS :800 - Pg - 57, Table 6.1A – ~~6~~ 6.1F

for $F_y = 250$, $\frac{T}{t} > 2$, or $\frac{d_1}{t} > 85$; $\frac{l_e}{r_y}$

Where, D = overall depth

T = Mean thk. of comp. flange

r_y = min. radius of gyration (always r_y)

l_e = eff. length of comp. ~~member~~. flange

t = thk. of web.

Depending upon above factor choose a table from 6.1A to 6.1F & find σ_{bc} (MPa)

* Effective length of compression flange [IS:800 / Pg. 76 / cl. 6.6]

For simply supported Beam :- (L = Span)		Not restrained against torsion
End condition of beam	Eff. length	(20%) ↑
Restrained against Torsion.	↓	
1) Unrestrained against lateral bending	→ L	→ 1.2L
2) Partially restrained against lateral bending	→ 0.85L	→ 1.05L
3) Fully restrained against lateral bending	→ 0.7L	→ 0.9L
4) Effective lateral bracing at interval along length	→ Dist. betⁿ inter-section of bracing with member	

NOTE ⇒ when end of Beam are not restrained @ Torsion above eff. length will be increased by 20%

For Cantilever Bem:- (L = Projecting length)	
* (A) Built in support →	
1) free at end	→ 0.85L
2) Restrained against torsion at end by continuous construction. Fig (8.8)	→ 0.75L
3) Restrained against lateral deflection & torsion against free end.	→ 0.5L
* (B) free at end & continuous at support:-	
1) Unrestrained against torsion at support	→ 3L
2) Partly restrained against torsion at support	→ 2L
3) fully restrained	→ L

[Please Note: Very short ; but effective Table] → Read carefully.

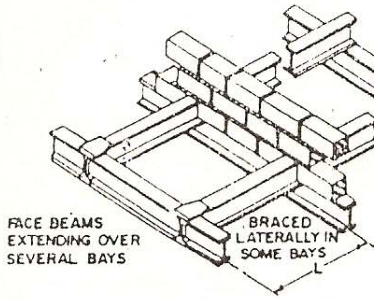
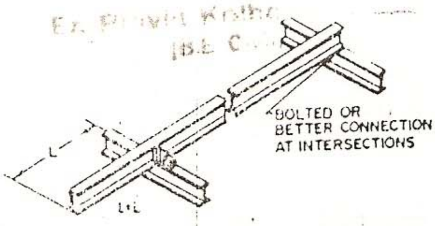
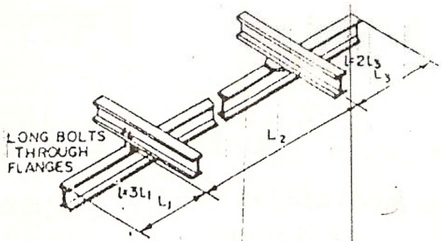


FIG. 8.9. CANTILEVER BUILT-IN AT SUPPORT, RESTRAINED LATERALLY AT THE END.



(c) CANTILEVER SPAN CONTINUOUS AT THE SUPPORT, FULLY RESTRAINED AGAINST TORSION AT THE SUPPORT AND UNRESTRAINED AT THE FREE END



CANTILEVER L₁ CONTINUOUS AT THE SUPPORT, UNRESTRAINED AGAINST TORSION AT THE SUPPORT AND UNRESTRAINED AT THE END

(b) CANTILEVER L₃ CONTINUOUS AT THE SUPPORT, PARTIALLY RESTRAINED AGAINST TORSION AT THE SUPPORT AND UNRESTRAINED AT THE END

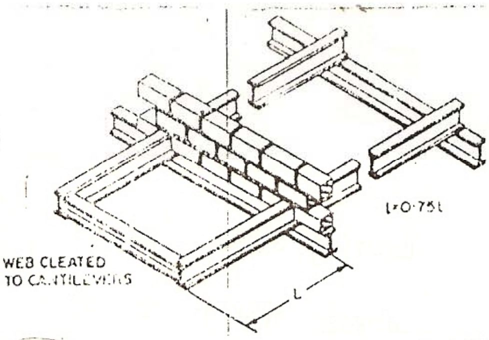
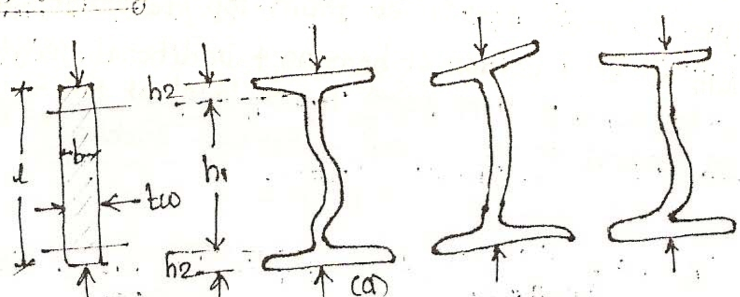


FIG. 8.8. CANTILEVER BUILT-IN AT SUPPORT, RESTRAINED AGAINST TORSION AT THE END

(2)

* Web buckling -



considered as col^m

Buckling failure

Procedure of check:-

At intermediate loads $B = b + h$

At end bearing $B = b + h/2$

let h_1 = clear depth of web betn root fillets $= h - 2h_2$; $h_2 <$ from steel Table or $2t_f$

This length acting as colm of thk. (t_w) as web.

$\therefore$ Area of colm $= A = B \times t_w$.

Eff. length of colm $h_1\,eff = \frac{d_1}{2} = \frac{h_1}{2}$ for fig (a)

$$r_{yy} = r_{min} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{\frac{1}{2} B \cdot t_w^3}{B \cdot t_w}} = \frac{t_w}{2\sqrt{3}}$$

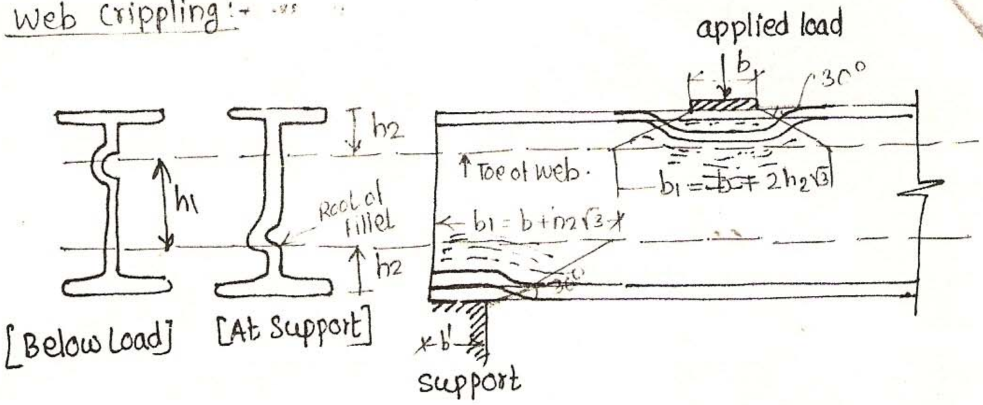
$$\therefore \lambda = \frac{l_e}{r_{min}} = \frac{h_1\,eff}{r_{min}} = \frac{h_1/2}{t_w/2\sqrt{3}} = \frac{h_1}{t_w} \cdot \sqrt{3}$$

$$\therefore \boxed{\lambda = \frac{\sqrt{3}\, h_1}{t_w}}$$

$\therefore$ find σ_{ac} cal $= \frac{P}{t_w \cdot B} < \sigma_{ac}$ permissible [IS Table]

$\therefore$ O.K.

* Web crippling :-



Bearing stress $\sigma_{p\,cal} = \frac{P}{\text{Bearing area pro.}} = \frac{P}{b_1 \cdot t_w}$

where $b_1 = b + 2h_2\sqrt{3}$ .... for intermediate / Pt. load
$= b + h_2\sqrt{3}$ ... for reaction at support (see fig) } $\not< 100$ mm

t_w is thk. of web (mm)

h_2 < from steel table or $2t_w$ = Thk. of web.

b = Bearing length (mm)

h_2 = Depth of the root of fillet from top / bottom of flange (mm)

b_1 = Length of stiff portion of bearing + additional length given by 30° dispersion (45°) to the level of NA + thk. of seating angle (if any). [For web Buckling]

h_1 = clear depth of web bet^n root fillet.

check $\sigma_{p\,cal} < \sigma_{p\,per.} = 0.75 f_y$.

GATE

In short :-

$\sigma_{p\,cal} = \frac{W = \text{Conc. load}}{t_w(b + 2h_2\sqrt{3})} \not> \sigma_{p\,per.}$	For intermediate support.
$\sigma_{p\,cal} = \frac{R = \text{End Reac}^n}{t_w(b + h_2\sqrt{3})} \not> \sigma_{p\,per}$	--- For end support

Normally these checks are not reqd.

(ii) Permissible bending stresses for beams, plate girders, Gantry Girder (with equal or unequal flange)

For beams & PG (with or without unequal flanges) bent @ the axis of max. strength (x-x axis), the max. per. bending comp. stress (σ_{bc}) is given in IS: 800 - Table 6.2 w.r.t. elastic critical stress in bending (f_{cb}) given by following eq^n when $I_y < I_x$ of the bem.

$\therefore f_{cb}$ = Elastic critical stress in bending

$\therefore f_{cb} = k_1 \cdot (X + k_2 \cdot Y) \cdot \frac{c_2}{c_1}$ — [Pg. 63 / 6.2.4]

for detailed explaination, please refer IS: 800/ pg 65
All the symbols & Tables are given.

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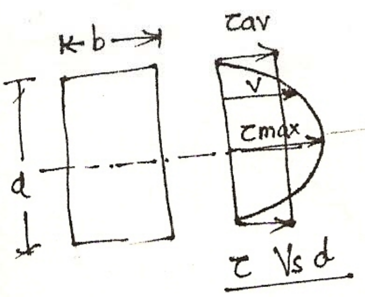
* check for shear:-

The shear stress τ at any point on the c/s of a beam is given by,

$$\tau = \frac{V \cdot A \cdot \bar{y}}{I \times b}$$

V = SF at the sect^n
$A \times \bar{y}$ = Moment of area @ the level above where shear stress is calculated @ the NA of sect^n.
I = M.I. of sect^n
b = width of sect^n where shear stress is to be calculated.



NOTE : See FM or IE : simillar to velocity distribution diagram of a channel section in which velocity distribut^n is parabolic & max. vel. is at midpoint where as avg. vel. is assumed as shown.
Same "funda" is applicable here.
τ is max. @ NA & zero at edges.

$$\tau_{va} = \text{Avg. shear stress} = \frac{SF}{\text{Depth of beam} \times \text{Thk. of web}}$$

τ_{vm} = Permissible shear stress = 0.45 fy. → yield stress.

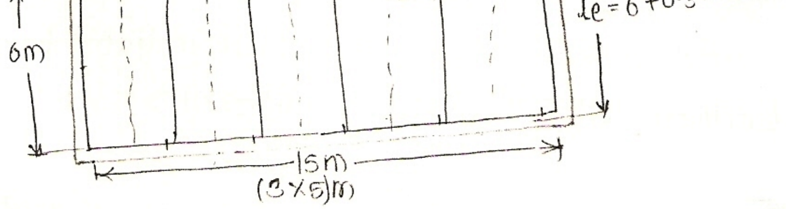
Thk. of web (Nominal) (mm)	τ_{va} (MPa)
up to 20	100
>20 ≤ 40	96
>40	92

* check for deflection:-

$$\delta_{max} \not> \frac{L}{325}$$

for exact value of δ_{max}. refer L.S. Negi.

Pro: 1] A hall of clear dimensions 15×6 m is covered by RCC slab resting over RS joist placed at a spacing of 3m, c/c. Terrazo finishing 2 cm thk. is to be provided over RCC slab. flooring 12 cm thk. The LL on the slab is 4 kN/m². The joists are resting over 30 cm thk. wall. Design the floor joists if the per. stresses in bending & shear are 165 MPa & 100 MPa resp. The unit wt. of RCC & finishing is 24 kN/m²



① Each beam will carry load of 3m wide slab.

② Load per m length of beam ⟹ ① RCC slab $= (l \times t) \times \gamma = 3 \times 0.12 \times 24 = 8.64$

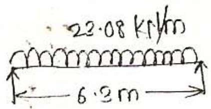
② Floor finish $= l \times t \cdot \gamma = 3 \times 0.02 \times 24 = 1.44$ kN/m

③ LL $= l \times LL = 3 \times 4 = 12$ kN/m

④ SW (assumed) $= 1$ kN/m.

Total udl on beam $= 23.08$ kN/m.

③ Effective span of beam $= l = 6.0 + 0.3$
$= 6.3$ m



④ $BM_{max} = \frac{wl^2}{8} = \frac{23.08 \times 6.3^2}{8} = 114.5 \times 10^6$ Nmm.

⑤ $Z_{reqd} = \frac{M}{\sigma_{bc} \text{ or } \sigma_{bt}} = \frac{114.5 \times 10^6}{165} = 693.9 \times 10^3 \text{ mm}^3$.

⑥ Try ISLB 350 @ 49.5 kgf/m having,
$Z_x = 751.9 \text{ cm}^3$ $a = 63.01 \text{ cm}^2$, $h = 350$ mm
$t_w = 7.4$ mm, $I_x = 13158.3 \text{ cm}^4$.

⑦ Check for shear:-

~~$\tau = \frac{Va\bar{y}}{I}$~~ =

∴ Avg. shear stress $= \frac{SF}{h \cdot t_w} = \frac{wl/2}{h \cdot t_w} = \frac{23.08 \times 6.3 \times 0.5 \times 10^3}{350 \times 7.4}$

$= 28.07$ ~~kN~~ MPa $\ngtr$ 100 MPa
∴ Safe.

⑧ Check for deflection:-

Max. deflection $\delta = \frac{5}{384} \cdot \frac{wl^3}{E \cdot I_x}$

$E = 2 \times 10^4 \text{ kN/cm}^2$

$\therefore \delta = \frac{5}{384} \times \frac{145.4 \times 630^2}{2 \times 10^4 \times 13158.3} = 1.8$ cm.

$\delta_{allowable} = \frac{l}{325} = 1.938 \text{ cm} < 1.8$ cm

∴ O.K.

⑨ Check for web crippling:-

Bearing stress at support $= \frac{R}{t_w(a + h_2\sqrt{3})}$

$R = \frac{wl}{2} = \frac{23.08 \times 6.3}{2} = 72.7$ kN.

$t_w = 7.4$ mm

a = width of support $= 30$ cm $= 300$ mm

$h_2 = 30.85$ mm.

∴ Bearing stress $= \frac{72.7 \times 10^3}{7.4 \times (300 + 30.85\sqrt{3})} = 27.8 \text{ MPa} \ngtr 187.5$ MPa

∴ Safe.

⑩ Check for web buckling:-

* slenderness ratio for web $= \frac{h_1}{t_w}\sqrt{3} = \frac{288.3 \times \sqrt{3}}{7.4} = 67.48$

from Table 5.1, $\sigma_{ac} = 122 - (122 - 112)\frac{7.48}{10} = 114.52$ MPa.

∴ Load carrying capacity of the web under concentrated load

$= \sigma_{ac} \times t_w \times B = \frac{114.52 \times 7.4}{1000} \times \left(300 + \frac{350}{2}\right) = 402.5 \text{ kN} > 72.7$ kN. ∴ Safe.

$(a + h/2)$

[∴ Provide ISLB 350 @ 49.5 kgf/m may be provided at 3m c/c.]