DESIGN OF FLEXURAL MEMBERS

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Beam is basically a structural member which is subjected to transverse loading, that means the load is perpendicular to its axis. And because of this transverse loading the members produces bending moment as well as shear force. So we have to design a beam against bending moment and shear force.

In case of steel structure the beam is not only failed due to bending or due to shear but also failed due to lateral buckling, due to local buckling, due to torsional moment, so many things will come into picture.

In case of RC structure because in case of RC structure generally we provide rectangular section, where such type of problems will not come. But in case of steel structure we provide certain rolled section where the thickness of the member is quite less (I section the thickness of the flange the thickness web is quite less). So there will be chances of local buckling of the flange, web which we need to take care.

Beams are basically two types, primary beam and secondary beam. So secondary beam are rested on the primary beam and in case of bridge structure, we often use a term girder and this bridge structures are designed considering beam as a plate girder, where the girder dimensions are decided on the basis of the bending moment and other forces.

http://wasatchsteel.blogspot.in/

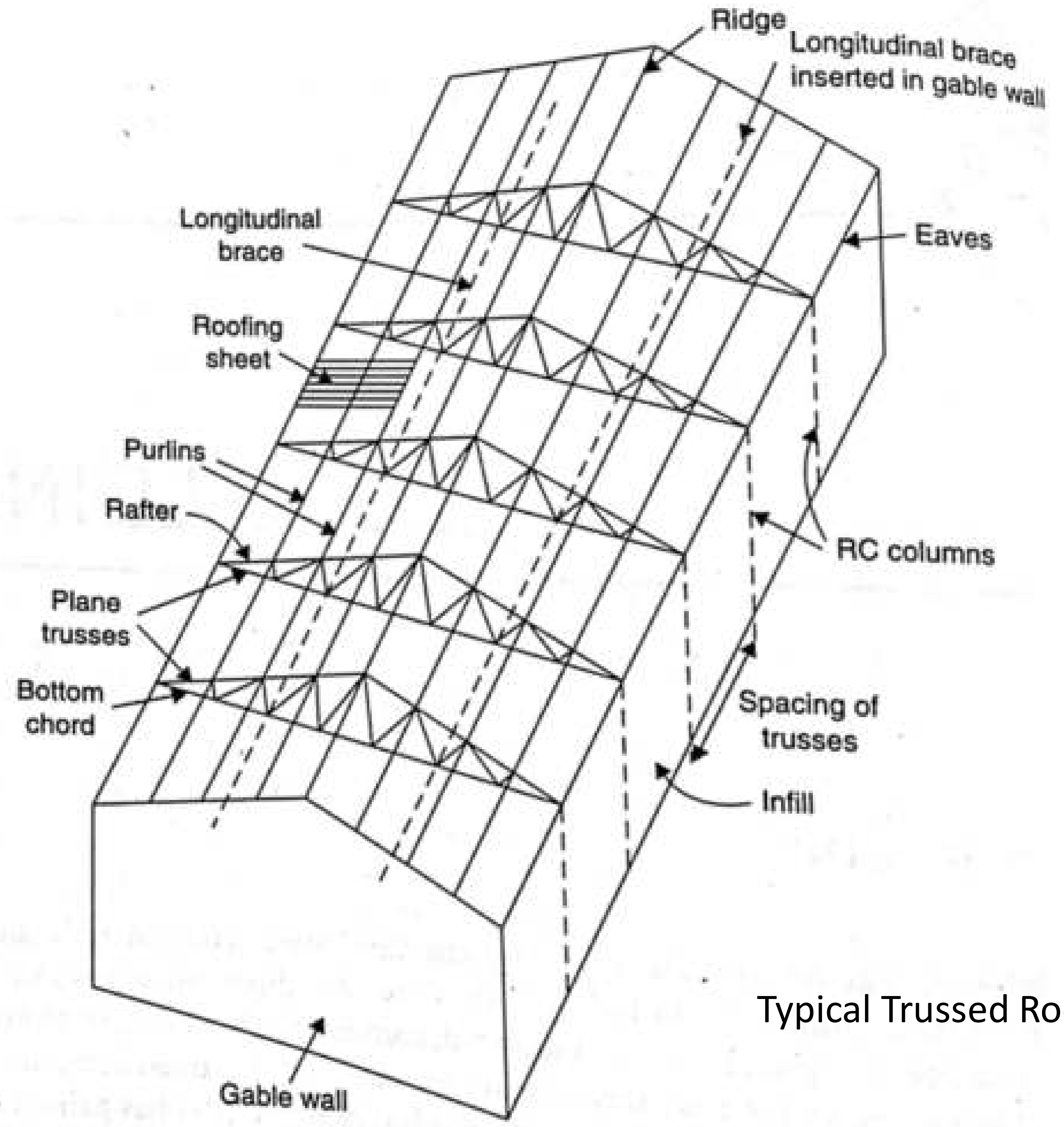
http://www.steel-bridges.com/

DIFFERENT TYPES OF BEAMS

- **SPANDREL BEAM:** In a building, a beam on the outside perimeter of a floor, supporting the exterior walls and outside edge of the floor
- **GIRT:** A horizontal beam spanning the wall columns of industrial buildings used to support wall coverings is called a **GIRT**.
- **RAFTER:** A roof beam usually supported by purlins.
- **LINTELS:** This type of beams are used to support the loads from the masonry over the openings .

DIFFERENT TYPES OF BEAMS

- **JOIST:** A closely spaced beams supporting floors or roofs of building but not supporting the other beams.
- **GIRDER:** A large beam, used for supporting a number of joists.
- **PURLIN:** Purlins are used to carry roof loads in trusses.
- **STRINGER:** In building, beams supporting stair steps; in bridges a longitudinal beam supporting deck floor & supported by floor beam.
- **FLOOR BEAM:** A major beam supporting other beams in a building; also the transverse beam in bridge floors.



Typical Trussed Roof

Primary modes of failure of beams are as follows:

1. Bending failure

2. Shear failure

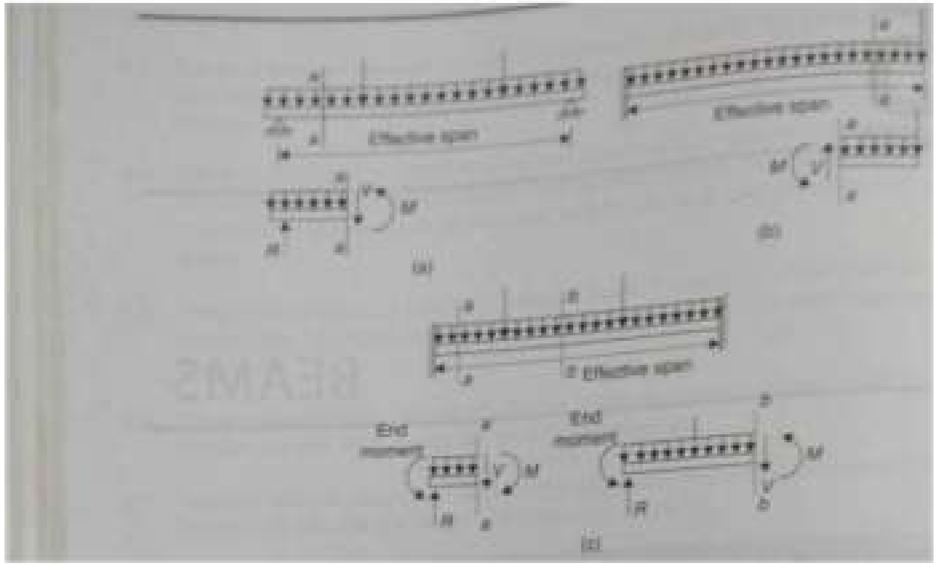
3. Deflection failure

1. **Bending failure:** Bending failure generally occurs due to crushing of compression flange or fracture of tension flange of the beam.
2. **Shear failure:** This occurs due to buckling of web of the beam near location of high shear forces. The beam can fail locally due to crushing or buckling of the web near the reaction of concentrated loads.
3. **Deflection failure:** A beam designed to have adequate strength may become unsuitable if it is not able to support its load without excess

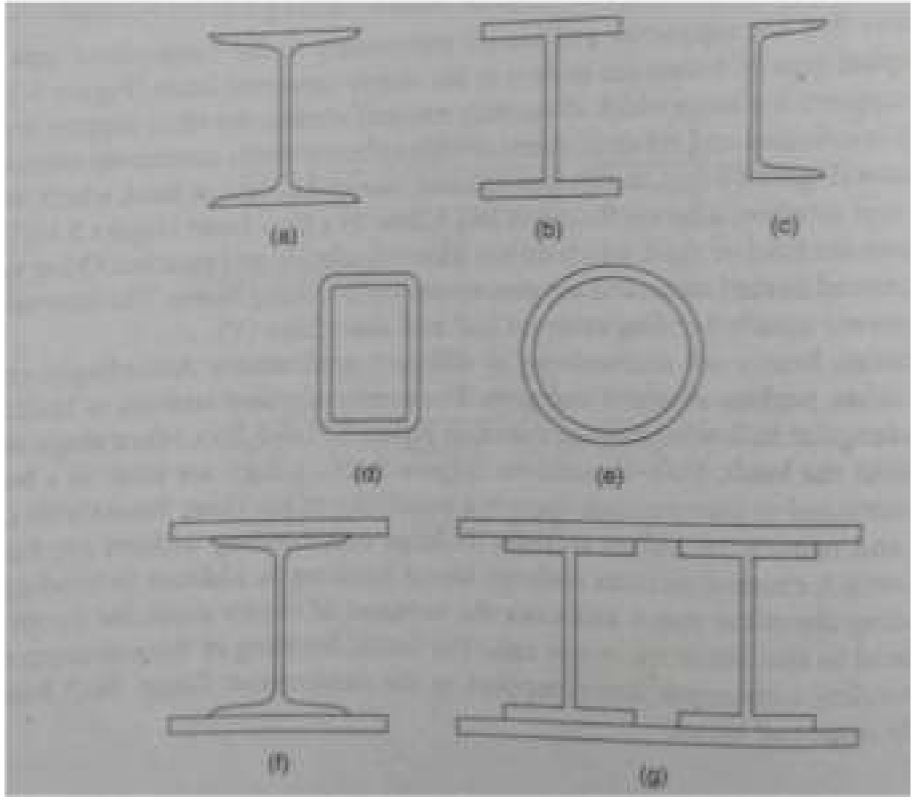
Beam should be proportional for strength in bending keeping in view of the lateral and local stability of the compression flange.

Now the selected shape should have capacity to withstand the essential strength in shear and local bearing. So, whatever shape we will select because different type of shape we can select like I section, channel section, some other section. That shape should have capacity to withstand essential strength in shear because the shear will be taken by the web, so web thickness should be sufficient enough to take care the shear force and local buckling.

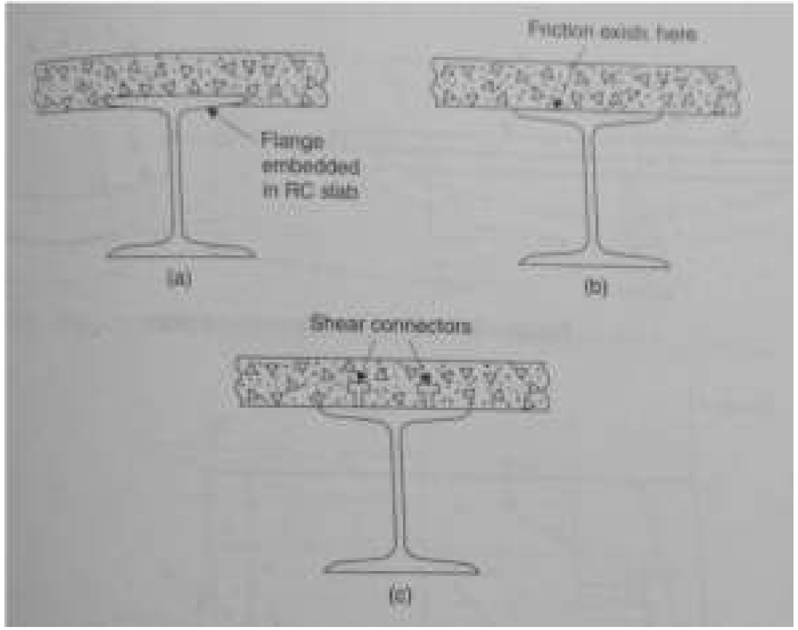
Then the beam dimension should be suitably proportional to stiffness, keeping in mind their deflections and deformations under service conditions.



Types of Supports



Types of Sections



Laterally restrained Beam

LIMITATIONS OF ANGLES , T-SECTIONS AND CHANNELS

- Angles and T-sections are weak in bending.
- Channels only be used for light loads.
- The rolled steel channels and angle sections are used in those cases where they can be designed and executed satisfactory.
- This is because the load is not likely to be in the plane, which removes torsional eccentricities .
- Also, it is complicated to calculate the lateral buckling characteristics of these sections .

MAIN FAILURE MODES OF HOT-ROLLED BEAMS

Category –I:

Excessive bending triggering collapse

Category –II:

Lateral torsional buckling of long beams

Category –III:

Failure by local buckling of

(i) flange in compression

(ii) Web due to shear

(iii) Web under compression

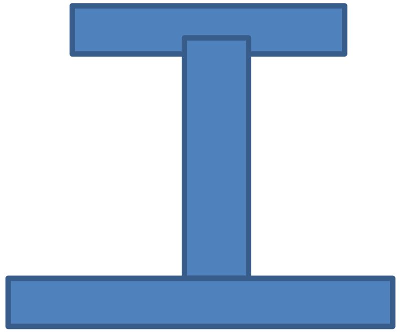
Category –IV:

Local failure by

(i) shear yield of web

(ii) Local crushing of web

(iii) Buckling of thin flanges



Bending failure is the basic failure mode and in this case, the beam is prevented from lateral buckling and the component elements are list compact so that they do not buckle locally. So such beams will collapse due to plastic deformation.

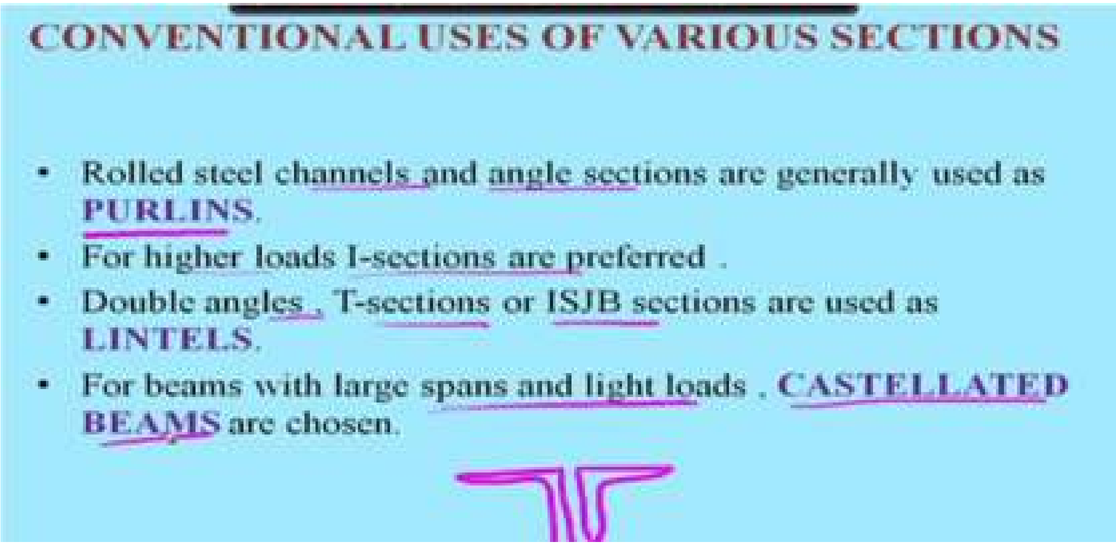
Another type of failure is lateral torsional buckling, which is an important failure criteria for steel flexural member. So lateral torsional buckling comes in picture when the beam is quite long.

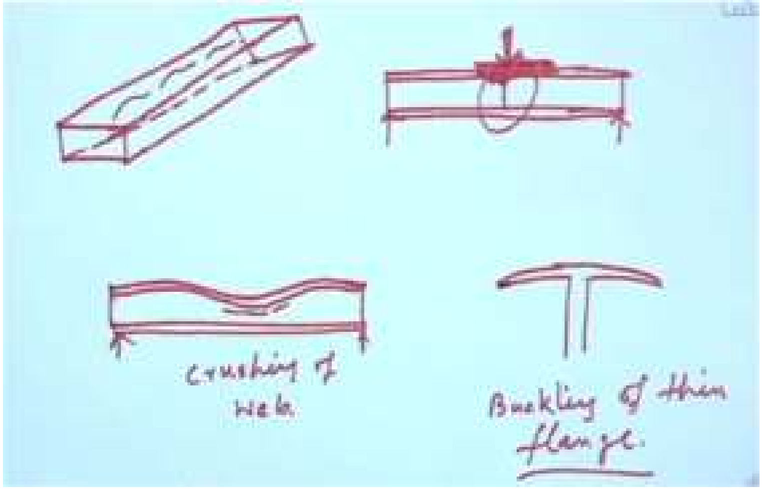
if an I-section have long length then it may fail due to lateral torsional buckling. So here, if load is acting in transverse direction and support conditions are there then it may buckle laterally and this lateral buckling occurs due to combination of lateral deflection and twist. The proportion of the beam support conditions and the load applied on it are the certain factors, which affect the failure due to lateral torsional buckling. say for example, if the load is not concentric twisting will occur because of the torsional moment across the section and because of that lateral torsional buckling take place.

The next category is failure by local buckling i.e. failure of flange in compression, failure of web due to shear and compression. These are the certain modes of failure, which come into this category. Say for example, if we have a box section, then it may fail in its flange due to compression. So, box sections may require flange stiffening to prevent premature collapse. In addition, it may fail due to web under shear and compression.

If we have a member under concentrated load then at the point of application of concentrated load the force is heavy, because load cannot disperse throughout it section. So therefore the failure may occur due to compression. This can be overcome by the use of additional bearing plate, which will disperse the load.

So local crushing of web means if we have a section and if it is under concentrated load then it may fail due to local crushing. Sometimes the flange width is quite high compares to its thickness. Therefore, it may buckle due to the very thin flange width. However, this type of failure may overcome, if we use additional plate at the flange by welding so that width to thickness ratio increase.





DESIGN CRITERIA

1. Based on deflection
2. Based on stress due to bending
3. Based on Shear

DEFLECTION CRITERIA

The amount of maximum deflection depends on:

1. Span
2. Moment of inertia of the section
3. Distribution of load
4. Modulus of elasticity &
5. Support condition

DESIGN PROCEDURE

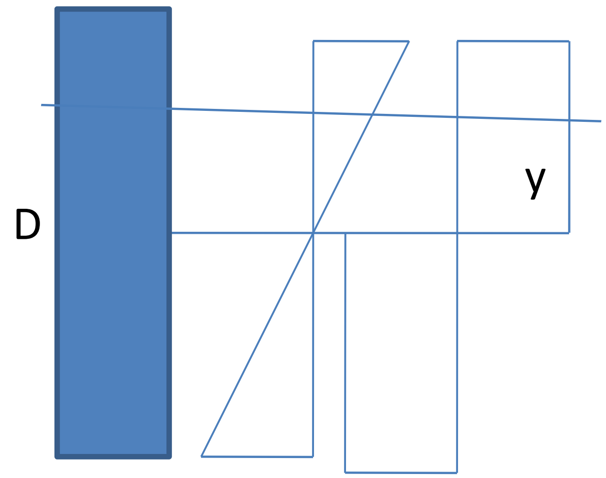
The design procedure can be divided into three parts and they are :-

- Structural :Bending moment, shear force, deflection and stability.
- Secondary effects : Local buckling, and secondary forces and connections.
- Practical limitations : Durability , fabrication tolerances , erection.

$M/I = f/Y$

BEAMS BETWEEN SUPPORTS
(Table 15, Clause 8.3.1, IS 800: 2007)

Support Conditions	Effective Length KL
Compression flange at the ends unrestrained against lateral bending (free to rotate in plan)	L
Compression flange partially restrained against lateral bending (partially free to rotate in plane at the bearings)	0.85L
Compression flange fully restrained against lateral bending (rotation fully restrained in plan)	0.7L



CLASSIFICATION OF CROSS-SECTIONS

- **Class 1 (Plastic)**

  Cross-sections which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism fall under this category. The width to thickness ratio of plate elements shall be less than that specified under Class 1 (Plastic), in Table 2 of IS 800:2007.

- **Class 2 (Compact)**

  Cross-sections which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling come under this class. The width to thickness ratio of plate elements shall be less than that specified under Class 2 (Compact), but greater than that specified under Class 1 (Plastic), in Table 2 of IS 800:2007.

- **Class 3 (Semi-compact)**

  Cross-sections in which the extreme fiber in compression can reach yield stress but can not develop the plastic moment of resistance, due to local buckling. The width to thickness ratio of plate elements shall be less than that specified under Class 3 (Semi-compact), but greater than that specified under Class 2 (Compact), in Table 2 of IS 800:2007.

- **Class 4 (Slender)**

  Cross-sections in which the elements buckle locally even before reaching yield stress. The width to thickness ratio of plate elements shall be greater than that specified under Class 3 (Semicompact), in Table 2 of IS 800:2007. In such cases, the effective sections for design shall be calculated either by following the provisions of IS 801 to account for the post-local-buckling strength or by deducting width of the compression plate element in excess of the semi-compact section limit.

Table 2 Limiting Width to Thickness Ratio

(*Clauses* 3.7.2 *and* 3.7.4)

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	$33.5\,\varepsilon$	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
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 $9.4\,\varepsilon$	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		$88\varepsilon^2$

NOTES

1 Elements which exceed semi-compact limits are to be taken as of slender cross-section.

2 $\varepsilon = (250 / f_y)^{1/2}$.

3 Webs shall be checked for shear buckling in accordance with **8.4.2** when $d/t > 67\varepsilon$, where, b is the width of the element (may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate), t is the thickness of element, d is the depth of the web, D is the outer diameter of the element (*see* Fig. 2, **3.7.3** and **3.7.4**).

4 Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favourable classification.

5 The stress ratio r_1 and r_2 are defined as:

$$r_1 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of web alone}}$$

$$r_2 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of overll section}}$$

TYPES OF ELEMENTS

- IS 800:2007 classifies elements in to three types, as per Cl. 3.7.3., as follows.
- **Internal elements**

  These are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners, for example, web of I-section and flanges and web of box section.
- **Outside elements or outstands**
- These are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of plane, for example flange overhang of an I-section, stem of T section and legs of an angle section.
- **Tapered elements**

  These maybe treated as flat elements having average thickness as defined in SP 6 (Part 1).
- **MAXIMUM EFFECTIVE SLENDERNESS RATIO**

  The maximum effective slenderness ratio, as per Cl. 3.8 of IS 800:2007, KL/r values of a beam, strut or tension member shall not exceed those given in Table 3 of IS 800:2007. 'KL' is the effective length of the member and 'r' is appropriate radius of gyration based on the effective section as defined in Cl. 3.6.1 of IS 800:2007. This data is reproduced here in Table .

Laterally Supported Beams

Beam can be designed on the basis of laterally supported or laterally unsupported. If web is supported laterally so that the lateral torsional buckling may be prevented.

The design criteria of such beam is given in clause 8.2.1 of IS 800-2007, the detail has been discussed where the design bending strength can be calculated in two cases, one is for low shear another is for high shear. When the shear force is less than the 0.6 times that design shear strength then it is called low shear, that means if Vd is the design shear strength of the cross section and V is less than 0.6Vd then it is a case of low shear. So in case of low shear we can find out the design bending strength simply by from this formula $M_d = \beta_B Z_P f_y / \gamma_{m0}$

To avoid irreversible deformation under serviceability loads, following conditions are to be satisfied.

$M_d \leq 1.2 * Z_e * f_y / \gamma_{m0}$ *for simply supported beams*

$M_d \leq 1.5 Z_e * f_y / \gamma_{m0}$ *for cantilever beams;*

Where,

β_b *= 1.0 for plastic and compact sections;*

β_b *= Ze / Zp for semi-compact sections;*

Zp , Ze = plastic and elastic section moduli of the cross-section, respectively;

f_y *= yield stress of the material; and* γ_{m0} *= partial safety factor* however if we see that the shear force is more than the 0.6 times design shear strength of the beam section then we can use this formula,

$M_d = M_{dv}$

Where, M_{dv} is the design bending strength under high shear and it is calculated as,

(a) Plastic or compact section

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2\frac{Z_e f_y}{\gamma_{m0}}/\gamma_{m0}$$

Where $\beta = (2\frac{V}{V_d} - 1)^2$

*Vd = design shear strength as governed by web yielding or web buckling = Av *f v*

fv = design shear strength

Av = shear area = $D*t_w$ for rolled sections

= dt_w for welded/built up sections

V = factored shear force

M_d = plastic design moment of the whole section disregarding high shear force effect and considering web buckling effects.

M_{fd} = plastic design strength of the area of the cross section excluding the shear area

$$M_{fd} = \frac{d^2 t_w}{4} f_y \quad \text{for built up sections}$$

$$M_{fd} = \frac{D^2 t_w}{4} f_y \quad \text{for rolled sections}$$

$d = D - 2t_f$

D is the overall depth and d is the effective depth.

So after designing for bending we will go for design for shear. Clause 8.4, IS 800:2007 describes the criteria. In clause 8.4, it says that the factored design shear force should satisfy,

$$V \leq \frac{V_n}{\gamma_{m0}}$$

Where V_n = nominal shear strength of a section

$$V_n = \frac{A_v f_{yw}}{\sqrt{3}}$$

Where A_v = shear area

f_{yw} = yield strength of the web

Now shear areas (*Av) can be calculated as given in clause 8.4.1.1, IS 800:2007 for different* types of section.

Shear Areas of different Sections (Cl. 8.4.1.1, IS 800: 2007):

Section	Shear Area A_v
Hot rolled (major axis bending)	Dt_w
Welded (major axis bending)	dt_w
Hot rolled or Welded (minor axis bending)	$2bt_f$
Rectangular hollow Sections (loaded parallel to height)	$AD/(b+D)$
Rectangular hollow Sections (loaded parallel to width)	$Ab/(b+D)$
Circular hollow tubes	$2A/\pi$
Plates & solid bars	A

Web Buckling

- The web behaves like a column if placed under concentrated load.
- The Web is quite thin and therefore is subjected to buckling.
- Web buckling occurs when the intensity of vertical compressive stress near the center of section becomes greater than the critical buckling stress for the web acting as column.

Web Buckling

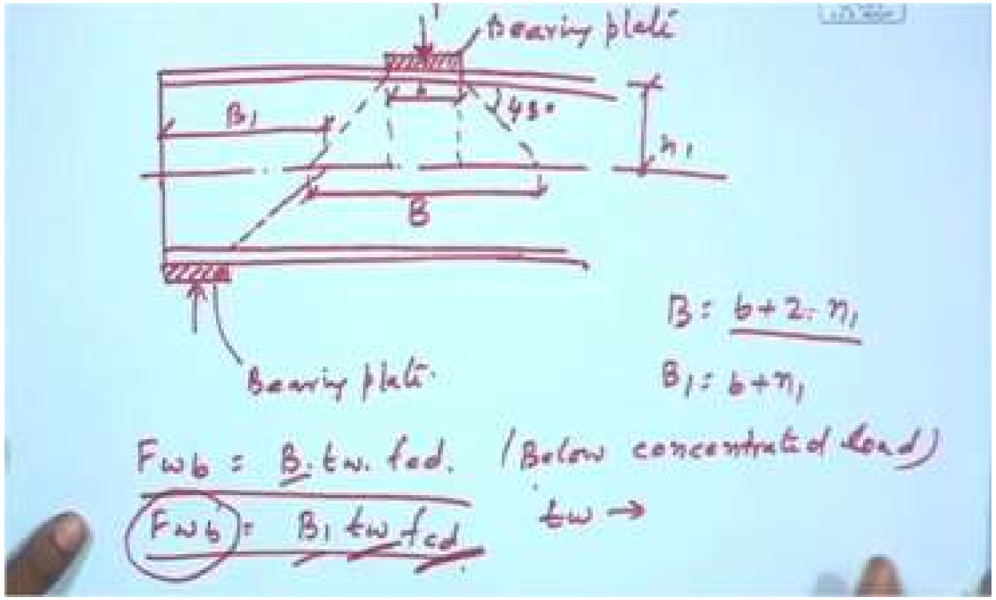
For all cases, bottom flange is assumed to be restrained against lateral deflection and rotation. For the top flanges, the end restraints and the effective depth of the web to be considered are as follows:

1. Restrained against lateral deflection and rotation, the effective depth = $d_1/2$

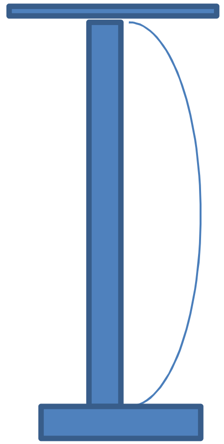
2. Restrained against lateral deflection but not against rotation, the effective depth= $(2/3)\ d_1$

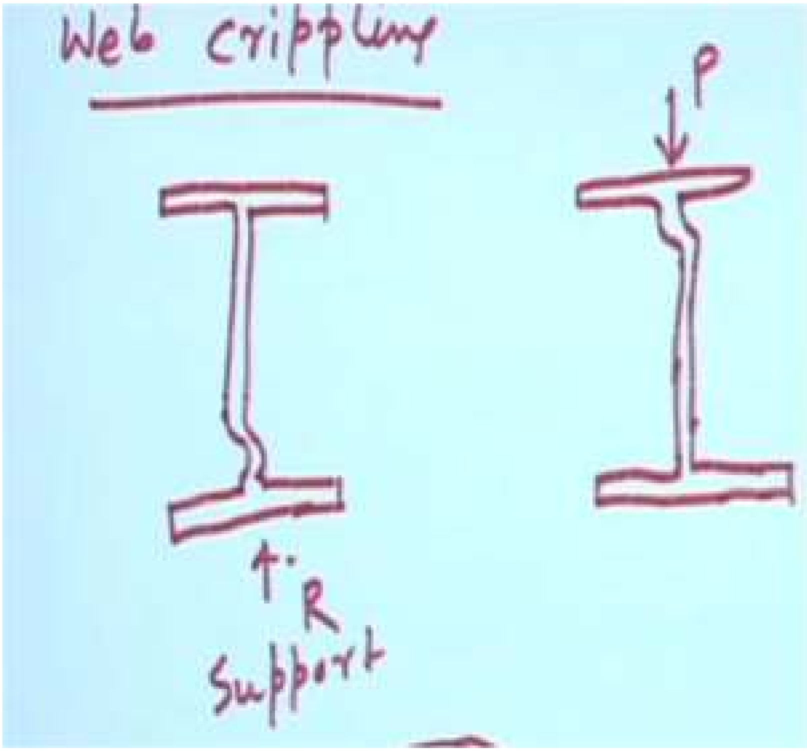
3. Retrained against rotation but not against lateral deflection, effective depth = d_1

4. Not restrained against rotation and lateral deflection, the effective depth = $2d_1$

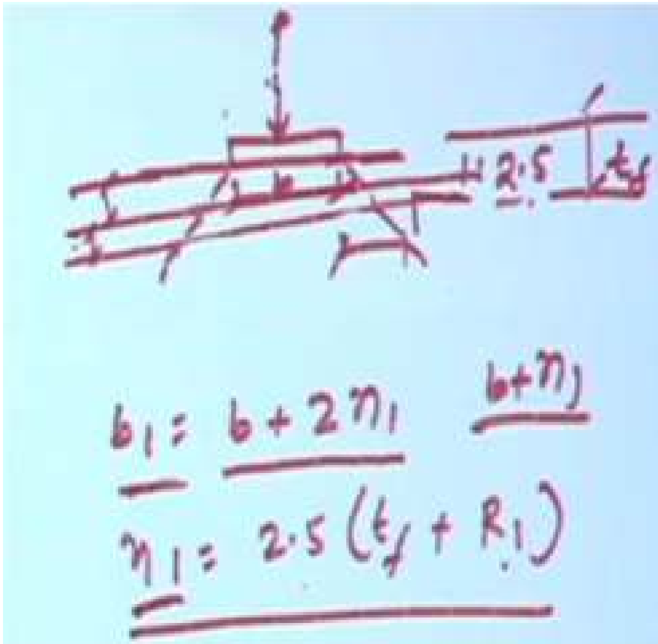


Bearing plate
45°
B1
n1
B
Bearing plate
B = b + 2.n1
B1 = b + n1
Fwb = B.tw.fcd. (Below concentrated load)
tw →
Fwb = B1 tw fcd





Web crippling
P
R
Support



P
b
2.5
tf
b1 = b + 2n1
b+n1
n1 = 2.5 (tf + R1)

So the web buckling strength can be calculated by,

$$F_{wb} = B t_w f_{cd}$$

(below concentrated load)

$$F wb = B_1 t_w f_{cd}$$

(at support)

Where,

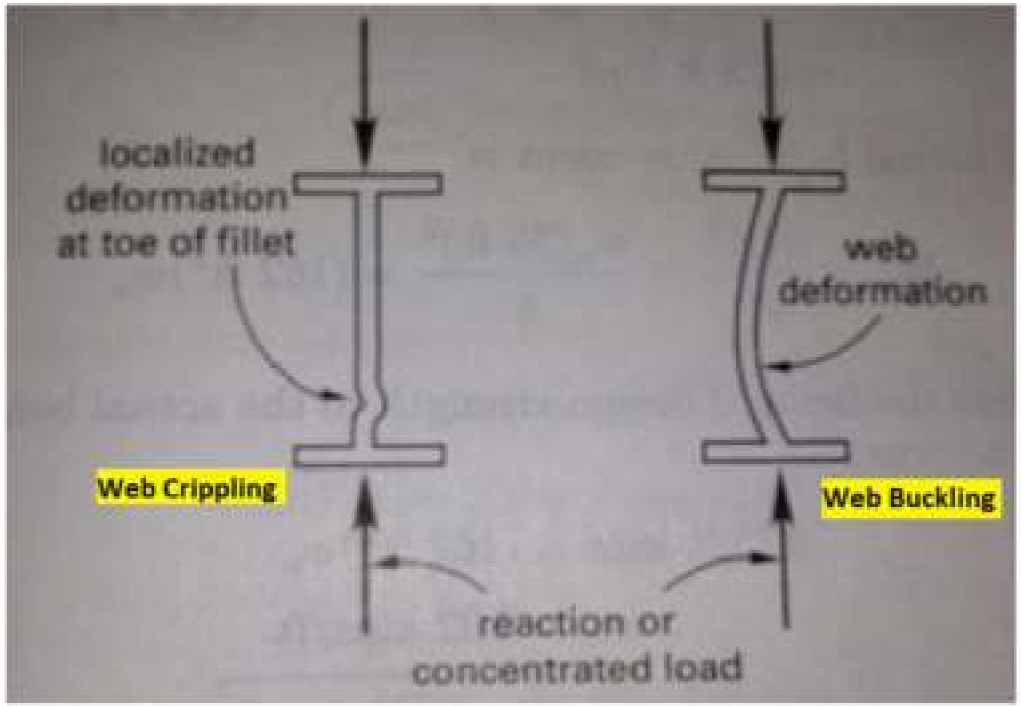
F_{wb} = web buckling strength at the support

$B = b + 2n_1$, $B_1 = b + n_1$

n_1 = length from dispersion at 45° to the level of neutral axis
t_w = thickness of the web
f_c = allowable compressive stress corresponding to assumed web strut according to buckling curve *c.*

Web Buckling: It is the sudden sideways deflection of a structural member under the application of compressive load. The load at which a compression member buckles is less than that member's ultimate strength. At buckling, the member exhibits *more than one Equilibrium states*.

Web Crippling: It is again the same thing however, it occurs when load concentration is more at a particular point in the member (usually new the supports).



localized
deformation
at toe of fillet
web
deformation
Web Crippling
Web Buckling
reaction or
concentrated load

Here, the effective length of strut will be $l_e = 0.7d$

Thus, the slenderness ratio $\lambda = \frac{l_e}{r_y} = \frac{0.7\,d}{r_y}$

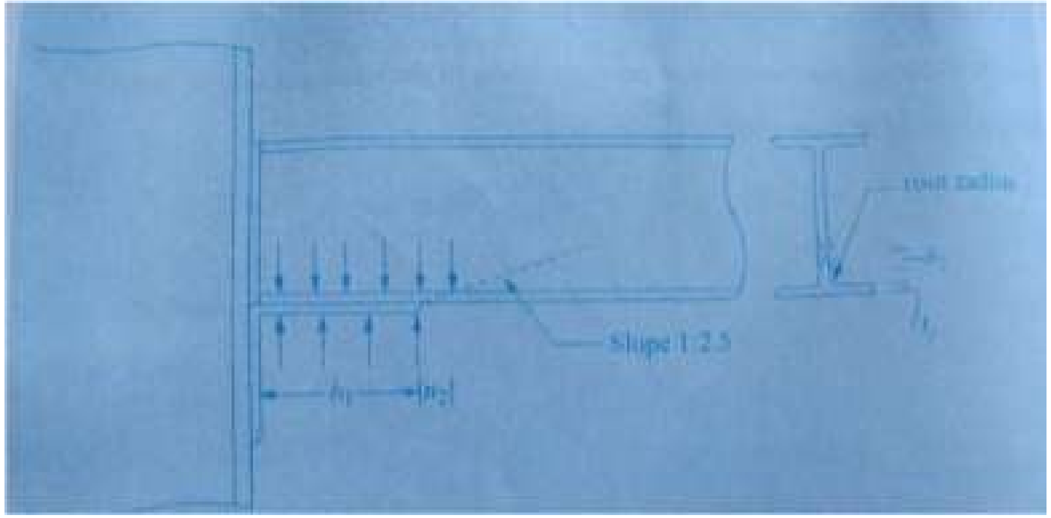
The radius of gyration, $r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{b\,t^3}{12 \times b \times t}} = \frac{t}{\sqrt{12}}$

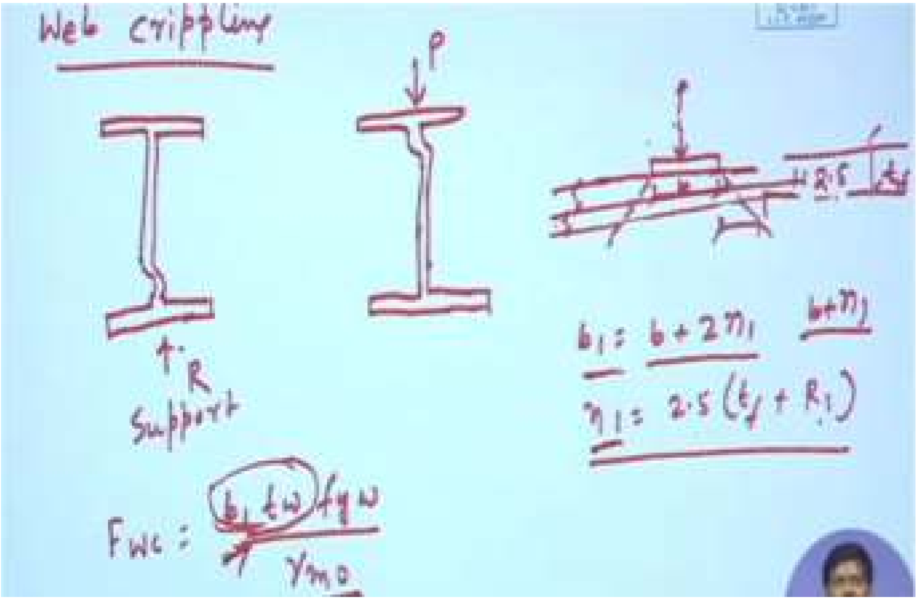
Hence, $\lambda = \frac{0.7\,d}{r_y} = \frac{0.7\,d \times \sqrt{12}}{t} \approx \frac{2.5\,d}{t}$

Thus, the slenderness ratio of the idealized web-strut is taken as

$$\lambda = \frac{2.5\,d}{t}$$

WEB CRIPPLING





$$F_{wc} = \frac{b_1 t_w f_{yw}}{\gamma_{m0}}$$

Where

F_{wc} = web crippling strength

b_1 = bearing length

= $b+2n_1$ under concentrated load

= $b+n_1$ under reactions at support

Minimum bearing length = 100 mm

n_1 = dispersion through the flange to the flange-to-web connection at a slope of 1:2.5

to the plane of the flange i.e. $n_1 = 2.5(t_f + R_1)$

t_w = thickness of the web

f_{yw} = design yield strength of the web

DESIGN STEPS FOR LATERALLY SUPPORTED BEAMS

1) The loads acting on the beam are calculated by multiplying the appropriate partial load factors.

2) The distribution of B.M. & S.F. along the length of the beam is determined. The maximum B.M. & S.F. is calculated

3) A trial plastic section for the beam is worked out from the following equation:

$$Z_p = \frac{M_d}{f_y/\gamma_{m0}}$$

4) A suitable section is selected which has plastic section modulus greater than the calculated value. ISMB, ISLB, ISWB sections are in general preferred.

5) The section is classified as plastic, compact or semi compact depending upon the specified limits of b/t_f and d/t_w as specified in **Table 2, IS 800: 2007**.

6) Calculate the design shear strength (V_d) from the relation:

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{m0}} h t_w$$

D

7) The beam is checked for high/low shear. If $V < 0.6\ V_d$, the beam will be low shear and if $V > 0.6\ V_d$, the beam will be high shear.

8) The trial section is checked for design bending strength

For low shear:

$$M_d = \beta_b Z_p f_y / \gamma_{m0}$$
$$\leq 1.2 Z_e f_y / \gamma_{m0} \text{ (for simply supported beams)}$$
$$\leq 1.5 Z_e f_y / \gamma_{m0} \text{ (for cantilever beams)}$$

For high shear:

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2\frac{Z_e f_y}{\gamma_{m0}}$$

(for plastic and compact section)

$$M_{dv} = Z_e \frac{f_y}{\gamma_{m0}}$$

(For semi-compact section)

8) If $M > M_d$, increase the section size and repeat from step 5.

9) The design shear strength (V_d) should be greater than the maximum factored shear force developed due to external load. If $V > V_d$, redesign the section by increasing the section size.

10) The beam is checked for deflection as per **Table 6, IS 800: 2007**.

11) The beam is checked for web buckling:

If, $\frac{d}{t_w} \leq 67\epsilon$ (for web without stiffeners) the web is assumed to be safe in web buckling and the shear strength of the web is governed by plastic shear resistance.

The web should be checked for buckling in case of high shear even if this limit is satisfied. The web buckling strength of the section, $f_{wb} = A_b \times f_{cd}$

Here, A_b = area of the web at the neutral axis of the beam = Bt_w and f_{cd} = design compressive stress

The web buckling strength should be greater than the design shear force

12) The beam is checked for web crippling.

$$F_{wc} > V$$

$$F_{wc} = \frac{b_1 t_w f_{yw}}{\gamma_{m0}}$$

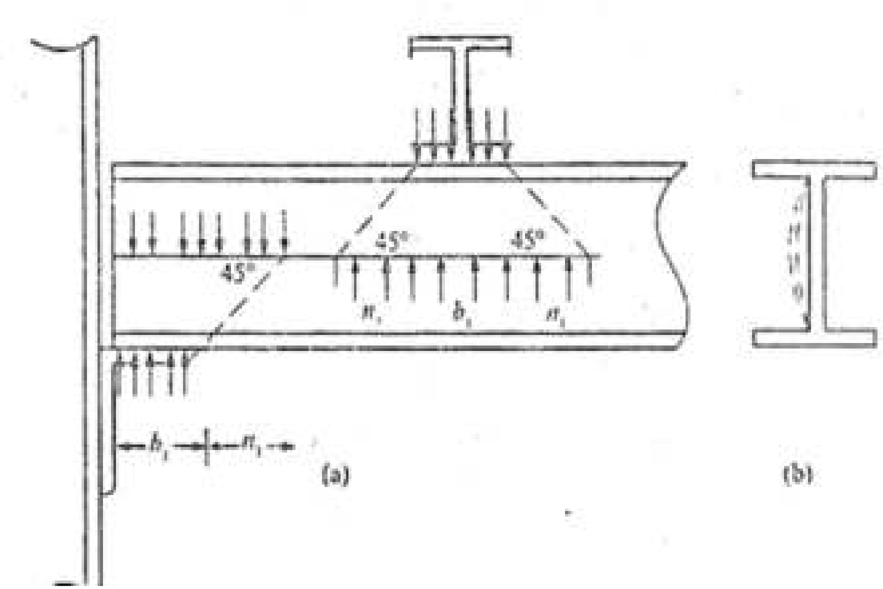
WEB BUCKLING STRENGTH

Certain portion of beam at supports acts as column to transfer load from beam to support. Under compressive load the web may buckle. This may also happen under a concentrated load on the beam. The load dispersion angle may be taken as 45^0. Rolled sections are provided with suitable thickness for web so that web buckling can be avoided. For built up sections it is necessary to check for web buckling.

As per IS 800-2007 effective web buckling strength is to be found based on cross section of web $(b_1+n_1)*t_w$

Where b1= width of stiff bearing on flange and n1=h/2, where h depth of section.

$F_{cdw}=(b_1+n_1)t_w*f_c$



F_{cdw}=Web buckling strength

f_c=Allowable compressive stress corresponding to assumed web column

ctive leng...

$$r_y = \sqrt{\frac{I_y}{A}} \text{ of web}$$

$$= \sqrt{\frac{\frac{1}{12}(b_1+n_1)t_w^3}{(b_1+n_1)t_w}} = \frac{t_w}{2\sqrt{3}}$$

$$\text{Slenderness ratio} = \frac{\text{Effective length}}{r_y}\ 0.7d \cdot \frac{2\sqrt{3}}{t_w} \approx 2.5\frac{d}{t_w}$$

Example: A cantilever beam of length 4.5 m supports a dead load (including self weight) of 18 kN/m and a live load of 12 kN/m. Assume a bearing length of 100 mm. Design the beam.

Solution:

Step 1: Calculation of load

Dead load = 18 kN/m

Live load = 12 kN/m

Total load = (18 + 12) = 30 kN/m

Total factored load =1.5 (18 + 12) = 45 kN/m

Step 2: Calculation of BM and SF

BM = $wl^2/2$= $45 \times 4.5^2/2$= 456 kN-m

SF = $w \times l = 45 \times 4.5 = 202.5$ kN

Step 3: Choosing a trial section

$$Z_{p,reqd} = \frac{M \times \gamma_{m0}}{f_y} = \frac{456 \times 10^6 \times 1.1}{250} = 2006.4 \times 10^3 \text{ mm}^3$$

Let us select the section ISLB 550 @ 0.846 kN/m

$Z_{pz} = 2228.16 \times 10^3$ mm^3

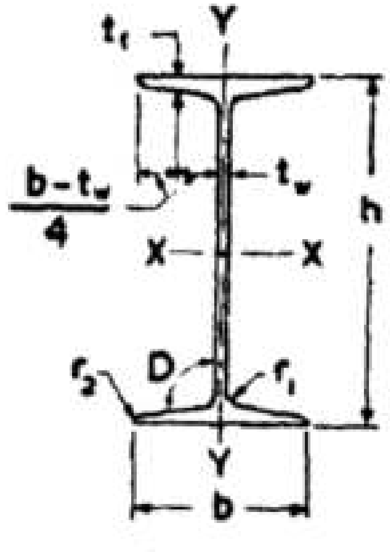
$Z_{ez} = 1933.2 \times 10^3$ mm^3

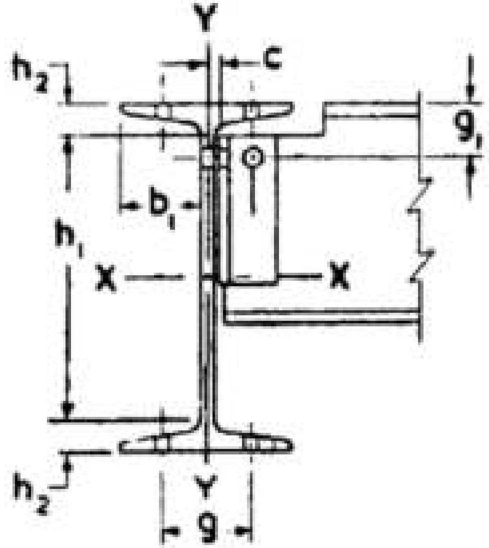
$h = 550$ mm, $b_f = 190$ mm, $t_f = 15$ mm, $t_w = 9.9$ mm, $r_1 = 18$ mm

$d = 550 - 2\times(15 + 18) = 484$ mm

$I_{zz} = 53161.6 \times 10^4$ mm^4

Designation	Weight per Metre w	Sectional Area a	Depth of Section h	Width of Flange b	Thickness of Flange t_f	Thickness of Web t_w	Moments of Inertia I_{xx}	Moments of Inertia I_{yy}	Radii of Gyration r_{xx}	Radii of Gyration r_{yy}
	kg	cm^2	mm	mm	mm	mm	cm^4	cm^4	cm	cm
ISLB 550	86·3	109·97	550	190	15·0	9·9	53 161·6	1 335·1	21·99	3·48
ISLB 600	99·5	126·69	600	210	15·5	10·5	72 867·6	1 821·9	23·98	3·79





Let us select the section ISLB 550 @ 0.846 kN/m

$Z_{pz} = 2228.16 \times 10^3$ mm3

$Z_{ez} = 1933.2 \times 10^3$ mm3

$h = 550$ mm, $b_f = 190$ mm, $t_f = 15$ mm, $t_w = 9.9$ mm, $R = 18$

$d = 550 - 2\times(15 + 18) = 484$ mm

$Izz = 53161.6 \times 10^4$ mm^4

Moduli of Section		Radius at Root	Radius at Toe	Slope of Flange	Connection Details						Maximum Size of Flange Rivet	Designation
Z_{xx}	Z_{yy}	r_1	r_2	D	h_1	h_2	b_1	C	g	g_1 (*Min*)		
cm^3	cm^3	mm	mm	degrees	mm	mm	mm	mm	mm	mm	mm	
1 933·2	140·5	18·0	9·0	98	476·1	36·95	90·05	6·45	100	70	32	**ISLB 550**
2 428·9	173·5	20·0	10·0	98	520·2	39·90	99·75	6·75	140, 100	75	25, 32	**ISLB 600**

$$\frac{\frac{b_f}{2}}{t_f}=\frac{95}{15}=6.33 \quad <9.4$$

$$\frac{d}{t_w}=\frac{484}{9.9}=48.9 \quad <84$$

Step 4: Calculation of shear capacity of the sectioi

$$V_d=\frac{f_y}{\gamma_{m0}\times\sqrt{3}}\times h\times t_w=\frac{250}{1.1\times\sqrt{3}}\times 550\times 9.9$$

$$¿714.47 \quad kN$$

$$0.6\,V_d=0.6\times 714.47=428.68 \quad kN>202.5\ kN$$

Hence. Low shear

Step 5: Design capacity of the section

$$M_d=\frac{Z_p\times f_y}{\gamma_{m0}}=\frac{2228.16\times 10^3}{1.1}\times 250$$ =506.4 kNm > BM so, ok.

$$\leq\frac{1.5\times Z_e\times f_y}{\gamma_{m0}}=\frac{1.5\times 1933.2\times 10^3\times 250}{1.1}$$ =659.04 kNm

Step 6: Check for deflection

$$\delta = \frac{w l^4}{8EI} = \frac{30 \times 4500^4}{8 \times 2 \times 10^5 \times 53161.6 \times 10^4} = 14 \quad .5 \text{ mm}$$

Allowable deflection = $L/150$ = 4500/150 = 30 mm

OK.

Step 7: Web buckling

Cross sectional area of web for buckling $A_b = (b_1 + n_1) t_w$

b_1 = 100 mm

n_1 = D/2=550/2 = 275 mm

A_b = (100 + 275) × 9.9

= 3712.5 mm^2

Effective length of the web = 0.7 × d = 0.7 × 484= 338.8 mm

$$I = \frac{b \times t_w^3}{12} = \frac{100 \times 9.9^3}{12} = 8085.8 \quad \text{mm}^3$$

A = 100 × 9.9 = 990 mm^2

$$r_{min} = \sqrt{\frac{8085.8}{990}} = 2.86 \text{ mm}$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{338.8}{2.85} = 119$$

As it is a rectangular section, buckling class will be "C"

Allowable stress f_{cd} = 84.8 N/mm^2

Capacity of the section = 84.8× 3712.5 = 314.8 kN > 202.5 kN

Hence, the section is safe against web buckling.

Step 8 : Check for web crippling

$$F_w = \frac{(b_1 + n_2) \times t_w \times f_y}{\gamma_{m0}}$$

$n_2 = 2.5\ (R + t_f) = 2.5(18 + 15) = 82.5$ mm

$$F_w = \frac{(100 + 82.5) \times 9.9 \times 250}{1.1} = 410.6 \text{ kN} > 202.5 \text{ kN}$$

So the section is safe against web crippling

BEAM WITH HIGH SHEAR

Example: Design a laterally supported beam of effective span 5 m for the following data.
Grade of steel: Fe 410
Factored maximum B.M. = 180 kN-m
Factored maximum S. F. = 220 kN
Check for deflection is not required

Solution:

For Fe 410 grade of steel: *fy = 250 Mpa,* Partial safety factor: $\gamma_{m0} = 1.1$

Factored Max. B.M. = 180 kNm

Factored Max. S.F. = 220 kN M=Zp*fy/1.1

Plastic section modulus required, Zp, reqd= $M * \gamma_{m0} / f_y$ =180×106×1.1/250=792×10^3 mm^3

Let us select a section, ISLB 350 @ 0.485 kN/m

$Z_{pz} = 851.11 \times 103\ mm^3$ $Ixx = 13158 \times 10^4$

$Z_{ez} = 751.9 \times 103\ mm^3$

h = 350 mm, bf = 165 mm, tw = 7.4 mm, tf = 11.4 mm

R1= 16 mm

d = D - 2(tf + R1) = 350 - 2(11.4 + 16) = 295.2 mm

$$\frac{\frac{b_f}{2}}{t_f}=\frac{165/2}{11.4}=7.23 \quad < 9.4$$

$$\frac{d}{t_w}=\frac{295.2}{7.4}=39.9 \quad < 84$$

Hence, the section is plastic.

Check for shear capacity:

Design shear strength of the section,

$$V_d=\frac{f_y}{\sqrt{3}\gamma_{m0}}D t_w=\frac{250}{\sqrt{3}\times 1.1}\times 350\times 7.4\times 10^{-3}=339.8 \quad \text{kN} > \text{V=220 kN}$$

$0.6V_d = 0.6 \times 339.8 = 203.9$ kN < V=220 kN

So, it is the case of high shear.

Check for design bending strength:

$$M_d=Z_{pz}\frac{f_y}{\gamma_{m0}}=851.11\times 10^3\times\frac{250}{1.1}\times 10^{-6}=193.43 \quad \text{kN-m} \quad \beta=\left(2\frac{V}{V_d}-1\right)^2=\left(2\frac{220}{339.8}-1\right)^2=0.087$$

$$Z_{fd}=Z_{pZ}-A_w Y_w=851.11\times10^3-(350\times7.4)\times\frac{350}{4}$$ =624.4 9×*103 mm*[3]

$$M_{fd}=624.49\times10^3\times\frac{250}{1.1}=141.93 \quad \text{kNm}$$

$$M_{dv}=M_d-\beta\left(M_d-M_{fd}\right)\leq1.2Z_e\frac{f_y}{\gamma_{m0}}$$

$$M_{dv}=193.43-0.087\times\left(193.43-141.93\right)=188.95 \quad \text{kNm}$$

$$\leq1.2Z_e\frac{f_y}{\gamma_{m0}}=1.2\times751.9\times10^3\times\frac{250}{1.1}\times10^{-6}=205.06 \quad \text{kNm}$$

Hence, $M_{dv}=188.95$ kNm > Mu=180 kNm, OK

Check for web buckling (at support)

Web buckling check is not required in general as

$$\frac{d}{t_w}=\frac{295.2}{7.4}=39.9 \quad < 67\epsilon$$

However, it is a case of high shear, web buckling check should be applied.

Assume a stiff bearing length, b = 100 mm

$$A_b=B_1 t_w=(b+n)t_w=(100+350/2)\times 7.4=2035 \quad \text{mm}^2$$

Effective length of web, $KL = 0.7d = 0.7\times 295.2 = 206.64$ mm

$$I_{eff} \text{ of web } ¿ \frac{b t_w^3}{12}=\frac{100\times 7.4^3}{12}=3376.87 \quad \text{mm}^4$$

A_{eff} of web = bt_w = 100 × 7.4 = 740 mm

$$r=\sqrt{\frac{3376.86}{740}} \quad =2.136 \text{ mm}$$

Slenderness ratio, $\lambda=\frac{KL}{r}=\frac{206.64}{2.136}=96.74$

For λ= 96.74, f_{yw} = 250 N/mm^2 , and buckling curve c , the design compressive stress from **Table 9(c), IS 800: 2007**.

f_{cd} = 111.56 N/mm^2

Capacity of web section $F_{wb} = A_b f_{cd}$ = 2035 × 111.56 × 10^{-3} = 227 kN

> 220 kN

Which is alright.

WEB CRIPPLING

$$F_w = (b + n_1) t_w \frac{f_{yw}}{\gamma_{m0}}$$

$$n_1 = 2.5 (t_f + R_1)$$

$= 2.5 \times (11.4 + 16) = 68.5$ mm

Stiff bearing length has been assumed, $b = 100$ mm

$F_w = (100 + 68.5) \times 7.4 \times 250 / 1.1 \times 10^{-3} = 283.4$ kN

> 220 kN, OK

FLEXURAL MEMBER-2

Dr. G.C. BEHERA

Laterally Unsupported Beams

design strength of laterally unsupported beam will be calculated based on the codal provisions, which is given in clause 8.2.2 of IS 800-2007. Now in case of laterally unsupported beam, the lateral torsional buckling will play an important role and because of lateral torsional buckling, the full plasticity of the section will not be developed that means the member will fail before it’s full bending stress of the section.

The cross sectional shape, support conditions and effective length will play an important role for the calculation of bending strength. So depending on all these bending strength of laterally unsupported beam will be calculated.

The design bending strength for laterally unsupported beams is

$M_d = \beta_b Z_p f_{bd}$

Where,

Zp = Plastic section modulus of the cross-section

β_b = 1.0 for compact & plastic sections

= Ze/Zp for semi-compact sections

f_{bd} = design bending compressive stress given by,

$f_{bd} = X_{LT} f_y / \gamma_{m0}$

X_{LT} = bending stress reduction factor to account for lateral torsion buckling

Now bending stress reduction factor to is calculated by,

$$X_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{LT}^2}} \leq 1.0$$

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

α_{LT} = imperfection factor for lateral torsional buckling of beams

= **0.21 for rolled steel sections**

= **0.49 for welded steel sections**

Suppose, if we use plate to make a I section with the use of welding, then for such type of section, we can use *α_{LT} as 0.49 otherwise for the rolled section we can α_{LT} as 0.21*

λ_{LT} = *non-dimensional slenderness ratio given by,*

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} \leq \sqrt{1.2\,\frac{Z_e f_y}{M_{cr}}}$$

$$\sqrt{\frac{f_y}{f_{cr,b}}}$$

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} \leq \sqrt{1.2\,\frac{Z_e f_y}{M_{cr}}} = \sqrt{\frac{f_y}{f_{cr,b}}}$$

Where,

*Mcr = elastic lateral buckling moment (**Cl. 8.2.2.1) is given by,***

$$M_{cr} = \sqrt{\left(\frac{\Pi^2 E I_y}{L_{LT}^2}\right)\left[G I_t + \frac{\Pi^2 E I_w}{L_{LT}^2}\right]} = \beta_b Z_p f_{cr,b}$$

I_t = *torsional constant* $=\Sigma b_i\, t_i^3/3$ for open section

I_w = *warping constant* Warping constant, $I_w = (1-\beta_f)\beta_f I_y h_f^2$

I_y = *moment of inertia about weaker axis*

Here, h_f = c/c distance between flanges $= D - t_f$ $\beta_f = I_{fc}/[I_{fc}+I_{ft}]$

r_y = *radius of gyration about weaker axis*

L_{LT} =*effective length for lateral torsional buckling (Clause 8.3)*

h_f = *centre-to-centre distance between flanges*

t_f= *thickness of flange*

G = shear modulus

I_w = The warping constant, given by:

$(1-\beta_f)\,\beta_f\,I_y\,h_y^{\,2}$ for I-sections mono-symmetric about weak axis

= 0 for angle, Tee, narrow rectangle section and approximately for hollow sections

β_f = $I_{fc}/(I_{fc}+I_{ft})$ where I_{fc}, I_{ft} are the moment of inertia of the compression and tension flanges, respectively, about the minor axis of the entire section.

I_t = torsion constant, given by:

= $\sum b_i t_i^{\,3}/3$ for open section

= $4A_e^{\,2}/\sum(b/t)$ for hollow section

where

A_e = area enclosed by the section, and

b, t = breadth and thickness of the elements of the section, respectively.

$f_{cr,b}$ is the extreme fiber bending compressive stress and is given by,

$f_{cr,b}$ = extreme fiber bending compressive stress corresponding to elastic lateral buckling moment and is given by

$$f_{cr,b} = \frac{1.1\,\Pi^2 E}{\left(\frac{L_{LT}}{r_y}\right)^2}\sqrt{1 + \frac{1}{20}\left(\frac{\frac{L_{LT}}{r_y}}{\frac{h_f}{t_f}}\right)^2}$$

For different values of *KL/r_y & h_f/t_f corresponding values of fcr,b is given in* ***Table 14, IS 800:2007. Values of** **fbd can also be found from Table 13(a) and 13(b), IS 800: 2007*** corresponds to different values of *$f_{cr,b}$ and f_y*

$$M_{cr} = \frac{\Pi^2 E I_y h_f}{2(L_{LT})^2}\sqrt{1 + \frac{1}{20}\left(\frac{\frac{L_{LT}}{r_y}}{\frac{h_f}{t_f}}\right)^2}$$

However, *Mcr for different beam sections, considering loading, support condition and nonsymmetric* section, shall be more accurately calculated using the method given in Annex E of IS: 800-2007.

Example: Calculate the design bending strength of ISLB 300 @ 0.369 kN/m considering the beam to be

(a) *Laterally supported*

(b) *Laterally unsupported*

Assume the design force is less the design shear strength and is of low shear. The effective length of the beam (L_{LT}) is 4 m. Assume Fe410 grade of steel.

Solution:

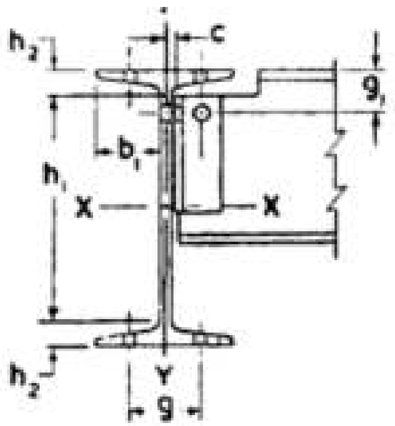
The relevant properties of ISLB 300

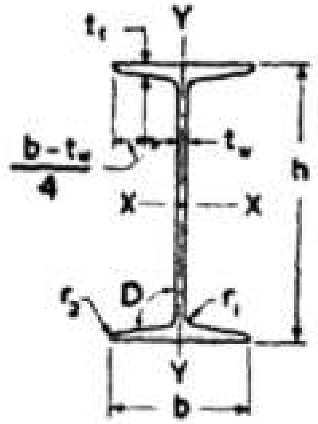
$D = 300$ mm, $b_f = 150$ mm, $t_w = 6.7$ mm, $t_f = 9.4$ mm,

$R_1 = 15.0$ mm

$r_x = 124$ mm, $r_y = 28$ mm, $Z_{pz} = 554.32 \times 10^3$ mm3, $Z_{ez} = 488.9 \times 10^3$ mm^3,
$Iz = 7333 \times 10^4$ mm^4, $Iy = 376 \times 10^4$ mm^4 $d = D - 2(tf + R1) = 300 - 2(9.4 + 15) = 251.2$ mm
For rolled section: $\alpha_{LT} = 0.21$, For Fe 410 grade of steel: $f_y = 250$ MPa
Partial safety factor: $\gamma_{m0} = 1.10$

Designation	Weight per Metre	Sectional Area	Depth of Section	Width of Flange	Thickness of Flange	Thickness of Web	Radii of Gyration		Section Modulus	Plastic Modulus	Shape Factor
			(D)	(b_f)	(t_f)	(t_w)	(r_z)	(r_y)	(Z_{ez})	(Z_{pz})	(Z_{pz}/Z_{ez})
	kg/m	cm^2	mm	mm	mm	mm	cm	cm	cm^3	cm^3	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
ISHB 250	51.0	64.96	250	250	9.7	6.9	10.91	5.49	618.9	678.73	1.096 7
ISMC 350	*42.1	53.66	350	100	13.5	8.1	13.66	2.83	571.9	672.19	1.175 4
ISMB 300	*44.2	56.26	300	140	12.4	7.5	12.37	2.84	573.6	651.74	1.136 2
ISLC 350	*38.8	49.47	350	100	12.5	7.4	13.72	2.82	532.1	622.95	1.170 7
ISLB 300	*37.7	48.08	300	150	9.4	6.7	12.35	2.80	488.9	554.32	1.133 8
ISHB 225	46.8	59.66	225	225	9.1	8.6	9.58	4.84	487.0	542.22	1.113 4





Designation	Weight per Metre	Sectional Area	Depth of Section	Width of Flange	Thickness of Flange	Thickness of Web	Moments of Inertia		Radii of Gyration	
	w	a	h	b	t_f	t_w	I_{xx}	I_{yy}	r_{xx}	r_{yy}
	kg	cm^2	mm	mm	mm	mm	cm^4	cm^4	cm	cm
ISLB 300	37·7	48·08	300	150	9·4	6·7	7 332·9	376·2	12·35	2·80
ISLB 325	43·1	54·90	325	165	9·8	7·0	9 874·6	510·8	13·41	3·05
ISLB 350	49·5	63·01	350	165	11·4	7·4	13 158·3	631·9	14·45	3·17

Moduli of Section		Radius at Root	Radius at Toe	Slope of Flange	Connection Details						Maximum Size of Flange Rivet	Designation
Z_{xx}	Z_{yy}	r_1	r_2	D	h_1	h_2	b_1	C	g	g_1 (*Min*)		
cm^3	cm^3	mm	mm	degrees	mm	mm	mm	mm	mm	mm	mm	
488·9	50·2	15·0	7·5	98	245·1	27·45	71·65	4·85	90	60	22	ISLB 300

$$\frac{b}{t_f}=\frac{150/2}{9.4}=7.98 \quad < 9.4$$

$$\frac{d}{t_w}=\frac{251.2}{6.7}=37.49 \quad < 84$$

Hence, the section is plastic.

Since, $\frac{d}{t_w}=\frac{251.2}{6.7}=37.49 \quad < 67\epsilon$

Webs shall be checked for shear buckling in accordance with 8,4.2 when *d/t> 67E, where, b is the width of the element (may be* taken as clear distance between lateral supports or between lateral support and free edge, as appropriate), *t is the thickness of* element, *d is the depth of the web, D is the outer diameter of the element (see Fig. 2,3.7.3 and 3.7.4).*

Shear buckling check of web will not be required.

(a) ***Laterally supported beam***

For low shear,

$$M_d=\beta_b Z_p \frac{f_y}{\gamma_{m0}}=1.0\times 554.32\times 10^3\times\frac{250}{1.1}=125.98 \quad \text{kN-m}$$

$$\leq 1.2\, Z_e \frac{f_y}{y_{m0}}=1.2\times 488.9\times 10^3\times\frac{250}{1.1}=133.34 \quad \text{kN-m}$$

Hence, design bending strength = 125.98 kN

(b) Laterally unsupported beam

$$M_{cr}=\sqrt{\left(\frac{\pi^2 E I_y}{(L_{LT})^2}\right)\left[G I_t+\frac{\pi^2 E I_w}{(L_{LT})^2}\right]}$$

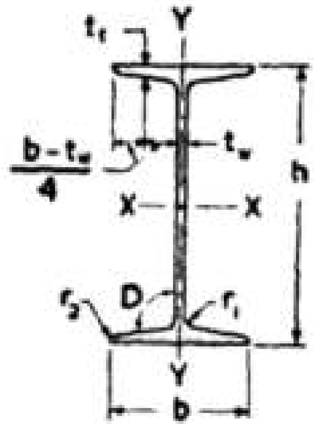
$$M_{cr}=\sqrt{\left(\frac{\Pi^2 E I_y}{L_{LT}^2}\right)\left[G I_t+\frac{\Pi^2 E I_w}{L_{LT}^2}\right]}=\beta_b Z_p f_{cr,b}$$

L_{LT} = 4000 mm

$$G=\frac{E}{2(1+\mu)}=\frac{2\times10^5}{2\times(1+0.3)}=76.92\times10^3$$

Torsional constant, $I_t=\sum\frac{b_i t_i^3}{3}$ $\quad 2\times\frac{150\times9.4^3}{3}+\frac{(300-2\times9.4)\times6.7^3}{3}$

11.12×10^4 $\quad$ mm^4



Warping constant, $I_w=(1-\beta_f)\beta_f I_y h_f^2$

Here, h_f = c/c distance between flanges $=D$ - t_f =300-9.4 = 290.6
$\beta_f=I_{fc}/[If_c+I_{ft}]$==0.5 [Since $I_{fc}=I_{ft}$]
Thus, $I_w=(1-0.5)\times0.5\times376\times10^4\times290.6^2$== 7.94 × 10^{10} mm^6

$$M_{cr}=\sqrt{\left(\frac{\Pi^2*2*10^5*376*10^4}{4000^2}\right)\left[76.92*10^3*11.22*10^4+\frac{\Pi^2*2*10^5*7.94*10^{10}}{4000^2}\right]}=92.45\ kNm$$

- or

$$M_{cr} = \frac{\Pi^2 E I_y h_f}{2(L_{LT})^2}\sqrt{1 + \frac{1}{20}\left(\frac{\frac{L_{LT}}{r_y}}{\frac{h_f}{t_f}}\right)^2} \quad M_{cr} = \frac{\Pi^2 * 2 * 10^5 * 376 * 10^4 * 290.6 I_y h_f}{2(4000)^2}\sqrt{1 + \frac{1}{20}\left(\frac{\frac{4000}{28}}{\frac{290.6}{9.4}}\right)^2}$$

$$= 96.92\ kNm$$

$$\lambda_{LT} = \sqrt{\frac{1 * 554.32 * 10^3 * 250}{92.45 * 10^6}} = 1.22 > 0.4$$

So, the effect of lateral torsional buckling has to be considered.

8.2.2 ***Laterally Unsupported Beams***

Resistance to lateral torsional buckling need not be checked separately (member may be treated as laterally supported, *see* **8.2.1**) in the following cases:

a) Bending is about the minor axis of the section,
b) Section is hollow (rectangular/ tubular) or solid bars, and
c) In case of major axis bending, λ_{LT} (as defined herein) is less than 0.4.

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2$$
$$\Phi_{LT} = 0.5[1 + 0.21(1.22 - 0.2) + 1.22^2 = 1.35$$

$$X_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{LT}^2}} \leq 1.0$$

$$X_{LT} = \frac{1}{1.35 + \sqrt{1.35^2 - 1.22^2}} = 0.518 \cong 0.52 \leq 1.0$$

$$f_{bd} = X_{LT} * \frac{f_y}{1.1} = 0.52 * \frac{250}{1.1} = 118.18\ N/mm^2$$

$$M_d = f_{bd} * Z_p = 118.18 * 554.32 * 10^3 = 65.51\ kNm$$

Calculations Using the Table

1. *$KL/r_y = 4000/28 = 142.86$*, h_f/t_f==290.6/9.4=30.9

For KL/ry=140 for hf/tf=30 fcr,b=160.2 ,

For KL/ry=150 for hf/tf=30 fcr,b=144.8 ,

For KL/ry=142.86 for hf/tf=30 fcr,b=160.2+(144.8-160.2)*(142.86-140)/(150-140)=155.7956

For KL/ry=140 for hf/tf=35 fcr,b=148.7 ,

For KL/ry=150 for hf/tf=35 fcr,b=133.7 ,

For KL/ry=142.86 for hf/tf=35 fcr,b=148.7+(133.7-148.7)*(142.86-140)/(150-140)=144.41

For KL/ry=142.86 for hf/tf=30 fcr,b=155.7956

For KL/ry=142.86 for hf/tf=35 fcr,b=144.41

For KL/ry=142.86 for hf/tf=30.9 fcr,b=155.7956+(144.41-155.7956)*(30.9-30)/(35-30)=153.74

From formulae

$$f_{cr,b} = \frac{1.1\,\Pi^2 E}{\left(\frac{L_{LT}}{r_y}\right)^2}\sqrt{1+\frac{1}{20}\left(\frac{\frac{L_{LT}}{r_y}}{\frac{h_f}{t_f}}\right)^2}$$

$$f_{cr,b} = \frac{1.1\,\Pi^2 * 2 * 10^5}{\left(\frac{4000}{28}\right)^2}\sqrt{1+\frac{1}{20}\left(\frac{\frac{4000}{28}}{\frac{290.6}{9.4}}\right)^2} = 153.11$$

For, *fcr,b = 153.11 N/mm*^{2} *and* f_y *= 250 N/mm*^{2} *and* α_{LT} *= 0.21 , from* **Table 13(a), IS 800: 2007**

For fcr,b=150, fbd=108.7, For fcr,b=200, fbd=134.7,

For 153.11=108.7+(134.7-108.7)*3.11/(200-150)=110.32 N/mm^2

M_d=Z_p*f_{bd}=110.32* 553.34*10^3=61.15 kNm

So the design bending strength of the member when lateral torsional buckling is considered is 61.15 kNm and earlier we found M_d as 125.98kNm when the beam is laterally supported.

DESIGN STEPS FOR LATERALLY UNSUPPORTED BEAM

1. Calculate service load, factored load, factored BM.
2. Trial plastic section modulus means, $Z_p = M_d / (f_y / \gamma_{m0})$ this is considering the section to be laterally supported. But in case of laterally unsupported beam a major amount of stress is reduced due to lateral torsional buckling. Take higher section modulus which is necessary to account for lateral torsional buckling. Increase section modulus 40 to 50%.
3. Take a suitable section.
4. Check the beam for shear.
5. Check the beam for deflection.
6. Check the beam for web buckling.
7. Check the beam web crippling.

Example: Design a simply supported steel joist of 5 m effective span, carrying a uniformly distributed load 12 kN/m if compression flange of the joist is laterally unrestrained.

Solution

Step-1: BM & SF on beam

Load on the beam = 12 kN/m

Factored load = 12 × 1.5 = 18 kN/m

Max. B. M. = $18 \times 5^2/8$ kN-m= 56.25 kN-m

Max S. F. = 18 × 5/2= 45 kN

Step-2: Selection of initial section,

$Z_p = M/(f_y/\gamma_{m0}) = 56.25 \times 10^6/(250/1.1) = 247.5 \times 10^3$

Increasing 50%, the required Z_p *will be* $1.5 \times 247.5 \times 10^3 = 371.25 \times 10^3\ mm^3$

Step-3 : Calculate bending strength of section,

Select ISLB 325

D = 325 mm ry = 30.5 mm Zpz = 687.76 × 103 mm3

bf = 165 mm R1 = 16 mm Zez = 607.7 × 103 mm3

tf = 9.8 mm Ixx = 9870 × 104 mm4

tw = 7.0 mm Iyy = 510.8 × 104 mm4

d = 325 – 2×(9.8+16) = 273.4 mm

Section classification:

b/tf = 82.5/9.8 = 8.41 < 9.4, d/tw = 273.4/7.0 = 39 < 84

Hence, section is plastic.

Calculation of bending strength:

KL/ry = 5000/30.5= 164, *hf/tf = (325 – 9.8)/9.8*= 32.16

From Table 14, IS 800: 2007

fcr,b = 122.82 N/mm2

From Table 13(a), IS 800: 2007,

fbd = 93.17 N/mm2

So, $Md = 1 \times 687.76 \times 10^3 \times 93.17$

= 58.57 kN-m > 56.25 kN-m OK.

Step-4: Check for shear:

Design shear strength of the section,

$V_d=[f_y/(\sqrt{3} * \gamma_{m0})]Dtw=[250/(\sqrt{3} \times 1.1)] \times 325 \times 7 \times 10^{-3}=299\ kN > V=45\ kN$

Step-5: Check for deflection:

$\delta= 5wl^4/384\ EI$

$= 5 \times 12 \times 5000^4/\{384 \times 2 \times 10^5 \times 9870 \times 10^4=4.9\ mm$

Allowable maximum deflection, *L/300 = 5000/300 = 16.67 mm.*

Hence, safe

Step-6: Check for web buckling:

Assuming stiff bearing length 100 mm

n1 = D/2 = 325/2 = 162.5 mm

C/S area for web buckling *Ab = (b + n1) × tw*= (100 + 162.5) × 7.0 = 1837.5 mm^2

Effective length of web, *leff = 0.7 × 273.4 = 191.38 mm*

I = 100 × 73/12 = 2858.33 mm3

λ = leff/rmin =0.7*2*sqrt(3)*d/tw=0.7*2*sqrt(3)*273.4/7= 94.71

From **Table 9(c), IS 800: 2007,** ***fcd = 114.364 N/mm2***
Capacity of the section, *Ab × fcd = 1837.5 × 114.364 = 210 kN*> 45 kN
Hence, the section is safe against web buckling.

Step-7: Check for web crippling:

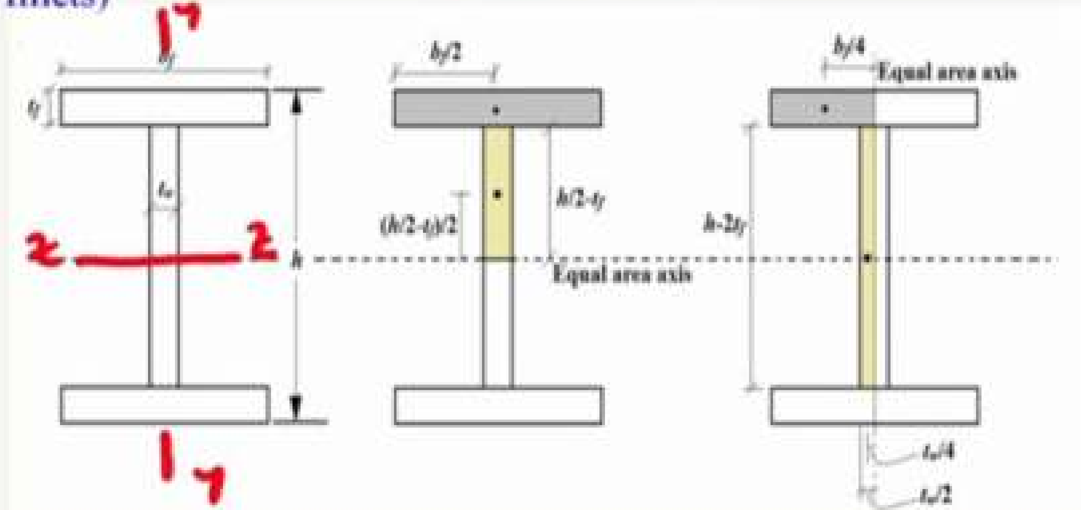
$F_{crip}=(b_1+n_2)\times t_w\times f_y/\gamma_{m0}$

$n_2=2.5\ (16+9.8)=64.5\ mm$

$Fcrip=(100+64.5)\times 7\times 250/1.1=261.70$ kN > 45 kN

CALCULATION OF PLASTIC SECTION MODULUS

Example: Determine the plastic section modulus of ISLB 300 @ 0.369 kN/m about the strong and weak axis (neglecting the fillets)



For symmetrical I-section the equal area axis *zz* and *yy* will pass through the centroid of the section.

For symmetrical I-section the equal area axis *zz* and *yy* will pass through the centroid of the section.

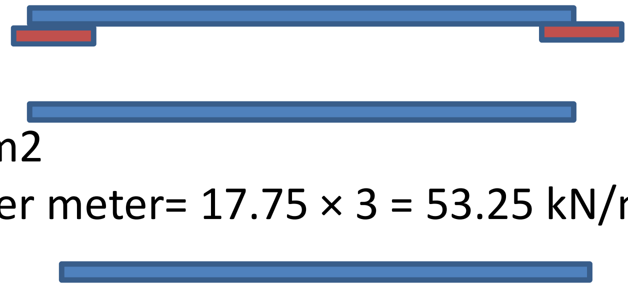
$$Z_{pz}=2(b_f t_f)\times\frac{(D-t_f)}{2}+2\left[\left(t_w\times\left(\frac{D}{2}-t_f\right)\right)\times\frac{\left(\frac{D}{2}-t_f\right)}{2}\right] \qquad b_f t_f(D-t_f)+\frac{t_w(D-2t_f)^2}{4}$$

$$Z_{py}=4\times\left[\left(\frac{b_f}{2}\times t_f\right)\times\frac{b_f}{4}\right]+2\times\left[\left((D-2t_f)\times\frac{t_w}{2}\right)\times\frac{t_w}{4}\right] \qquad \frac{t_f b_f^2}{2}+\frac{(D-2t_f)t_w^2}{4}$$

BUILT UP SECTION

Example: Steel beams having a clear span of 8 m are resting on 200 mm wide end bearings. The beams spacing is 3 m and the beams carry a dead load of 4.5 kN/m^2 including the weight of the section. The imposed load on the beam is 13.25 kN/m^2. The beam depth is restricted to 500 mm and the yield strength of the steel is 250 N/mm^2 and is laterally supported.

Solution:



Factored loads:

Total (Dead Load + Imposed load) = (4.5+ 13.25)=17.75 kN/m2

The beams are spaced at 3 m intervals, therefore the load per meter= 17.75 × 3 = 53.25 kN/m^2

Total factored load = 1.5× 53.25 = 80 kN/m

Eff. Span = 8 + 2×0.1 = 8.2 m

Mid span moment = 80 × 8.2^2/8 = 672.8 kN-m

Reactions at support = 8.2 × 80/2 = 328 kN

Selection of section:

Plastic section modulus required $Z_p = \frac{M \times \gamma_{m0}}{f_y} = \frac{672.8 \times 10^6 \times 1.1}{250} = 2960.32 \times 10^3 \quad mm^3$

The section with largest plastic modulus under 500 mm depth restriction is ISHB 450 @ 0.907 kN/m with plastic section modulus 2030.95 × 10^3 mm^3 which is less than required value. The section must be strengthened with additional plates to provide the required plastic section modulus.

The stiffness required to be provided can be calculated as follows:

Max. deflection = Eff. span/360 = 8200/360 = 22.78 mm

So, required moment of inertia of the beam due to un-factored imposed load,

$$I_z = \frac{5}{384} \times \frac{53.25 \times 82004}{2 \times 10^5 \times 22.78} = 68807 \times 10^4 \quad mm^4$$

Additional plastic section modulus to be provided by the plate =(2960.32×10^3 – 2030.95×10^3) = 929.37×10^3 mm^3

Assume thickness of the plate is 14 mm

Thus, the total depth of the beam = 478 mm.

Distance between the c/c of the plates = 464 mm.

So, required area of plate = 929.37×10^3 /464 = 2003 mm^2

So provide area of plate = 2200 mm^2.

Thus the width of plate = 2200/14 = 158 mm

Thus let provide plate of size 200×14=2800 mm^2

Thus plastic section modulus of the built up section = 2030.95×10^3 +200×14×(464/2)×2 =3330×103 mm^3

Check for deflection:

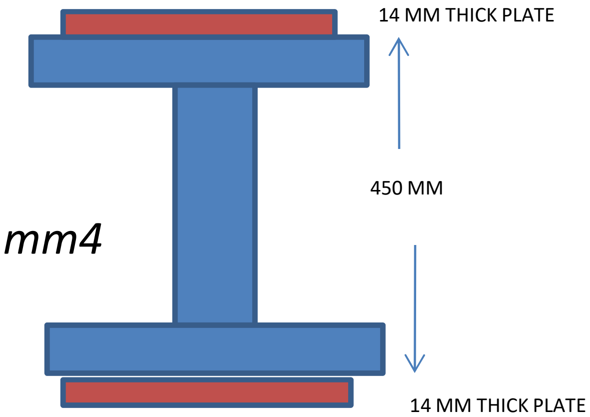
Maximum *Iz required is 68807×104 mm4*

Iz provided by ISHB 450, 40349.9×104 mm4

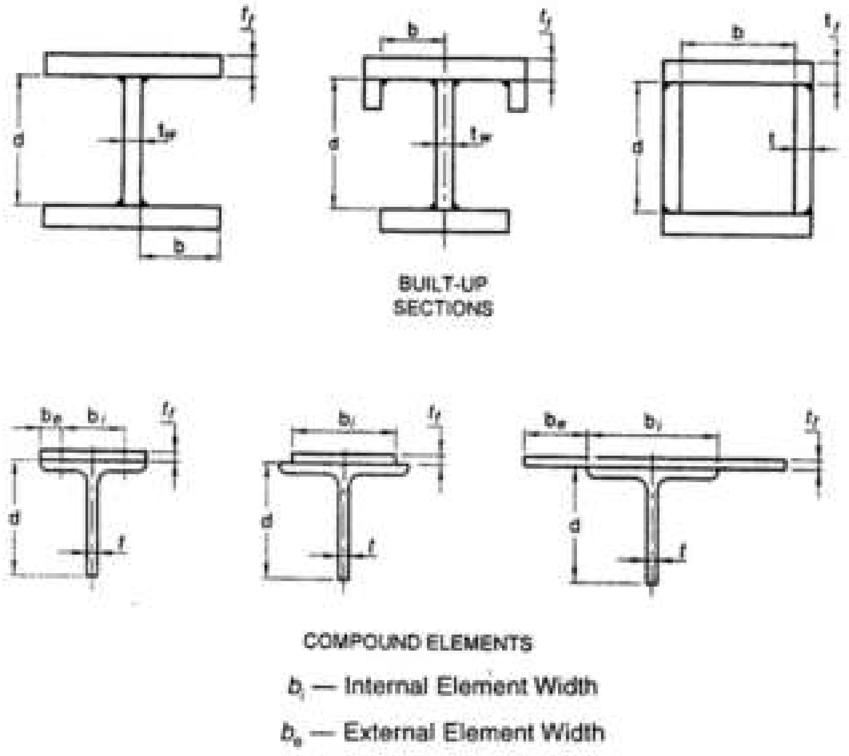
Iz provided by plate = 2×200×14×(225+7)2 = 30141× 104 mm4

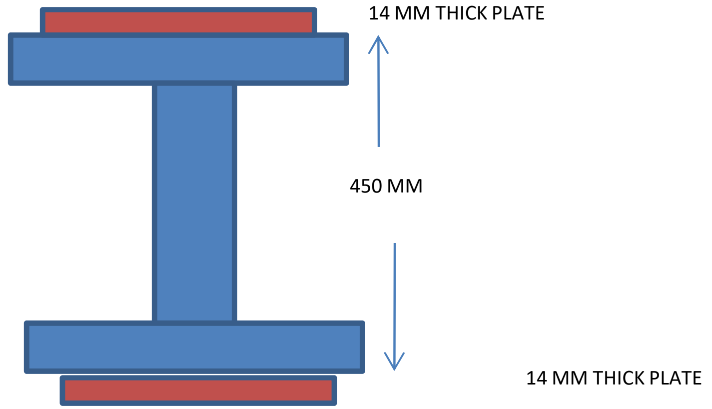
Total *Iz provided = (40349.9×104 + 30141× 104) = 70490.9×104 mm4 greater than Iz required*

(= 68807×*104) OK*



b_f=250 mm t_f=13.7 mm t_w=11.3 mm E_1=15mm
be=(250-200)/2=25 mm bi=200 mm
be/tf=25/14=<9.4 bi/tf=200/14 <29.3
d/tw=450/11.3=39.82<84 so, plastic





Moment capacity of the beam ISHB 450,

M = 2030.95 × 10^3 × 250/1.1 = 461.58 kN-m

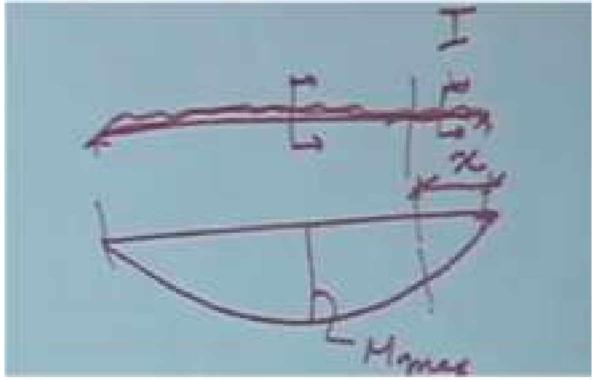
At any point distance *x from the support,*

$461.58 \times 106 = 328 \times 10^3 x - 80x^2/2$

or, $x = 6396.5\ ,\ 1803.05$ *mm*

Hence the theoretical cut off point is 1800 mm from either side.

Check for Shear:



Shear capacity of section,

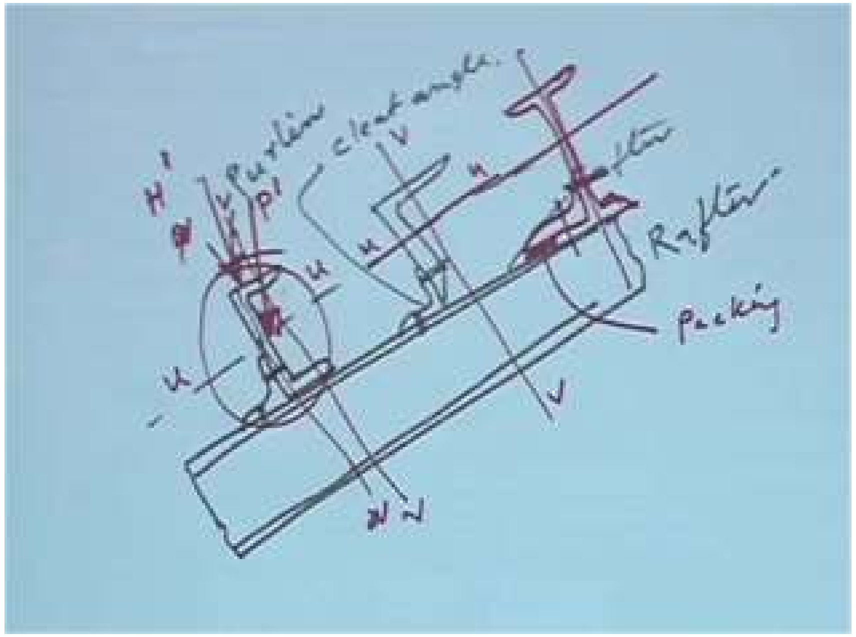
$$V_d = \frac{f_y}{\gamma_{m0} \times \sqrt{3}} \times D \times t_w = \frac{250}{1.1 \times \sqrt{3}} \times 450 \times 11.3 = 667.23 \quad \text{kN}$$

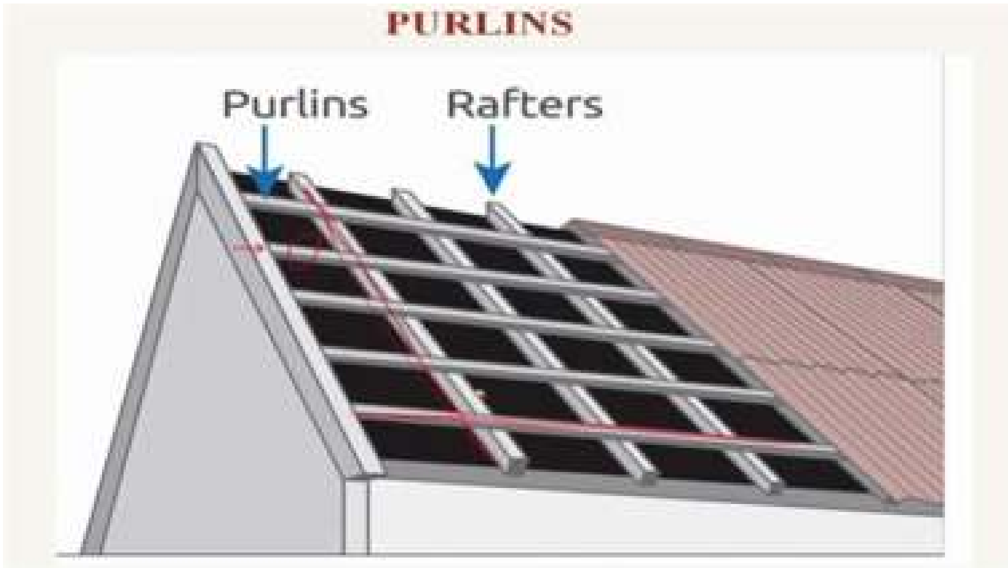
$0.6V_d = 0.6 \times 667.23 = 400.33$ kN > 328 kN. Low shear OK

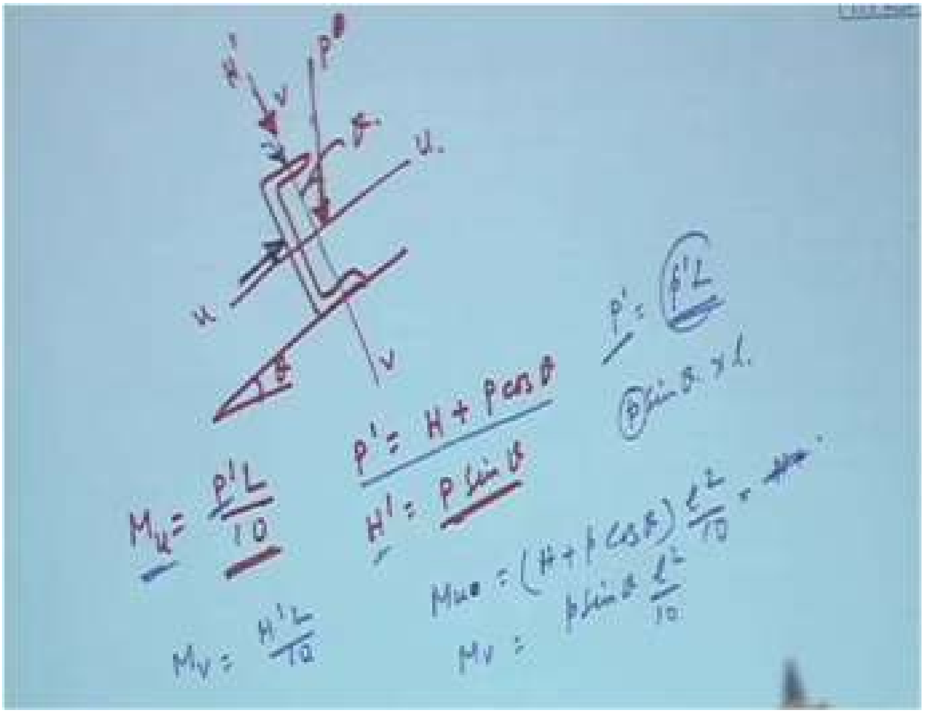
cut-off point is 1800 mm from After this portion steel plate will be provided.

PURLIN DESIGN

Purlins are basically a flexural member in which transverse load act, in case of purlins the moments from both the axis occur as a result purlins are needed to be designed for biaxial moment. So we need to check the bending moment carrying capacity against both the axis and then we have to check the interaction formula so that the purlin is designed and these purlins are basically connects the transverse members in the roof structure to support the roof sheets and other materials and these purlins are placed on the rafter.







H'
P
u
v
u
θ
P' = H + P cos θ
H' = P sin θ
P' = P'L
P sin θ × L
M_u = P'L / 10
M_v = H'L / 10
M_uo = (H + P cos θ) L² / 10
P sin θ L² / 10
M_v =

So if we see for an example say for channel section if we see here that load is basically two type one is the wind load (H’) which are acting perpendicular to the roof. Another load is acting vertically downward i.e. self-weight (P’). Codal provision says that we should design purlin as an continuous beam because purlins are connected to the truss members in different places.

So the moment can be calculated as, $Mu=P' L/10$ *and* $M_v=H' L/10$

M_u = *maximum bending moment about u-u axis.*

M_v = *maximum bending moment about v-v axis.*

P’ = gravity loads acting along v-v axis, including sheeting, self-weight of purlins, LL & snow loads = $H+P\cos\theta$.

H’ = loads acting along u-u axis, including wind loads= $P\sin\theta$ *L = span of the purlin, i.e.* c/c distance of adjacent trusses

$Muu=(H+P\cos\theta) L/10$

$Mvv=(P\sin\theta) L/10$

For biaxial moment of channel and I-sections the interaction formula is given by

$(Mu/Mdu)+(Mv/Mdv)\leq 1.0$

Where,

M_{du} = *design bending moment about u-u axis*

M_{dv} = *design bending moment about v-v axis*

Purlins are subjected to bi-axial bending. A trial section may be obtained arbitrarily or the expression given by **Gaylord *et al.* *(1992) as follows:***

$$Z_{pz} = \frac{M_z \gamma_{m0}}{f_y} + \frac{M_y \gamma_{m0}}{f_y} \times 2.5 \times \frac{d}{b_f}$$

Where,

Z_{pz} = required plastic section modulus

M_y = factored bending moment about y-y axis

M_z = factored bending moment about z-z axis

f_y = Yield stress of the material

d = depth of the section

bf = width of the section

We have to assume certain d and bf value initially and on the basis of that we can find out Zpz,value and once we find out Zpz value we can find out a particular section say channel section,or I section, or angle section.

So after knowing the actual d and bf we can again find out what is the actual requirement Zpz and whether it is satisfying that or not,

Design procedures for channel/I section purlin:

1. The span of the purlin is taken as c/c distance of adjacent trusses
2. The gravity loads P and wind loads H are computed. The component of these loads in the direction parallel & perpendicular to the sheeting are determined. These loads are multiplied with partial safety factor for loads as per Table 4 of the code for various load combinations
3. The maximum B.M. (*Mz or Muu and My or Mvv) and S.F. (Fz and Fy) using the factored* loads are determined.
4. The required value of plastic section modulus of the section may be determined by using the following equation

$$Z_{p,reqd} = \frac{M_z \gamma_{m0}}{f_y} + \frac{M_y \gamma_{m0}}{f_y} \times 2.5 \times \frac{d}{b_f}$$

where

My = Factored bending moment about y-y axis

Mz = Factored bending moment about z-z axis

fy = Yield stress of steel

γm0 = Partial safety factor = 1.10

d = Depth of the trial section

bf = Width of the trial section

5. Check for the section classification as per **Table 2: IS 800: 2007 .**
6. Check for shear capacity of the section for both z and y axes taken as (Moris & Plum 1996)

$$V_{dy}=\frac{f_y}{\sqrt{3}\gamma_{m0}}A_{vy} \quad \text{and} \quad V_{dz}=\frac{f_y}{\sqrt{3}\gamma_{m0}}A_{vz}$$

Avz=Dt_w and A_{vy}=2b_f*t_f*
where
D = height of the section
t_w = thickness of the web
b_f = breadth of the flange
t_f = thickness of the flange
7. Compute the design capacity of the section in both the axes using

$$M_{dz}=\frac{Z_{pz}f_y}{\gamma_{m0}}\leq 1.2\frac{Z_{ez}f_y}{\gamma_{m0}} \quad M_{dy}=\frac{Z_{py}f_y}{\gamma_{m0}}\leq 1.2\frac{Z_{ey}f_y}{\gamma_{m0}}$$

8. Check for local capacity using the interaction formula $\frac{M_Z}{M_{dz}}+\frac{M_y}{M_{dy}}\leq 1.0$

9. Check whether deflection is under permissible limits *(l/180) as per Table 6, IS 800:* 2007.

Design of Angle Section Purlins:

The following procedure is adopted for the design :

1. The vertical and the wind loads are determined. These loads are assumed to be normal to roof truss.
2. The maximum bending moment is computed.

$Mu = w\ L^2/10 \vee\ W\ L/10$

where *L = span of purlin*

w = uniformly distributed load

W = concentrated load at centroid

3. The required section modulus is calculated by

$Zp,\ reqd = M/[1.33 \times 0.66 \times f_y]$

4. Assuming the depth = 1/45 of the span and width = 1/60 of the span, a trial section of angle purlins is arrived by.

The depth and width must not be less than the specified values to ensure the deflection criteria.

5. A suitable section is then selected for the calculated value of leg lengths of angle section.

The modulus of section provided should be more than modulus of section calculated.

Example: Design an I-section purlin, for an industrial building situated in the outskirt of *Kolkata, to support a galvanized iron sheet roof for the following data:*

Slope of truss = 30°

Spacing of truss c/c = 5.0 m, Span of truss = 12.0 m, spacing of purlins c/c = 2 m

wind speed = 50 m/s, Weight of galvanized sheets = 120 N/m², Grade of steel = Fe 410

Solution:

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

Weight of galvanized corrugated iron sheets = 120 × 2 = 240 N/m

Assume dead load of purlin = 100 N/m

Total dead load = 240 + 100 = 340 N/m

The dead load acts vertically downwards.

The component of dead load parallel to roof = 340sin30° = 170 N/m

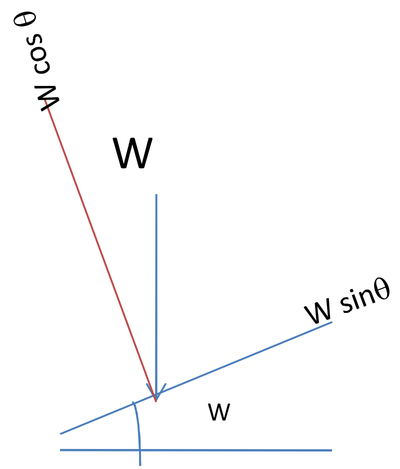
The component of dead load normal to roof = 340cos30° = 294.5 N/m

Wind pressure = $p_z = 0.6V_z^2 = 0.6 \times 50^2 = 1500$ N/m2

Wind load is assumed to act normal to the roof. θ

Wind load = 1500×2×1 = 3000 N/m

Total load on purlin normal to roof = 3000+294.5 = 3294.5 N/m



Wind load is assumed to act normal to the roof.

Wind load = 1500×2×1 = 3000 N/m

Total load on purlin normal to roof = 3000+294.5 = 3294.5 N/m

Factored load normal to roof, P = 1.5×3294.5 = 4941.75 N/m

Factored load parallel to roof, H = 1.5×170 = 255 N/m

Maximum moment,

$Muu = Mz = PL/10 = (4941.75\times5)\times5\times10^{-3} /10 = 12.35$ kNm

$Mvv = My = HL/10 = (255\times5)\times5\times10^{-3} /10 = 0.6375$ kNm

Let us try a section with flange width *bf = 75 mm and depth, d = 125 mm.*

Plastic section modulus required,

$$Z_{pz,reqd} = M_z \frac{\gamma_{m0}}{f_y} + 2.5\left(\frac{d}{b}\right)\left(M_y \frac{\gamma_{m0}}{f_y}\right)$$

$$Z_{pz,reqd} = 12.35\times10^6\times\frac{1.1}{250} + 2.5\left(\frac{125}{75}\right)\left(0.6375\times10^6\times\frac{1.1}{250}\right)$$

$$= 66\times10^3 \ \text{mm}^3$$

Select a section ISLB 150 with $Zpz = 104.5 \times 10^3$ mm^3

A= 1808 mm^2, $D = 150$ mm, $b_f = 80$ mm, $t_f = 6.8$ mm, $t_w = 4.8$ mm

$R_1 = 9.5$ mm, $d = 150 - 2(6.8+9.5) = 117.4$ mm

$Iz = 688.2 \times 10^4$ mm4 , $Iy = 55.2\times 104$ mm^4, $Z_{ez} = 91.8 \times 103$ mm^3, $Z_{ey} = 13.8 \times 10^3$ mm^3

Section classification

$\epsilon = sqrt(250/f_y) = sqrt(250/250) = 1$

$b/t_f = 40/6.8 = 5.88 < 9.4$, $d/t_w = 117.4/4.8 = 24.5 < 84$, *Tble-2*

Hence the section is plastic.

Check for bending strength

Mdz=Zpz(f y/γm0)*=104.5 × 10^3× *250/*1.1×10^{-6}*=23.75 kN-m*
< 1.2Z_{ez}**f y/γm0*=1.2×*91.8*×10^3× *250/*1.1×10^{-6}*=25.04 kN-m*
Which is alright.
M_{dz}*=23.75 kN-m > Md = 12.35 kNm; OK*

$$M_{dy}=Z_{py}\times\frac{f_y}{\gamma_{m0}}\leq\gamma_f Z_{ey}\frac{f_y}{\gamma_{m0}}$$

$$Z_{py}=4\times\left[\left(\frac{b_f}{2}\times t_f\right)\times\frac{b_f}{4}\right]+2\times\left[\left((D-2t_f)\times\frac{t_w}{2}\right)\times\frac{t_w}{4}\right]$$

$$Z_{py}=\frac{t_f b_f^2}{2}+\frac{(D-2t_f)t_w^2}{4}=\frac{6.8*80^2}{2}+\frac{(150-2*6.8)4.8^2}{4}=22546\ mm^3$$

Mdy = 22546 × 250/1.1 × 10-6 = 5.12 kN-m
< 1.5 × 13.8 × 10^3 × 250/1.1 × 10-6 = 4.7 kN-m
(1.2 is replaced by γ_f *=1.5 since Zpy/Zey (=1.6)> 1.2)*
Hence, *Mdy = 4.7 kN-m > Md = 0.6375 kN-m; OK*

Check for overall member strength (local capacity)

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

$$\frac{12.35}{23.75}+\frac{0.6375}{4.7}=0.66<1 \quad ; OK$$

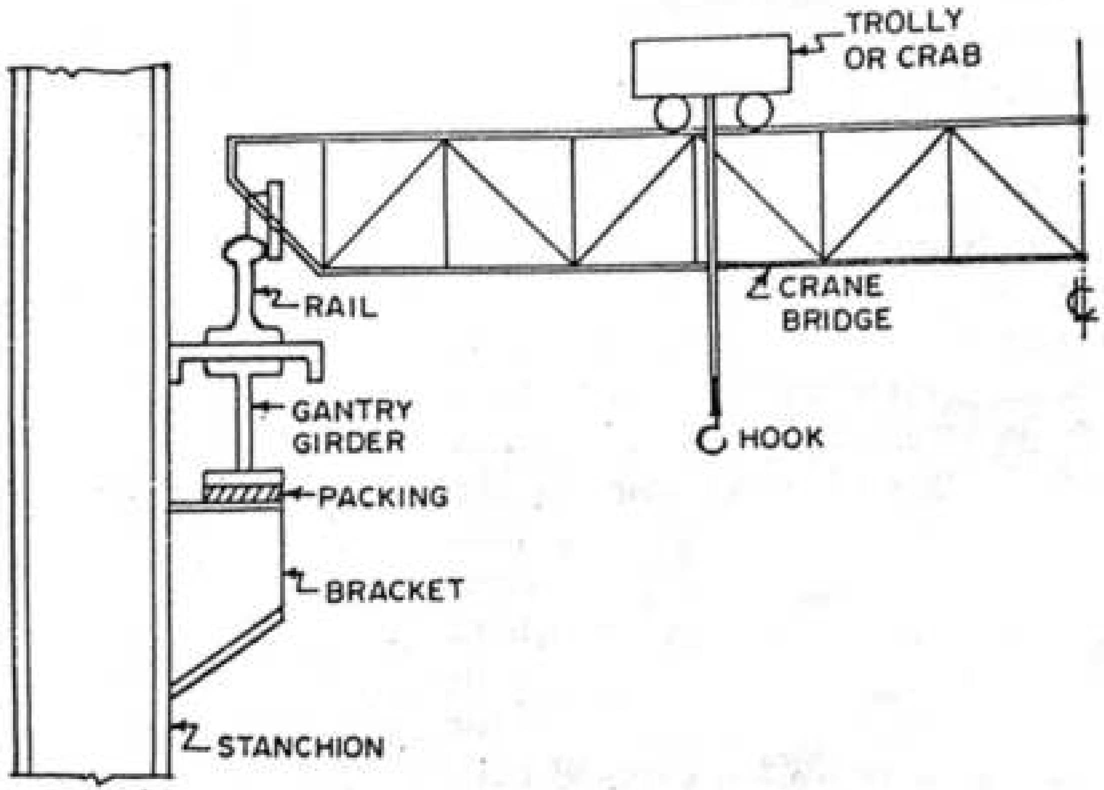
Check for deflection

$\Delta=5wl^4/384EI=5*3294.5*(5000)^4/[384*200000*682*10^4]=19.5$ mm

Δallowed=L/180=5000/180=27.78 mm

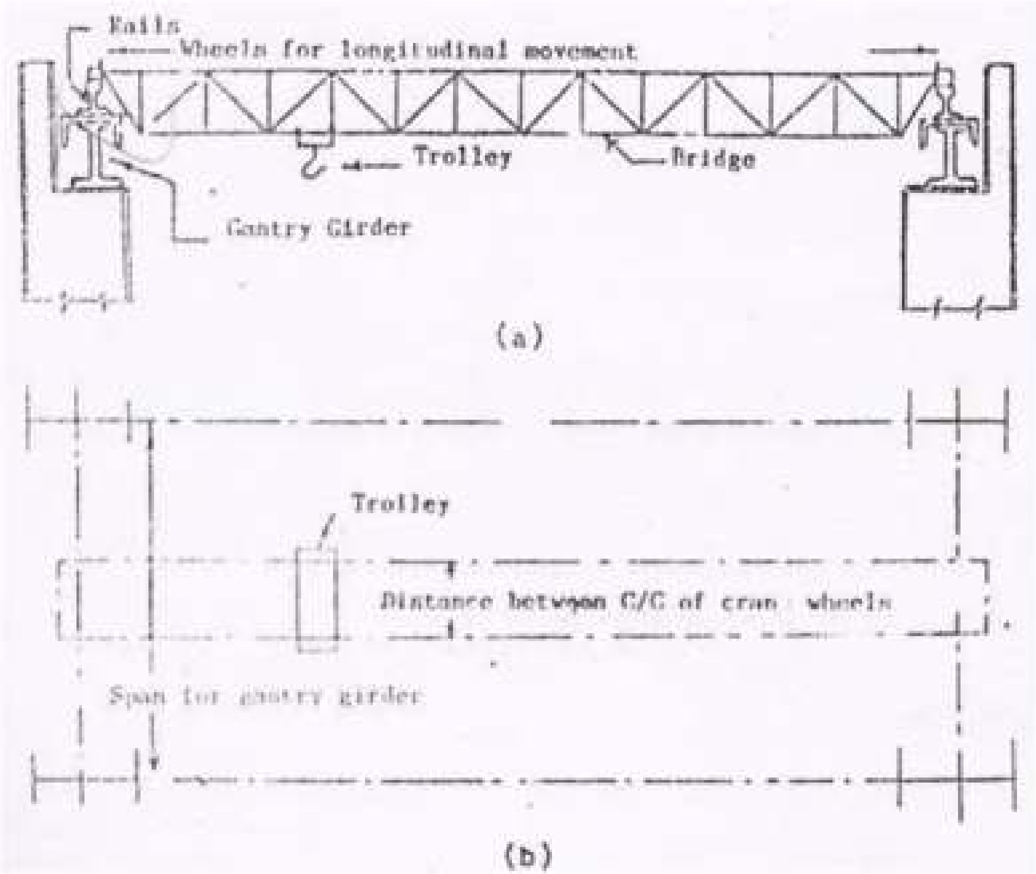
GANTRY GIRDER

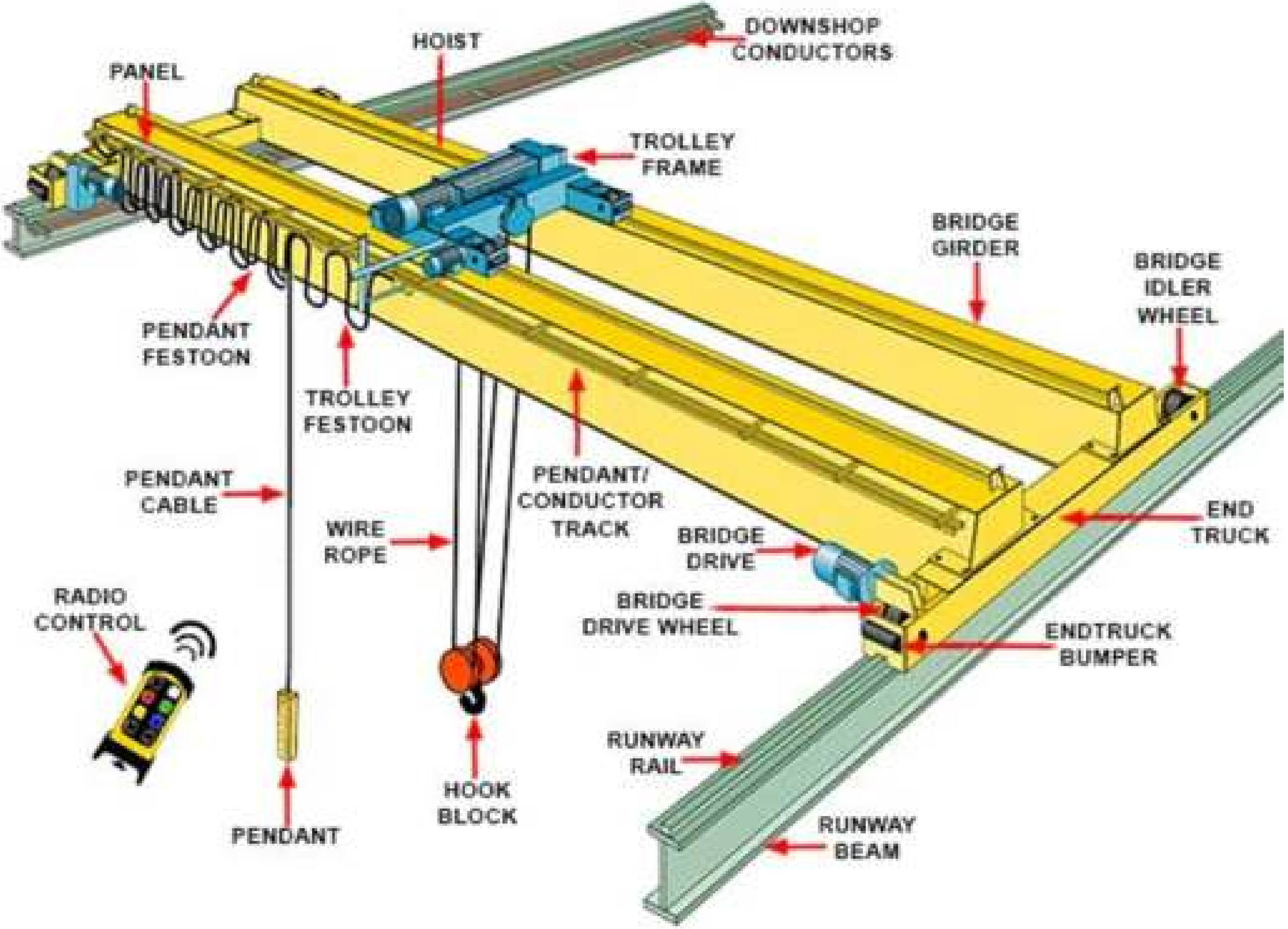
Dr. G.C. BEHERA



. COMPONENTS OF CRANE SYSTEM.

Overhead travelling cranes are used in factories and workshops to lift heavy materials, equipments, etc and to carry them from one place to the other. These cranes are either hand operated or electrically operated. The crane consists of a bridge spanning the bay of the shop. A trolley or a crab is mounted on the bridge. The trolley moves along the bridge. The bridge as a whole moves longitudinally on rails provided at the ends. The rails on either side of the bridge rest on crane gantry girders. The gantry girders are the girders which support the loads transmitted through the travelling (moving) wheels of the cranes as shown in figure below.





HOIST
DOWNSHOP CONDUCTORS
PANEL
TROLLEY FRAME
BRIDGE GIRDER
BRIDGE IDLER WHEEL
PENDANT FESTOON
TROLLEY FESTOON
PENDANT/ CONDUCTOR TRACK
PENDANT CABLE
WIRE ROPE
END TRUCK
BRIDGE DRIVE
BRIDGE DRIVE WHEEL
RADIO CONTROL
ENDTRUCK BUMPER
RUNWAY RAIL
HOOK BLOCK
PENDANT
RUNWAY BEAM

In factories and workshops, overhead traveling cranes are generally used to lift the heavy materials, equipments, etc and also to carry it from one place to another. Such types of cranes are mostly hand driven or electrically operated. The crane mostly consists of the bridges which have the span at the bay of the shop. The trolley or some type of cab is generally mounted on the bridge. Then the bridge as the whole unit moves longitudinally on the rails which are provided at the ends.
The rail on either side of the bridge rests on the crane gantry girder. The gantry girders are girders which supports the loads that are transmitted through the traveling wheels of the crane.

The crane girder spans from column to column, this usually do not have any lateral support at the intermediate points excepting when a walkway is formed at the top of the girder. Therefore under normal circumstances, the crane girder should be designed as laterally unsupported beam carrying vertical as well as the horizontal load at the top flange. So, the girder should be provided but having very heavy and wide compression flange is necessary. The wide flange beam without any other reinforcement is used for the shorter span and light crane loads. The cover plate should be provided on the compression face so that the lateral buckling strength of the beam improves while larger moment of inertia about the vertical axis against any lateral loading is also provided. To increase the property of I_{yy} then the channel can be provided instead of the cover plate. To increase the torsional stiffness of the girder channel is provided just below the compression flange of the wide flange beam and supported by brackets.

The stresses in the fiber of gantry crane girders should be computed by considering the bi-axial bending combined with the torsion. Generally the torsion is produced by the lateral force which applied at the top flange. The lateral moment is resisted by the top flange bending horizontally without any assistance from the bottom flange. The crane girders are supported on brackets which are connected to columns of uniform sections, it can also rested on stepped columns. The stepped columns are used for heavy crane loads and taller columns while the brackets are used for lighter crane loads. To restrain from lateral bending and twisting at the support point the girder is supported on suitably formed seat which are connected to the column just near the top flange. Also due to effect of temperature and deflection, to permit the horizontal movement in the crane girder slotted holes are used to connect the channels with column. To provide the restraint the vertical plates are mostly provided in the crane girder. If the roof leg as well as the crane leg is of column load and the shear action due to the effect of bending under crane load and wind load.

The crane columns should be properly braced in the longitudinal direction of the crane girder so that it can take the longitudinal forces acting due to moving crane. This kind of bracing should be provided at every fourth or fifth bay, while the other bays should be provided with the struts to transmit the longitudinal force to the bracing frame.

Gantry Girders are used in mill and heavy industrial buildings such as factories and workshops, where Gantry Girders are supported by columns and carrying cranes. Gantry girders are utilised to transport the goods and equipment from one place to another place in the workshop. Gantry Girders are typical example for laterally unsupported beam in industrial buildings. Also Gantry Girders undergo bending moment under both the direction. one is vertically and other is laterally. So biaxial bending movement has to be also checked for design of Gantry Girders. Therefore the when we will be going to design Gantry Girders we need to consider two things, one is the Gantry Girders is a laterally unsupported beam, so we have to design the Gantry Girders considering the lateral torsional buckling effect and also we have to consider the biaxial bending that means that interaction formula

$$\frac{M_Z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1.0$$

In Gantry Girders, the loads are moving from one place to another place therefore we need to know little about influence line diagram that means, we have to see the position of load in which the maximum bending moment and maximum shear force is going to occur. So as the wheel is moving from one place to another place, wheels are placed in such a way that maximum shear force and the maximum bending moment can be achieved. The gantry girder is designed against that maximum shear force and bending moment.In gantry girder certain impact load will come into picture so some additional

In gantry girder certain impact load will come into picture so some additional load has to be added in the load calculation. It has been recommended in codal guidelines to add certain percentage of additional load. Also certain percentage of load will be acting as a lateral load for which lateral bending moment and lateral shear force will be produced and because of that the lateral bending strength and shear strength should also be checked so that it is not going to fail under this lateral load.

The overhead travelling crane running system consists few components like crane, then this crane is compromising the crab or trolley, power transmitting device and the cap which houses the control and operators and also the crane rails and their attachment, also the gantry girder and column with brackets supporting gantry girder.

Gantry girder is placed on the column either with the support of a bracket or with the step column. Above the step column packing plates are placed, then gantry girders are onsisting of I Section is placed above packing plate and at the top planes along with I section we provide another channel section to take the heavy load coming from crane girder. Crane girder is placed on top of the crane rails. A crab trolley is kept in the crane girder, with the help of the trolley loads are being shifted from one place to another. A diaphragm is used to keep the I section the gantry girder in position throughout its length. Gantry girder is supported between two stepped columns in two sides.

The following imposed loads should be considered in the design.

1. Vertical loads from the cranes because crane will be carrying certain instruments, certain heavy machines.
2. Impact loads from crane because during operation certain impact will come into picture.
3. Longitudinal horizontal force along the crane rail.
4. Lateral thrust across the crane rail In calculating the above forces crane should be positioned such that it gives maximum design forces in the girder.

ADDITIONAL IMPACT LOADS

Type of load	*Additional load*
Vertical loads	
(i) For electric overhead cranes	25% of the maximum static wheel load
(ii) For hand operated cranes	10% of the maximum static wheel load
Horizontal forces transverse to rails	
(i) For electric overhead cranes	10% of the wt of crab & the wt lifted on the crane
(ii) For hand operated cranes	5% of the wt of crab & the wt lifted on the crane
Horizontal forces along rail	5% of the static wheel load

LATERAL LOAD

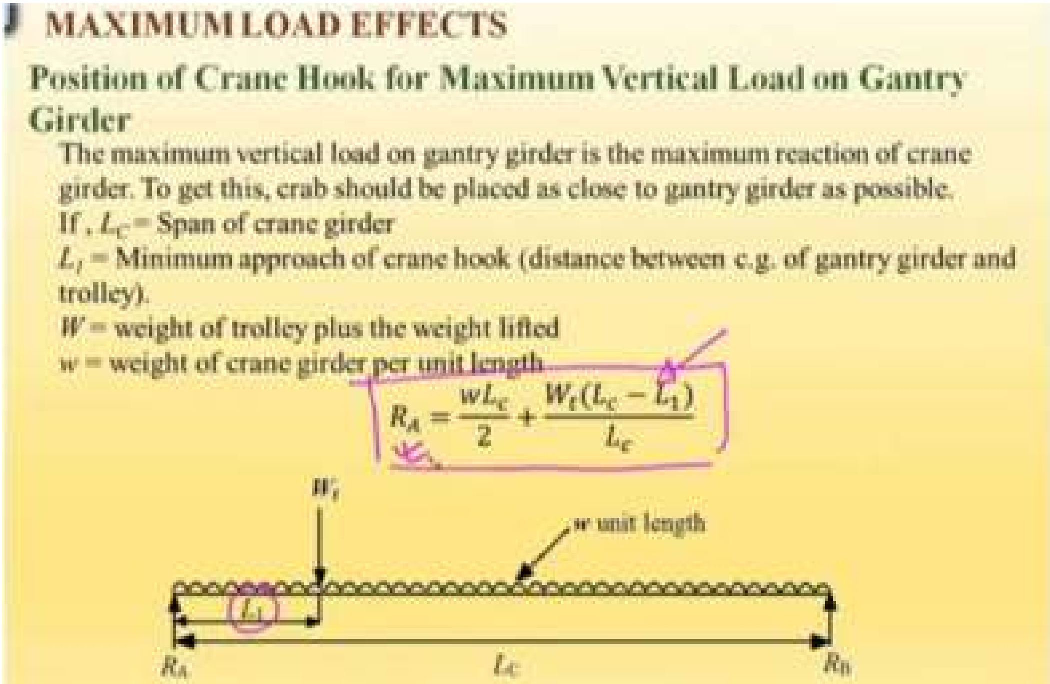
- As the crane moves with the load, a lateral load (transverse to the rail) is developed due to application of brakes or sudden acceleration of trolley.
- IS 875 recommends **10% of *W*** for EOT cranes as horizontal loads, where *W* is the total weight including lifted weight and the trolley weight.

LONGITUDINAL LOAD

- As the crane moves longitudinally, loads parallel to the rails are caused due to the braking (stopping) or acceleration and swing (starting of the crane). This load is called the longitudinal load and is transferred at the rail level.
- The longitudinal load per wheel = 5% of the wheel load.

$$W_g = 5W/100$$

	Category	*Max. Deflection*
a.	Where manually operated cranes are operated and for similar loads.	L/500
b.	Where electric overhead traveling cranes operate, up to 50t.	L/750
c.	Where electric overhead traveling cranes operate, over 50t	L/1000
d.	Other moving loads such as charging cars, etc.	L/600
e.	**Lateral deflection** Relative between rails	10 mm or L/400



Ra*LC=Wt*(Lc-L1)+w*Lc*Lc/2

$$R_A = \frac{W L_c}{2} + \frac{W_t(L_c - L_1)}{L_c}$$

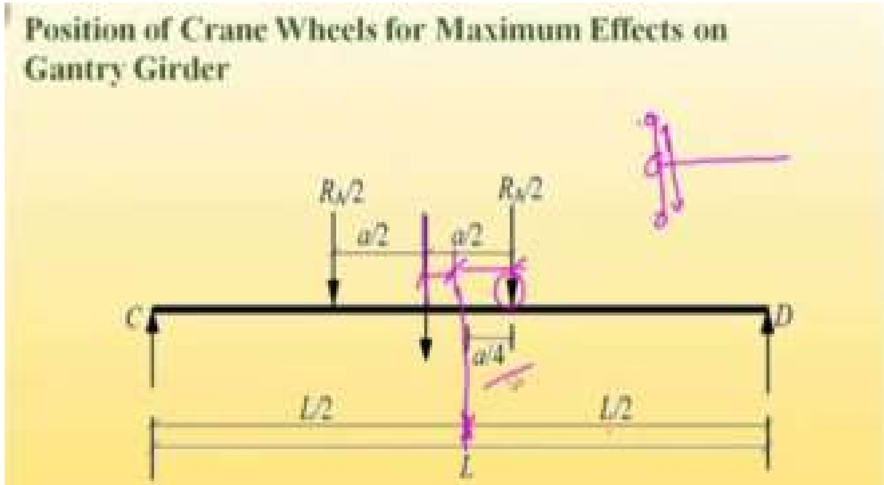
The maximum vertical load on gantry girder is the maximum reaction of crane girder. To get this, crab should be placed as close to gantry girder as possible.

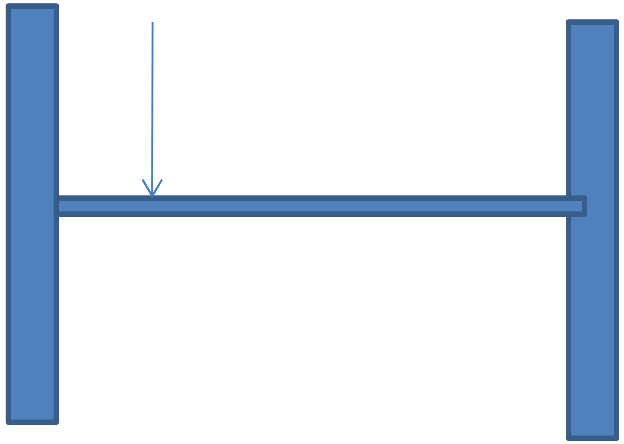
If , L_C = *Span of crane girder*

L_1 = *Minimum approach of crane hook (distance between c.g. of gantry girder and trolley).*

W = weight of trolley plus the weight lifted

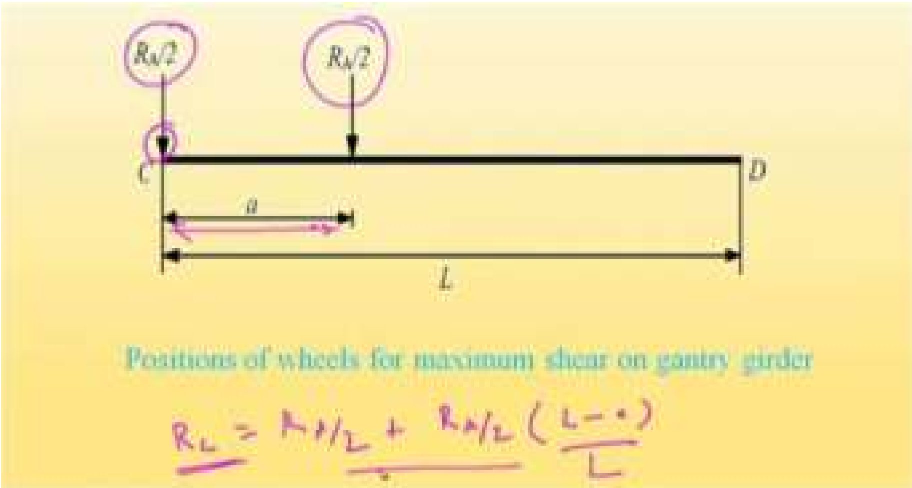
w = weight of crane girder per unit length





Position of wheels for maximum moment on gantry girder

So, once the wheel load is found, maximum bending moment and maximum shear force in gantry girder can be obtained. Maximum bending moment occur when the mid span of the gantry girder intersects the distance between C.G. of the wheels of and one of the wheel load. Then, the maximum bending moment can be achieved at a position of the nearest wheel load from the mid span.



Maximum shear force can be achieved by placing one of the wheel loads on the support.

Gantry girders cause moving loads that cause fatigue. Fatigue effects for light and medium duty cranes need not to be checked, if normal and shear stress ranges,

$$f \le \frac{27}{\gamma_{mft}}$$

Or, if actual number of stress cycle,

$$N_{SC} < 5 \times 10^{6}\left(\frac{27/\gamma_{mft}}{\gamma_{m} f}\right)^{3}$$

For heavy duty crane, the gantry girder must be checked for fatigue.

γ_{mft} = partial safety factor for strength (Table 25 of IS 800-2007)

f = actual fatigue stress range

γ_m = partial safety factor for material = 1.10

For heavy duty crane the gantry girder must be checked for fatigue.

Normal stress range

$$f_f = f_{fn} \sqrt[3]{5 \times 10^6 / N_{sc}} \text{ for } N_{sc} \leq 5 \times 10^6$$

$$f_f = f_{fn} \sqrt[5]{5 \times 10^6 / N_{sc}} \text{ for } 5 \times 10^6 \leq N_{sc} \leq 10^8$$

Shear stress range

$$\tau_f = \tau_{fn} \sqrt[5]{5 \times 10^6 / N_{sc}}$$

Where, $f_f, \tau_f =$ ¿design normal and shear fatigue stress range of the details , respectively for life cycle of N_{SC}

f_{fn} , τ_{fn}=. normal and shear fatigue strength of the details for 5×10^6 cycles for the detail category.

DESIGN STEPS

1. Maximum wheel load will come when one wheel is close to the gantry girder. The wheel can move along crane girder, maximum effect will occur when it will be closest to gantry girder.
2. In second step, maximum moment and shear force on gantry girder can be calculated after suitable proportioning of crane. Contribution of impact load should be taken care of. Though the maximum moment 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 UDL on girder for simplification and design moment is found.
3. find out the maximum shear force due to this vertical load .
4. shear force will be obtained maximum when one of the wheel is placed at the support of the gantry girder. So similar way we can find out maximum bending moment and shear force due to lateral load with similar positions.
5. In next step, we have to find out section modulus. Generally, an I-section with channel section is chosen, though an I-section with a plate at the top flange may be used for light cranes. $Z_p = M_u/f_y$. When the gantry is not laterally supported, the following formula may be used to select a trial section: *Zp (trial)=k Zp* ***(k = 1.30 – 1.60)*** Generally, the economic depth of a gantry girder is about (1/12)th of the span. The width of the flange is chosen to be between (1/40)th and (1/30)th of the span to prevent excessive lateral deflection.
6. Next step, a suitable section is chosen and the properties IZZ , IYY and Zez ,Zey, Zpy ,Zpz are found. Then the section is classified according to *b/tf and d/tw ratios.*

7. When lateral support is provided at the compression flange, the chosen section should be checked for the moment capacity of the whole section:

$$M_{dz} = \beta_b Z_{pz} f_y / \gamma_{m0} < 1.2 Z_{ez} f_y / \gamma_{m0}$$

However, for laterally unsupported compression flange, the buckling resistance is to be checked with design bending compressive stress ***fbd .Bending strength about yy axis is*** calculated because of lateral loading:

$$M_{dy} = \beta_b Z_{py} f_y / \gamma_{m0} < 1.2 Z_{ez} f_y / \gamma_{m0}$$

Combined local capacity can be checked as $\left(\frac{M_y}{M_{dy}}\right) + \left(\frac{M_z}{M_{dz}}\right) \leq 1$

Then section is to be checked against shear and local buckling will be checked under wheel load. The girder needs to be checked for bearing. Bearing stiffness will be provided if necessary. The maximum deflection under working load must be checked and the girder is checked for fatigue strength.

Example:

Design a simply supported gantry girder to carry electric overhead travelling crane, given:

Span of gantry girder = 6 m

Span of crane girder = 15 m

Crane capacity = 200 kN

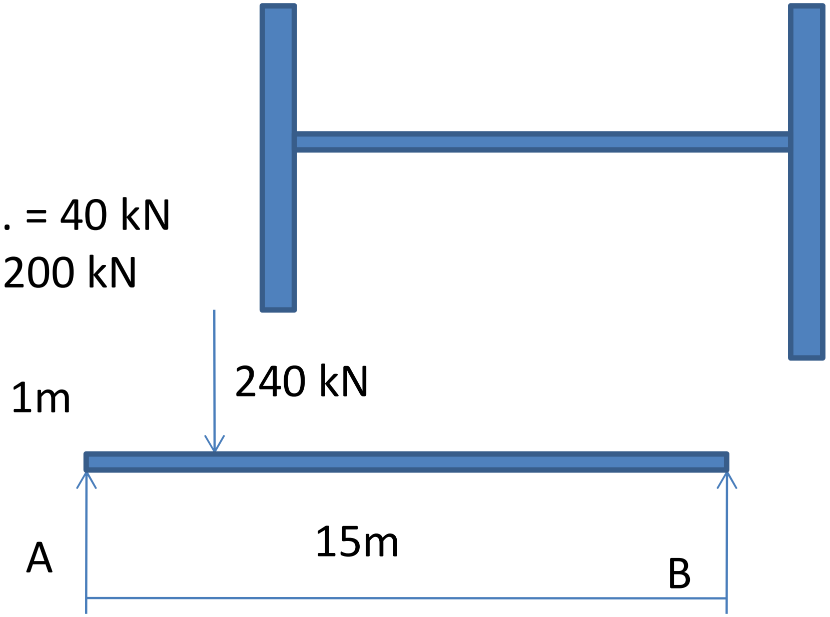
Self weight of trolley, hook, electric motor etc. = 40 kN

Self weight of crane girder excluding trolley = 200 kN

Minimum hook approach = 1.0 m

Distance between wheels = 3 m

Self weight of rails = 0.2 kN/m



Solution:

Maximum moment due to vertical force

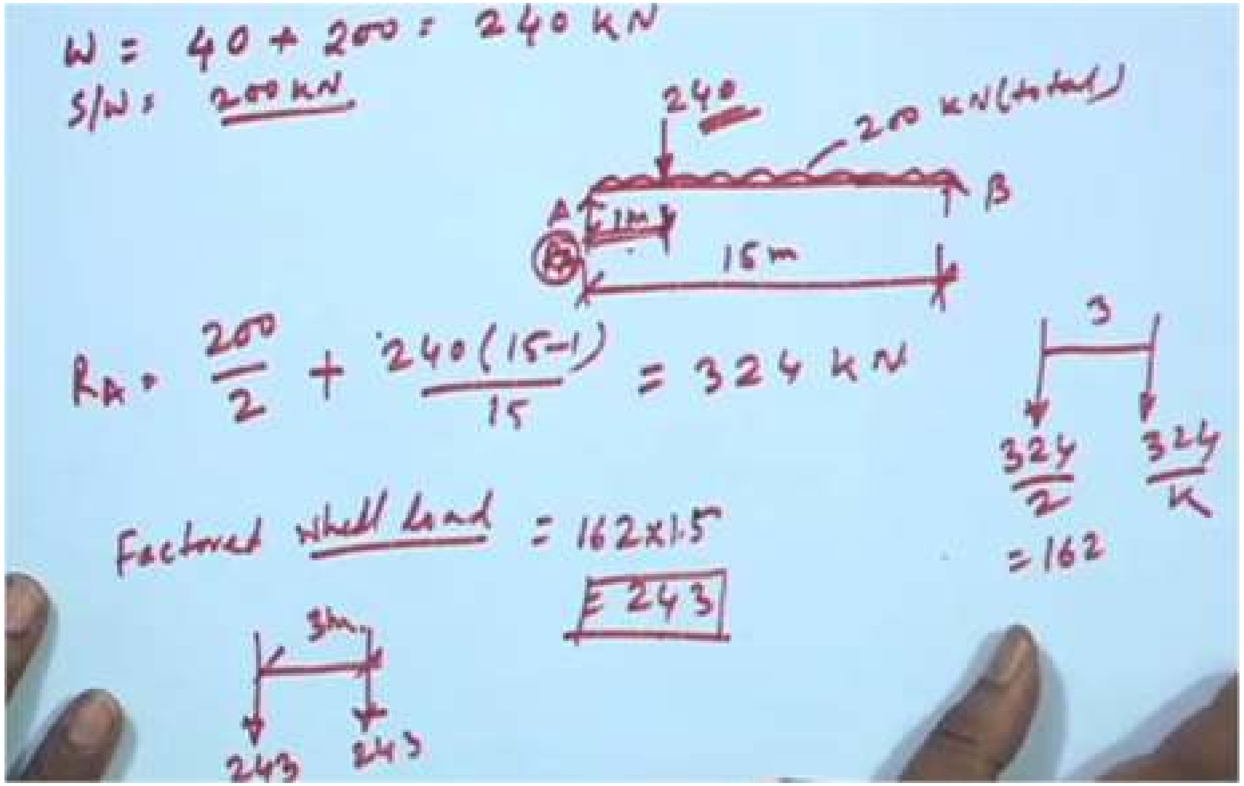
Weight of trolley + lifted load = 40 + 200 = 240 kN

Self weight of crane girder = 200 kN

For maximum reaction on gantry girder, the moving load should be as close the gantry as possible.

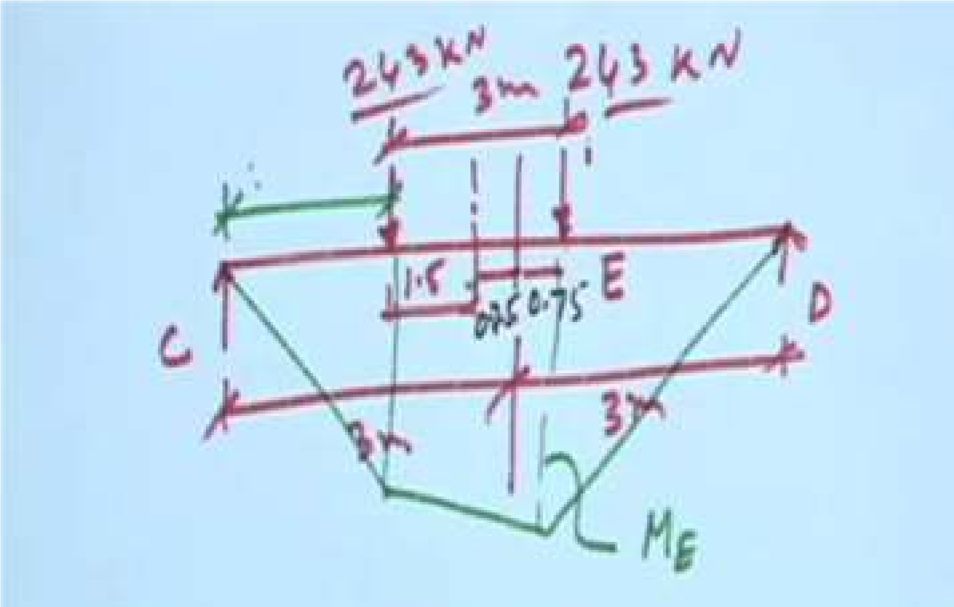
$$R_A = \frac{W L_c}{2} + \frac{W_t (L_c - L_1)}{L_c}$$

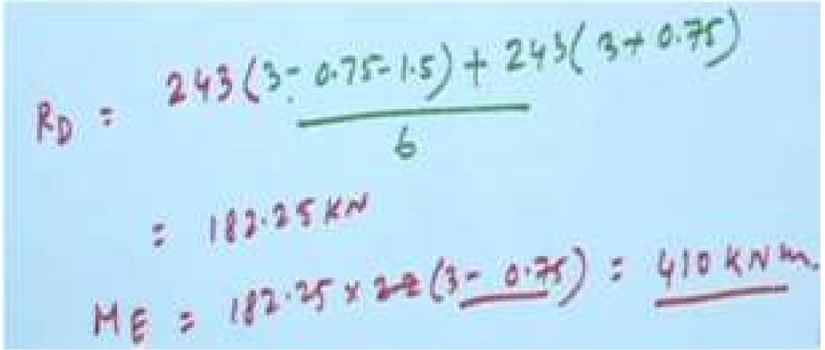
$$R_A = \frac{240 \times 14}{15} + \frac{200}{2} = 324 \quad \text{kN}$$

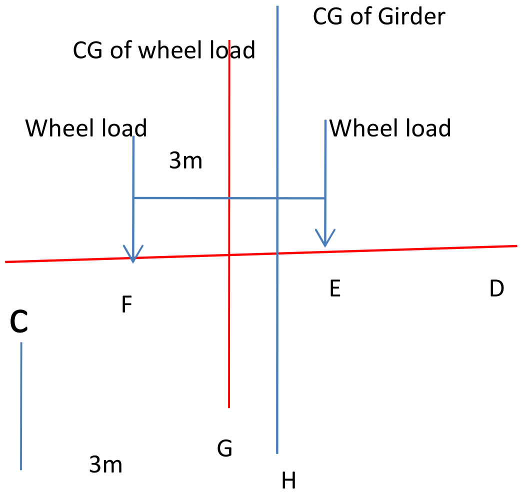


This load is transferred to gantry girder, through two wheels, the wheel base being 3 m.
So load on gantry girder from each wheel = 324/2 = 162 kN
Factored wheel load = 162 × 1.5 = 243 kN

Maximum moments due to moving loads occur under a wheel when the c.g. of wheel load and the wheel are equidistant from the centre of girder. This is shown in figure:







Moment due to impact = 0.25 × 410 = 102.5 kN-m

Assume self weight of girder = 2 kN/m

Dead load due to self weight + rails = 2 + 0.2 = 2.2 kN/m

Factored DL = 2.2 × 1.5 = 3.3 kN/m

Moment due to DL = $3.3 \times 6^2/8$ = 14.85 kN-m

Factored moment due to all vertical loads,

M_z = 410 + 102.5 + 14.85 = 527.35 kN-m

Maximum moments due to moving loads occur under a wheel when the c.g. of wheel load and the wheel are equidistant from the centre of girder.

If G and E are equidistant from H.

CH= 3m, FE=3 m, FG=1.5 m, GE=1.5 m, GH=HE=0.75 m

CF=CH-FH=CH-(FG+GH)=3-(1.5+0.75)=0.75m

CE=CH+HE=3+0.75=3.75 m

Maximum moment due to lateral force

Horizontal force transferred to rails = 10% of weight of trolley plus load lifted = (10/100) × (200 + 40) = 24 kN

This is distributed over 4 wheels.

So, horizontal force on each wheel = 24/4 = 6 kN

Factored horizontal force on each wheel = 1.5 × 6 = 9 kN

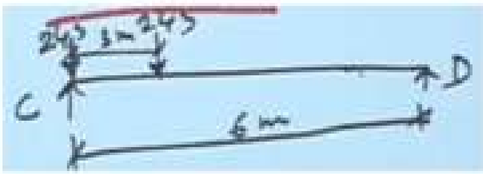
For maximum moment in gantry girder the position of loads is same as earlier except that it is horizontal. Hence by proportioning we get,

M_y = (9/243) × 410 = 15.18 kN-m

Shear force

For maximum shear force on the girder, the trailing wheel should be just on the girder as shown in figure below

Vertical shear due to wheel loads = 243 + (243 × 3)/6 = 364.5 kN



Vertical shear due to impact = 0.25 × 364.5 = 91.125 kN

Vertical shear due to self weight = (3.3 × 6)/2 = 9.9 kN

Total vertical shear = 364.5 + 91.125 + 9.9 = 465.52 kN

By proportioning lateral shear due to surge = (9/243) × 465.52 = 17.24 kN

Preliminary Section

Minimum economic depth, $L/12 = 6000/12 = 500$ mm

Width of the compression flange may be taken as (1/40) to (1/30)th of the span

So, flange width can be taken, $L/40 = 6000/40 = 150$ mm to $L/30 = 6000/30 = 200$ mm

Required $Z_p = 1.4 \times M/f_y = 1.4 \times 527.35 \times 10^6/250 = 2953.16 \times 10^3$ mm^3

Let us try a ISMB 550 with ISMC 250 on compression flange.

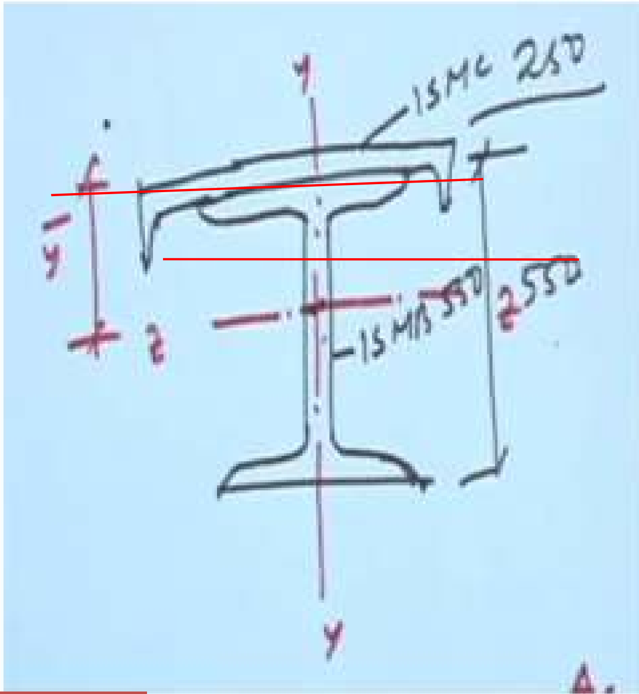
Properties of ISMB 550 @ 1.02 kN/m	Properties of ISMC 250 @ 0.3 kN/m
$A = 13200$ mm^2	$A = 3900$ mm^2
$h = 550$ mm	$h = 250$ mm
$b = 190$ mm	$b = 80$ mm
$t_f = 19.3$ mm	$t_f = 14.1$ mm
$t_w = 11.2$ mm	$t_w = 7.2$ mm
$I_{zz} = 64900 \times 10^4$ mm^4	$I_{zz} = 3880 \times 10^4$ mm^4
$I_{yy} = 1830 \times 10^4$ mm^4	$I_{yy} = 211 \times 10^4$ mm^4
$R_1 = 18$ mm	$C_{yy} = 23$ mm

Let the distance of N. A. from the tension flange be $\acute{y}$,

Then, $\acute{y}=\frac{13200\times 275+3900\times(550+7.2-23)}{13200+3900}=334.11$ mm

$I_{zz} = 64900 \times 10^4 + 13200 \times (334.11-275)^2 + 211 \times 10^4 + 3900 \times (550+7.2-23-334.11)^2 = 853.37 \times 10^6$ mm^4

$Z_{ez} = 853.37 \times 10^6 / 334.11 = 2554.15 \times 10^3$ mm^3



For compression flange about y-y axis,

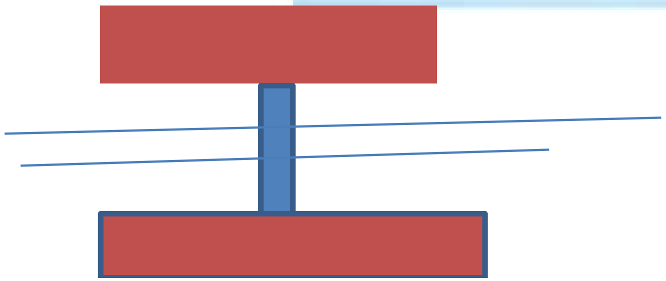
$I = 3880 \times 10^4 + 1/12 \times 19.3 \times 190^3 = 4984.15 \times 10^4$ mm^4

Z_{ey} for compression flange = $4983.15 \times 10^4 / 125 = 398.73 \times 10^3$ mm^3

Total area of section = 13200 + 3900 = 17100 mm^2

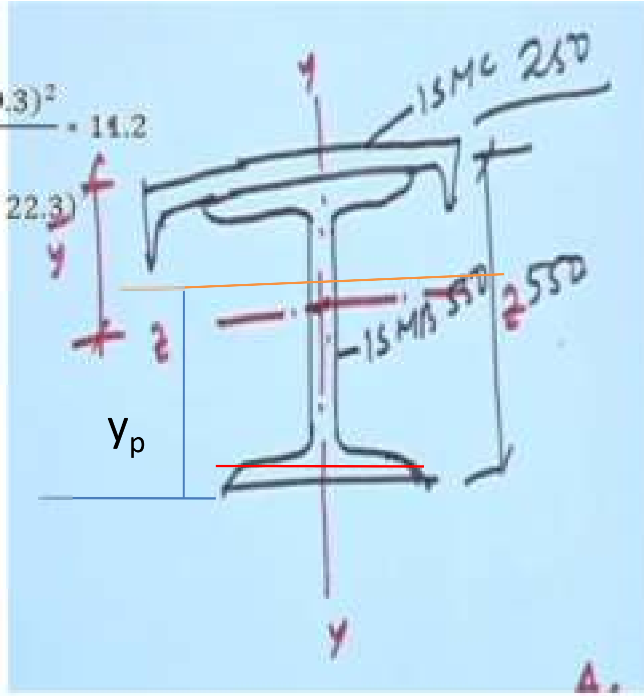
Let Plastic N.A. be at a distance Y_p from tension flange. Then,

$(Y_p - 19.3)\times 11.2 + 190 \times 19.3 = 17100/2$



Y_p=455.28 mm

$$Z_{pz} = (190*19.3)*\left(455.28-\frac{19.3}{2}\right)+\frac{(455.28-19.3)^2}{2}*11.2+\frac{(550-455.28-19.3)^2}{2}*11.2$$
$$+190*19.3*\left(550-455.28-\frac{19.3}{2}\right)+3900*(550-455.28+7.2-22.3)$$
$$=3352.89*10^3\ mm^3$$



For Top Compression Flange

$$Z_{py} = 19.3*\left(\frac{190}{2}\right)*\left[\frac{\frac{190}{2}}{2}\right]*2+2*\left[\frac{(250-2*14.1)}{2}\right]*7.8*\frac{\left[\frac{(250-2*14.1)}{2}\right]}{2}+2$$
$$*\left[80*14.1*\left(\frac{250}{2}-\frac{14.1}{2}\right)\right]=536.203*10^3\ mm^3$$

Section classification

b/t of flange of ISMB 550 = (190-11.2)/(2 ×19.3) = 4.63 < 9.4

d/t of web of ISMB 550 = (550-2 ×(19.3+18))/11.2 =42.44 < 84

And b/t of flange of channel = (80-7.2)/14.1 = 5.16 < 9.4

Hence the section is plastic.

Check for local moment capacity

Local moment capacity for bending in vertical plane:

$M_{dz} = f_y Z_p / 1.1 = 250 \times 3367.74 \times 10^3 / 1.1 = 765.31$ kN-m

$1.2 Z_{ez} f_y / 1.1 = 1.2 \times 2554.15 \times 10^3 \times 250 / 1.1 = 696.58$ kN-m

So, $M_{dz} = 696.58$ kN-m

For top flange,

$M_{dy} = 250 \times 536.203 \times 10^3 / 1.1 = 121.86$ kN-m

$1.2 Z_{ey} f_y / 1.1 = 1.2 \times 332.21 \times 10^3 \times 250 / 1.1 = 90.6$ kN-m

So for top flange $M_{dy} = 90.6$ kN-m

Check for combined local capacity

$527.35 / 696.58 + 15.18 / 90.6 = 0.92 < 1$

Check for buckling resistance

$M_d = \beta_b Z_p f_{bd}$

For plastic section $\beta_b = 1$

$$f_{cr,b} = \frac{1.1\pi^2 E}{\left(\frac{L_{LT}}{r_y}\right)^2}\sqrt{1+\frac{1}{20}\left(\frac{\frac{KL}{r_y}}{\frac{h}{t_f}}\right)^2}$$

$L_{LT} = 6000$ mm, E = 2×10^5 N/mm^2

$h_f = 550 - (19.3/2) + (14.1/2) = 547.4$ mm

$I_y = 1830 \times 10^4 + 3880 \times 10^4 = 5710 \times 10^4$ mm^4

$A = 13200 + 3900 = 17100$ mm^2

$r_y = (I_y/A)^{1/2} = (5710 \times 10^4/17100)^{1/2} = 57.78$ mm

$$f_{cr,b} = \frac{1.1\pi^2 \times 2 \times 10^5}{\left(\frac{6000}{57.78}\right)^2}\sqrt{1+\frac{1}{20}\left(\frac{\frac{6000}{57.78}}{\frac{547.4}{19.3}}\right)^2} = 260.23 \text{ N/mm}^2 \quad \lambda_L = \sqrt{\frac{250}{260.23}} = 0.98$$

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

$$\Phi_{LT} = 0.5[1 + 0.21(0.96 - 0.2) + 0.96^2] = 1.04$$

$$X_{LT} = \frac{1}{\emptyset_{LT} + \sqrt{\emptyset_{LT}^2 - \lambda_{LT}^2}} \leq 1.0$$

$$X_{LT} = \frac{1}{1.04 + \sqrt{1.04^2 - 0.96^2}} = 0.694 \cong 0.694 \leq 1.0$$

f_{bd}=0.694*250/1.1=157.83 N/mm^2

M_{dz}=β_b*Z_p*f_{bd}=1*3367.74*10^3*157.83=531.53 kNm

$$Z_{py} = 19.3 * \left(\frac{190}{2}\right) * \left[\frac{\frac{190}{2}}{2}\right] * 4 + 2 * \left[\frac{(250 - 2 * 14.1)}{2}\right] * 7.8 * \frac{\left[\frac{(250 - 2 * 14.1)}{2}\right]}{2} + 2 * \left[80 * 14.1 * \left(\frac{250}{2} - \frac{14.1}{2}\right)\right] + 2 * (550 - 2 * 19.3) * \left(\frac{11.2}{2}\right) * \left(\frac{11.2}{2}\right)/2 = 726.42 * 10^3\ mm^3$$

f_{bd}=0.694*250/1.1=157.83 N/mm^2

M_{dy}=β_b*Z_{py}*f_{bd}=1*726.42*10^3*157.83=114.58 kNm

FOR COMBINED BENDING EFFECT

531.53

$$\varphi_{_L} = 0.5\left[1 + 0.21 \times (0.98 - 0.2) + 0.98\,2\right] = 1.062$$

$$X_{_L} = \frac{1}{\left[1.062 + (1.062\,2 - 0.98\,2)\,0.5\right]} = 0.68$$ N/mm^2 $f_{bd} = 0.733 \times 250/1.1 = 164.11$

$M_{dz} = 1.0 \times 166.6 \times 3367.74 \times 10^3 = 531.06$ kN-m

Since lateral force is also acting, the beam must be checked for bi-axial bending.

So, $M_{dy} = 250/1.1 \times (1830 \times 10^4 + 3880 \times 10^4)/125 = 103.81$ kN-m

Hence, $$\frac{527.35}{561.06} + \frac{15.18}{103.81} = 1.086$$

The section is unsafe against torsional buckling.

CHECK FOR SHEAR

V_z = 465.52 kN

Shear capacity = $\frac{A_v f_{yw}}{\sqrt{3} \times 1.1} = \frac{550 \times 11.2 \times 250}{\sqrt{3} \times 1.1} = 808.29$ kN > 465.52 kN

Now, 0.6 × 808.29 = 484.974 kN

So, it is a case of low shear.

Check for Web buckling

Assuming b_1 = 150 mm

n_1 = 225 + 7.6 = 232.6 mm

d = 550 - 2(19.3+18) = 475.4 mm, t = 11.2 mm

$$\lambda = 2.42 \frac{d}{t} = 2.42 \times \frac{475.4}{11.2} = 102.72$$

For buckling class a, from **Table 9(a), Is 800: 2007**

F_{cd} = 128.6 N/mm^2

Buckling resistance = (150+232.6)×11.2×128.6 = 551.66 kN > 465.52 kN

Check for deflection at working load

(i) Vertical deflection

Serviceability vertical wheel load excluding impact = 162 kN

Deflection at mid-span

Where, $a = (L - c)/2 = (6000\text{-}3000)/2 = 1500$ mm

$$\delta = \frac{WL^3\left[\frac{3a}{4L} - \frac{a^3}{L^3}\right]}{6EI}$$

$$= 162 * 1000 * \frac{6000^3\left[\frac{3 * 1500}{4 * 6000} - \frac{1500^3}{6000^3}\right]}{[6 * 2 * 10^5 * 853.37 * 10^6]}$$

$$= 5.87\ mm$$

Allowable deflection=L/750=6000/750= 8 mm
So, OK.

(ii) Horizontal deflection

$I = (I_z)_{ch} + I_F = 4983.15 \times 10^4$ mm4

$$\delta = \frac{WL^3\left[\frac{3a}{4L} - \frac{a^3}{L^3}\right]}{6EI}$$

$$= 6 * 1000 * \frac{6000^3\left[\frac{3 * 1500}{4 * 6000} - \frac{1500^3}{6000^3}\right]}{[6 * 2 * 10^5 * 4983.15 * 10^4]}$$

$$= 7.788\ mm$$

MAXIMUM ALLOWABLE =10 MM

Design for weld

The required shear capacity of the weld, $q = VA\dot{y}/I_z$

V = 465.52 kN

A = 3900 mm^2 (Area of the channel section)

$\dot{y}$ = (550 - 334.11 + 14.1/2) = 222.94 mm

$I_{zz} = 853.37 \times 10^6$ mm^4

q = 465.52 $\times 10^3 \times 3900 \times 222.94 / 853.37 \times 10^3$ = 474.3 N/mm

So, $(0.7s \times 410)/(\sqrt{3} \times 1.5) = 474.3$

or, s = 4.29 mm

So use 5 mm fillet weld for the connection.