What is Limit State?

Acceptable limit for the safety and serviceability requirements before failure occurs is called a Limit state

Highlights

IS : 800 - 1984

Working stress method

- Factor of safely for yield stress, allowable stresses are less than ‘f_y’.
- Pure elastic approach for analysis of structures under working loads.
- Yielding or buckling never occurs at working loads
- Deformations are evaluated at working loads.

IS : 800 – 2007

Limit State Method

- Partial safety factor for material (γ_m) for yield and ultimate stress.
- Working loads are factored (increased) as per partial safely factor (γ_f) causing Limit State of strength.
- Post buckling and post yielding plays important role in estimating capacity of structural elements at Limit State.
- Deformations are evaluated at working loads.

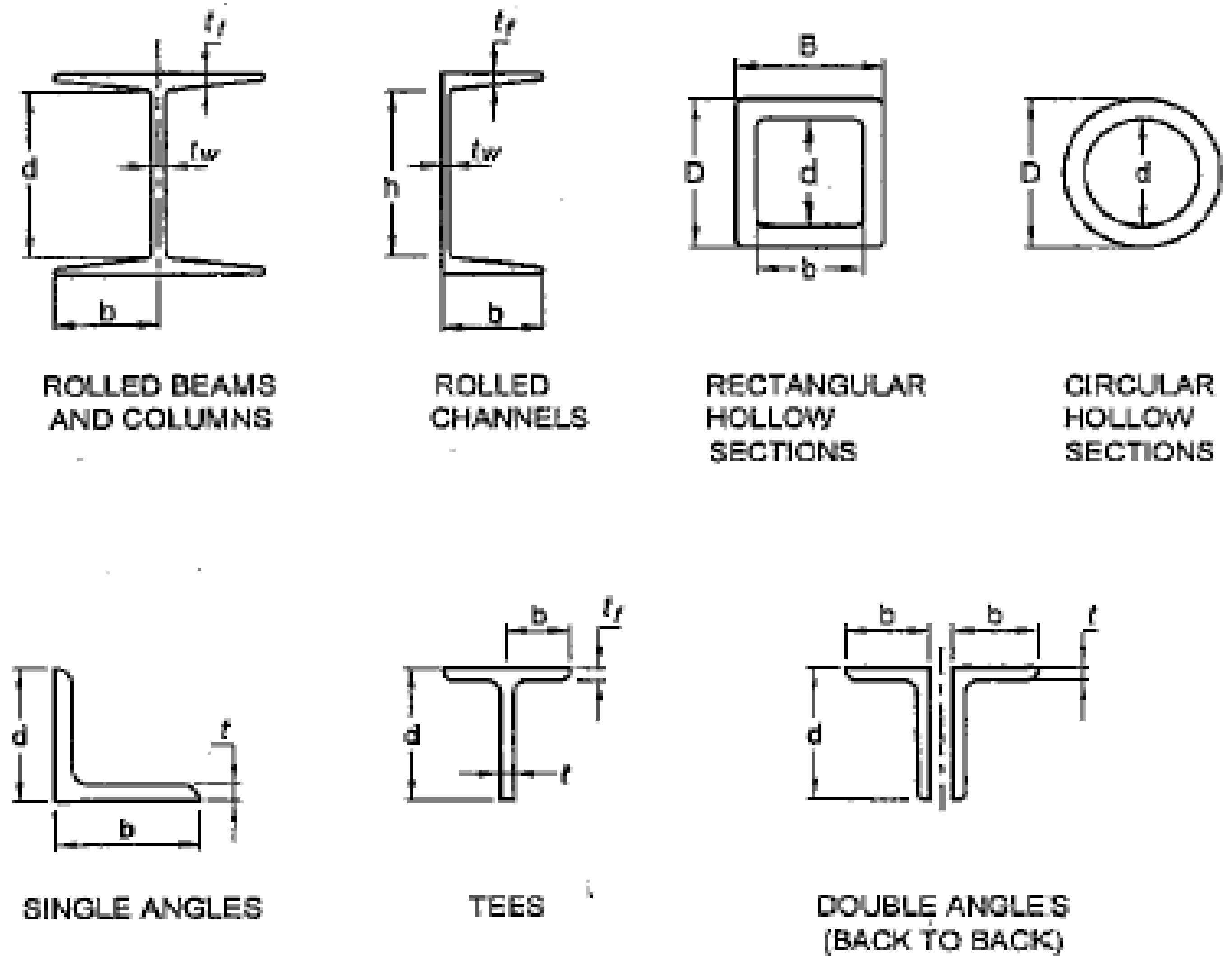
Classification of cross sections

- Structural elements in axial compression, bending compression tend to buckle prior yielding. To avoid this, elements of cross section such as width of flange, depth of web of I and channel section, width of legs of angle section, width of flange and leg of Tee section, width and height of Box section need to be proportioned in relation with thickness of element of section.

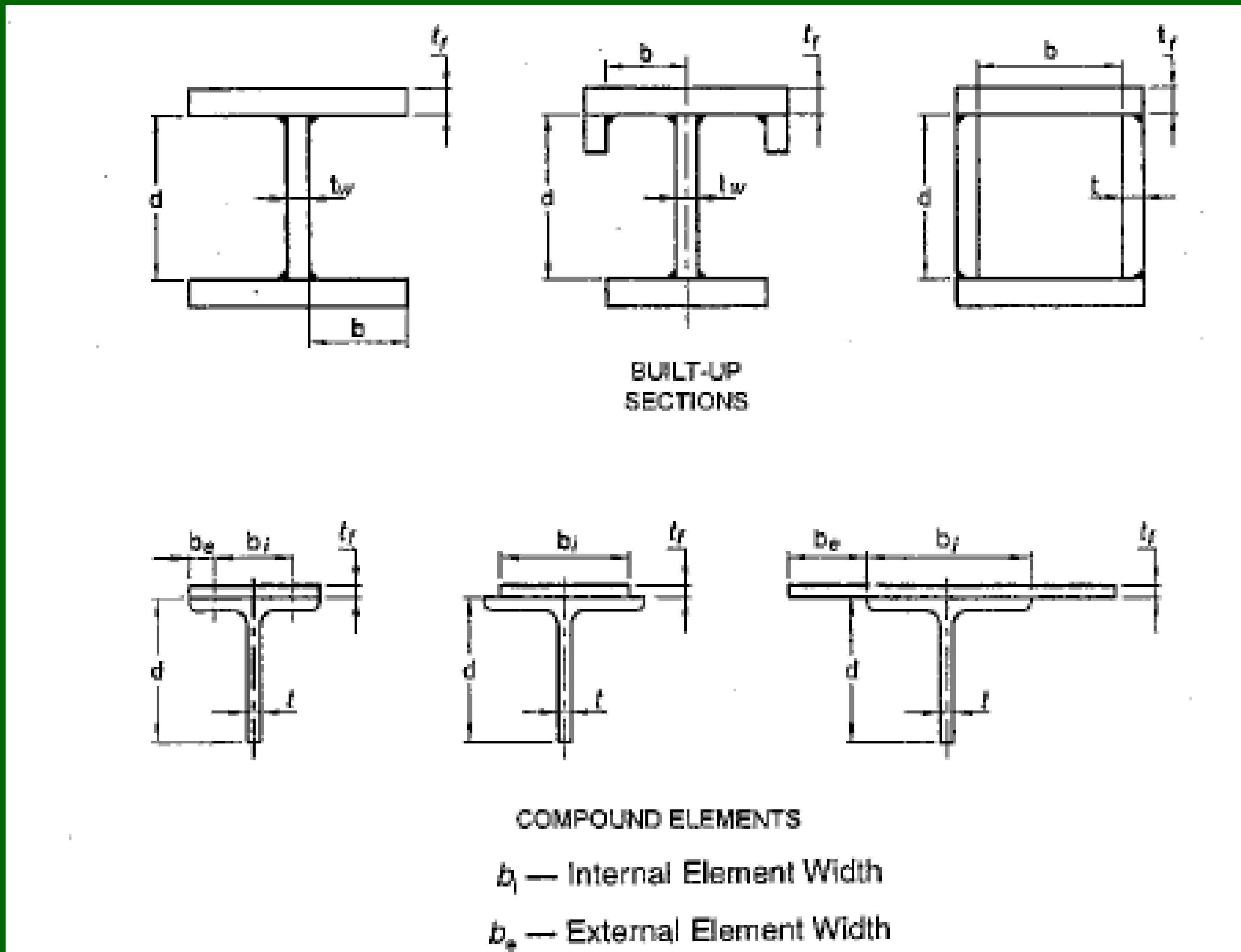
Classification of cross sections

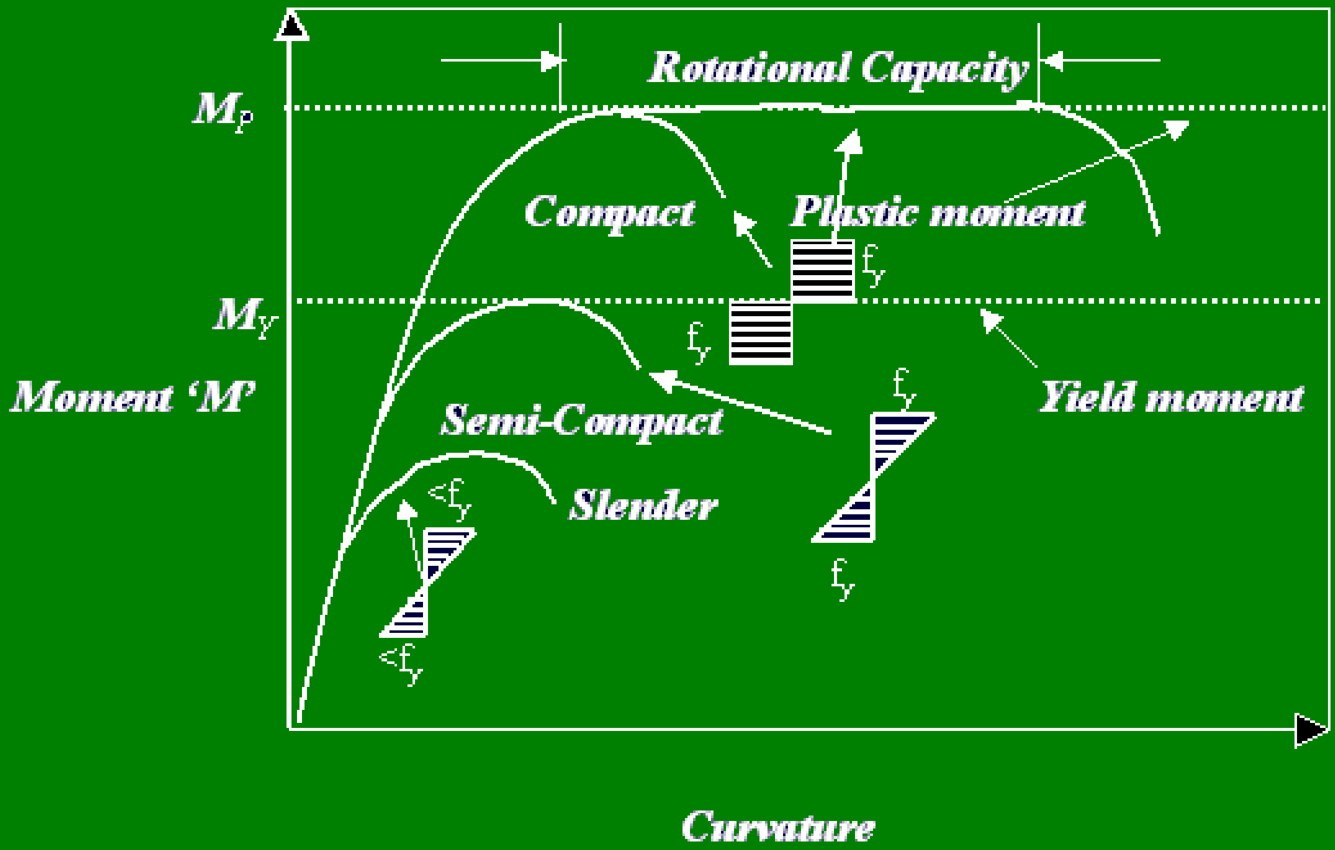
- A table of classification shows three distinct varieties of cross section such as plastic, compact and semi compact section.
- Section in which width to thickness ratio exceeds the limits of semi compact section is known as slender section. These sections are to be avoided.
- Slender section if at all used needs to ignore excess area to arrive at effective cross section as semi compact section.
- If two elements of cross section fall under two different classifications then section is classified into most unfavourable classification.

Elements of cross section

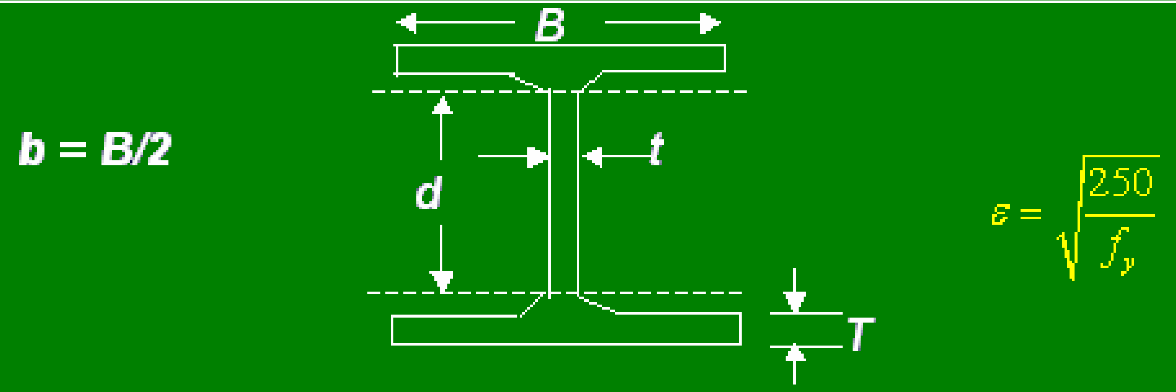


Elements of cross section





Flexural member performance using section classification



Section type	Flange criterion (b/T)	Web criterion (d/t)
Slender	>15.7	> 126
Semi-compact	<15.7 $\geq$ 10.5	< 126 $\geq$ 105
Compact	<10.5 $\geq$ 9.4	< 105 $\geq$ 84
Plastic	<9.4	< 84

Sectional Classification for Indian Conditions

Classification of section

Compression Element (1)			Ratio (2)	Class of Section		
				Class 1 Plastic (3)	Class 2 Compact (4)	Class 3 Semi-compact (5)
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I, H or box section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\leq 42\varepsilon$	$\frac{105.0\varepsilon}{1+r_1}$	$\frac{126.0\varepsilon}{1+2r_2}$ but $\leq 42\varepsilon$
		If r_1 is positive :	d/t_w		$\frac{105.0\varepsilon}{1+1.5r_1}$ but $\leq 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		42ε

Classification of section CONTD

Web of a channel	d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)	b/t d/t	9.4ε 9.4ε	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)	b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member	d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component	d/t	9.4ε	10.5ε	15.7ε
Stem of a T-section, rolled or cut from a rolled I-or H-section	D/t_f	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to: a) moment	D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
b) axial compression	D/t	Not applicable		$88\varepsilon^2$

Table showing various γ_f factors for Limit States

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL[1)]		WL/EL	AL	DL	LL[1)]		WL/EL
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL+LL+CL	1.5	1.5	1.05	—	—	1.0	1.0	1.0	—
DL+LL+CL+	1.2	1.2	1.05	0.6	—	1.0	0.8	0.8	0.8
WL/EL	1.2	1.2	0.53	1.2					
DL+WL/EL	1.5 (0.9)[2)]	—	—	1.5	—	1.0	—	—	1.0
DL+ER	1.2 (0.9)[2)]	1.2	—	—	—	—	—	—	—
DL+LL+AL	1.0	0.35	0.35	—	1.0	—	—	—	—

1) When action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.
2) This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.

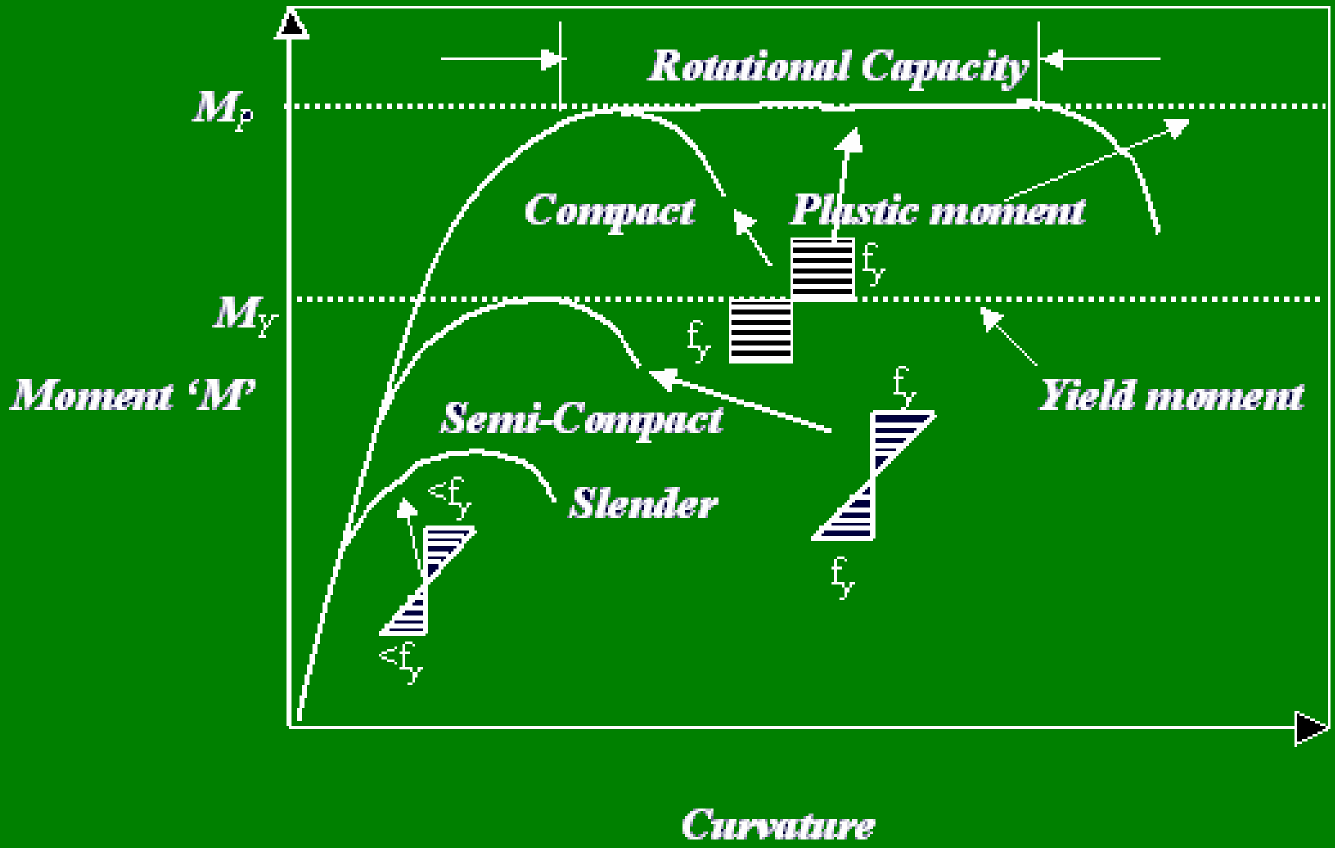
Abbreviations:
DL = Dead load, LL = Imposed load (Live loads), WL = Wind load, CL = Crane load (Vertical/Horizontal), AL = Accidental load, ER = Erection load, EL = Earthquake load.
NOTE — The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis as in 4.

Table showing Partial safety factors for materials γ_m

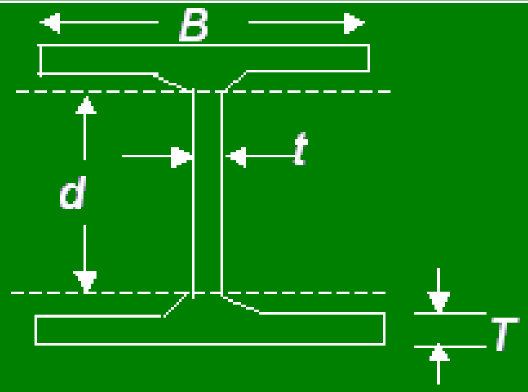
Sl No.	Definition	Partial Safety Factor	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	*Shop Fabrications*	*Field Fabrications*
	a) Bolts-Friction Type, γ_{mf}	1.25	1.25
	b) Bolts-Bearing Type, γ_{mb}	1.25	1.25
	c) Rivets, γ_{mr}	1.25	1.25
	d) Welds, γ_{mw}	1.25	1.50

THE END

DESIGN OF FLEXURAL MEMBER AND BENDING WITH HIGH SHEAR



Flexural member performance using section classification



b = B/2

$$\varepsilon = \sqrt{\frac{250}{f_y}}$$

Section type	Flange criterion (b/T)	Web criterion (d/t)
Slender	>15.7	> 126
Semi-compact	<15.7 $\geq$ 10.5	< 126 $\geq$ 105
Compact	<10.5 $\geq$ 9.4	< 105 $\geq$ 84
Plastic	<9.4	< 84

Sectional Classification for Indian Conditions

Flexural members
Laterally supported beam

Elastic Analysis

$$Me = 0.66\,fy.Ze$$

Plastic Analysis

$$Md = \beta b \frac{fy}{\gamma_{mo}} Zp$$

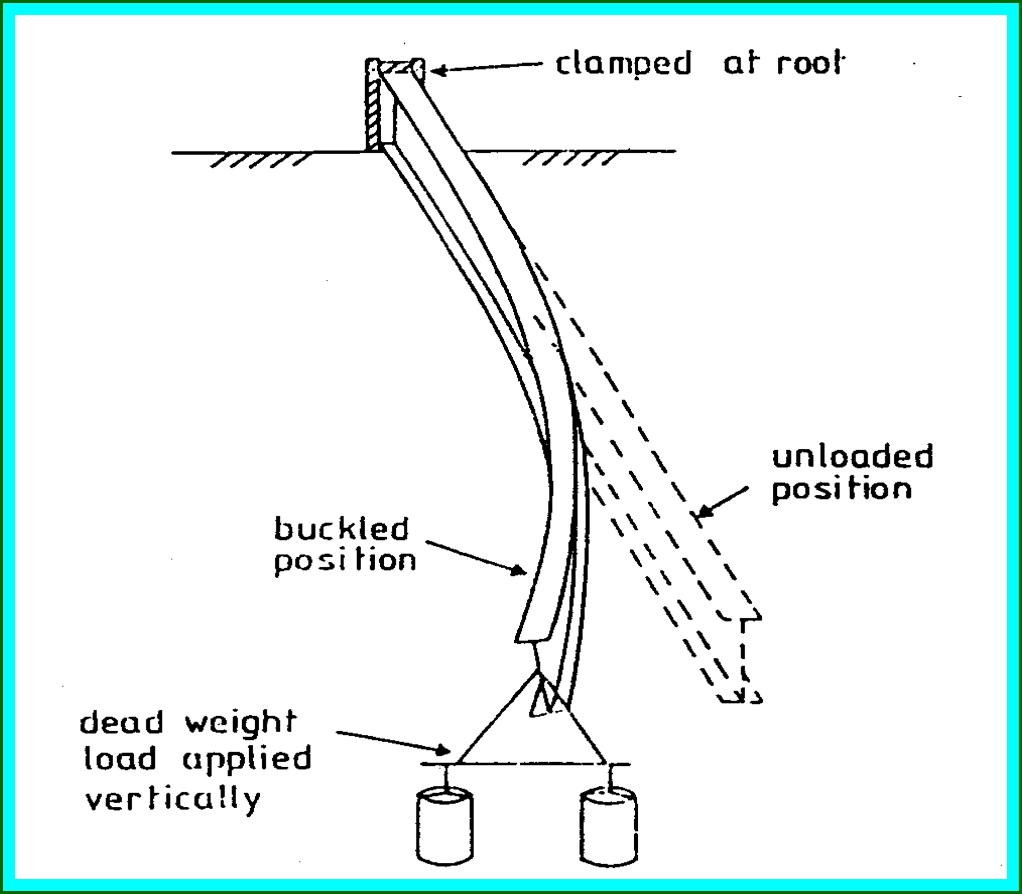
- When factored design shear $\leq 0.6V_d$ and

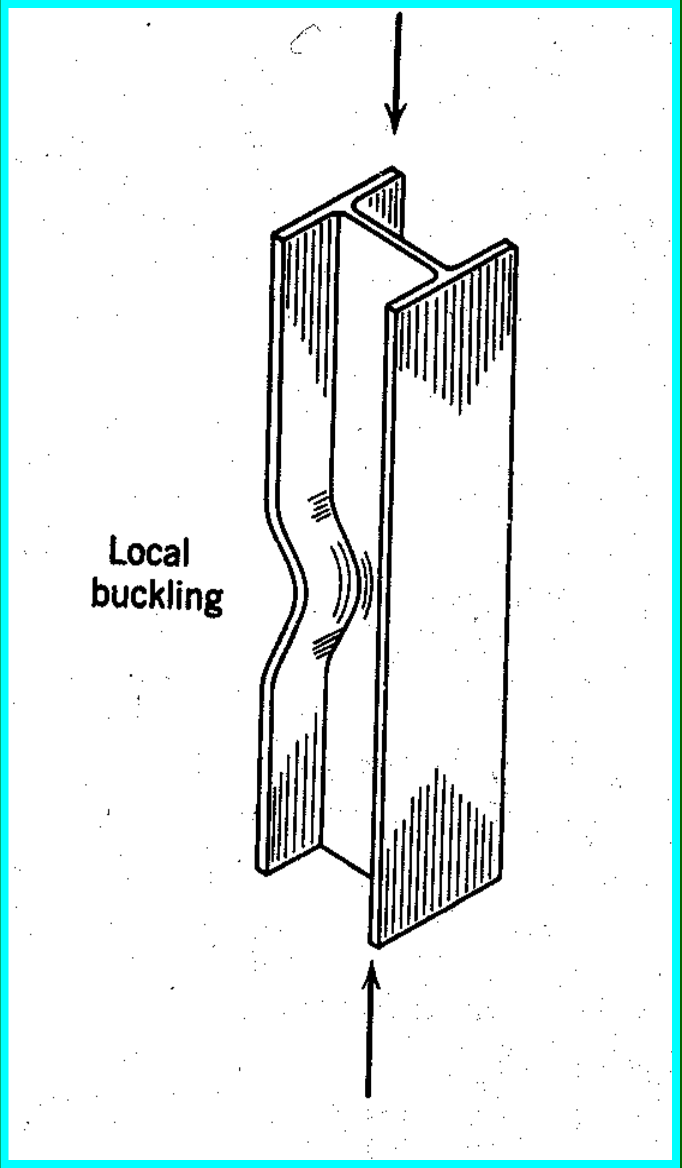
$$\frac{d}{tw} \leq 67\varepsilon$$

Conditions to Qualify as a Laterally Restrained Beam

- It should not laterally buckle
- None of its element should buckle until a desired limit state is achieved
- Limit state of serviceability must be satisfied
- Member should behave in accordance with the expected performance of the system

Lateral Stability of Beams





Local Buckling

In IS:800 (1984) the local buckling is avoided by specifying b/t limits. Hence we don't consider local buckling explicitly

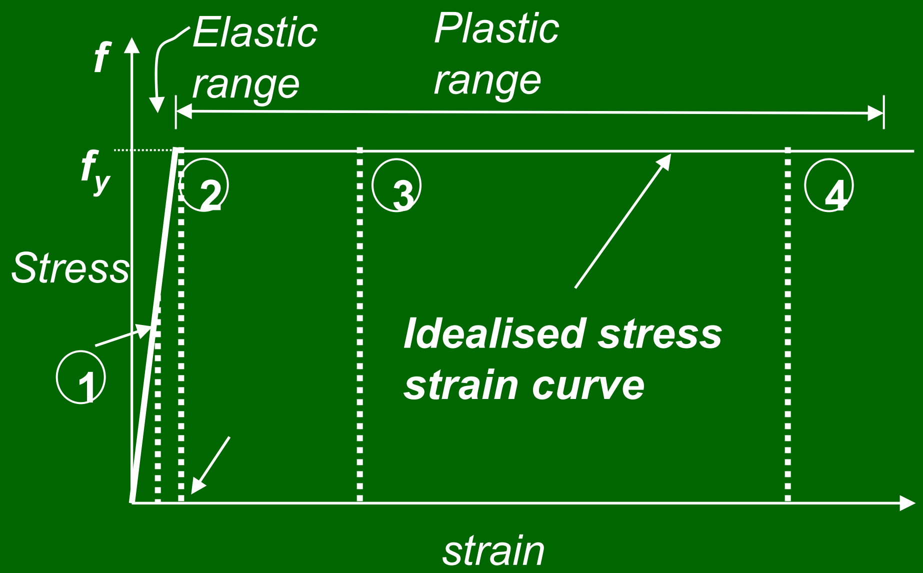
However in IS:800(2007) limit state design, the local buckling would be the first aspect as far as the beam design is concerned

How do we consider?

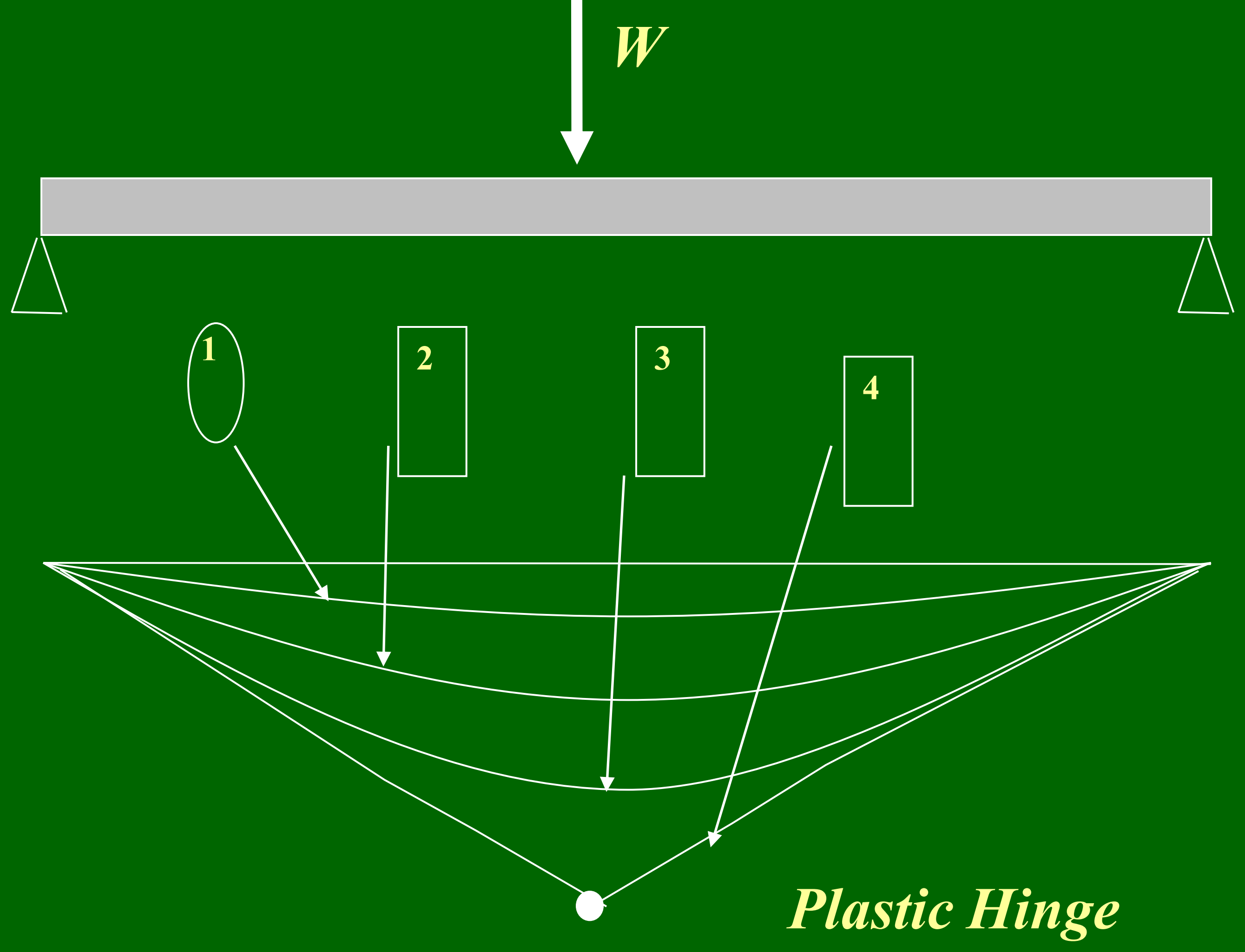
By using section classification

Limit states for LR beams

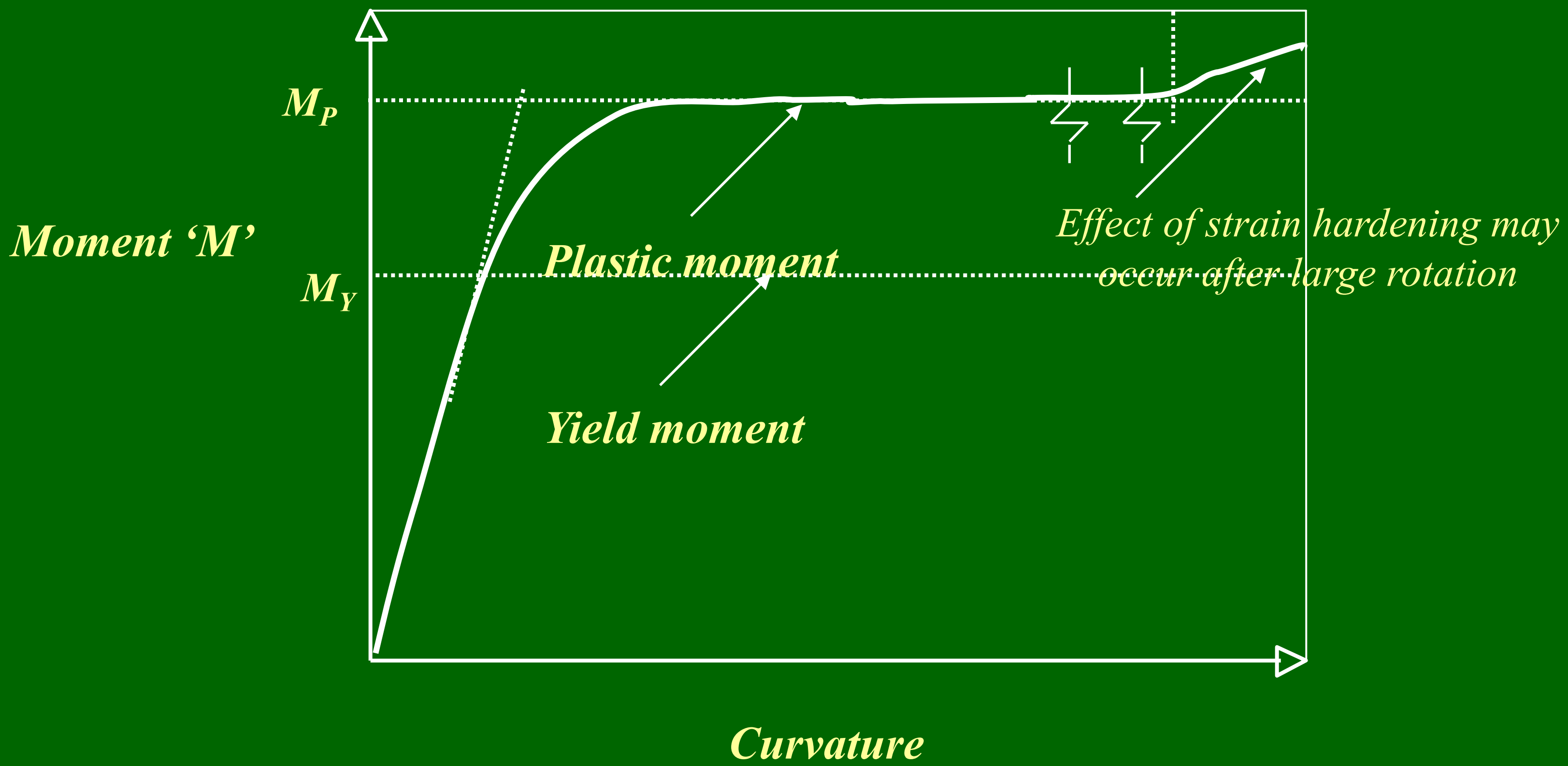
- Limit state of flexure
- Limit state of shear
- Limit state of bearing
- Limit state of serviceability



Idealized elasto- plastic stress stain curve for the purpose of design

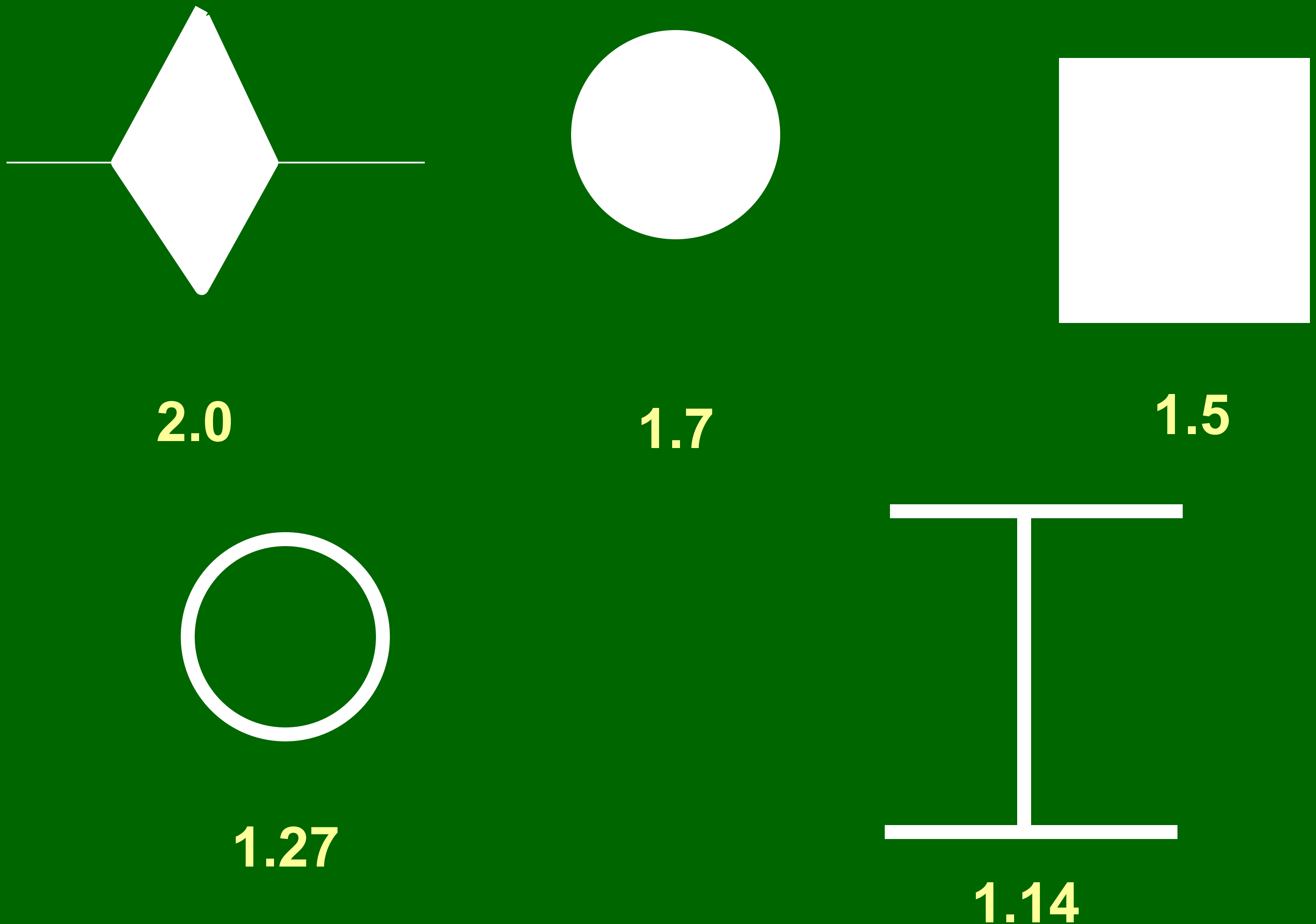


Simply supported beam and its deflection at various stages



Moment curvature characteristics of the simply supported beam

Some typical shape factor

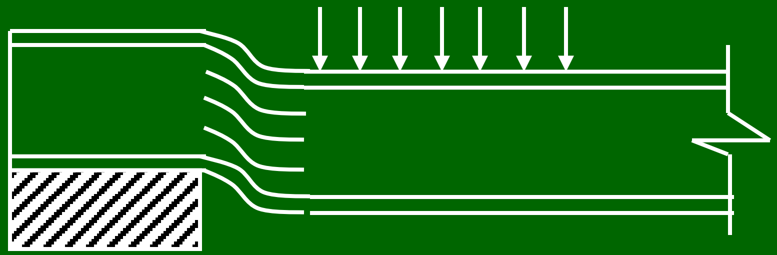


EQUATIONS FOR SHEAR CAPACITY

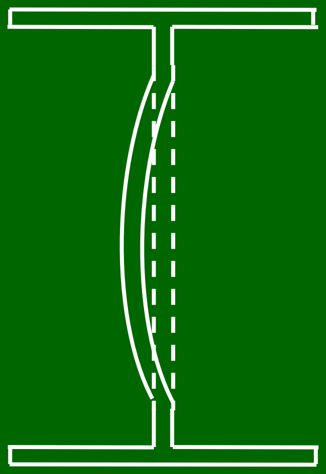
$$\tau_y = \frac{f_y}{\sqrt{3}} = 0.577 f_y$$

$$V_p = f_y t_w d_w / \sqrt{3}$$

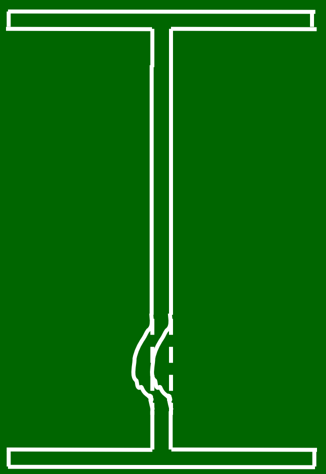
$$Vd = \frac{Vp}{\gamma_{mo}}$$



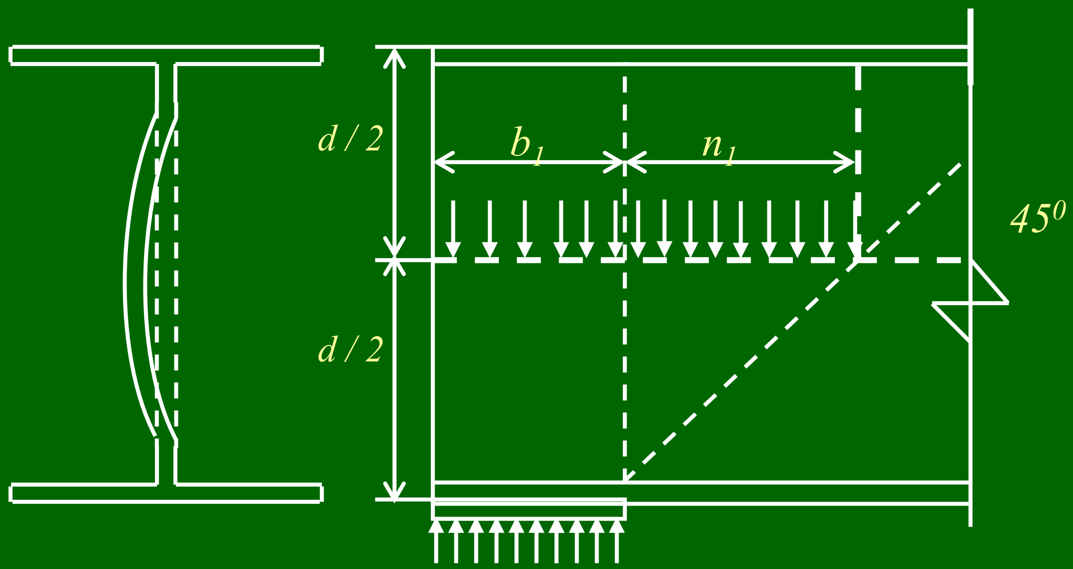
Shear yielding near support



Web buckling



Web crippling



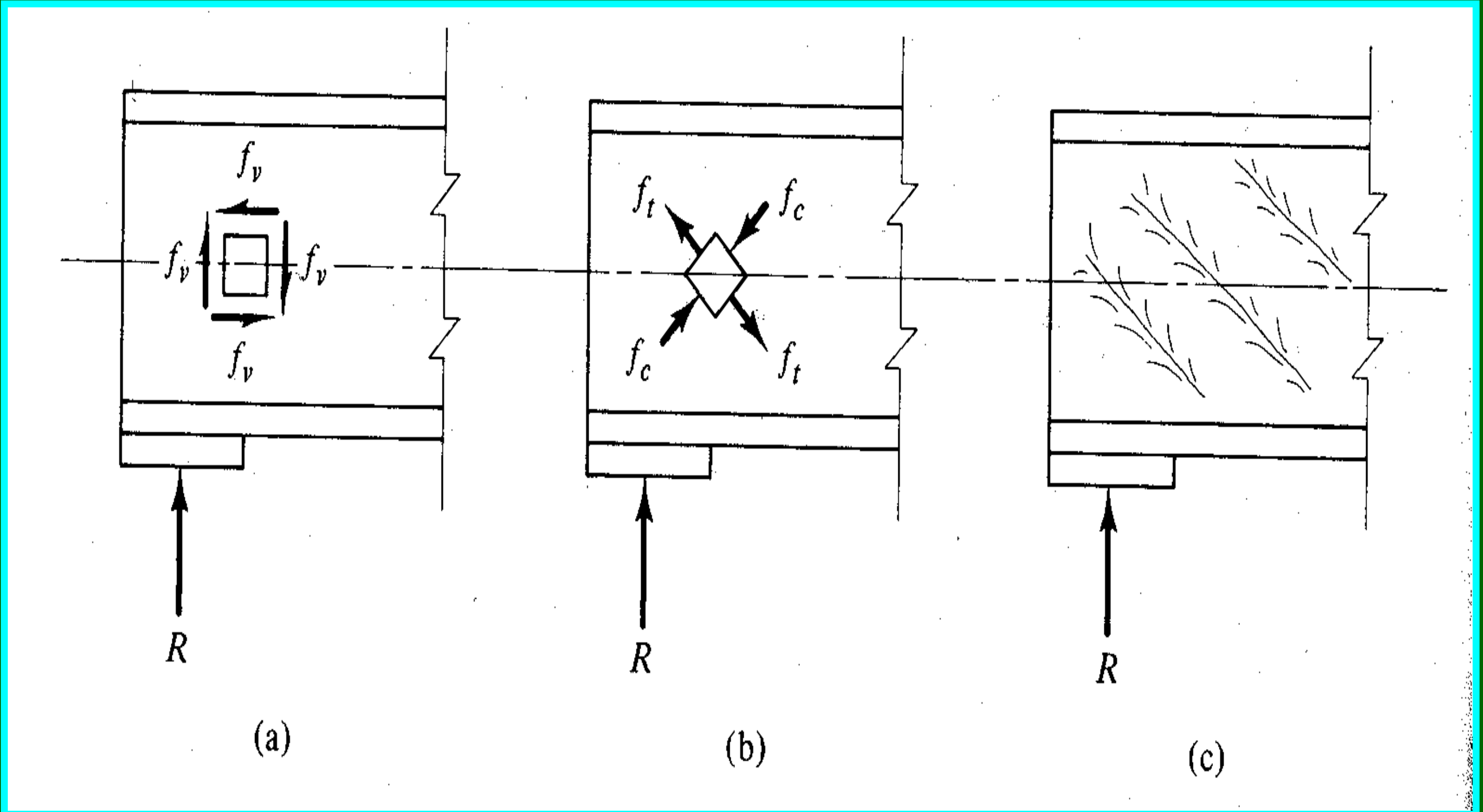
Effective width for web buckling

$$P_{wb} = (b_1 + n_1)\ t\ f_c$$

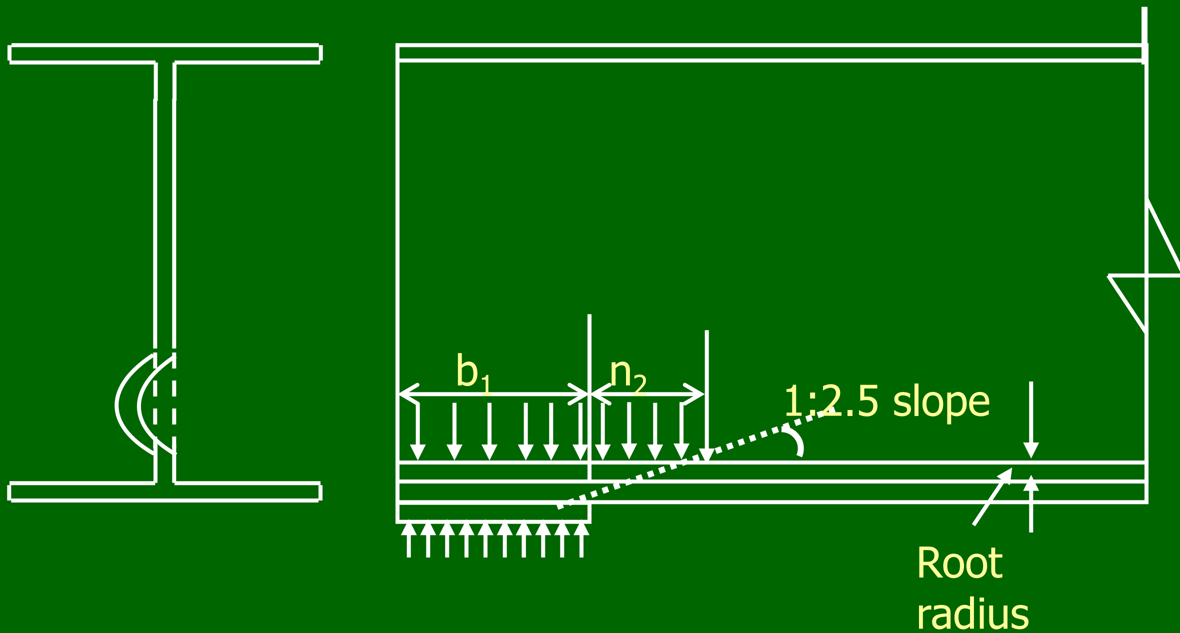
$$\lambda = \frac{L_E}{r_y} = \frac{0.7d}{r_y}$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{t^3}{12t}} = \frac{t}{2\sqrt{3}}$$

$$\frac{L_E}{r_y} = 0.7d\,\frac{2\sqrt{3}}{t} \approx 2.5\,\frac{d}{t}$$



f_v
f_v
f_v
f_v
R
(a)
f_t
f_c
f_c
f_t
R
(b)
R
(c)



$$P_{crip} = (b_1 + n_2)\, t\, f_{yw}$$

Effective width of web bearing

<u>Web Crippling in beams</u>

Design of Laterally Supported Beam

Limit State Method – As per IS: 800 - 2007.

Example No : 1

Design a suitable I beam for a simply supported span of 5 m. and carrying a dead load of 20 kN/m and imposed load of 40 kN/m. Take fy = 250 MPa

Design load calculations :

Factored load = γ_{LD} x 20 + γ_{LL} x 40

Using partial safety factors for D.L γ_{LD} = 1.50 and for L.L γ_{LL} = 1.5

(*Cl. 5.3.3 Table 4, Page 29)*

Total factored load = 1.50 x 20 + 1.5 x 40 = 90 kN/m

Factored Bending Moment M = 90 x 5 x 5 / 8

= 281.25 kN.m

Zp required for value of *fy* = 250 MPa and

γ_{mo} = 1.10

(*Table 5, Page 30*)

Zp = (281.25 x 1000 x 1000 x 1.1) / 250 = 1237500 mm^3

= 1237.50cm3

Using shape factor = 1.14, *Ze* = 1237.50/1.14 =1085.52 *cm3*

Options ISWB 400 @ 66.7 *kg/m* or ISLB 450 @ 65.3 *kg/m*

Try ISLB 450

Ze = 1223.8 cm^3 > 1085.52

Geometrical Properties : ISLB 450

D = 450 mm , B = 170 mm , tf = 13.4 mm , tw = 8.6 mm , h1 = 384 mm , h2 = 33 mm

Ixx = 27536.1 cm4

As *fy = 250 MPa ,* $\varepsilon = \sqrt{\frac{250}{fy}} = 1$

Section Classification :

B/2tf = 85 / 13.4 = 6.34 < 9.4ε

h1 / tw = 384/8.6 = 44.65 < 83.9 ε

Section is Classified as *Plastic*

Zp = 1.14 x 1223.8 = 1395.132 cm3

Design Bending Strength: Md

$$M_d = \frac{\beta_b Z_p fy}{\gamma_{mo}} = \frac{1.0x1395.132x1000x250}{1.10} = 317.075 kN.m$$

> 281.25 kN.m

βb = 1.0 for plastic section (Cl. 8.2.1.2, Page 53)

Check for Serviceability – Deflection

Load factor = γLD and γLL = 1.00 both , (Cl. 5.6.1, Page 31)

Design load = 20 + 40 = 60 kN/m

$$\delta = \frac{5x60x(5000)^4}{384x2x10^5x27536.1x10^4} = 8.866mm$$

Limiting deflection = Span/360 *(Table. 5.3, Page 52)*

= 5000/360 = 13.889 mm….*OK*

Hence Use ISLB 450

Working Stress Method
IS : 800 - 1984

Max Bending Moment = 60 x 5 x 5/8 = 187.5 kN.m

Max Shear Force = 60 x 5/2 = 150 kN

$$Zreq = \frac{187.5x10^6}{165} = 1136.3\,cm^3$$

Select ISLB 450 Zxx = 1223.8 Moment Capacity

= 201.927 kN.m

Check for Shear

$$q_{av} = \frac{150x1000}{450x8.6} = 38.76MPa < 100\ MPa$$

Check for Deflection

$$\delta = \frac{5x60x(5000)^4}{384x2x10^5x27536.1x10^4} = 8.866mm$$

Limiting deflection = Span/325 = 5000/325

= 15.38 mm…*OK*

Comparison of ISLB 450 Section

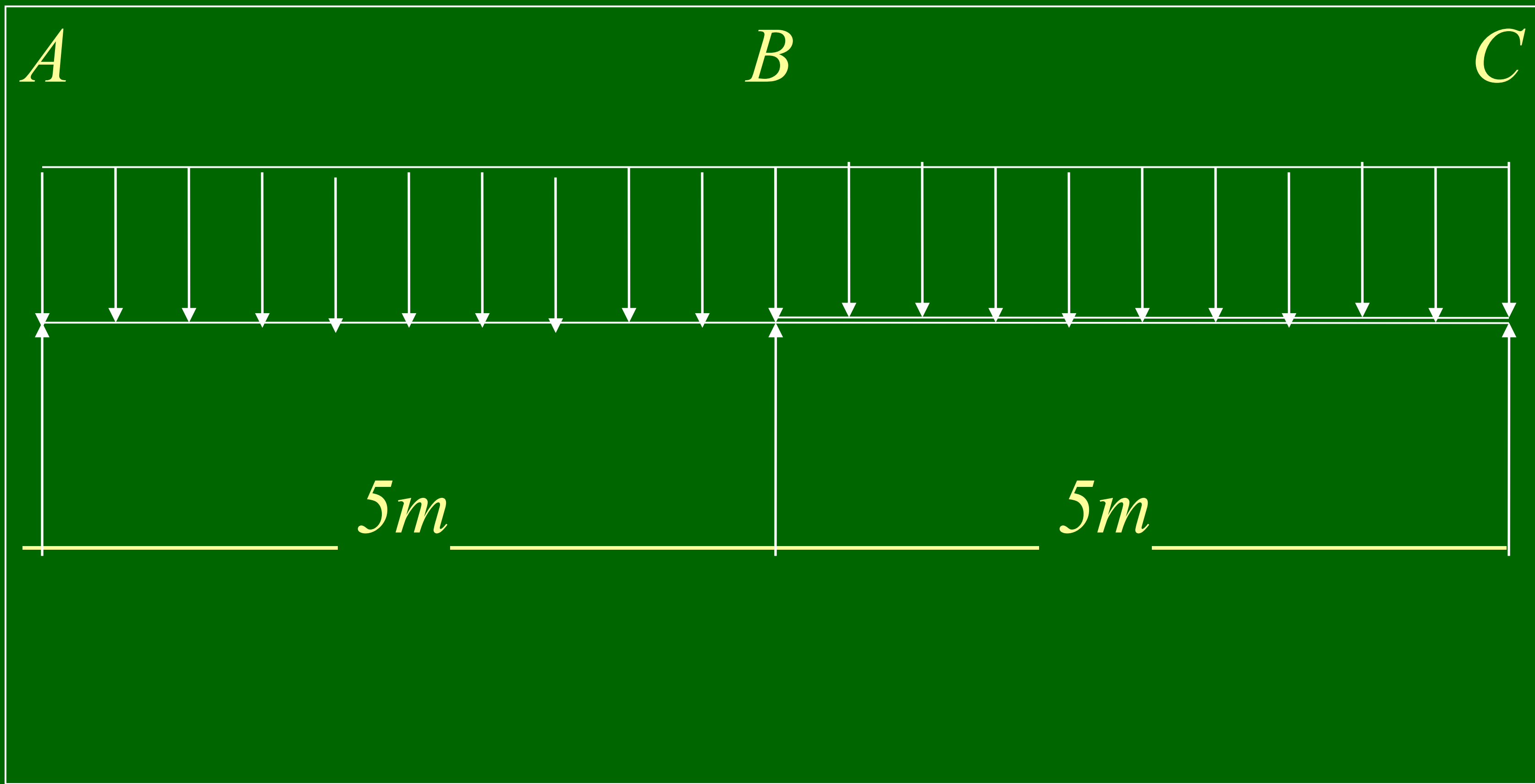
	Working Stress Method	Limit State Method
Moment Capacity	201.927 kN.m > 187.5 KNm	317.075 KNm > 281.25 KNm
Shear Capacity	387 KN > 150 KN	507.497KN > 225 KN
Section Designed	ISLB 450@ 65.3 Kg/m	ISLB 450 @ 65.3 kg/m

The Section designed as per LSM is having more reserve capacity for both BM and SF as compared to WSM

Design of Beam with High Shear LSM

Example No. 2

Factored Load 100 KN/m



Plastic Analysis

Degree of Redundancy = r = 1

No. of plastic hinges required to transform structure into mechanism = r + 1 = 2

Failure of any span is failure of continuous beam.

Failure mechanism of AB & BC is identical due to symmetry & this is similar to failure mechanism of propped cantilever beam with udl.

$$wp = 11.656\ Mp / l^2$$

$$\therefore\ Mp = wp.l^2 / 11.656$$

$$= 100 \times 25 / 11.656$$

$$= 214.48\ KNm.$$

As both spans fail simultaneously actual no of plastic hings are three – two hinges each at 0.414 l from A & C & third at B.

$\therefore$ as $n = 3 > 2$ required

Collapse is over complete

$$Z_p = 214.48 \times 10^6 \times 1.10 / 250 \quad mm^3$$

$$= 943.72\ cm^3$$

$$Z_e = 943.72 / 1.14 = 827.82\ cm^3$$

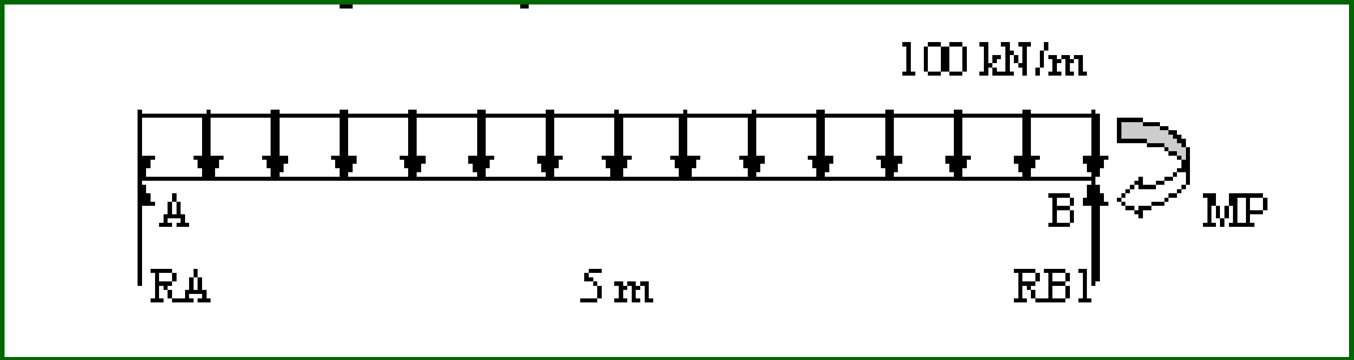
Select ISLB 400 $\quad Z_{xx} = 965.3\ cm^3$

$$M_d = 1.0 \times 1.14 \times 965.3 \times 250 / 1.10 = 250.1\ KNm$$

$$> 214.48$$

Reaction at A

Considering free body of AB



$M_p = 214.48$ KNm

$M_p + RA \times 5 = 100 \times 5 \times 5/2$ $\therefore RA = 207.1$ KN

$RB1 = 500 - 207.1 = 292.9$ KN

Due to symmetry in loading

Maximum shear is at B = 292.9 KN= V

$V_d = 0.577 \times 400 \times 8 \times 250 / 1.1 = 419.636$ KN

Where $400 \times 8 = D.t_w$ of ISLB 400

As $V/V_d = 292.9 / 419.636 = 0.697 > 0.6$

As per *C1.9.2.2 Page No. 70*

Effect of shear is to be considered for reduction in moment capacity

$M_{dv} = M_d - \beta(M_d - M_{fd})$

$\beta = (2V/V_d - 1)2 = 0.156$

M_{fd} = Plastic moment capacity of flanges only

$= 165 \times 12.5 (400 - 12.5) \times 250 / 1.1 = 181.64$ KNm

$\therefore M_{dv} = 250.1 - 0.156 (250.1 - 181.64)$

$= 239.42$ KNm

As $M_{dv} = 239.42 > M_p = 214.48$ ------- Ok

Select ISLB 400 @ 56.9 kg / m

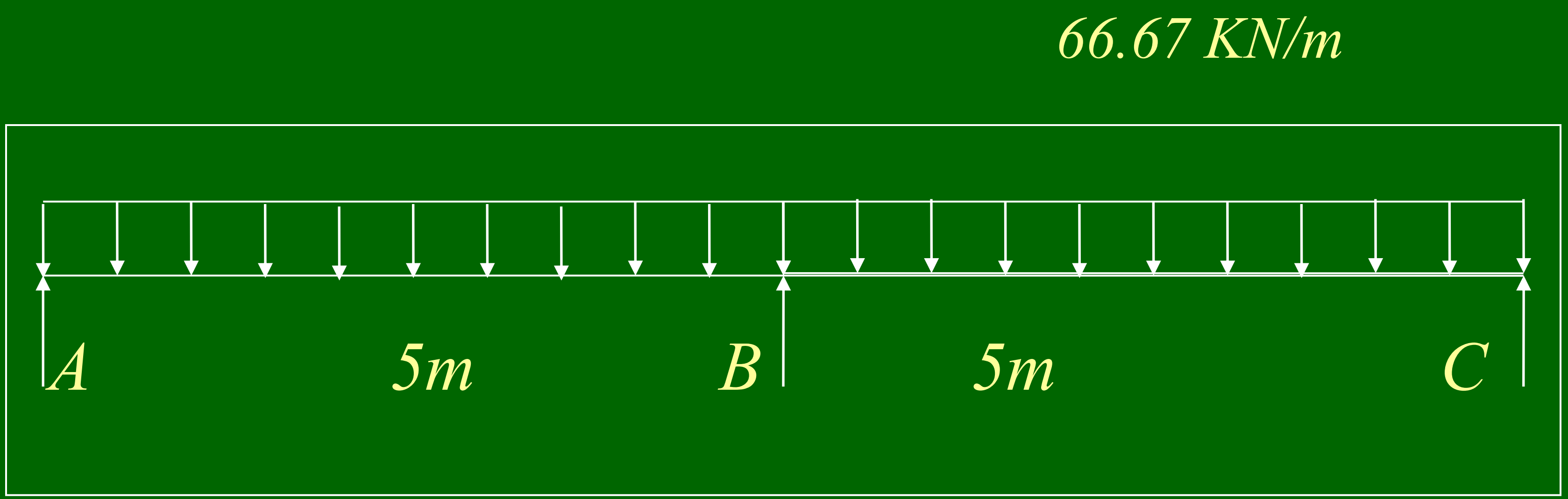
Laterally supported beam

Design of Beams with High Shear by WSM

Factored load in LSM is 100 KN/m

$\therefore$ Working load in WSM = 100 / 1.5

= 66.67 KN/m



Reactions -

RB = 5/8 x 66.67 x 10 = 416.66 kN ,

RA = RC = 125.0 kN

Maximum Bending Moment

At continuous support = 125.0 x 5 – 66.67 x 5 x 5/2

= -208.33 kN.m

Design Shear = 208.33 kN

Design Moment = 208.33 kN.m

As per IS:800 – 1984, 6bc = 0.66fy = 0.66 x 250 = 165 MPa

Z required = (208.33 x 106) / 165

= 1262.62 cm3

Try ISMB 450 @ 72.4 kg/m.

Zxx = 1350 cm2 > 1262.62

Cheak for shear tw = 9.4 mm

qav = (208.33 x 1000) / (450 x 9.4) = 49.25 N/mm2 < 0.4fy i.e. 100 N/mm2

Comparison of WSM vs LSM

	Working Stress Method	Limit State Method
Moment Capacity	222.75 KNm > 208.33 KNm	239.42 KNm > 214.48
Shear Capacity	423 KN > 208.33 KN	419.636 KN > 292.90 KN
Section Designed	ISMB 450 @ 72.4 kg/m	ISLB 400 @ 56.9 kg/m

Design of beam by LSM is more economical

DESIGN OF GANTRY GIRDER

FEATURES

- Design of Gantry Girder is a classic example of laterally unsupported beam.
- It is subjected to in addition to vertical loads horizontal loads along and perpendicular to its axis.
- Loads are dynamic which produces vibration.
- Compression flange requires critical attention.

IS:800-2007 PROVISIONS

- Partial safety factor for both dead load and crane load is 1.5 (Table 4, p. no. 29).

- Partial safety factor for serviceability for both dead load and crane load is 1.0 (Table 4, p. no. 29).

- Deflection limitations (Table 6, p. no. 31).

  Vertical loads

  i) Manually operated… Span/500

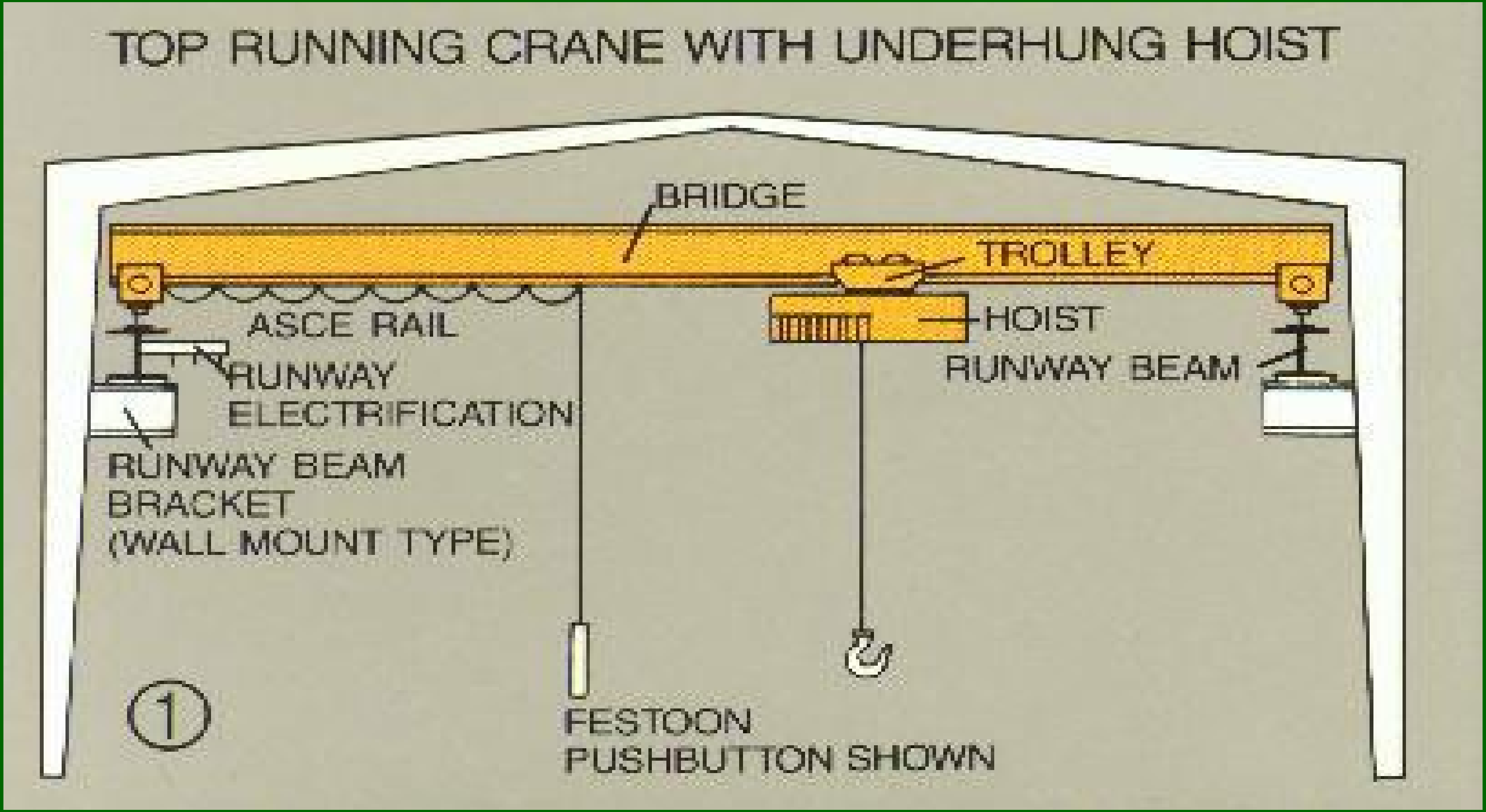
  ii) Electric operated.... Span/750
  up to 50t

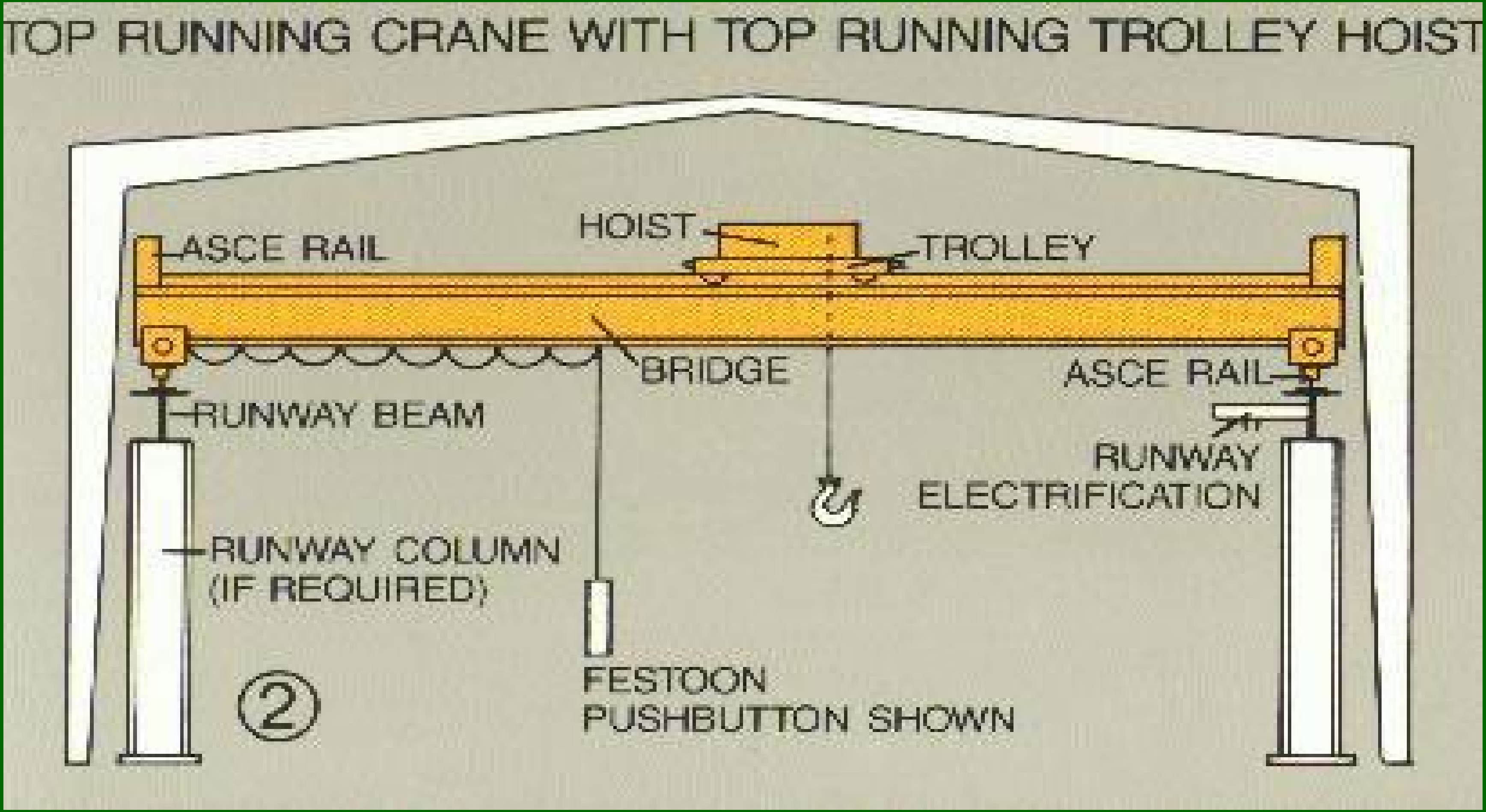
  iii) Electric operated… Span/1000
  over 50t

OTHER CONSIDERATIONS

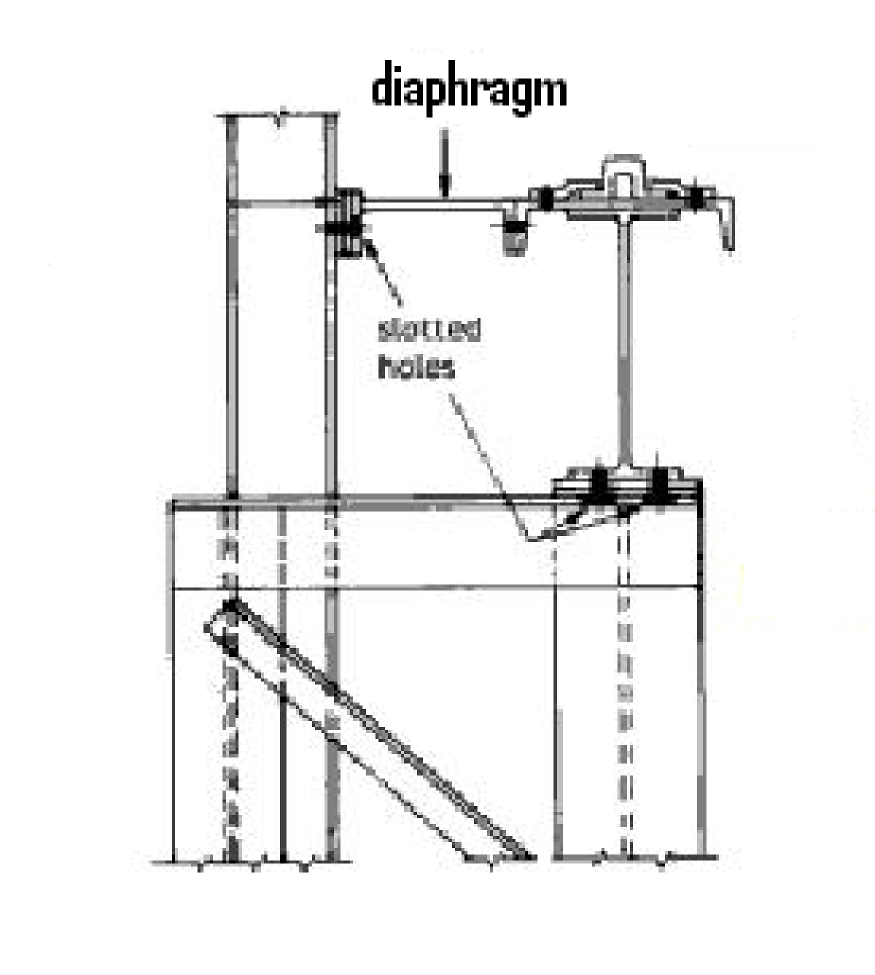
- Diaphragm must be provided to connect compression flange to roof column of industrial building to ensure restraint against lateral torsional buckling.

- Span is considered to be simply supported to avoid bumpy effect.

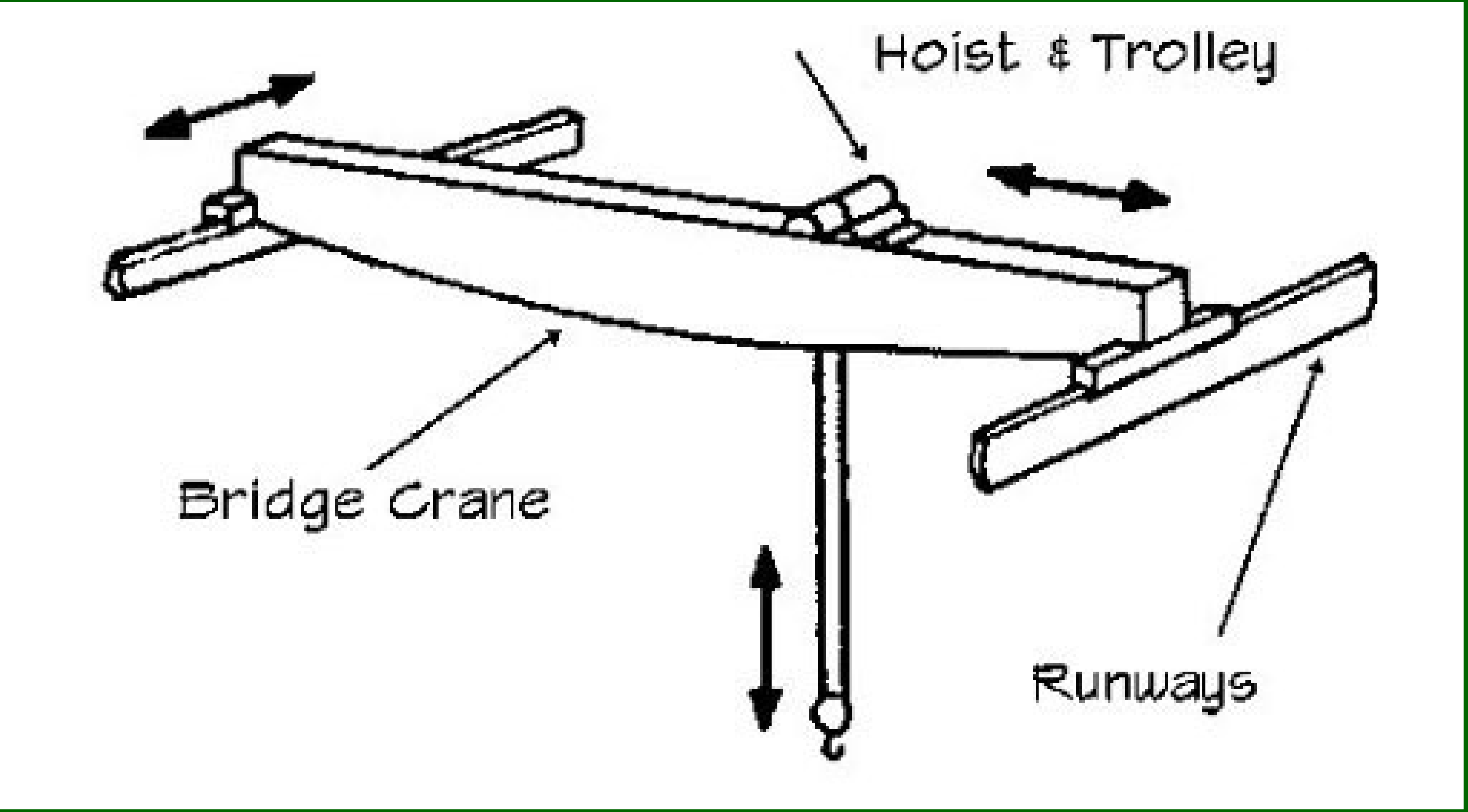

TOP RUNNING CRANE WITH UNDERHUNG HOIST
BRIDGE
TROLLEY
ASCE RAIL
HOIST
RUNWAY ELECTRIFICATION
RUNWAY BEAM
RUNWAY BEAM BRACKET (WALL MOUNT TYPE)
1
FESTOON PUSHBUTTON SHOWN


TOP RUNNING CRANE WITH TOP RUNNING TROLLEY HOIST
HOIST
TROLLEY
ASCE RAIL
BRIDGE
ASCE RAIL
RUNWAY BEAM
RUNWAY
ELECTRIFICATION
RUNWAY COLUMN
(IF REQUIRED)
2
FESTOON
PUSHBUTTON SHOWN

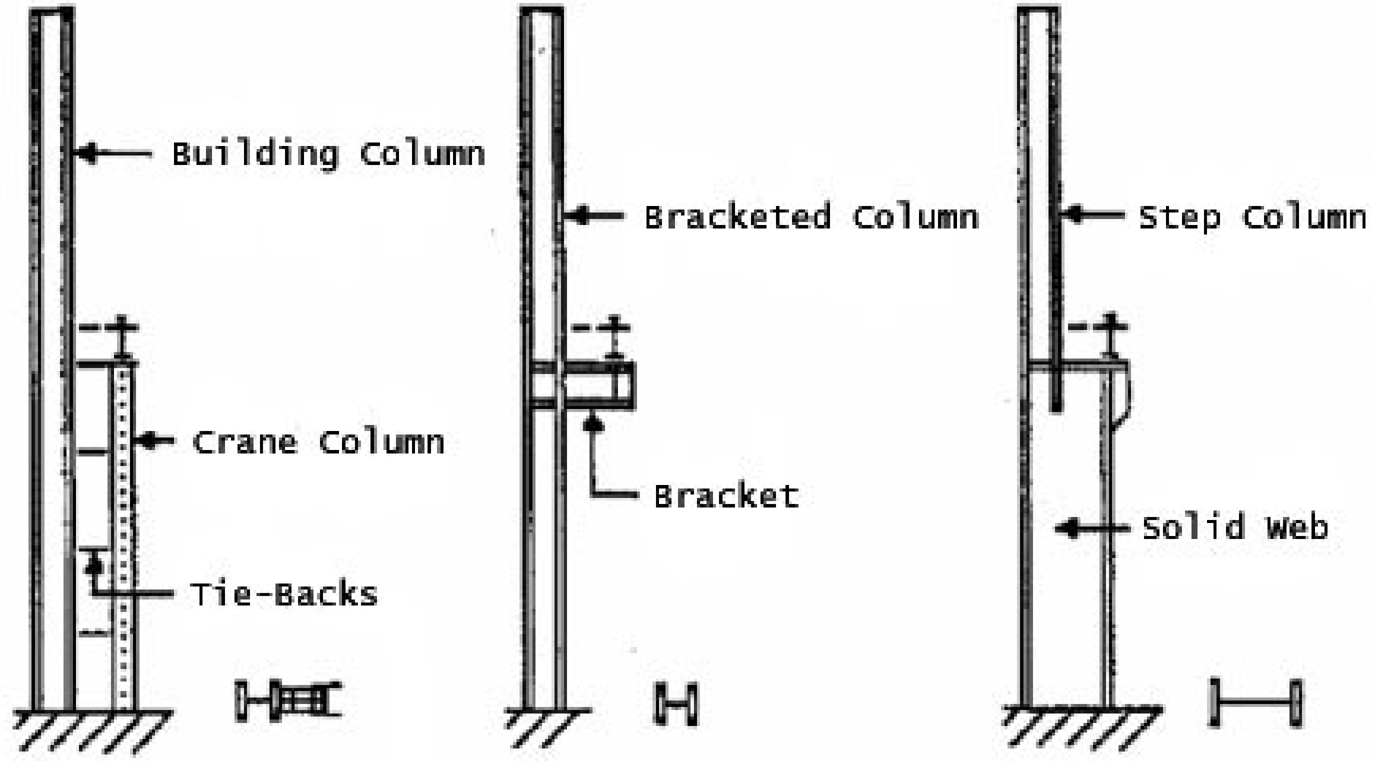
TYPICAL GANTRY GIRDER DETAILS



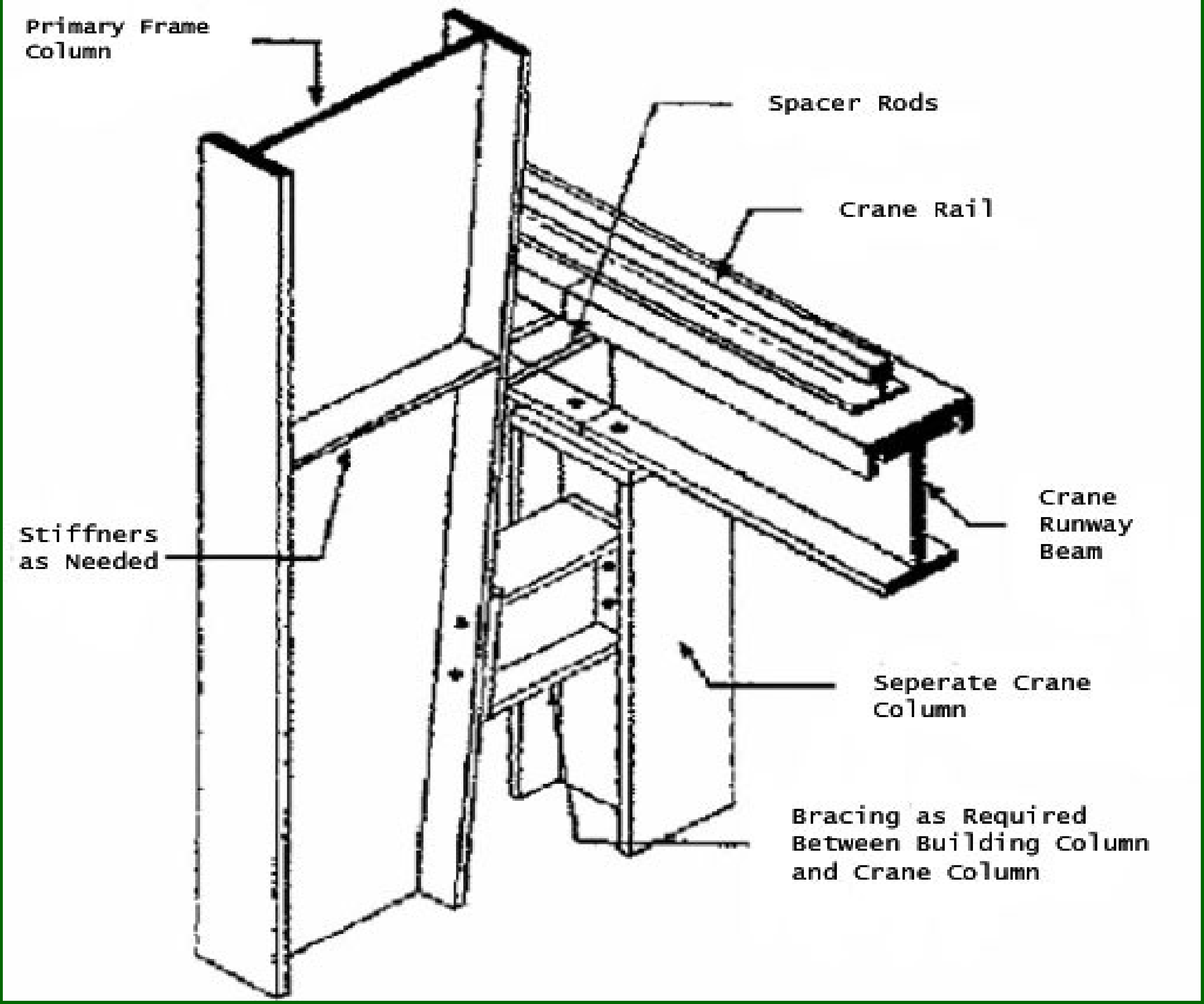
FORCES AND MOTIONS



VARIOUS TYPES OF SUPPORTS



(A) The Building/Crane Column (B) The Bracketed Column (C) The Stop Column-Solid Web


Primary Frame
Column
Spacer Rods
Crane Rail
Stiffners
as Needed
Crane
Runway
Beam
Seperate Crane
Column
Bracing as Required
Between Building Column
and Crane Column

IMPACT FACTORS

Type of load	Additional load
• *Vertical loads*	
a) EOT crane…	25% of static wheel load
b) HOT crane…	10% of static wheel load
• *Horizontal forces transverse to rails*	
a) EOT crane…	10% of wt. of crab & wt. lifted
b) HOT crane…	05% of wt of crab & wt. lifted
• *Horizontal forces along the rails*	
For both EOT & HOT cranes	05% of static wheel load

Note: *Gantry Girder & their vertical supports are designed under the assumption that either of the horizontal forces act at the same time as the vertical load.*

GANTRY GIRDER DESIGN

Data

a) Wt. of crane girder/truss… 180kN

b) Crane capacity… 200kN

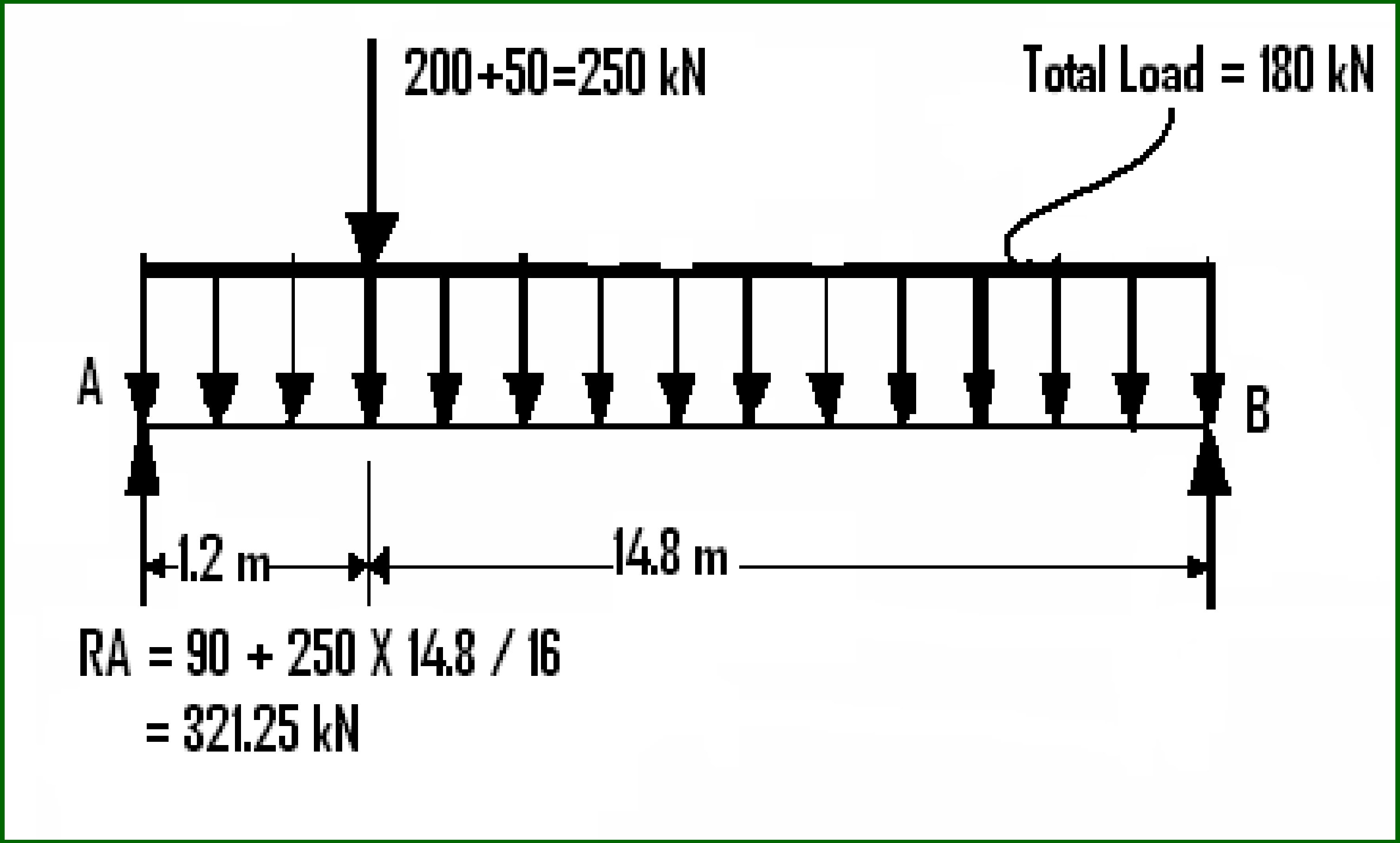
c) Wt. of crab + motor… 50kN

d) Span of crane girder/truss… 16m

e) Min hook approach… 1.2m

f) c/c distance bet^{n} grantry columns… 6m

g) Wt. of rail… 0.25kN/m



- Maximum vertical static wheel load = $R_A/2$
  $=160.625$ kN

Wheel load with impact = 1.25 X 160.625

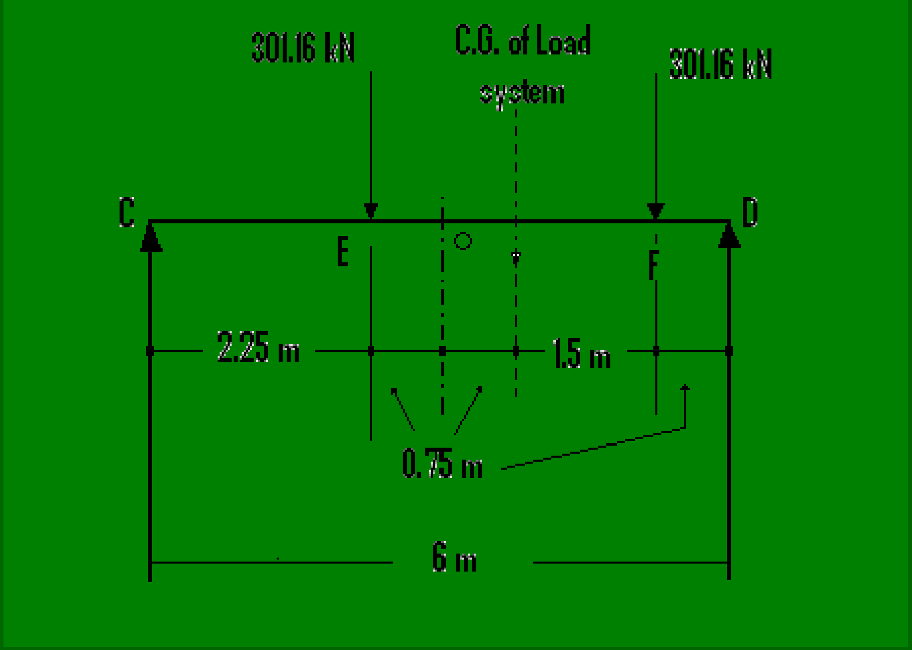
= 200.775 kN

Factored load = 1.5 X 200.775

= 301.16 kN

Absolute max bending moment in Gantry Girder

This will occur under any wheel load when distance bet^n that load and C.G. of load system is equidistant from the centre of the Gantry Girder span.



Absolute max bending moment = 508.21 kNm

Md = Design moment for laterally unsupported beam

$= \beta_b \cdot Z_p \cdot f_{bd}$ *(Clause 8.2.2, p. no. 54)*

Where β_b = 1.0 for plastic section (assumed)

Z_p = plastic modulus of section

f_{bd} = design bending compressive stress

Assuming f_{bd} = 200 Mpa

Z_p required = $(508.21 \times 10^6) / (1.0 \times 200)$

$= 2.54 \times 10^6$ mm^3

Using I and channel section and assuming 80% of Z_p is contributed by I section

Z_p by I section = 2.032×10^6 mm^3

using shape factor of I section = 1.14

Z_e required = 2032 / 1.14 = 1766.95 cm^3

select ISWB 500 @ 0.94 kN/m

Z_e provided = 2091.6 > 1766.95 cm^3 …. OK

Width of the flange of ISWB 500 = 250 mm

Select channel section having clear web depth more than 250 mm.

Select ISLC 350 @ 0.38 kN/m

having h_1 = 291.9 mm > 250 mm ….. OK

Total dead load intensity = 0.94 + 0.38 + 0.25

= 1.57 kN/m

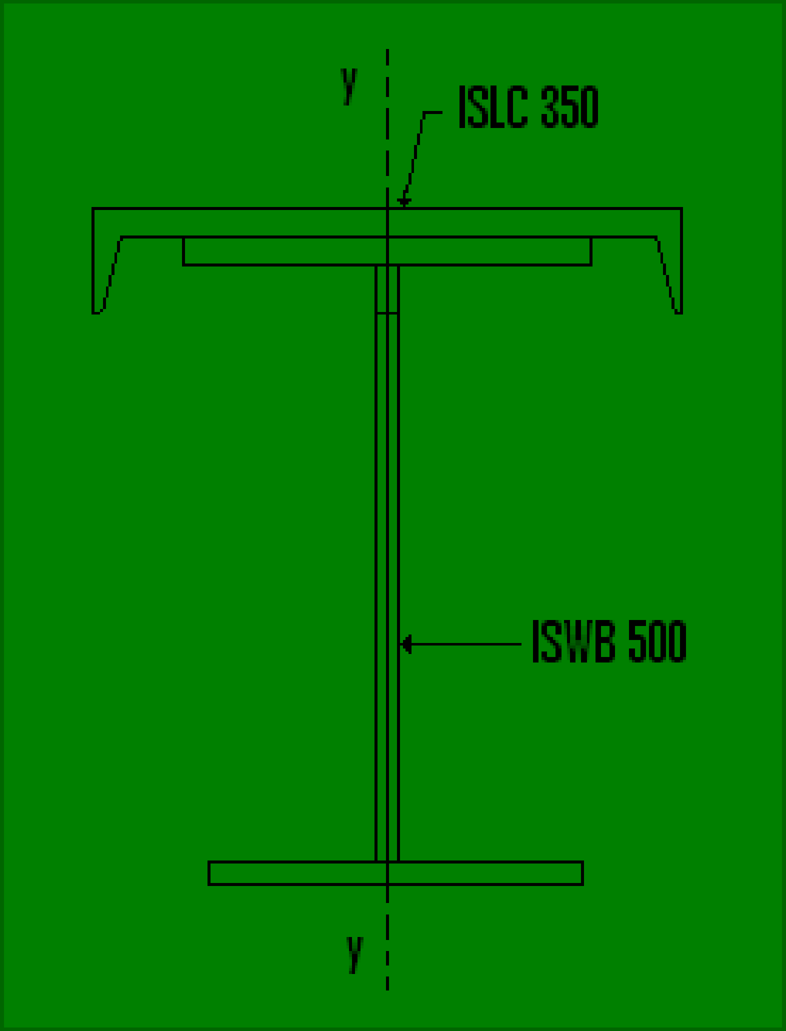
Factored dead load intensity = 1.5 X 1.57

= 2.355 kN/m

Bending moment @ E = 9.93 kNm

Total bending moment due to DL + CL = 518.14 kNm

SELECTED CROSS SECTION



Refer Annexure E *(p. no. 128)*

Elastic lateral torsional buckling moment

Elastic critical moment of a section symmetrical about minor axis yy is given by E-1.2 of Annexure E *(p. no. 128)* in which various factors and geometrical values of Gantry Girder section are involved.

These are as under

c_1, c_2, c_3, = factors depending upon the loading and end restraint conditions, *Refer table 42(p. no. 130)*

K = effective length factor = 0.8

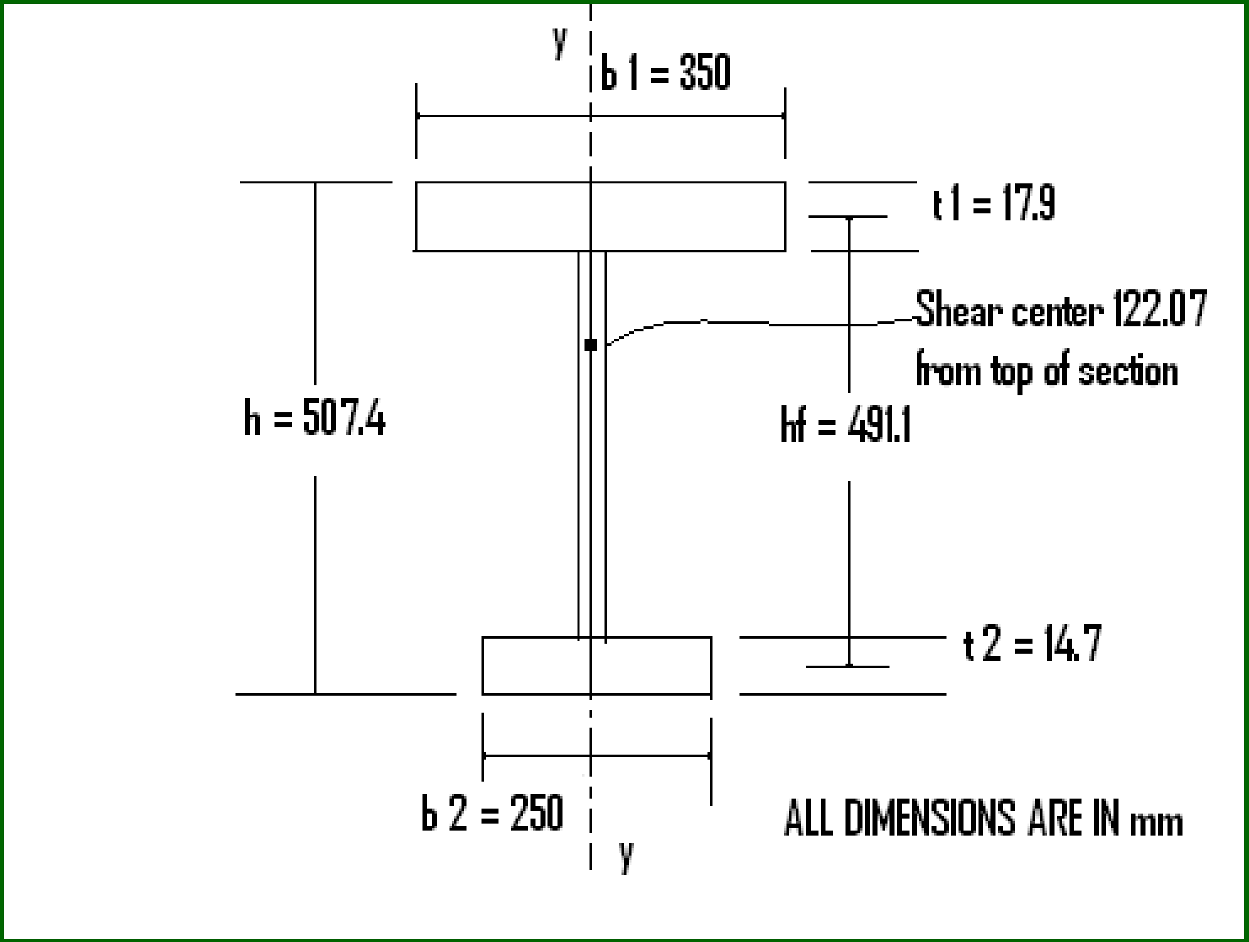
Therefore $c_1 = 1.03$, $c_2 = 0.422$ & $c_3 = 1.22$

K_w = warping restraint factor = 1.0

y_g = y distance bet^n the point of application of the load & shear centre of the cross section (+ve when load acts towards Shear centre)

= 122.07 mm

LOCATION OF SHEAR CENTRE



y_j for lipped flanges of channel section which depends on ratio of β_f

Where $\beta_f = I_{fc} / (I_{fc}+I_{ft})$.

$= 0.7$

$y_j = 94.055$

Iyy = Iyy of ISWB 500 + Ixx of ISLC 350

$= 2987.8 + 9312.6 = 12300.4 \text{ X } 10^4 \text{ mm}^4$

$L_{LT} = K . L = 0.8 \text{ X } 6000 = 4800 \text{ mm}$

I_w = warping constant

$= (1- \beta_f)\, \beta_f . I_y . (h_y)^2$

$= 6.23 \text{ X } 10^{12} \text{ mm}^6$

I_t = Torsion constant

= $\sum bt^3/3$ = 10.86 X 10^5

G = 0.77 X 10^5

$$Mcr = c_1 \frac{\pi^2 EIy}{(L_{LT})^2} \{[(\frac{k}{k_w})^2 \frac{I_w}{I_Y} + \frac{GI_t (L_{LT})^2}{\pi^2 EI_y} + (c_2 y_g - c_3 y_j)^2]^{0.5} - (c_2 y_g - c_3 y_j)\}$$

= 2950 kNm

To find Z_p of Gantry Girder section we need to find equal area axis of the section.

This axis is at a depth of 48.74 mm from the top of the section.

Taking moments of areas about equal area axis.

$\sum A \cdot y = Z_p$ = 29.334 X 10^5 mm^3

Refering clause 8.2.2 for laterally unsupported beam

(p. no. 54)

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}}$$

$= 0.4984$

$\alpha_{LT} = 0.21$ for rolled section

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2] = 0.655$$

$$\chi_{LT} = 1/(\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]) = 0.925$$

Therefore $f_{bd} = \chi_{LT} \cdot f_y / \gamma_{m0}$

$= 0.925 \text{ X } 250 / 1.1 = 210.22 \text{ N/mm}^2$

$M_{dZ} = \beta_b \cdot Z_p \cdot f_{bd} = 616.66$ kNm $> M_d = 508.21$ kNm…

OK

Horizontal Action

Total horizontal force perpendicular to span of Gantry Girder = 10 % (crane capacity + wt. of crab and motor)

= 10% (200+50) = 25 kN.

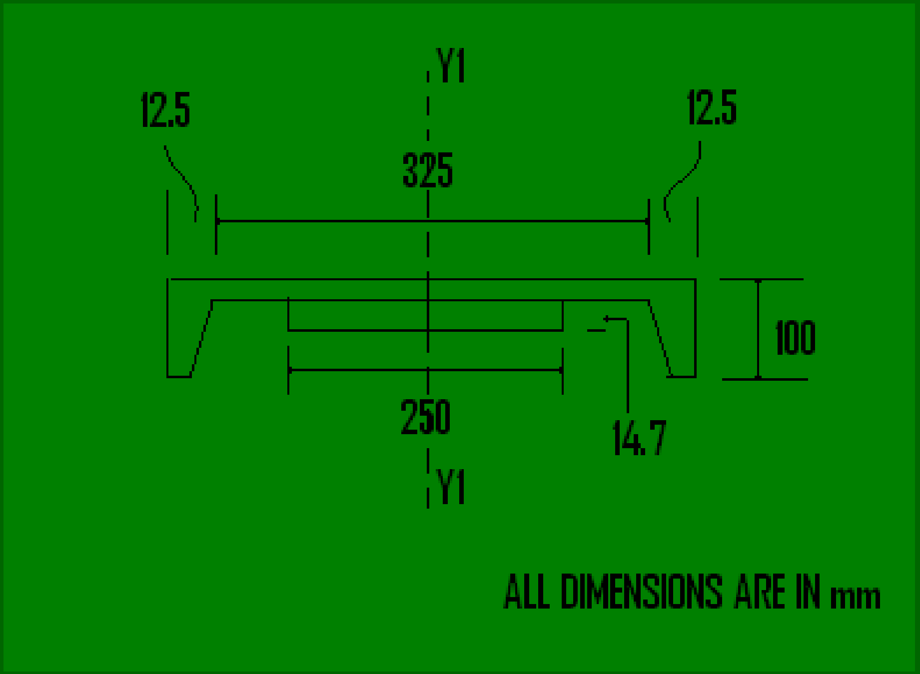
As wheels are having double flanges

Horizontal force / wheel = 25/4 = 6.25 kN

Therefore max^{m} horizontal BM in proportion to vertical bending moment

$$M_y = (6.25 / 301.16) \times 508.21 = 10.546 \text{ kNm}$$

This is resisted by ISLC 350 with top flange of ISWB 500



$$Z_{py1y1} = 100 \text{ X } 12.5 \text{ X } 337.5^2 + (1/4)\ 7.4 \text{ X } 325^2 + (1/4) \text{ X } 14.7 \text{ X } 250^2$$

$$= 8.47 \text{ X } 10^5 \text{ mm}^3$$

Plastic moment capacity about $y_1 y_1$ axis

$M_{dy} = \beta_b \,.\, f_y \,.\, Z_p / \gamma_{mo}$

$= 192.5$ kNm

Check for biaxial moment

Reffering clause 9.3.1.1 (p. no. 70)

$(M_z/M_{dz}) + (M_y/M_{dy})$

$= (518.14 / 614.57) + (10.54 / 192.5)$

$= 0.897 < 1.0$ …….. OK

Hence select section for the gantry Girder as ISWB 500 and ISLC 350 over it.

DESIGN OF BEAM COLUMN

DESIGN OF BEAM COLUMN

Combined action of bending and axial force (tension or compression) occurs in following situations.

- Any member in a portal frame.
- Beam transferring reaction load to column.
- Effect of lateral load on a column due to wind, earthquake
- Effect of eccentric load by crane loading due to bracket connection to column.
- In case of principal rafter, purlins not placed exactly over joint of roof truss.

IS : 800 – 2007 CODAL PROVISIONS

- Minimum eccentricity of load transferred by beam to column is specified by clause 7.3.3 (p. no. 46)
- Section-9, Member subjected to combined forces.

  clause 9.3 for combined axial force and bending moment (p. no. 70) recommends check for section

  a) By material failure clause 9.3.1

  b) By overall buckling failure clause 9.3.2

DESIGN OF BEAM COLUMN

DATA

A column in a building 4m in height bottom end fixed, top end hinged.

reaction load due to beam is 500 kN at an eccentricity of 100 mm from major axis of section.

DESIGN

Column is subjected to axial compression of 5×10^5 N with bending moment of 50×10^6 Nmm.

Taking design compressive stress for axial loading as 80 Mpa.

A_e reqd = 500 X 10^3 / 80 = 6250 mm^2

To account for additional stresses developed due to bending compression.

Try ISHB 300 @ 0.58 kN/m

A_g = 7485 sq.mm, r_{xx} = 129.5 mm, r_{yy} = 54.1 mm

f_y = 250 Mpa

Classification of section

$b/t_f = 125 / 10.6 = 11.79 > 10.5$ (limit for compact section)

Flange is semicompact

$h_1/t_w = 249.8 / 7.6 = 32.86 < 84$

Web is plastic

Therefore overall section is semicompact.

a) Section strength as governed by material failure *(clause 9.3.1)*

Axial stress = N/A_e = 500 X 10^3 / 7485

= 66.80 N/mm^2

Bending stress M_z/Z_e = 50 X 10^6 / 836.3 X 10^3

= 59.78 N/mm^2

As the section is semicompact use clause 9.3.1.3 *(p. no. 71)*

Due to bending moment at top, horizontal shear developed 'V' is 18.75 kN = 18750 N

Shear strength of section $V_d = ((f_y / \sqrt{3}) \cdot h \cdot t_w) / 1.10$

= 299 kN

As $V/V_d = 18750 / 299 \times 10^3 = 0.062 < 0.6$

Reduction in moment capacity need not be done.

As per clause 9.3.1.3 *(p. no. 71)*

Total longitudinal compressive stress

$f_x = 66.80 + 59.78$

$= 126.58 < f_y/\gamma_{mo} = 227.27$…… OK

Alternately

$N = 500$ kN

$N_d = A_g \cdot f_y / \gamma_{mo} = 7485 \times 250 / 1.1 = 1701.136$ kN

$M_z = 50 \times 10^6$ Nmm $= 50$ kNm

$M_{dz} = Z_e \cdot f_y / \gamma_{mo} = 836.3 \times 10^3 \times 250 / 1.10$

$= 190.068$ kN

Hence, $(500 / 1701.136) + (50 / 190.068)$

$= 0.557 < 1$ ……. OK

b) Member strength as governed by buckling failure *clause 9.3.2 (p. no. 71)*

In the absence of M_y, equations are reduced to

$$\frac{P}{P_{dy}} + k_{LT}\frac{M_z}{M_{dz}} \leq 1$$

$$\frac{P}{P_{dz}} + k_z\frac{C_{mz}M_z}{M_{dz}} \leq 1$$

Where, $P = 500 \times 10^3$ N

$M_z = 50 \times 10^6$ Nmm

$M_{dz} = \beta_b \cdot Z_p \cdot f_{bd}$

$\beta_b = Z_e / Z_p$ as section is semicompact

Therefore $M_{dz} = Z_e f_{bd}$

$f_{bd} = \chi_{LT} f_y / \gamma_{mo}$

χ_{LT} = bending stress reduction factor to account torsional buckling.

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \leq 1$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$

$\alpha_{LT} = 0.21$ for rolled section

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{cr,b}}}$$

$f_{cr,b}$ depends on following factors

$k_L / r_{yy} = 0.8 \text{ X } 4000 / 54.1 = 59.15$

$h / t_f = 300/10.6 = 28.30$

Using table 14, (p. no. 57)

$f_{cr,b} = 691.71 \text{ N/mm}^2$

$$\lambda_{LT} = \sqrt{\frac{250}{691.71}} = 0.060 < 0.4$$

As per *clause 8.2.2 (p. no. 54)*

Resistance to lateral buckling need not be checked and member may be treated as laterally supported.

$M_{dz}=Z_e \, . \, f_y / \gamma_{mo} = 190$ kNm

Evaluation of P_{dy} buckling load @ yy axis

Referring table 10 (p. no. 44)

$h/b_f=300/250 = 1.2$

buckling @ yy axis is by class 'c'

$t_f = 10.6$ mm < 100mm

buckling @ zz axis is by class 'b'

$l_y / r_y = 3200/54.1 = 59.15$

For $f_y = 250$ and using *Table 9 (c), (p. no. 42)*

$F_{cdy} = 169.275 \text{ N/mm}^2$

$P_{dy} = A_g . f_{cdy}$

$= 1267.02 \text{ kN}$

Evaluation of P_{dz} buckling @ zz axis

$l_z / r_z = 3200 / 129.5 = 24.71$

For $f_y = 250$ and using *Table 9 (b), (p. no. 41)*

$f_{cdz} = 220.76 \text{ N/mm}^2$

Therefore $p_{dz} = A_g . f_{cdz}$

$= 1652.38 \text{ kN}$

$K_z = 1 + (\lambda_z - 0.2)n_z$

Where,

$$\lambda_z = \sqrt{\frac{f_y}{f_{cr,z}}}$$

$l_z / r_z = 24.71$, $h/t_f = 300 / 10.6 = 28.30$

From table 14 (p. no. 57)

$f_{cr,z} = 4040\ N/mm^2$

Ratio of actual applied load to axial strength,

$n_z = 500 / 1625.38 = 0.30$

$n_y = 500 / 1267.02 = 0.39$

$\lambda_z = \sqrt{250/4040} = 0.246$

$K_z = 1 + (\lambda_z - 0.2)\, n_z = 1.0138 < 1 + 0.8\, n_z$

< 1.24…. OK

ψ = ratio of minimum to maximum BM

$\psi = -25 / 50 = -1 / 2$

$C_{mz} = 0.6 + 0.4 \times (\psi) = 0.4$

$$K_{LT} = 1 - \frac{0.1 \lambda_{LT} n_y}{C_{mLT} - 0.25}$$

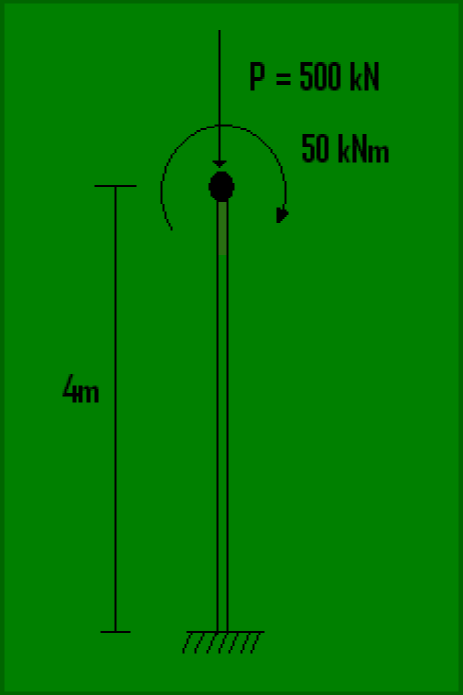
$= 0.844$

$$\frac{P}{P_{dy}} + K_{LT}\frac{M_z}{M_{dz}} = 0.612 \qquad < 1 \ \ldots\ldots \text{ OK}$$

$$\frac{P}{P_{dz}} + K_z\frac{C_{mz}M_z}{M_{dz}} = 0.406 \qquad < 1 \ \ldots\ldots \text{ OK}$$

Hence select ISHB 300 @ 0.58 kN/m as a section for eccentrically loaded column.

Design of Beam Column Working Stress Method IS : 800 - 1984



Checking section ISHB 300 @ 0.58 kN/m

A = 7485 sq mm

$\sigma_{ac,cal}$ = P/A = 66.80 N/mm^2

slenderness ratio = L / r_{yy} = 59.15

for f_y = 250 Mpa, σ_{ac} = 121.15N/mm^2

from table 5.1 (p. no. 39)

β=ratio of smaller to larger moment = 0.5

Therefore, $C_{mx} = 0.6 – 0.4 \times 0.5 = 0.4 \geq 0.4$ OK

$\sigma_{bcx,cal.} = 50000 / 836.3 = 59.78$ N/mm^2

f_{cc} = elastic critical stress in compression

$= \pi^2 E / \lambda^2 = 563.6$ N/mm^2

σ_{bcx} = Permissible bending stress in compression. As column is laterally unsupported following ratios are evaluated.

$D/T = 28.30$, $L / r_{yy} = 59.15$

As $T / L = 10.6 / 7.6 < 2$

for $f_y = 250$ using *table 6.1 B (p. no. 58)*

$\sigma_{bcx} = 150$ N/mm^2

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{C_{mx}\sigma_{bcx,cal}}{\left\{1 - \frac{\sigma_{ac,cal}}{0.6 f_{ccx}}\right\}\sigma_{bcx}} = 0.7486$$

< 1 ….. OK

Hence requirement of section for a column under eccentric load is same as ISHB 300 @ 0.58 kN/m

Beam Column

LSM	WSM
1) Interaction bet^n axial & uniaxial bending is considered taking buckling due to axial loading about both axes of c/s	1) Interaction is countered only by taking buckling due to axial load @ weaker axis with bending @ major axis.
2) $C_{mx} = 0.4$	2) $C_{mx} = 0.4$
3) Combined interaction is considered for buckling @ both axes of cross section.	3) Combined interaction is considered for buckling @ yy axis only.
4) Interaction values are @ yy axis… 0.612 @ zz axis… 0.406	4) Interaction value is @ yy axis… 0.7486

Thus reserve strength in a section by LSM is more than WSM.