ANNAMACHARYA INSTITUTE OF TECHNOLOGY & SCIENCES
(AUTONOMOUS)
UTUKUR (P), C. K. DINNE (V&M), KADAPA, YSR DIST.
Approved by AICTE, New Delhi & Affiliated to JNTUA, Anantapuramu.
Accredited by NAAC with 'A' Grade, Bangalore.

Design of Steel Structures

(23HPC0117)

Lecture Notes

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CIVIL ENGINEERING					
III B.Tech – II Semester					
Course Code	DESIGN OF STEEL STRUCTURES	L	T	P	C
23HPC0117		3	0	0	3

Course Objectives:

The objectives of this course are to make the student to:

1. **Understand** the properties, types, and applications of structural steel in construction.
2. **Analyze** the behavior and design of bolted and welded connections for steel structures.
3. **Design** tension and compression members, including built-up members and column bases.
4. **Develop** steel structural elements such as beams, plate girders, roof trusses, and gantry girders.
5. **Apply** plastic analysis concepts to the design of continuous beams and portal frames.

Course Outcomes:

Upon successful completion of this course, students will be able to:

1. **Explain** the properties of structural steel, types of sections, and the concept of limit state design.
2. **Analyze and design** bolted and welded connections for structural steel members.
3. **Design** tension and compression members, including built-up sections and column bases.
4. **Develop** design solutions for beams, plate girders, roof trusses, and gantry girders.
5. **Perform** plastic analysis and design of continuous beams and portal frames.

CO – PO Articulation Matrix

Course Outcomes	PO 1	PO 2	PO 3	PO 4	PO 5	PO 6	PO 7	PO 8	PO 9	PO 10	PO 11	PO 12	PSO1	PSO2
CO -1	3	2	2	2	1	1	1	-	1	-	2	2	2	2
CO -2	3	2	2	2	1	1	1	-	1	-	2	2	1	1
CO -3	3	2	2	2	1	-	-	-	1	-	2	2	2	1
CO -4	3	2	2	2	1	-	-	-	1	-	2	2	1	1
CO -5	3	2	2	2	1	-	-	-	1	-	2	2	2	2

UNIT – I

INTRODUCTION to STRUCTURAL STEEL and DESIGN of CONNECTIONS

General -Types of Steel -Properties of Structural Steel - I.S. Rolled Sections, Section classification – Concept of Limit State Design – Design of Simple and Eccentric Bolted and Welded Connections – Types of Failure and Efficiency of Joint – Prying Action – Introduction to HSFG bolts

UNIT – II

DESIGN of TENSION and COMPRESSION MEMBERS

Behavior and Design of Simple and Built-Up Members Subjected to Tension – Shear Lag Effect Design of Lug Angles – Tension Splice – Behavior of Short and Long Columns – Euler's Column Theory Design of Simple and Built-Up Compression Members With Lacings and Battens – Design of Column Bases – Slab Base and Gusseted Base

UNIT – III
DESIGN of BEAMS
Design of Laterally Supported and Unsupported Beams – Design of Built-Up Beams – Design of Plate Girders

UNIT – IV
INDUSTRIAL STRUCTURES
Design of Roof Trusses – Loads On Trusses – Purlin Design Using Angle and Channel Sections – Truss Design, Design of Joints and End Bearings–Design of Gantry Girder – Introduction to Pre-Engineered Buildings

UNIT – V
PLASTIC ANALYSIS and DESIGN
Introduction to Plastic Analysis – Theory of Plastic Analysis – Design of Continuous Beams and Portal Frames Using Plastic Design Approach.

TEXT BOOKS:

1. Duggal S.K., Design of Steel Structures, Tata McGraw Hill, Publishing Co. Ltd., New Delhi, 2010
2. Bhavikatti S.S, Design of Steel Structures, Ik International Publishing House, New Delhi, 2017.

REFRENCE BOOKS:

1. Gambhir M L, Fundamentals of Structural Steel Design, McGraw Hill Education India Pvt Limited, 2013
2. Jack C. McCormac& Stephen F. Csernak – Structural Steel Design, Pearson, 7th Edition, 2023.
3. William T. Segui&FaridSoleimani – Steel Design, Cengage, 7th Edition, 2023.
4. SarwarAlamRaz, Structural Design in Steel, New Age International Publishers, 2014
5. Subramanian N, Design of Steel Structures, Oxford University Press, New Delhi, 2016

Online Learning Resources:
https://nptel.ac.in/courses/105105162

1. Intro to Structural Steel & Design of Connections

Steel Structure : It is a metal structure made up different members of steel to carry external loads & members.

Steel → Iron (>98%) + Carbon (0.1 - 1.1%) + Sulphur, P, Si, Mn, Ni, Cr

Stainless Steel : Steel + Nickel (8%) + Chromium (18%)

Uses of Steel Structure :

→ It has high strength to weight ratio meaning it has high strength/unit mass.

→ It is a fast construction, easy to repair, modify & more economical.

Types of Steel Structures & Structural Units :

* Towers
* Roof Trusses
* Water Tanks
* Bridges
* Gravity Gudders
* Chimneys
* Columns
* Building Frames

Codes for practices for use of Structural Steel in Building Construction :

→ For LSM : IS 800 : 2007

→ For WSM : IS 800 : 1984

Physical Properties of Strctrl Steel :

Property	As per IS 800 : 2007
* Specific Gravity	7.85
* Unit Mass	$\rho_s = 7850\ Kg/m^3$
* Youngs Modulus	$E = 2 \times 10^5\ N/mm^2$
* Modulus of Rigidity	$G = 0.769 \times 10^5\ N/mm^2$
* Coeff of Thermal Expansion	$12 \times 10^{-6}/^\circ c$
* Poissons Ratio	$\mu = 0.30$

Types of Rolled Structural Steel Sections :

* I section
* Tee section
* Channel section
* Angle Section
* Bars
* Flats
* Plates
* Sheets

From I section :

ISJB - Indian Standard Junior Beam

ISLB - Ind Stnd Light Beam

ISWB - Ind Stnd Wide Flanged Beam

ISHB - Ind Stnd Heavy Beam

ISMB - Ind Stnd Medium Beam

From Tee Section :

ISNT - Ind Stnd Normal Tee Section

ISDT - Ind Stnd Deep legged T section

ISLT - Ind stnd Slif light T - section

ISMT - Ind Stnd Medium Weight T section

ISHT - Ind Stnd Heavy Flange T section

From Chanel Section :

ISJC - Ind Stnd Junior C section or Channel

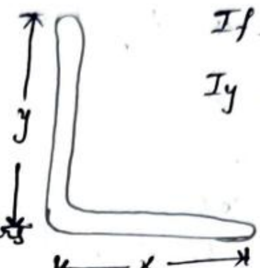
ISLC - Ind Stnd Light Channel

ISMC - Ind Stnd Medium Weight Channel

ISHC - Ind Stnd Heavy weight channel

From Ange Section (ISA) :

ISA - Ind Stnd Angle Section



If $y = x \rightarrow$ Equal Angle

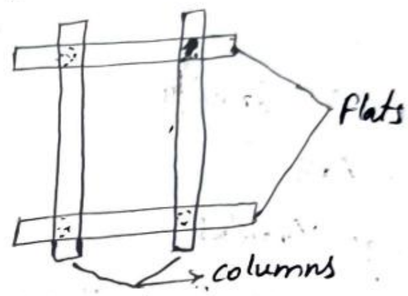
Iy $y \neq x \Rightarrow$ Unequal Angle

Bars :

ISRO - Ind Stnd Rounded Bars

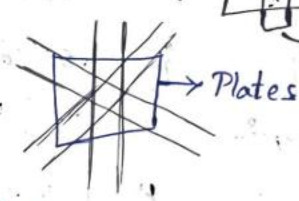
ISSQ - ~~Ind Stnd~~ Square Bars

Flats : ISF - Ind Stnd Flats



Plates : ISPL - Ind Stnd Plates

$\Rightarrow$ Its stnd thickness is 6 - 10mm



Loads & Load Combinations :

Dead Loads - DL

Imposed ld - LL

Wind Ld - WL

Earthquake ld - EL

Accidental ld - AL

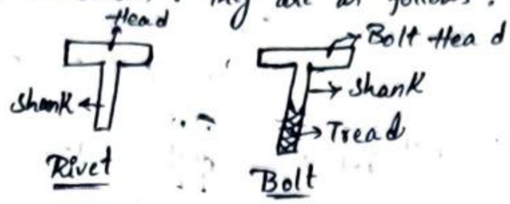
Temp ld - TL

Crane Ld (vertical or horizontal) - CL

Erection Ld - ER

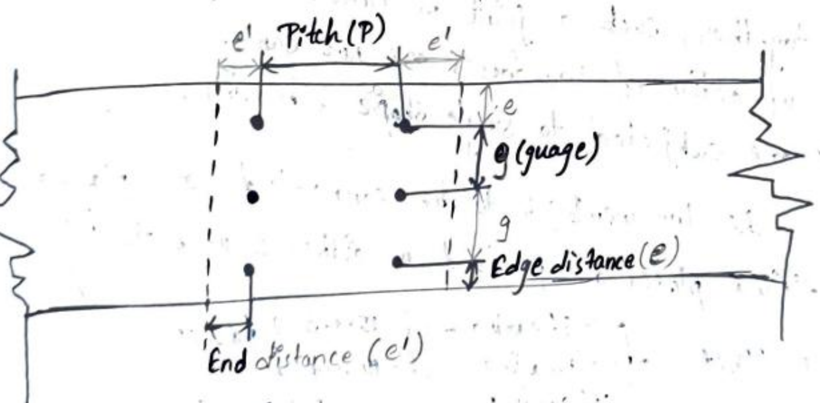
Types of Connections :

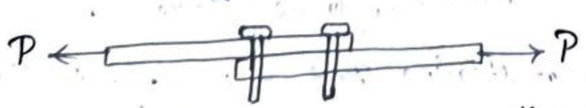
There are three types of connections. They are as follows :
a. Rivet Connection
b. Bolted Connection
c. Welded Connection



a. Rivet Connection : A rivet connection is a ductile steel pin with a manufactured head at one end & strip portion known as shank.

b. Bolted Connection : A bolted connection is defined as a metal pin with head at one end & a shank, thread at the other end..





(P) Pitch : It is the distance b/w two consecutive bolts measure parallel to the direction of force.

(g) Guage : It is the distance b/w two consecutive bolts measure ⊥ to the direction of force.

(e') End Distance : It is the distance b/w centre of bolt & end of the plate measure parallel to the direction & force.

(e) Edge Distance : It is the distance b/w centre of bolt & end of the plate measure ⊥ to the direction of force.

Specifications of IS800:2007

Minimum Edge/End Distance:

$1.5\ d_o$ → Machine Flame Cut

$1.7\ d_o$ → Hand Flame Cut

Minimum Pitch: $2.5\ \phi$

ϕ - Nominal dia (dia of bolt)

d_o - Gross dia

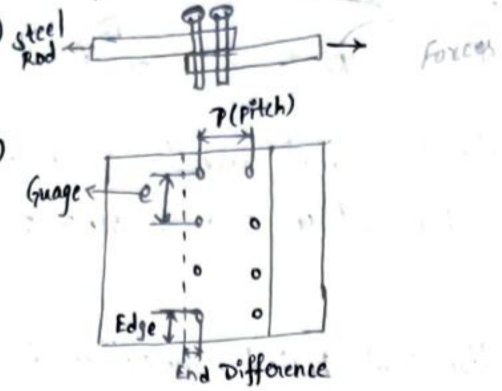
ϕ	d_o (clearness)
12 - 14mm	ϕ + 1mm
16 - 18 mm	ϕ + 2mm
> 24mm	ϕ + 3mm

Maximum Pitch:

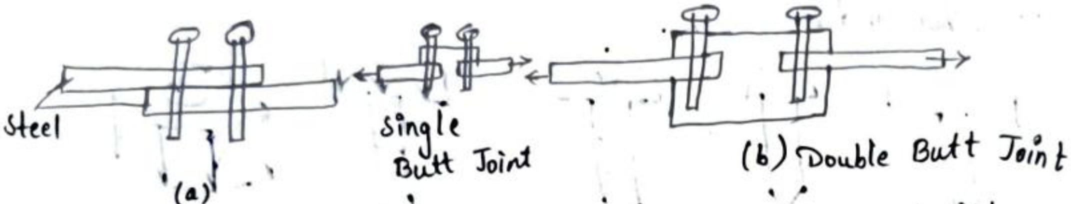
Compression Member: 12t or 200mm [whichever is less]

Tension Member: 16t or 200mm [whichever is less]

Tacking Bolts: 33t or 300mm [" " "]



Types of Joints: a) Lap Joint b) Butt Joint



Lap Joint: The two members to be connected are overlaped & connected together, such a joint is called lap joint.

* The line of action of two forces are not same.
* The bolts are subjected to single shear & bearing.

Butt Joint: The two members to be connected are placed end to end & additional plates are provided on either 1 or both sides.

Design of Bolt:
- → Shearing - Cl - 10.3.3 } pg - 75
- → Bearing - cl - 10.3.4 } pg - 75
- → Tearing - cl - 10.3.5 } pg - 76

a. **Design of strength of shearing Bolt:** From pg - 75; Cl - 10.3.3

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} \longrightarrow V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$$

γ_{mb} → Partial safety factor = 1.25

f_{ub} → Ultimate Tensile Strength of Bolt

A_{sb} → Sectional Area the shank of Bolt $= \frac{\pi}{4} d^2$

where, d = Dia of Bolt

A_{nb} → Net Tensile Area of threads

$$\boxed{\begin{array}{c} A_{nb} = 0.78 \times \frac{\pi}{4} d^2 \\ \text{or} \\ A_{nb} = 0.78 \times A_{sb} \end{array}}$$

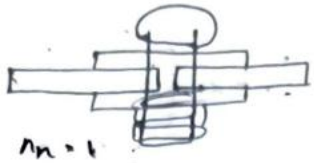
$A_{sb} = \frac{\pi}{4} d^2$

Where, $n_n \rightarrow$ No. of shear plane within the threads

$n_s \rightarrow$ No. of shear plane within the shunk

$n_n = 0$

$n_s = 1$



$n_n = 1$

$n_s = 1$

b. Design of Bearing :

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

Where, V_{npb} = Nominal Bearing strength of bolt $= 2.5\, K_b d t f_u$

$K_b = \frac{e}{3d_0}$ or $\frac{P}{3d_0} - 0.25$ or $\frac{f_{ub}}{f_u}$ or 1 - Whichever is less

e = End Distance

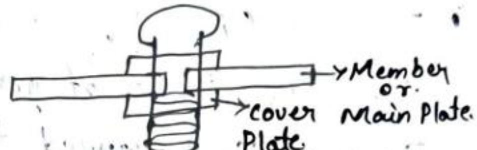
p = Pitch Distance

d_0 = Dia of hole

f_u = Ult. Tensile strength of bolt

f_{ub} = Ult. Tensile Strength of bolt

t = Thickness of connected plates

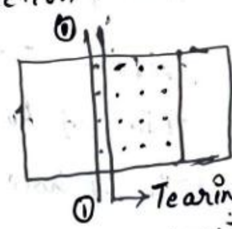


c. Design of Tearing or Tension Capacity : It is due to rupture @ the net section or critical section.

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}}$$

$$T_{nb} = 0.90\, f_{ub} A_n$$

$$T_{db} = \frac{0.90\, f_{ub} A_n}{\gamma_{mb}}$$



Tearing Failure for rupture.

f_{ub} : Ult. Tensile Stress of bolt

γ_{mo} = Partial Safety Factor

A_n = Net Sectional Area of the plate

$\therefore A_n = (b - n d_0) \times t \rightarrow$ for chaining bolt.

$A_n = \left[\{b - n d_0\} + \left(\frac{\sum P_{si}^2}{4 g_i} \right) \right] t \rightarrow$ for stagged bolting

n = No. of bolts @ the critical section

d. Efficiency (Joint/plate) :

$$\eta = \frac{\text{strength of joint/plate}}{\text{strength of solid plate}} \times 100$$

Bolts are arranged in parallel pattern :

1. Chain Pattern   2. Staggered Pattern or zig-zag pattern   3. Diamond Pattern

Min. Value of Bolt — Design Shearing / Design Bearing / Design Tearing } V_{db}

Note : Bolt Value (V_{db}) is defined as the value of bolt. The max. load that can be taken by the bolt in safe condition.

Hint : Service load given, then 1.5 × service load for taking total load. or Total Design Load given only consider the total load.

① Two flats [Fe 410], each 300 × 16mm are to be joined using 20mm dia of bolts of grade 4.6 to form a lap connection. The connection is supposed to transfer a service load of 375 KN. Calculate no. of bolts required for connection with min. pitch & end distance for bolts. Assume thread on the bolt does not intersect the shear plate.

Sol: Given : For Fe 410 grade steel

f_u = 410 N/mm²

Width = 300mm , Thickness = 16mm

Dia of bolts, d = 20 mm

Dia of hole bolts, d_o = Ø + 2mm

= 20 + 2

= 22 mm

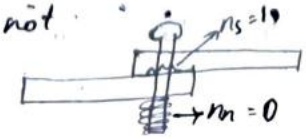
Service Load = 375 KN

Total Load, W = 1.5 × 375 = 562.5 KN

Bolt Grade - 4.6 [pg - 13 ; Table 1]

f_{ub} = 400 N/mm²

Assume thread on the bolt does not intersect the shear plate.



M20 bolts

Min. Pitch $= 2.5\,\phi$

$= 2.5 \times 20$

$P = 50mm$

Min. End Distance $= 1.5\, d_0$

$= 1.5 \times 22$

$e = 33mm$

$$n = \frac{\text{Design load or Total load or Factored load } (P)}{\text{Design of bolt } (V_{db})}$$

$$n = \frac{P}{V_{db}}$$

Design of Shearing : $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}} = \dfrac{\cancel{72549.83}}{1.25} = 58039.86\ KN$

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$$

$$A_{nb} = \frac{\pi}{4} d^2 \times 0.78 = \frac{\pi}{4}(20)^2 \times 0.78 = 245.04\ mm^2$$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4}(20)^2 = 314.15\ mm^2$$

$$V_{nsb} = \frac{400}{\sqrt{3}}\left[0 \times A_{nb} + 1 \times A_{sb}\right]$$

$$= \frac{400}{\sqrt{3}}\left[314.15\right]$$

$= 72549.83\ N$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{72549.83}{1.25} = 58039.86\ N = 58.039\ KN$$

Design of Bearing : $(V_{dpb}) = \dfrac{V_{npb}}{\gamma_{mo}} = \dfrac{164}{1.25} = 131.2\ KN$

$V_{npb} = 2.5\ K_b\, d\, t\, f_u$

$$K_b = \frac{e}{3d_0} = \frac{33}{3 \times 22} = 0.5$$

$$\frac{P}{3d_0} = \frac{50}{3 \times 22} = 0.75$$

$\therefore\ K_b = 0.5$

$V_{npb} = 2.5 \times 0.5 \times 20 \times 16 \times 410 = 164000\,N = 164\,KN$

$\therefore\ V_{db} = 58.04\ KN$

No. of bolts, $\boxed{n = \frac{W}{V_{db}}} = \frac{562.5}{58.04} = 9.69 \simeq 10$ Nos

from pg-75 :- Cl. 10.3.2

② Design a lap joint b/w the two plates each of width 120 mm, if the thickness of plate is 16 mm & the other is 12 mm. The joint has to transfer a design load of 160 KN. the plates are off Fe 410 grade. Use bearing type bolts. Use M16 bolts of grade 4.6.

Sol: Given : $b = 120$ mm

$t_1 = 16$ mm

$t_2 = 12$ mm

$W = 160$ KN

$n_n = 1$

$n_s = 0$

$f_u = 410$ N/mm^2

[For threads intercepting the shear plane common assumption] $d = 16$ mm

$d_o = \phi + 2 = 16 + 2 = 18$mm

Bolt Grade = 4.6

From pg-13 : Table-1

$f_{ub} = 400$ N/mm^2

Min. pitch = $2.5\phi = 2.5 \times 16 = 40$ mm

Min. End distance = $1.5 d_o = 1.5 \times 18 = 27$ mm

Design of Shearing: $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{36.21}{1.25}$

$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}[n_n A_{nb} + n_s A_{sb}]$ $\quad V_{dsb} = 28.97$ KN

$\frac{400}{\sqrt{3}}[1 \times 0.78 \times \frac{\pi}{4}(16)^2]$

$= 36.21$ KN

Design of Bearing: $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$

$V_{npb} = 2.5 K_b d t f_u$

Min. End Distance, $e = 1.5 d_o = 1.5 \times 18 = 27$ mm

Min. Pitch, $P = 2.5\phi = 2.5 \times 16 = 40$ mm

$K_b = \frac{e}{3d_o} = \frac{27}{3 \times 18} = 0.5$

$\frac{p}{3d_o} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.5$

$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{98.4}{1.25} = 78.72$ KN

Design strength of bolt,

$V_{db} = 28.97$ KN

Design of Tearing: $T_{db} = \frac{T_{nb}}{\gamma_{mb}}$

$T_{nb} = 0.90 f_{ub} A_n = 0.9 \times 400 \times 144 = 51.84$ KN

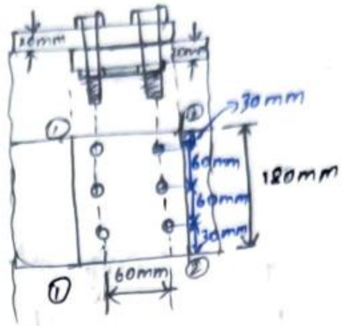
$A_n = (b - n d_o)t = (120 - 6 \times 18)12 = 144$ mm^2

$n = \frac{W}{V_{db}} = \frac{160}{31.15} = 5.13 \simeq 6$ Nos

$T_{db} = \frac{T_{nb}}{\gamma_{mb}} = \frac{51.84}{1.25} = 41.47$ KN

$V_{db} < T_{db}$

Q) Find the efficiency of the lap joint shown in fig. Use M20 bolts of grade 4.6 & Fe410 (E 250) plates are used.



Sol: $d = 20mm$

$d_0 = d + 2 = 20 + 2 = 22mm$

$f_u = 410\ N/mm^2$

$f_{ub} = 400 N/mm^2$ [From Table-1, Pg-13]

$\gamma_{mo} = 1.25$

$t = 20mm$ $\qquad b = 180\ mm$

$p = 60mm$

Gauge, $g = 60mm$

Edge $e = 30\ mm$

$n = 6$

$n_n = 1$

$n_s = 0$

Chain pattern = ①-①

Design of shearing: $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}} = \dfrac{339.537}{1.25} = 271.63\ KN$

$V_{nsb} = \dfrac{f_{ub}}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$

$A_{nb} = \dfrac{\pi}{4} d^2 \times 0.78 = \dfrac{\pi}{4}(20)^2 \times 0.78 = 245.04 mm^2$

$A_{sb} = \dfrac{\pi}{4} d^2 = \dfrac{\pi}{4}(20)^2 = 314.15\ mm^2$

$V_{nsb} = \dfrac{400}{\sqrt{3}}\left[6 \times 245.04 + 0 \times 314.15\right] = 339.537\ KN$

Design of bearing: $V_{dpb} = \dfrac{V_{npb}}{\gamma_{mo}} = \dfrac{184.5}{1.25} = 147.6 KN \times 6 = 885.6 KN$

$V_{npb} = 2.5\, K_b\, d\, t\, f_u$

$K_b = \dfrac{e}{3d_0} = \dfrac{30}{3 \times 22} = 0.45$ (or) $\dfrac{p}{3d_0} = \dfrac{60}{3 \times 22} = 0.45$

$V_{npb} = 2.5 \times 0.45 \times 20 \times 20 \times 410 = 184.5\ KN$

$\therefore V_{db} = 271.63\ KN \Rightarrow$ Design strength of bolt.

Design of tearing: $T_{db} = \dfrac{T_{nb}}{\gamma_{mb}} = \left[\dfrac{820.8}{1.25}\right] = 656.64 KN$

$T_{nb} = 0.90\, f_{ub}\, A_n = 0.90 \times 400 \times 2280 = 820.8\ KN$

$A_n = (b - n d_0)\, t = (180 - 3 \times 22)\, 20 = 2280\ mm^2$

$V_{db} < T_{db}$.

Design Strength of Solid Plate $= \dfrac{f_y}{\gamma_{m0}} \times A_g$ → Pg 190, Table-1; γ_{m0} → Pg-30, Table-5

$A_g = 180 \times 20 = 3600\ mm^2$

$= \dfrac{240}{1.1} \times 3600$

$= 785.45\ KN$

Efficiency $= \dfrac{\text{Design Strength of joint}}{\text{Design Strength of solid plate}} \times 100$

$= \dfrac{271.63}{785.45} \times 100$

$= 34.58\ \%$

(4) Two cover plates 10mm & 18mm thick are connected by a double cover butt joint using 6mm cover plates as shown in fig. Find the strength of the joint. Given M20 bolts of grade 4.6 & Fe415 plates are used,

Given

$d = 20\ mm$ $\quad f_{ub} = 400 N/mm^2$

$d_0 = 22\ mm$ $\quad$ Cover plate $= 6mm$

$f_u = 415\ N/mm^2$

$t_1 = 10mm$

$t_2 = 18mm$

$n_n = 01$

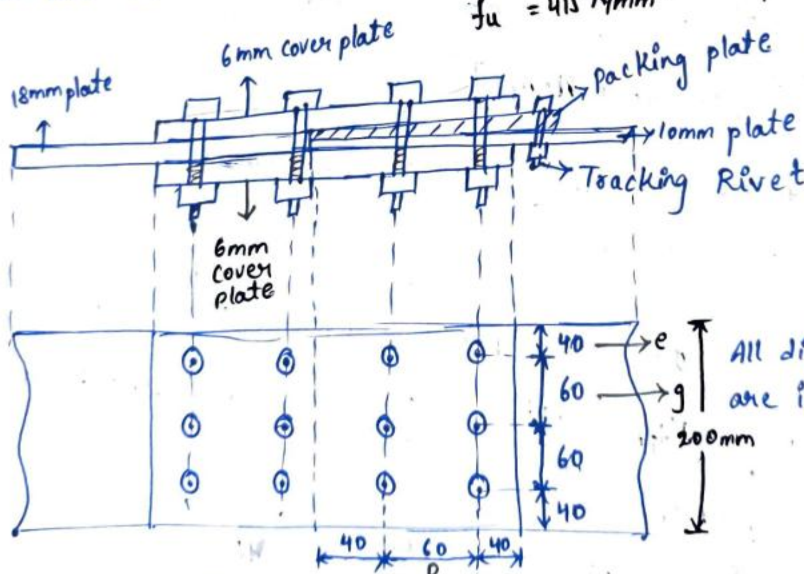
$n_s = 1$

$e = 40\ mm$

$g = 60\ mm$

$P = 60\ mm$

$b = 200\ mm$



All dimensions are in mm

Note: Strength of tracking rivets are not to be considered in the design.

In this problem, connection of packing plate of 8mm thickness is to be used.

Hence they shall be reduction in the shear strength of bolt.

The reduction factor is given by

$\beta_{pk} = 1 - 0.0125\, t_{pp} = 1 - 0.0125 \times 8 = 0.9$

[Pg-75, Cl. 10.3.3.2]

Design of Shearing : $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$

$$\cancel{V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left[n_n A_n + n_s A_s\right] = \frac{400}{\sqrt{3}}\left[1\times\frac{\pi}{4}(20)^2\times 0.78 + 6\times\frac{\pi}{4}(20)^2\right]} \times$$

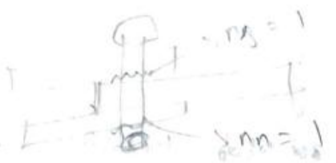
$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left[n_n A_n + n_s A_s\right]\times \beta_{pk}$$

$$= \frac{400}{\sqrt{3}}\left[1\times 0.78\times\frac{\pi}{4}(20)^2 + \frac{\pi}{4}(20)^2\times 1\right]\times 0.9$$

$$= 116.23 \text{ KN}$$

For 6 bolts :

$$V_{dsb} = \frac{116.23}{1.25}\times 6 = 557.90 \text{ KN.}$$



Design of Bearing : $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{124.5}{1.25} = 99.6 \text{ KN}\times 6$
$= 597.6 \text{ KN}$

$$V_{npb} = 2.5\, K_b\, d\, t\, f_u$$

$$K_b = \frac{e}{3d_0} = \frac{40}{3\times 22} = 0.6 \qquad \frac{P}{3d_0} - 0.25 = \frac{60}{3\times 22} - 0.25 = 0.659$$

$$V_{npb} = 2.5\times 0.6\times 20\times 10\times 415 = 124.5 \text{ KN}$$

Design of Tearing : $T_{db} = \frac{T_{nb}}{\gamma_{m0}} = \frac{244.8}{1.25} = 195.84 \text{ KN}$

$$T_{nb} = 0.90\, f_{ub}\, A_n = 0.90\times 400\times 680 = 244.8 \text{ KN}$$

$$A_n = (b - n d_0)\, t = (200 - 6\times 22)\, 10 = 680 \text{mm}^2$$

For 18mm plate :

$$A_n = (b - n d_0)t = (200 - 6\times 22)\, 18 = 1224 \text{mm}^2$$

$$T_{db} = \frac{0.90\times 400\times 1224}{1.25} = 352.512 \text{ KN}$$

$$\therefore T_{db} = 195.84 \text{ KN}$$

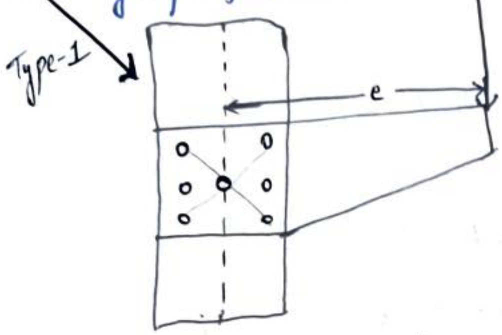
Eccentric Bolted Connections :

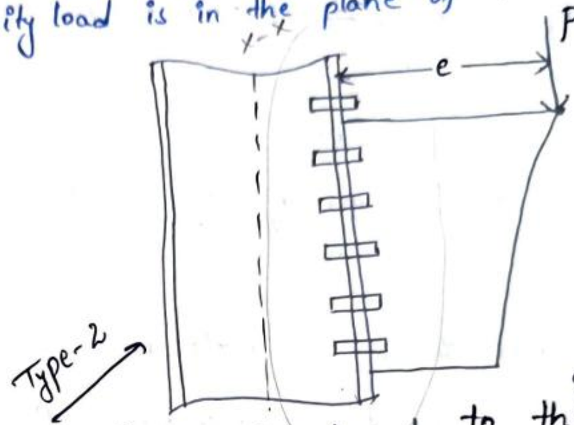
Concentrated Load : A load is said to be concentric load, when its line of action passes through the centre of gravity of bolt group or rivet group or weld group.

Eccentric Load : A load which does not pass through COG of bolt / rivet / weld group because of eccentricity additional moment is in the joint.

Types of Eccentric Connections :

Type-1 : Line of action of eccentricity load is in the plane of the group of bolts.





Type-2 : The line of action of eccentricity load is in the ⊥ to the group of plane of the bolts.

Types of Welded Joints :

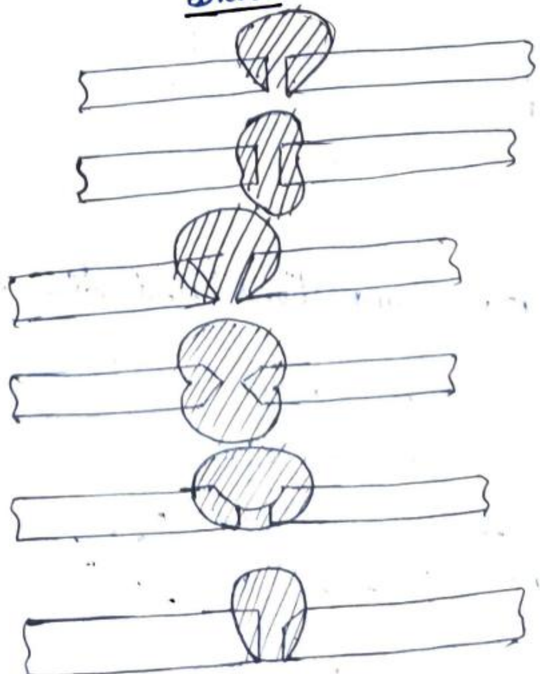
There are 3 types of welded joints

1. Butt Weld 2. Fillet Weld 3. Slot & Plug Weld

Strength of Welds - Butt unfillet welds.

Butt Weld : Also known as groove weld. Depending upon the shape of the groove made for welding welds are classified as below :

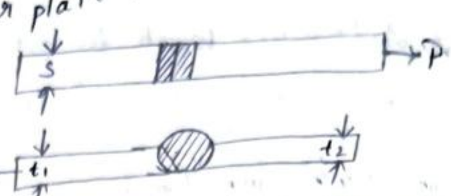
Type of Butt Weld	Sketch
⇒ Square BW on one side	
⇒ Square BW on two sides	
⇒ Single V-Butt joint	
⇒ Double V-Butt joint	
⇒ Single U-Butt joint	
⇒ Single J-Butt joint	



Design of Butt Weld :

* Size of butt weld specified by the effective throat thickness .
* For an incomplete penetration the effective throat thickness would be
 $\frac{5}{8}$ x thickness of the thinner plate .

 Ex : Single U, V, J weld .



* The eff. throat thickness of fully penetrated plates shall be taken as thickness of thinner plate joint .

 Ex : Double U , V , J weld

* Load carried capacity for single U, V, J weld are.

$$\sigma = \frac{Load}{Area}$$

$$Load = \sigma \times Area = \sigma \times b \times t_e$$

$$Load = \sigma \times b \times \frac{5}{8} \times \text{thickness of thinner plat}$$

$$Load = \sigma \times b \times \frac{5}{8} \times t$$

* Design Strength of Butt Weld : The design strength of butt weld in compression or tension member is governed by yielding .

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$f_{wn} = \frac{f_u}{\sqrt{3}} (L_w \times t_e)$$

Where ; f_u = Ultimate strength of weld or parent metal

γ_{mw} = Partial safety Factor for weld [pgno- 30; Table -5]

Shop weld: $\gamma_{mw} = 1.25$

Field Weld: $\gamma_{mw} = 1.5$

Type-1 Eccentric Connections Derivation :

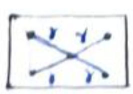
F_1 = Shear due to direct axial load P

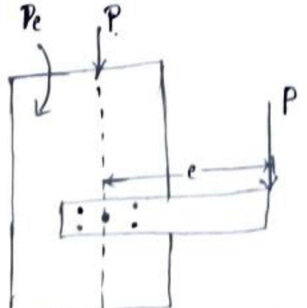
F_2 = Twisting moment due to $P \times e$

$$\boxed{\therefore F_1 = \frac{P}{n}}$$

P → Factored Load

n → No. of bolts





F_2 = It is directly proportional to radial distance (r).

$F_2 \propto r \Rightarrow F_2 = Kr$

Where K = constant

K can be found by equating resisting moment to applied moment

$\therefore F_2 = Pe$

$K r^2 = Pe$

$K \Sigma r^2 = Pe$

$$\boxed{K = \frac{Pe}{\Sigma r^2}}$$

$F_2 = K r$

$$\boxed{F_2 = \frac{Pe}{\Sigma r^2} \times r}$$

$\therefore$ The resultant of F_1 & F_2 act on the bolt. If θ is the angle b/w F_1 & F_2. Then the resultant F is given by

$$F_r = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

For the safety of connections $F_r \leq$ Design strength of bolt, the condition is $\boxed{n' = \sqrt{\frac{6M}{mP \cdot V_{db}}}}$

Where ; n' = No. of bolts required per each vertical line.

M = Twisting Moment

m = No. of bolts lies in vertical.

P = Pitch of the bolt

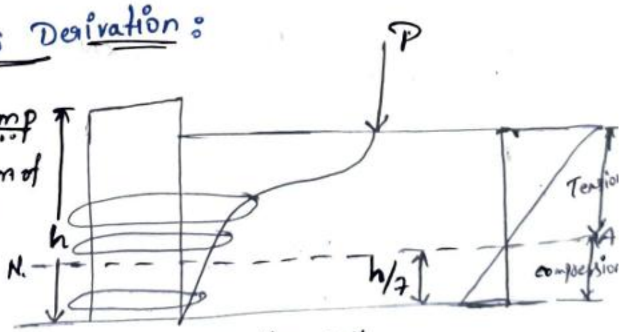
V_{db} = Design strength of the bolt.

Type-2 Eccentric Connections Derivation :

Total Force = Shear, Tension, Comp

Height of NA = $\frac{h}{7}$ (from bottom of angle)

h = Distance from bottom of angle to the center of top bolt.



Section x-x

For safety of connection :- pg no - 76, cl : 10.3.6

$$\left[\frac{V_{sb}}{V_{db}}\right]^2 + \left[\frac{T_b}{T_{db}}\right]^2 \leq 1.0$$

Where ; V_{sb} = Factored shear force acting on the bolt.

V_{db} = Design shear capacity (10.3.2)

T_b = Factored Tensile force acting on the bolt

T_{db} = Design Tension Capacity (10.3.5)

Moment about NA.

$$\boxed{M' = \frac{M}{\left[1 + \frac{2h \Sigma y_i}{3 I \Sigma y_i^2}\right]}}$$

Factored Tensile force acting on bolt

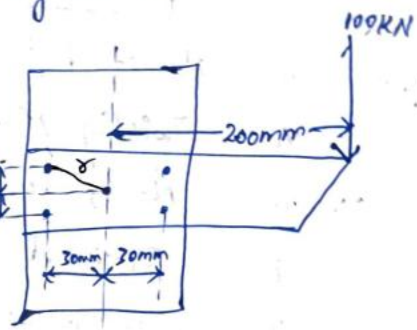
$$T_b = \frac{M' y_n}{\Sigma y_i^2}$$

Where h = Height of angle

y_1, y_2, y_n = Distance of each bolts in tension from axes.

(1) A bracket connection is made with 5 M16 bolts & supports a factored load of $P = 100 KN$ be an eccentricity of 200mm as in fig. The max. resultant force taken up by any bolt is.

Sol: Given Data :



$n = 5$

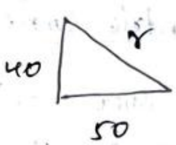
$d = 16mm$

$d_0 = 18mm$

$e = 200mm$

$P = 100KN$

$$F_1 = \frac{P}{n} = \frac{100}{5} = 20KN$$

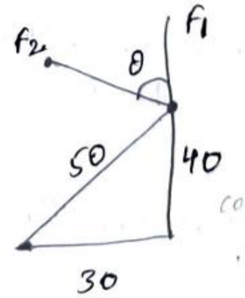


$$F_2 = \frac{Pe}{\Sigma r^2} \times r$$

$$r = \sqrt{40^2 + 50^2} = 50m$$

$$= \frac{100 \times 200}{4(50)^2} \times 50$$

$$= 100 KN$$



$$F_r = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

$$\cos\theta = \frac{adj}{hyp} = \frac{30}{50} = 0.6$$

$$= \sqrt{20^2 + 100^2 + 2(20)(100)(0.6)}$$

$$= 113.13 KN$$

(2) A bracket plate bolt to a vertical column is loaded as shown in fig. If M20 ~~grade of~~ bolts of grade 4.6 are used. Det the max. value of factored load P which can be carried safely.

Given : $d = 20$

$d_0 = \phi + 2 = 20 + 2 = 22\,mm$

Bolts grade 4.6

$f_{ub} = 400\ N/mm^2$

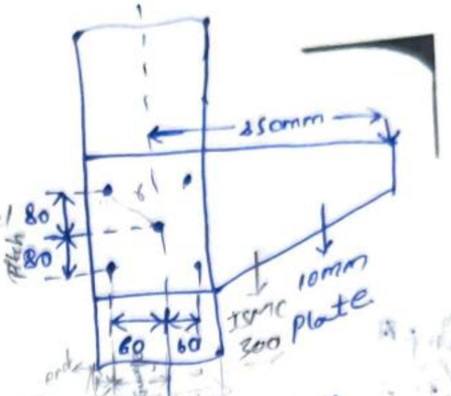
Eccentricity $e = 250\,mm$

$n = 5$

Assume Fe 410 steel

$f_u = 410\,N/mm^2$

$\gamma_{mb} = 1.25$



$n_n = 1 \quad n_s = 0$

Design of shear : $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{56.58}{1.25} = 45.26\,KN$

$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}[n_n A_n + n_s A_s] = \frac{400}{\sqrt{3}}[1 \times 245.04 + 0] = 56.58\,KN$

$A_n = \frac{\pi}{4} d^2 \times 0.78 = \frac{\pi}{4}(20)^2 \times 0.78 = 245.04\,mm^2$

$A_s = \frac{\pi}{4} d^2 = \frac{\pi}{4}(20)^2 = 314.16\,mm^2$

From steel tables
SMC 300
thickness web
$= 7.6\,mm$

Design of Bearing :

$K_b = \frac{e}{3d_0} = \frac{90}{3 \times 22} = 1.36 \qquad \frac{p}{3d_0} - 0.25 = \frac{80}{3 \times 22} - 0.25 = 0.96$

$V_{npb} = 2.5 K_b\, d\, t\, f_u = 2.5 \times 0.96 \times 20 \times 10 \times 410 = 196.8\,KN$

$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{196.8}{1.25} = 157.44\,KN$

$\boxed{V_{db} = 45.26\,KN}$

$F_1 = \frac{P}{n} = \frac{P}{5} = 0.2P$

$F_2 = \frac{P \times e}{\Sigma r^2} \times r$

$= \frac{P \times 250}{4 \times 100^2} \times 100$

Edge =
$150 - 60 = 90$

$= 0.625\,P$

$F_r = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$

$= \sqrt{0.2P^2 + 0.625P^2 + 2 \times 0.2P \times 0.625P \times 0.6}$

$r = 100$

$= P\sqrt{0.2^2 + 0.625^2 + 2 \times 0.2 \times 0.625 \times 0.6}$

$\cos\theta = \frac{60}{100} = 0.6$

$F_r = 0.762\,P$

Equating design strength of weld & resultant force

$V_{dw} = F_R$

$45.26 = 0.762\,P$

$P = 59.40$ KN

$P_u = 59.40 \times 1.5 = 89.11$ KN

Imp ☆

(3) A 18mm thick plate is jointed to a 16mm plate by 200mm long (effective) butt weld. Determine the strength of joint if

i) A double V butt weld is used

ii) A single V butt weld is used.

Assume that Fe 410 grade plates & shop welds are used.

Sol: Given Data:

$t_1 = 18\,mm$

$t_2 = 16\,mm$

Eff. Length, $L_w = 200\,mm$

Fe 410 $\Rightarrow f_u = 410\,N/mm^2$

For shop weld $\Rightarrow \gamma_{mw} = 1.25$ → pg-30

1. Double V butt weld

Eff. Thickness, $t_e = t$

$\therefore t_e = 16\,mm$

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$ [pg-79, cl 10.5.7.1.1]

$$f_{wn} = \frac{f_u}{\sqrt{3}}[L_w \times t_e]$$

$$= \frac{410}{\sqrt{3}}[200 \times 16]$$

$$= 757.48\ KN$$

$$f_{wd} = \frac{757.48}{1.25}$$

$$f_{wd} = 605.98\ KN$$

II. Single V Butt Weld :

Eff. Thickness, $t_e = \frac{5}{8} \times t$

$= \frac{5}{8} \times 16$

$\therefore t_e = 10mm$

Design strength, $f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$

$f_{wn} = \frac{f_u}{\sqrt{3}} [L_w \times t_e]$

$= \frac{410}{\sqrt{3}} [200 \times 10]$

$= 473.427 KN$

$f_{wd} = \frac{473.427}{1.25}$

$f_{wd} = 378.74 KN.$

4.14 pf(2)

(4) Two plates of 16mm & 14mm are to be joined by a ~~group~~ groove weld. The joint is subjected to a factored tensile force of 430 KN. Due to some reasons the eff. length of weld would be provided 175mm only. Check the safety of joint.

1. If single V groove weld is provided

2. If double V groove weld is provided.

Assume Fe 410 plate & shop welds.

$f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u (L_w \times t_e)}{\sqrt{3}\,\gamma_{mw}}$

$= \frac{410 (175 \times 8.75)}{\sqrt{3} \times 1.25}$

$f_{wd} = 289.97 KN < P_u$

Joint isn't safe

Sol: Given: $t_1 = 16mm$

$t_2 = 14mm$

$P_u = 430 KN$

$L_w = 175 mm$

$Fe\,410 \Rightarrow f_u = 410 N/mm^2$

Shop welds $\Rightarrow \gamma_{mw} = 1.25$

1. Single V groove Weld :

Eff. Throat Thickness,

$t_e = \frac{5}{8} \times t$

$= \frac{5}{8} \times 14$

$= 8.75mm$

[For single V groove with shop weld, is taken as $\frac{5}{8}$ of thinner plate thickness (t) when angle is b/w 60 - 90°]

II. Double V groove Weld :

Eff. Throat Thickness, $t_e = 14mm$

$f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u (L_w \times t_e)}{\sqrt{3}\,\gamma_{mw}}$

$= \frac{410 (175 \times 14)}{\sqrt{3} \times 1.25}$

$\therefore f_{wd} = 463.96 KN > P_u$

Safety of joint Check :

$f_{wd} \geq P_u$

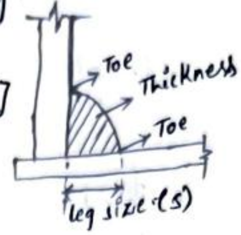
$463.96 > 430$

Joint with a double V groove weld is safe.

Specifications of Fillet Weld :

1. The size of fillet weld shall not be less than 3mm & shall not exceed 0.7t

2. Throat Thickness of fillet welds : The ⊥ distance from the root of the fillet weld to the line of joining its toes.



∴ Eff. Throat Thickness , $t_e = KS$

Where ; K = constant [pg-78, Table-22]

S = Size of weld

3. Eff. Length of fillet weld :

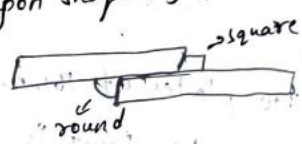
Actual Length = L

Eff. Length , $L_{eff} = L - 2S$

If $L_{eff} > 4S$ } always satisfactory
$L_{act} > 6S$

4. To find the min. size of fillet weld : Pg - 78 , Table - 21 - S_{min}

5. Max. Size of Fillet Weld : S_{max} depends upon shape of weld

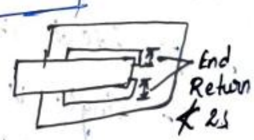


For square weld = $t_{min} - 1.5$ mm

For rounded weld = $3/4\ t_{min}$

Size of Fillet Weld, $S = \dfrac{S_{min} + S_{square} + S_{rounded}}{3}$

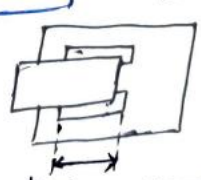
⑦ End Return:



6. Design Strength of Fillet Weld : pg - 79

$f_{wd} = \dfrac{f_{wn}}{\gamma_{mw}}$ → Nominal shear stress
→ Partial safety factor

⑧ For Overlap :

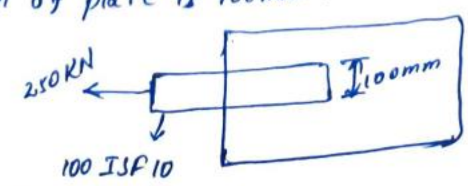


$f_{wn} = \dfrac{f_u}{\sqrt{3}}$

$f_{wd} = \dfrac{f_u}{\sqrt{3}\gamma_{mw}} \times \underbrace{L_w \times t_e}_{\text{Throat Area}}$

overlap ≮ 4t or 40mm

① Determine size, length & overlap b/w the plates of the fillet weld lap connection to transmit a factored load of 250KN as shown in fig. The yield & ultimate tensile strength of weld & steel are 250 Mpa & 410 Mpa respectively. Assume workshop welds & width of plate is 100mm.



WELDED CONNECTIONS :-

Welded connection consists of joining two pieces of metal by establishing a metallurgical bond between them. The elements to be connected are bought closer and the metal is melted by means of electric arc or oxyacetylene flame along with weld rod which adds metal to the joint. After cooling the bond is established between the two elements.

Advantages of welding connections :

1. Due to the absence of gusset plate, connecting angles etc, welded structures are lighter.
2. The absence of making holes for fasteners, makes welding process quicker.
3. Welding is more adaptable than bolting or riveting.
   Ex: even circular tubes can be easily connected by welding
4. It is possible to achieve 100 percent efficiency in the joint whereas in bolted connection it can reach a max of 70-80 percent only.
5. Noise produced in welding process is relatively less.
6. Welded connections have good aesthetic appearance.
7. Welded connections is airtight and watertight.
8. Welded joints are rigid.
9. Alterations in connections can be easily made in the design of welded connections.

Disadvantage of welding connections :

1. Due to uneven heating and cooling, members are likely to distort in the process of welding.
2. There is a greater possibility of brittle fracture in welding
3. A welded joint fails earlier than a bolted joint, if the structure is under fatigue stresses.
4. The inspection of welded joint is difficult and expensive it needs non-destructive testing
5. Highly skilled person is required for welding
6. Proper welding in field connections is difficult.
7. Welded joints are over rigid.

2 Fillet weld :

Fillet weld is a weld approximately triangular cross-section joining two surface approximately right angle to each other in a lap joint, tee Joint or corner joint. fig shows typical fillet welds.

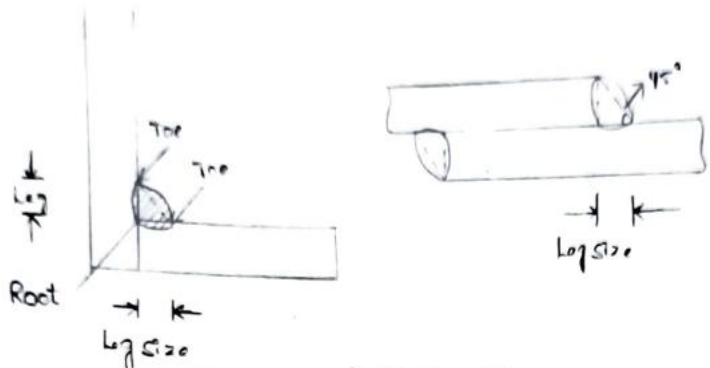
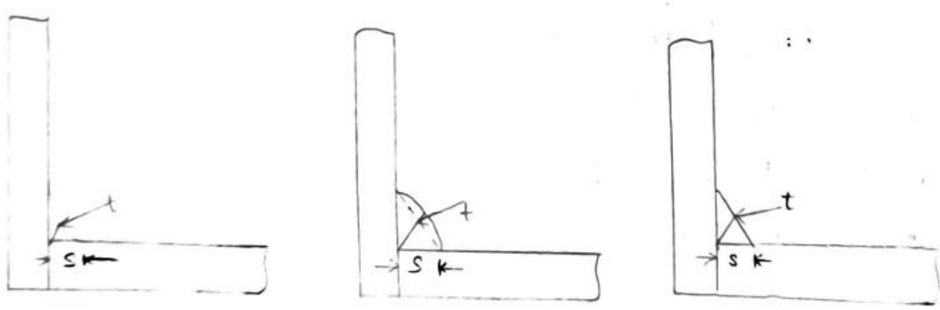


fig: Typical fillet welds.

When the cross-section of fillet weld is e. isoceles triangle with face at 45°, it is known as a standard fillet weld. In special circumstance 60° and 30° angles are also used.

A fillet weld is known as concave fillet weld, convex fillet weld or as mitre fillet weld depending upon the shape of weld face.



(a) concave (b) convex (c) mitre

(s - size) and (t - throat thickness)

fig. Types of fillet welds

Design Procedure of fillet weld :

Sol:

1. Size of fillet weld
2. The size of normal fillet shall be taken as minimum weld leg size.

For deep penetration welds, penetration less than 2.4mm then size of weld = minimum leg size + 2.4mm

3. weld penetration more than 2.4mm then size of weld = minimum leg size + actual penetration.

4. minimum size of fillet weld is 3mm

Note :-

1. If a fillet weld is applied to the edge of a plate then size of fillet weld = edge thickness - 1.5mm

2. If the fillet weld is applied to the rounded toe of a rolled section the size of fillet weld should not exceed 3/4th of the thickness of toe.

Step 02

Effective throat thickness :-

Throat thickness $t_e = k.s$

where ; k = constant value depends upon angle b/w fusion surface

In code book [Pg: 78 Table NO: 22]

Angle b/w fusion faces	60°-90°	91°-100°	101°-106°	107°-113°	114°-120°
Constant (k)	0.70	0.65	0.60	0.55	0.5

"t_e" shall not be less than (3mm) should not exceed 0.7xt.

where "t" is the thickness of thinner plate

Step 03

The actual length of plate or weld = Effective length of weld :-

The effective length should not be less than 4 times of size of weld $l + 4s$

Note : The min lap should be 4-times the thickness of thinner part joint $lap = 4t$

Step 04

Reduction factor : If the length of welded joint is greater than 150mm throughout the thickness a reduction in welding strength to an amount "β_{lw}"

(Pg NO: 79)

$$\beta_{lw} = 1.2 - \frac{0.2 l_j}{150 t} \leq 1.0$$

Problems

1) A Tie member in a truss girder is 250x10mm in it is welded to a gusset plate 10mm thick by a fillet weld the overlap of the member is 200mm the weld is 6mm in size. Find the strength of the joint if the welding is done as shown in fig. what is the increase in the strength if welding is done all around. Assume shop weld.

Specifications of Fillet Weld :

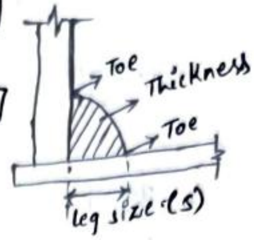
1. The size of fillet weld shall not be less than 3mm & shall not exceed 0.7t

2. Throat Thickness of fillet welds : The ⊥ distance from the root of the fillet weld to the line of joining its toes.

∴ Eff. Throat Thickness, $t_e = KS$

Where ; K = constant [pg-78, Table-22]

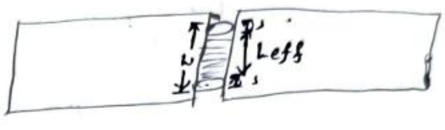
S = Size of weld



3. Eff. Length of fillet weld :

Actual Length = L

Eff. Length, $L_{eff} = L - 2S$



If $L_{eff} > 4S$, $L_{act} > 6S$ } always satisfactory

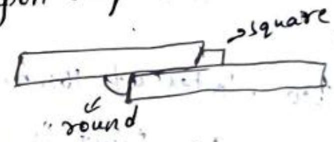
4. To find the min. size of fillet weld : Pg - 78, Table. - 21 - S_{min}

5. Max. Size of Fillet Weld : S_{max} depends upon shape of weld

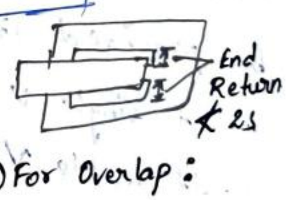
For square weld = t_{min} - 1.5 mm

For rounded weld = $3/4\ t_{min}$

Size of Fillet Weld, $S = \dfrac{S_{min} + S_{square} + S_{rounded}}{3}$



7) End Return:

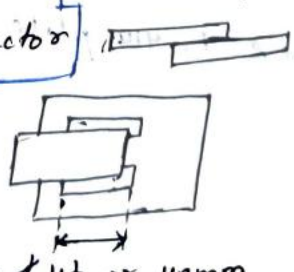


≮ 2S

6. Design Strength of Fillet Weld : Pg - 79.

$f_{wd} = \dfrac{f_{wn}}{\gamma_{mw}}$ f_{wn} → Nominal shear stress, γ_{mw} → Partial safety factor

8) For Overlap :

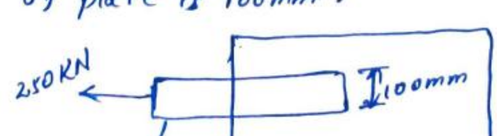


$f_{wn} = \dfrac{f_u}{\sqrt{3}}$.

$f_{wd} = \dfrac{f_u}{\sqrt{3}\gamma_{mw}} \times \underbrace{L_w \times t_e}_{\text{Throat Area}}$

overlap ≮ 4t or 40mm

① Determine size, length & overlap b/w the plates of the fillet weld lap connection to transmit a factored load of 250KN as shown in fig. The yield & ultimate tensile strength of weld & steel are 250 Mpa & 410 Mpa respectively. Assume workshop welds & width of plate is 100mm.



Sol: Given : Factored Load , $P = 250 KN$

$f_y = 250 N/mm^2$

$f_u = 410 N/mm^2$

Assume workshop welds

$\gamma_{mw} = 1.25$

Width of plate = 100mm

Given 100 ISF 10 ; $t = 10 mm$

Minimum size Weld : $S_{min} = 3mm$ [Table - 21, Pg - 78]

Maximum Size of Weld :

i) For Square Weld, $S_{max} = t_{min} - 1.5$

$= 10 - 1.5$

$= 8.5 mm$

ii) For Rounded Weld, $S_{max} = \frac{3}{4} t_{min}$

$= \frac{3}{4} \times 10$

$= 7.5 mm$

Size of Weld, $S = \frac{3 + 8.5 + 7.5}{3} = 6.3 mm$

$\boxed{\therefore S = 6mm}$

Design Strength of fillet weld :

$$f_{wd} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \times L_w \times t_e$$

Eff. Throat Thickness :

$t_w = K \times S = 0.7 \times 6 = 4.2mm$

$$250 \times 10^3 = \frac{410}{\sqrt{3} \times 1.25} \times L_w \times 4.2$$

L_w or L_{eff} = 314.32 mm

According to IS 800:2007, the overlap ≮ 4t or 40mm

$t = 10mm \rightarrow 4t = 40mm$

$$\text{overlap} = \frac{\text{Eff. Length} - \text{Actual length}}{2}$$

$$= \frac{314.32 - 100}{2} = 107.16 mm$$

H.W

(2) A tie member of size 75x8mm is to transmit a factored load of 145 KN. Design a fillet weld & calculate the necessary overlap. Assume shop weld & Fe 410 steel.

Sol: Given : $P_u = 145$ KN

Tie member size $= 75 \times 8$ mm

Fe410 $\Rightarrow f_u = 410$ N/mm²

Shop weld $\Rightarrow \gamma_{mw} = 1.25$

Min. size of fillet weld for 8mm plate is $S_{min} = 3$mm

Max. size of fillet weld: $S_{square} = t_{min} - 1.5 = 8 - 1.5 = 6.5$ mm

$S_{round} = \frac{3}{4} t_{min} = \frac{3}{4} \times 8 = 6$mm

Size of fillet weld, $S = \frac{3+6.5+6}{3} = 5.167 \simeq 5$mm

Design strength of fillet weld :

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$t_w = K.S = 0.7 \times 5 = 3.5 \text{ mm}$$

$$f_{wd} = \frac{f_u \times l_w \times t_w}{\sqrt{3} \times \gamma_{mw}}$$

$$145 \times 10^3 = \frac{410 \times l_w \times 3.5}{\sqrt{3} \times 1.25}$$

$$l_w = \frac{145 \times 10^3 \times \sqrt{3} \times 1.25}{410 \times 3.5}$$

$$l_w = 218.77 \text{ mm}$$

As per IS 800:2007,

overlap $\nless$ 4t or 400 mm

t = 8mm $\Rightarrow$ 4t = 32 mm

$$\text{overlap} = \frac{l_w - l}{2} = \frac{218.77 - 75}{2} = 71.88 \text{ mm}$$

⑧ Design a suitable longitudinal fillet weld to connect the plates as shown in fig to transmit a pull equal to the full strength of small plate. Given plates are 12mm thick. Grade of plates Fe410 & welding to be made workshop.

Sol: $s_{min} = 5mm$

$s_{square} = t_{min} - 1.5 = 12 - 1.5 = 10.5$

$s_{rounded} = \frac{3}{4} t_{min} = \frac{3}{4} \times 12 = 9mm$

$s_{max} = \frac{5 + 10.5 + 9}{3} = 8.16$

$s_{max} \simeq 8mm$

Full strength of Small Plate $= \frac{A_g f_y}{\gamma_{m0}}$

A_g = Area of plate $= 12 \times 100 = 1200 mm^2$ $= \frac{1200 \times 250}{1.1}$

$f_y = 250 Mpa = 250 N/mm^2$

$\gamma_{m0} = 1.1$ $= 272.72 KN$

Eff. Length : $L_{eff} = ?$

Eff. Throat Thickness :

$t_w = KS = 0.7 \times 8 = 5.6 mm$

Design strength of Weld :

$f_{wd} = \frac{f_u}{\sqrt{3}\, \gamma_{mw}} \times t_w \times L_{eff}$ $\quad \frac{f_{wn}}{\gamma_{mw}}$

$272.72 \times 10^3 = \frac{410}{\sqrt{3} \times 1.25} \times 5.6 \times L_{eff}$

$L_{eff} = 257.16 mm$

$L_{eff} = 257 mm.$

Eccentricity of Welded Connections :

① Design the size of weld as shown in fig below. Use Fe410 & shop welds.

☆☆☆ PF③

Mid

... the size of weld as shown in fig. Use Fe410 for shop welds.

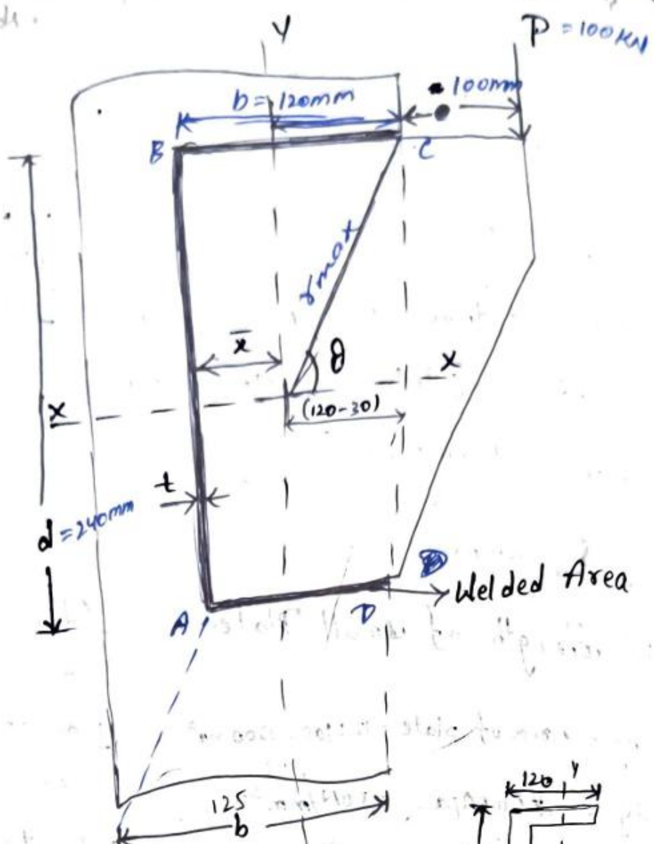
Sol: Given :

$d = 240mm$

$e = 100mm$

$b = 120mm$

$P = 100KN$



Let t be the throat thickness.

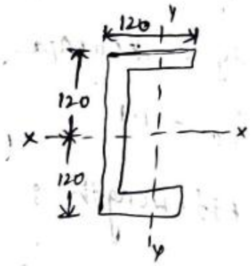
$\bar{x}$ is the distance of COG of weld

$$\bar{x} = \frac{A_1 y_1}{A}$$

Total Area,

$A = 240 \times t + 120 \times t \times 2$

$A = 480t \ mm^2$



$$\bar{x} = \frac{A_1 y_1}{A} = \frac{120 \times t \times 60 \times 2}{480t} = 30mm$$

Moment of Inertia (I_{zz}):

$$I_{zz} = I_{xx} + I_{yy}$$

$$I_{xx} = \frac{bd^3}{12} + Ay^2$$

$$= \frac{t \times 240^3}{12} + (120t \times 120^2) \times 2$$

$$= 4608000 \ t \ mm^4$$

$$I_{yy} = \frac{b^3 d}{12} + Ax^2$$

$$= 2 \times \left[\frac{120^3 \times t}{12} \right] + 240 \times t \times 30^2$$

$$= 720000 \ t \ mm^4$$

$$I_{zz} = I_{xx} + I_{yy} = 5328000 \ t \ mm^4$$

$$r_{max} = \sqrt{p^2 + q^2} = \sqrt{120^2 + 90^2}$$

$$= 150mm$$

$\tan\theta = \frac{opp}{adj} = \frac{120}{90} = 1.33$

$\theta = 53.06°$

Eccentricity, $e = 100 + 120 - 30 = 190mm$

~~Design of Throat thickness:~~

~~Design~~ Direct Shear Stress, $f_1 = \frac{Load}{Area}$

$f_1 = \frac{100 \times 10^3}{480\,t}$

$f_1 = \frac{208.33}{t}\ N/mm^2$

Max. Shear Stress, $F_2 = \frac{P \times e \times r_{max}}{I_{zz}}$

$= \frac{100 \times 10^3 \times 190 \times 150}{5328000\,t}$

$= \frac{534.90}{t}\ N/mm^2$

Resultant Force, $f_r = \sqrt{f_1^2 + f_2^2 + 2f_1f_2\cos\theta}$

$= \sqrt{\left[\frac{208.33}{t}\right]^2 + \left[\frac{534.90}{t}\right]^2 + 2\left[\frac{208.33}{t}\right]\left[\frac{534.90}{t}\right]\cos 53.06}$

$= \frac{1}{t}\sqrt{43.40\times10^3 + 286.11\times10^3 + 133.94\times10^3}$

$= \frac{680.77}{t}\ N/mm^2 \rightarrow (1)$

Design Strengt of Weld:

$f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3}\times\gamma_{mw}} = \frac{410}{\sqrt{3}\times1.25} = 189.37\ N/mm^2 \rightarrow (2)$

Equating (1) & (2)

$\frac{680.77}{t} = 189.37$

$t = 3.594$

size weld $= 0.7t$

$0.7t = 3.594$

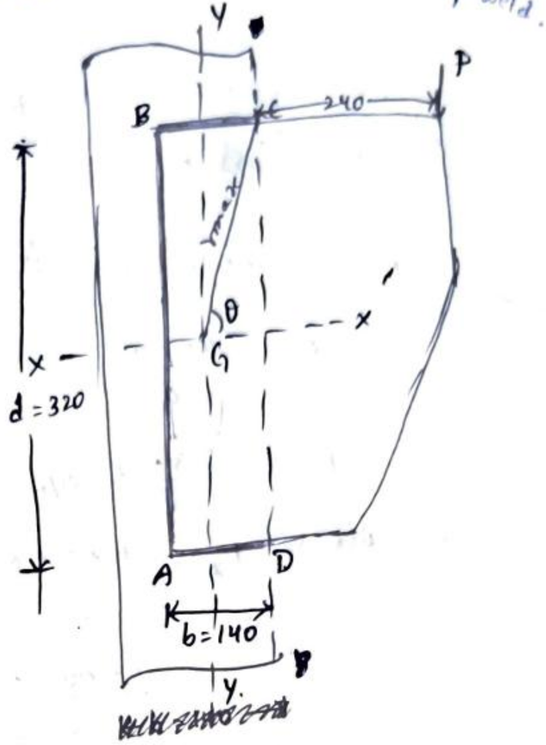
$t = 5.134$

$t \simeq 6mm$

Provide 6mm throat thickness

HW

(2) Det the max load that can be resisted by the bracket as shown in fig. by fillet weld size is 6mm if it is a shop weld.

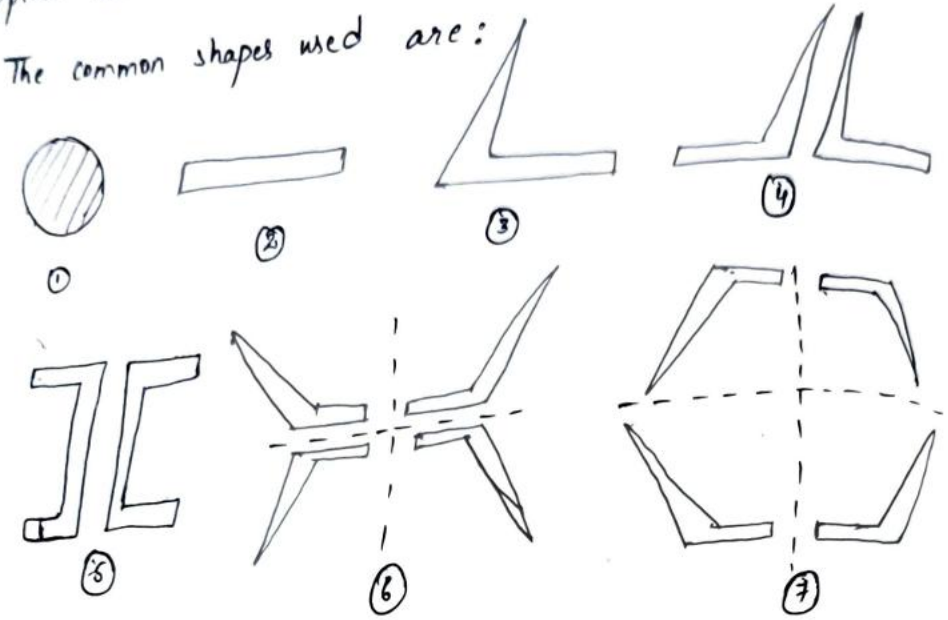


03/01/26

2. Tension Members

Tension Members : A structural member subjected to two pulling forces applied at its ends is called tension members or tie members.

* The common shapes used are :



→ In general, the section should be compact & in order to minimize stress concentration, it should be arranged that has large portion of it as possible is connected to the desert plate.

Design Procedure or Design Strength of a tension member:

* The design strength of tension member as follows : pg-32

1. Design Strength due to yielding of cross-section (T_{dg}):
2. Rupture Strength of critical section (T_{dn})
3. Block Shear (T_{dh})

1. Design Strength due to yielding of cross-section (T_{dg}):

The strength is given by : Pg-32, Cl-6.2

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$$

Where, f_y = yield stress of the material.

A_g = Gross Area of cross section

γ_{m0} = Partial safety factor for failure in tension by yielding Table-5

Design strength due to Rupture of Critical section :

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \Rightarrow \text{for Plates}$$

where ; γ_{m1} = Partial safety factor for failure at ultimate stress.

f_u = Ultimate stress of the material

A_n = Net effective Area of the member

$$A_n = \left[b - n d_h + \sum_i \frac{P_{si}^2}{4 g_i}\right] t \Rightarrow \text{For staggered \& diamond pattern.}$$

where ; b, t = Width & Thickness of plate respectively

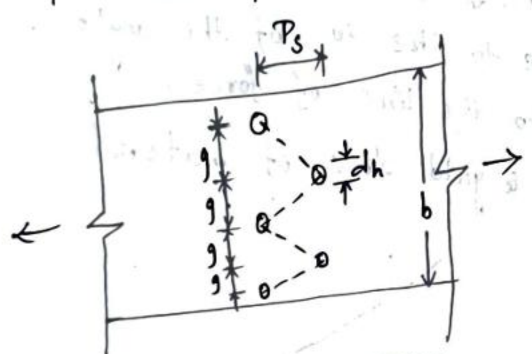
d_h = Dia of bolt hole (2mm in addition to the dia of hole in case the directly punched holes

g = Gauge length b/w the bolt holes as shown in figs

P_s = Staggered pith length b/w line of bolt holes as shown in figs

n = No. of bolt holes in the critical section

i = Subscript for summation of all the inclined legs



⇒ The rupture strength of an angle is

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

Where ; $\beta = 1.4 - 0.076\left[\frac{w}{t}\right]\left[\frac{f_y}{f_u}\right]\left[\frac{b_s}{L_c}\right] \le \left[\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}\right] \ge 0.7$

L_c = Length of the end connection

w = Outstanding leg width

b_s = shear lag width

For preliminary sizing ;

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}$$

Where ; α = 0.6 for 2 bolts

0.7 for 3 bolts

0.8 for 4 & more bolts.

A_n = Net area of total c/s.

A_{nc} = Net area of connected leg

A_{go} = Gross Area of the outstanding leg

t = Thickness of the leg.

3. Design Strength due to Block Shear :

→ Bolted Connections:

$$T_{db_1} = \frac{A_{vg} f_y}{(\sqrt{3}\, \gamma_{m0})} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$ Pg 33 Cl. 6.4.1

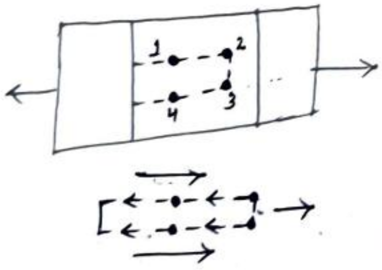
or

$$T_{db_2} = \frac{0.9 A_{vn} f_u}{(\sqrt{3}\, \gamma_{m1})} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

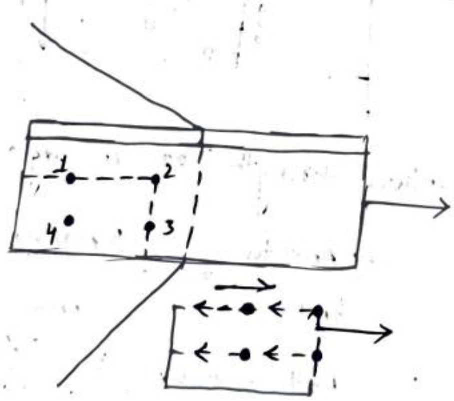
Where ; A_{vg}, A_{vn} = Min Gross & Net Area in shear along bolt line parallel to external force.

A_{tg}, A_{tn} = Min. Gross & Net Area in tension from the bolt hole to the toe of the angle, end bolt line ⊥ to the line of force.

f_u, f_y = Ultimate & yield stress of material.



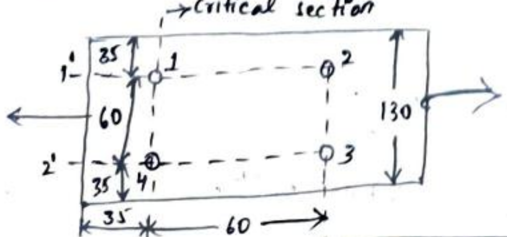
7A Plate



7B Angle

Block Shear Failure.

① Determine the design tensile strength of the plate 130mm x 12mm with the holes for 16mm dia bolts as shown in fig. Steel used is off Fe 410 grade



Q1: **Given Data:** $b = 130\ mm$

$t = 12 mm$

$d = 16 mm$, $d_0 = 18 mm$

Fe410 steel $\Rightarrow f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

$\gamma_{m0} = 1.1$; $\gamma_{m1} = 1.25$

Design strength due to yielding:

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1560 \times 250}{1.1} = 354545.45\ N = 354.54\ KN$$

$A_g = b \times t = 130 \times 12 = 1560\ mm^2$

Design strength due to Rupture:

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 1128 \times 410}{1.25} = 332.98 KN$$

$A_n = (b - n d_0) t = (130 - 2 \times 18) \times 12 = 1128\ mm^2$

Design strength due to Block shear:

$$\left.\begin{aligned} T_{db_1} &= \frac{A_{vg} f_y}{\sqrt{3}\, \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} \\ T_{db_2} &= \frac{0.9 A_{vn} f_u}{\sqrt{3}\, \gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}} \end{aligned}\right\} \text{Whichever is less}$$

$A_{vg} = b \times t = [(35+60) \times 12] \times 2 = 2280\ mm^2$

$A_{vn} = [[(35+60) - 18 - \frac{18}{2}] \times 12] \times 2 = 1632\ mm^2$

$A_{tg} = 60 \times 12 = 720\ mm^2$

$A_{tn} = [60 - \frac{18}{2} - \frac{18}{2}] \times 12 = 504 mm^2$

$$T_{db_1} = \frac{2280 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 504 \times 410}{1.25} = 447.95 KN$$

$$T_{db_2} = \frac{0.9 \times 1632 \times 410}{\sqrt{3} \times 1.25} + \frac{720 \times 250}{1.1} = 449.078 KN$$

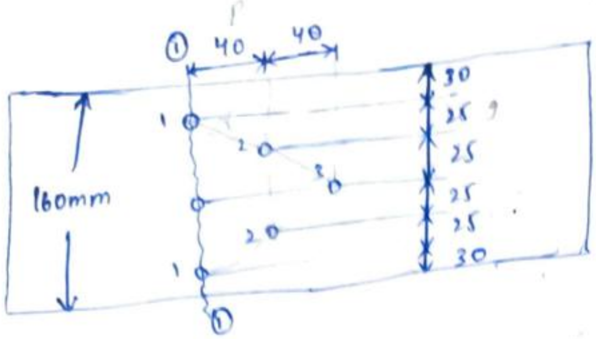
$T_{db} = 441.78\ KN$

$\therefore$ The strength of the plate is $332.98 KN$

H.W

② Determine the design tensile strength of $160 \times 8mm$ plate with the holes for 16mm bolts as shown in fig. Plates are of steel grade Fe410.

Determine the design tensile strength of 160 x 8 mm plate with the holes for 16 mm bolts as shown in fig. plates are of grade Fe410.



Sol: Given Data: $b = 160mm$, Pitch distance, $P = 40mm$

$t = 8mm$, Guage distance, $g = 25mm$

$d = 16mm$, $d_0 = 18mm$

Fe410 steel $\Rightarrow f_u = 410N/mm^2$ → Ult. Tensile strength

$\Rightarrow f_y = 250N/mm^2$ → Yield strength - standard for Fe410

$\gamma_{m0} = 1.1$; $\gamma_{m1} = 1.25$

Design strength due to yielding :

Gross Area of the plate, $A_g = b \times t = 160 \times 8 = 1280 mm^2$

$$T_{dg} = \frac{f_y A_g}{\gamma_{m0}} = \frac{250 \times 1280}{1.1} = 290.909 KN$$

Design strength due to rupture :

$$A_n = \left[b - nd_0 + \Sigma \frac{P_i^2}{4g_i}\right] \times t = \left[160 - 2 \times 18 + \frac{40^2}{4 \times 25}\right] \times 8 = 1120 mm^2$$

$$T_{dn} = \frac{0.9 f_u A_n}{\gamma_{m1}} = \frac{0.9 \times 410 \times 1120}{1.25} = 330.62 KN$$

Imp 17(4)

(3a) A single unequal angle ISA 90 60 @ 6mm is connected to a 10mm gusset plate at the end with 5 nos of 16mm bolts to tranfer tension refer fig below. Determine the tensile strength of the angle if the gusset plate is connected to 90mm leg.

Use Fe410 grade steel.

Given Data: Thickness of leg, $t = 6mm$

" of gusset plate, $= 10mm$

Fe 410 ⇒ $f_u = 410\ N/mm^2$

$f_y = 250\ Mpa$

$\gamma_{mo} = 1.1$

$\gamma_{m1} = 1.25$

Dia of bolts, $d = 16mm$

$d_o = 18mm$

No. of bolts $= 5$

Design strength due to yielding:

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{865 \times 250}{1.1} = 196.59\ KN$$

$A_g = \left[90 - \frac{6}{2}\right]6 + \left[60 - \frac{6}{2}\right]6$
$= 864\ mm^2$

$A_g = 865\ mm^2$ [from pg-13 of steel Tables]
(or)

$A_1 = \left[L_1 - \frac{t}{2}\right] \times t = \left[90 - \frac{6}{2}\right] \times 6 = 522\ mm^2$

$A_2 = \left[L_2 - \frac{t}{2}\right] \times t = \left[60 - \frac{6}{2}\right] \times 6 = 342\ mm^2$

⇒ $A_g = A_1 + A_2$
$= 522 + 342$
$= 864\ mm^2$

Design strength due to Rupture:

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{mo}}$$ [from pg-33, Cl. 6.3.3]

$A_{nc} = \left[90 - \frac{6}{2}\right] \times 6 = 522\ mm^2$

$A_{go} = \left[60 - \frac{6}{2}\right] \times 6 = 342\ mm^2$

$b_s = W + W_i - t$
$= 60 + 50 - 6$
$= 104\ mm$

i) $\beta = 1.4 - 0.076\left[\frac{W}{t}\right]\left[\frac{f_y}{f_u}\right]\left[\frac{b_s}{L_c}\right]$

$= 1.4 - 0.076\left[\frac{60}{6}\right]\left[\frac{250}{410}\right]\left[\frac{104}{200}\right]$

$= 1.159\ mm$

ii) $\beta = \frac{f_u \gamma_{mo}}{f_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.443$

$1.159 < 1.443 > 0.7$

$$T_{dn} = \frac{0.9 \times 522 \times 410}{1.25} + \frac{1.159 \times 342 \times 250}{1.1} = 244.18\ KN$$

Design strength due to Block shear:

$$T_{db_1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$T_{db_2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$A_{vg} = b \times t = (200+30) \times 6 = 1380\ mm^2$ $\quad L_v \times t$

$A_{vn} = \left[(230-(4\times18)) - \frac{18}{2}\right] \times 6 = 894\ mm^2$ $\quad [L_v - (n-1+0.5)d_0]\,t$

$A_{tg} = 40 \times 6 = 240\ mm^2$ $\quad L_t \times t$

$A_{tn} = \left[40 - \frac{18}{2}\right] \times 6 = 186\ mm^2$ $\quad (L_t - 0.5 d_0)\,t$

$$T_{db1} = \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 186 \times 410}{1.25} = 235.98\ KN$$

$$T_{db2} = \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{240 \times 250}{1.1} = 206.91\ KN$$

$\therefore T_{db} = 206.91\ KN$

Thus, when 90 mm leg is connected to gusset, the strength of the plate is the least of T_{dg}, T_{dn}, T_{db}. of the plate is the least of T_{dg}, T_{dn}, T_{db}.

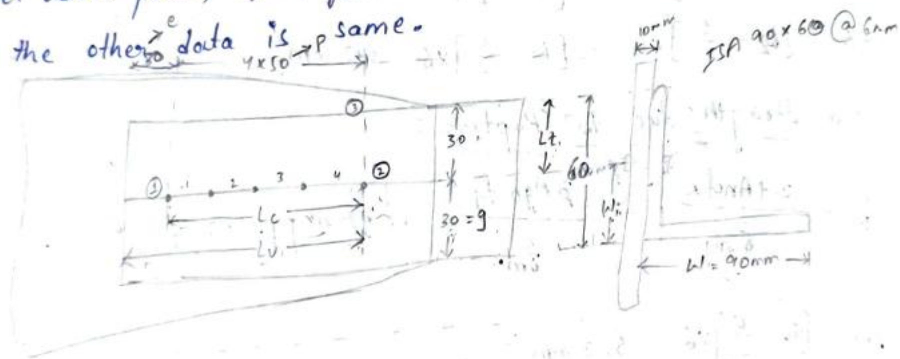
of the plate is 196.59 KN.

H.W $\therefore$ strength of plate is 196.59 KN.

H.W $\therefore$ strength of plate if the gusset is conneted to 60mm plate.

(3b) As per above poblm, if the gusset is conneted to 60mm plate.

★ Imp & all the other data is same.



Given: Thickness of leg, t = 6mm

of gusset plate = 10mm

d = 16 mm ; d_0 = 18 mm

Fe 410 ⟹ f_y = 250Mpa

[Tb-1, pg-14] f_u = 410 Mpa

[Tb-5, pg-30] $\gamma_{m0} = 1.1$

$\gamma_{m1} = 1.25$

No. of bolts = 5

Design strength due to yielding: Cl. 6.2 - pg: 32

$A_g = \left[60 - \frac{6}{2}\right] \times 6 + \left[90 - \frac{6}{2}\right] \times 6 = 864\ mm^2$ or $A_g = 865\ mm^2$ [Steel tables - pg: 13]

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{864 \times 250}{1.1} = 196.36\ KN$$

Design strength due to Rupture : Cl. 6.3.3 - pg: 33

A_{nc} = Net Area of connected leg = $\left[60 - \frac{6}{2}\right] \times 6 = 342\ mm^2$

A_{go} = Gross Area of outstanding leg = $\left[90 - \frac{6}{2}\right] \times 6 = 522\ mm^2$

$\beta = 1.4 - 0.076 \left[\frac{w}{t}\right]\left[\frac{f_y}{f_u}\right]\left[\frac{b_s}{L_c}\right]$

Outstanding leg width = 90 mm

Thickness of angle = 6 mm

$w + w_1 - t = 90 + 30 - 6 = 114$ mm

length of end connection = 4×50 = 200 mm

$\beta = 1.4 - 0.076 \left[\frac{90}{6}\right]\left[\frac{250}{410}\right]\left[\frac{114}{200}\right] = 1.004$

$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$

$= \frac{0.9 \times 342 \times 410}{1.25} + \frac{1.004 \times 522 \times 250}{1.1}$

$= 220.069$ KN

Shear failure occur along 1-2

Tension failure occur along 2-3.

Design strength due to block shear : Cl.6.4.1 - Pg : 33

Remaining Length in tension, $L_t = 60 - 30 = 30$mm

$A_{vg} = L_v \times t = [200 + 30] \times 6 = 1380$ mm^2

or $A_{vn} = [L_v - (n - 0.5) d_0] t$

$A_{vn} = [L_v - (n - 1 + 0.5) d_0] t = [230 - (5 - 1 + 0.5) 18] \times 6 = 894$ mm^2

$A_{tg} = L_t \times t = [60 - 30] \times 6 = 180$ mm^2

$A_{tn} = (L_t - 0.5 d_0) t = [30 - 0.5 \times 18] \times 6 = 126$ mm^2

$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3} \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$

$= \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 126 \times 410}{1.25}$

$= 218.273$ KN

$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3} \gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$

$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{180 \times 250}{1.1}$

$= 193.277$ KN

$\therefore T_{db} = 193.277$ KN

Hence Design Tensile strength, $T_d = 193.277$ KN [Min of T_{dg}, T_{dn}, T_{db}]

(n-1+0.5)

10-1+0.5

9+0.5 9.5

BODMAS

n-1+0.5

n-0.5

10-0.5

9.5

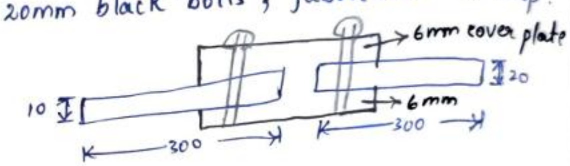
Tension Member Splice : A tension member splice connected to two sections to create a longer member. {Cover plates are called as Splices}.

* If a single piece of required length is not available tension members are spliced to transfer required tension from one piece to another.
* The strength of the splice plates & the bolts or welds connecting them should've strength atleast equal to the design load.
* The design shear capacity of bolts carrying shear through a packing plate in excess of 6mm shall be decreased by a factor [cl 10.3.3.3 from IS 800: 2007] pg - 75

$$\beta_{pk} = 1 - 0.0125\, t_{pk}$$

where t_{pk} = Thickness of the thicker packing in mm

① Design a splice to connect a 300 x 200 mm plate with a 300 x 10mm plate. The design load is 500 KN. Use 20mm black bolts, fabricated in shop.



Sol: Assume a double butt joint to connect the plates.

6mm cover plates.

Given : $d = 20mm$ $\qquad t_{pk} = 10\,mm$

$d_0 = 22mm$ $\qquad n_n = 1$

$n_s = 1$

Fe 410 ⇒ $f_u = 410$ Mpa, $f_y = 250$ Mpa } from Table - 1, pg - 14

$f_{ub} = 400$ Mpa → from Table - 1, pg - 13

Shop weld ⇒ $\gamma_{m0} = 1.1$

$\gamma_{m1} = 1.25$

Design Load = 500KN

Design Strength of bolt : a) shearing:

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} \times \beta_{pk} = \frac{132.37}{1.25} \times 0.88$$

$$= 93.18\ KN$$

$$V_{nsb} = \frac{f_u}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$$

$$= \frac{410}{\sqrt{3}}\left[0.78 \times \frac{\pi}{4}(20^2) + \frac{\pi}{4}(20)^2\right]$$

$$= 132.37\ KN$$

$$\beta_{pk} = 1 - 0.0125\, t_{pk} = 1 - 0.0125 \times 10 = 0.88$$

b) Bearing : $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{123}{1.25} = 98.4\ KN$

Let us assume Edge Distance , $e = 40mm$

Pitch Distance , $p = 60mm$

$$K_b = \frac{e}{3d_0} = \frac{40}{3\times22} = 0.6$$

$$\frac{p}{3d_0} - 0.25 = \frac{60}{3\times22} - 0.25 = 0.66$$

$$\frac{f_{ub}}{f_u} = \frac{410}{410} = 1$$

fub from Table - 1, pg - 14

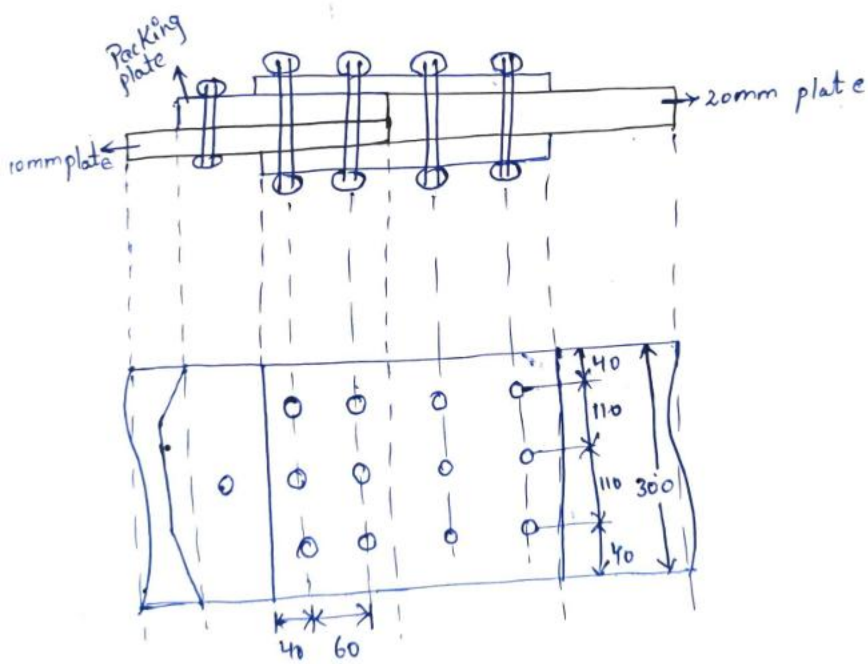
$\therefore K_b = 0.6$

$$V_{npb} = 2.5\ K_b\ d\ t\ f_u = 2.5 \times 0.6 \times 20 \times 10 \times 410 = 123\ KN$$

c) ~~Tearing~~

Design strength of bolt = 93.18 KN

$$\text{No. of bolts} = \frac{\text{Design load}}{\text{strength of bolt}} = \frac{500}{93.18} = 5.36 \simeq 6\text{Nos}$$



Design strength due to yielding : $A_g = b \times t = 300 \times 10 = 3000\ mm^2$

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{(300 \times 10) \times 250}{1.1} = 681.81\ KN$$

Design strength due to rupture :

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 2340 \times 410}{1.25} = 690.7\ KN$$

$A_n = (b - nd_0)\,t$
$= (300 - 3\times22)10$
$= 2340mm^2$

1. Laterally supported beam

2. Laterally unsupported beam

1. Laterally supported beams:

→ Laterally supported beam are mainly subjected to shear and bending beams.

→ beams supported against lateral torsion buckling is known as "laterally supported beam".

→ Laterally supported beam are classified into the following cases:-

* The strength of beam are mainly depends upon the load carrying capacity & moment capacity of beam.

$$\boxed{M_P = f_y \cdot Z_P}$$

where; Z_P = Plastic section

f_y = yield stress

* moment capacity mainly depends upon the plastic section modulus of beams.

"Z_P" is depends upon various crosssections.

Cross-section (Pg:80, table:2)

The various c/c depends upon the width of the thickness ratio of element.

$$\boxed{\frac{b}{t_f} = \frac{d}{t_w}}$$

Class-1 : plastic section

class-2 : compactte section

class-3 : Semi-compact section.

class-4 : Slender section

* Service loads are applied on the beams & applied on beams multiplied with load factor to get factored load.

* Calculate max. bending moment & shear force.

$(Zp)_{req} = \frac{M}{f_y} \times \gamma_{mo}$

(10)

where ; M - max bending moment

⇒ select the section with $Z \geq (Z)_{req}$ (Pg: 138)

⇒ An adopt the selection as been required.

⇒ The classified section checked & it is classified as Plastic compact & semicompact depending upon the width to thickness ratio

Section is checked for shear (Pg: 70)

$V \leq 0.6\ V_d$ for low shear

$V > 0.6\ V_d$ for high shear

The shear force $V_d = \frac{f_y}{\sqrt{3} \cdot \gamma_{mo}} \times h \times t_w$

trail section is checked for design bending strength for low shear.

$V \leq 0.6\ V_d$

$M_d > M$

Where ; $M_d = \beta_b \cdot Z_p \cdot \frac{f_y}{\gamma_{mo}} \leq 1.2\ Z_e \cdot \frac{f_y}{\gamma_{mo}}$ — for simply supporting beam.

(Pg: 53, 70)

$\leq 1.5\ Z_e \cdot \frac{f_y}{\gamma_{mo}}$ — for contineous beam.

$\beta_b = \frac{Z_p}{Z_e}$

for high shear $V > 0.6 V_d$

$M_{dv} \geq M$ for plastic, compact section.

$M_d \geq M$ for semi compact section

M_v, Plastic design moment of whole section.

$$\boxed{M_d = Z_e \cdot \frac{f_y}{\gamma_{mo}}}$$

Z_e = elastic section modulus

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

$M_{fd} = Z_{fd} \times \frac{f_y}{\gamma_{mo}}$

$Z_{fd} = Z_p - A_w \cdot y_w$

A_w = Area of web

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

$\sqrt{3}\ \gamma_{m0}$

A_v = ht_w – for hot rolled section

= dt_w – for welded section

f_y = yield stress of web

γ_{m0} = partial safety factor = 1.1

when $V \ngtr 0.6 V_d$

Design welding stress $M_d = \beta_b \cdot Z_p \dfrac{f_y}{\gamma_{m0}} \leq$

$\leq 1.2 \cdot Z_e \cdot \dfrac{f_y}{\gamma_{m0}}$ – S.S. beam

$\leq 1.5\, Z_e \cdot \dfrac{f_y}{\gamma_{m0}}$ – Cantilever beam.

$\beta_b = 1.0$

for Plastic & compact section. (fy : IS, table – 2)

$\beta_b = \dfrac{Z_e}{Z_p}$ for semi-compact section.

when design shear force $V > 0.6 V_d$

$M_d = M_{dv}$

M_{dv} = Design bending strength under high shear

$= M_d - \beta (M_d \cdot M_{fd}) \leq 1.2 \cdot Z_e \dfrac{f_y}{\gamma_{m0}}$ → for plastic of compact section.

$\beta = \left[2 \cdot \dfrac{V}{V_d} - 1\right]^2$

M_d = plastic design moment of hole section.

M_{fd} = plastic design moment of area of c/s

$M_{fd} = \dfrac{Z_{fd} \cdot f_y}{\gamma_{m0}}$

$Z_{fd} = Z_p - \dfrac{t_w h^2}{4}$

$M_{dv} = \dfrac{Z_e f_y}{\gamma_{m0}}$ = for semi-compact section.

Z_e = elastic section modulus of hole section

eff. of holes in tension zone :

$\dfrac{A_{nf}}{A_{gf}} \geq \dfrac{\left(\dfrac{f_y}{f_u}\right)\left(\dfrac{\gamma_{m1}}{\gamma_{m0}}\right)}{0.9}$

$\dfrac{\gamma_{m1}}{\gamma_{m0}} = \dfrac{\text{ultimate}}{\text{yield}}$

$= \dfrac{1.25}{1.1} = 1.136$

$\dfrac{A_{nf}}{A_{gf}}$ = $\dfrac{\text{Net area of flange (tension}}{}$

Angles : Length of the end connection of a heavy ... maybe reduced by using lug angles as shown in fig. By

Design of laterally unsupported ...

[Code book page No: 54, 8.2.2]

bending strength of laterally unsupported beam

* Design bending strength

$$M_d = \beta_b \cdot Z_p \cdot f_{bd}$$

$\beta_b = 1.0$ for plastic and compact sections

$\beta_b = \frac{Z_e}{Z_p}$ for semi compact sections

f_{bd} = Design bending comp strength.

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

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

$$\chi_{LT} = \frac{1}{\left[\phi_{LT} + (\phi_{LT}^2 - \lambda_{LT}^2)^{0.5}\right]} \le 1.0$$

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

α_{LT} = Imperfection factor.

α_{LT} = 0.21 for rolled steel section.

0.49 for welded sections

* The non-dimensional slenderness Ratio, λ_{LT}, is given by :-

$$\lambda_{LT} = \sqrt{\frac{\beta_b \cdot Z_p \cdot f_y}{M_{cr}}} \le \sqrt{\frac{1.2 \cdot Z_e \cdot f_y}{M_{cr}}} = \sqrt{\frac{f_y}{f_{cr.b}}}$$

M_{cr} = elastic critical moment calculated as ;

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

$\beta_b \times Z$: t_f = thickness of flange.

where,

I_t : torsional constant = $\frac{\Sigma b_i t_i^3}{3}$ for open section.

I_w = warping constant

I_y, r_y = moment of inertia and radius of gyration, respectively about the weaker axis

L_{LT}, r_y = effective length for lateral torsional buckling.

h_f = centre to centre d/s b/w flanges.

31/01/2

Internal shear stress :

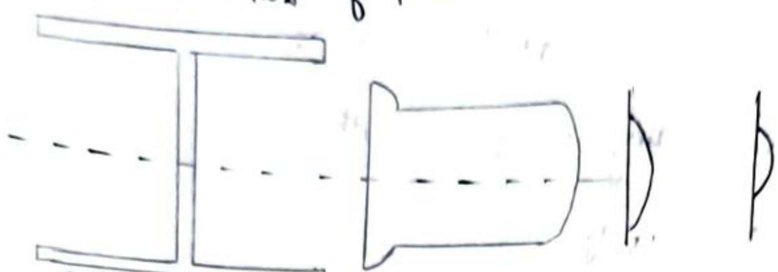
$$\tau = \frac{VQ}{I_z \cdot t}$$

V = shear force

Q = A·ȳ static moment of c/s

I_z = moment of intertia.

t = thickness of portion at which 'τ' is calculated.



The factored design shear force. $V < V_d$.

where ; $V_d = \frac{V_n}{\gamma_{m0}}$

$V_n = V_p \rightarrow$ Pure shear

$V_p = \frac{A_v \cdot f_{y \cdot w}}{\sqrt{3}}$

where : A_v = shear area.

$f_{y \cdot w}$ = yield strength of web.

Shear area for I-channel :-

1) Major axis bending :

Hot rolled section = $b \cdot t_w$

welded section = $d \cdot t_w$.

2) Minor axis bending :

Hot rolled (or) welded section equal to = $2 \cdot b \cdot t_f$

For rectangular :

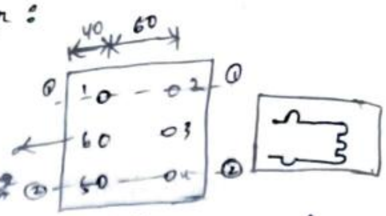
loaded parallel to depth $b = \frac{Ah}{b+h}$

loaded parallel to width $b = \frac{Ab}{b+h}$

for circular hallow section = $\frac{2A}{\pi}$

for plates & solids = A

Design strength due to block shear :



$A_{vg} = b \times t = [(40+60)\times 10]\times 2 = 2000 m^2$

$A_{vn} = [(e+p) - 1.5 d_0) t] = [[(40+60) - 1.5\times 22]\times 10]\times 2$
$= 3340 mm^2$

$A_{tg} = (110+110)\times 10 = 2200 mm^2$

$A_{tn} = [220 - \frac{22}{2} - \frac{22}{2} - \frac{22}{2}]10 = 1870 mm^2$

Shear failure occurs in 1, 2, 5, 4
Tension failure occurs in 2, 3, 4

$T_{db_1} = \frac{2000 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 1870 \times 410}{1.25} = 814.45 KN$

$T_{db_2} = \frac{0.9 \times 3340 \times 410}{\sqrt{3} \times 1.25} + \frac{2200 \times 250}{1.1} = 106.92 KN$

Along ① - ② - ③ - ④ - ⑥

$A_{vg} = (40+60) \times 10 = 1000$

$A_{vn} = [(40+60) - 1.5 \times 22] \times 10 = 670 mm^2$

$A_{tg} = (110+110+40) \times 10 = 2600 mm^2$

$A_{tn} = [260 - \frac{22}{2} - \frac{22}{2} - \frac{22}{2}] \times 10 = 2270 mm^2$

$T_{db_1} = \frac{1000 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 2270 \times 410}{1.25} = 801.319 KN > 500 KN$

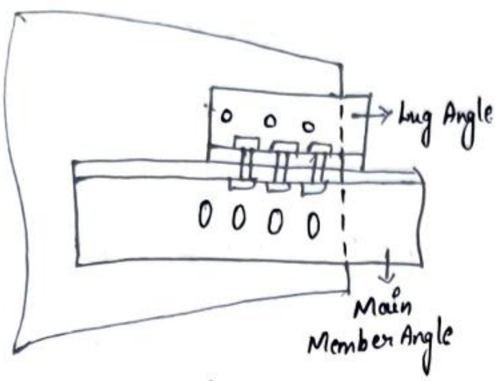
$T_{db_2} = \frac{0.9 \times 670 \times 410}{\sqrt{3} \times 1.25} + \frac{2600 \times 250}{1.1} = 705.09 > 500 KN$

Hence, Design of block shear $= 705.09 KN$.

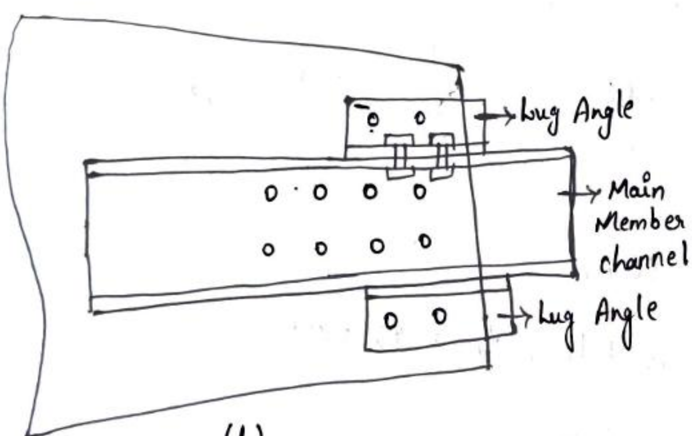
11/01/26

Lug Angles : Length of the end connection of a heavily loaded tension member maybe reduced by using lug angles as shown in fig. By using lug angles there will be saving in gusset plate, but it is upset by additional fasteners & angle required. Hence now-a-days it is not preferred. IS 800:2007 specifications for lug angles are [Cl 10.12]

Pg. 83, Cl.10.12



(a)



(b)

PP⑤

1. A tension member of a roof truss carries a factored axial tension load of 430 KN. Design the section & connection : a) Without using lug angle b) Using Lug Angle.

Sol: Given : Factored Load, P_u = 430 KN

Considering the strength in b, gross area required is given by :

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

Assume Fe410 grade ⇒ f_y = 250 Mpa ; f_u = 410 Mpa

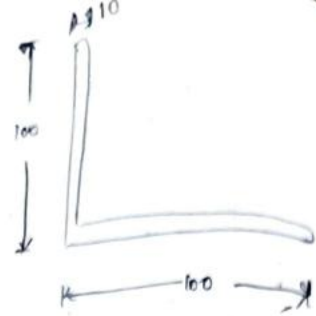
γ_{mo} = 1.1

$$430 \times 10^3 = \frac{A_g \times 250}{1.1}$$

$$A_g = 1892 \text{ mm}^2$$

Select ISA 100 x 100 x 10 mm

$A_g = 1903 \text{ mm}^2$



Use 20mm dia bolts.

$d = 20 \text{ mm} \quad ; \quad d_0 = 22\text{mm}$

Strength in Single Shear,

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_u[n_n A_{nb} + n_s A_{sb}]}{\sqrt{3} \cdot \gamma_{mb}} = \frac{410\left[0.78 \times \frac{\pi}{4}(20)^2 + 0\right]}{\sqrt{3} \times 1.25}$$

$V_{dsb} = 46.40 \text{ KN}$

Strength in Bearing,

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5\, K_b\, d\, t\, f_u}{\gamma_{mb}} = \frac{2.5 \times 0.45 \times 20 \times 10 \times 410}{1.25} = 73.8 \text{ KN}$$

K_b: $e = 1.5\, d_0 = 1.5 \times 22 = 33 \text{ mm}$

$P = 2.5\, \phi = 2.5 \times 20 = 50\text{mm}$

$$K_b = \frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.45 \qquad = \frac{P}{3d_0} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$$

$K_b = 0.45$

Let $e = 30\text{mm}$, $P = 50 \text{ mm}$

∴ The design strength of bolt, $V_d = 46.40\text{KN}$

a. Connection without Lug Angle:

No. of bolts required, $n = \frac{430}{46.40} = 9.26 \simeq 10$

Length of end connection, $L_c = n \times P = 9 \times 50 = 450 \text{ mm}$

From Cl.10.3.3.1, $15d = 15 \times 20 = 300$ {Pg - 75}

$L_c > 15d$

Then, $\beta_{lj} = 1.075 - 0.005\left[\frac{L_c}{d}\right]$

$= 1.075 - 0.005\left[\frac{450}{20}\right]$

$\beta_{lj} = 0.9625$

∴ Design of shear strength $= \beta_{lj} \times V_{db}$

$= 0.9625 \times 46.40$

$= 44.66 \text{ KN}$

No. of bolts $= \dfrac{430}{44.66} = 9.62 \simeq 10 \rightarrow$ Check OK.

Hence 10 bolts are sufficient.

Strength of yielding;

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1903 \times 250}{1.1} = \cancel{48/88}\ 432.5\ KN > 430\ KN$$

Hence OK.

Design strength due to rupture;

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}} = \frac{0.9 \times 730 \times 410}{1.25} + \frac{1.255 \times 950 \times 250}{1.1}$$

$$\boxed{T_{dn} = 486.46\ KN}$$

$$A_{nc} = \left[\left(100 - \frac{10}{2}\right) - 22\right] \times 10 = 730\ mm^2$$

$$A_{go} = b \times t = \left[100 - \frac{10}{2}\right] \times 10 = 950\ mm^2$$

$$b_s = w + w_i - t = 100 + 50 - 10 = 140$$

$$\beta = 1.4 - 0.076 \left[\frac{w}{t}\right]\left[\frac{f_y}{f_u}\right]\left[\frac{b_s}{L_c}\right]$$

$$= 1.4 - 0.076 \left[\frac{100}{10}\right]\left[\frac{250}{410}\right]\left[\frac{140}{450}\right]$$

$$= 1.255$$

Design strength due to block shear:

$$T_{db_1} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} \qquad T_{db_2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

Tearing length in tension, $L_t = 100 - 50 = 50\ mm$

$$A_{vg} = L_v \times t = [450 + 30] \times 10 = 4800\ mm^2$$

$$A_{vn} = [L_v - (n - 0.5) d_0]\, t = [480 - (10 - 0.5) \times 22] \times 10 = 2710\ mm^2$$

$$A_{tg} = L_t \times t = [100 - 30] \times 10 = 700\ mm^2$$

$$A_{tn} = (L_t - 0.5 d_0)\, t = [70 - 0.5 \times 22] \times 10 = 590\ mm^2$$

$T_{db_1} = 804\ KN$, $T_{db_2} = 620.96\ KN$

$\therefore T_{db} = 620.96\ KN$

Strength of the angle $= 432.5\ KN$

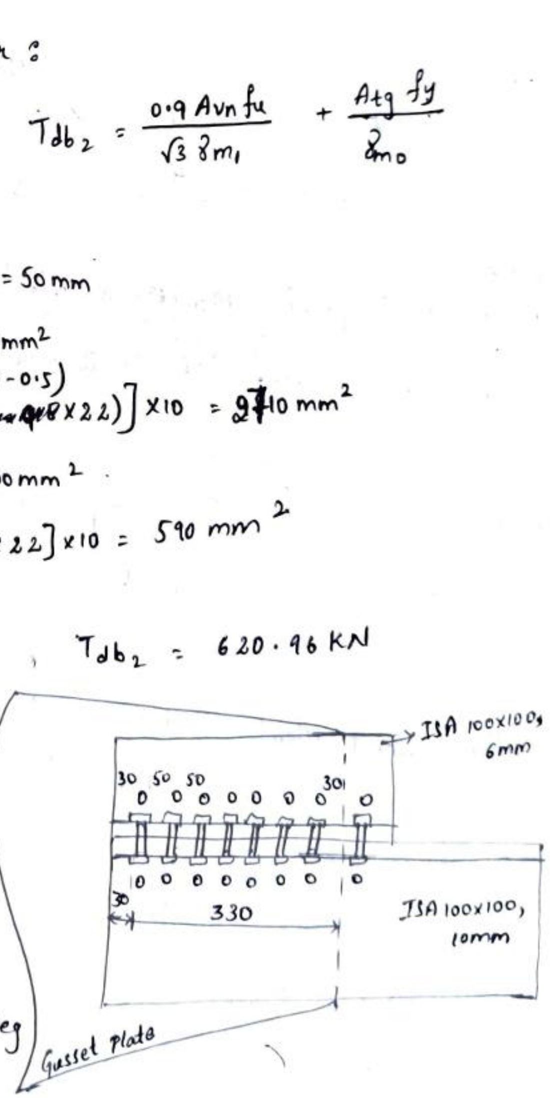
b. Connection With Lug Angle:

Given Load $= 430\ KN$

Load is distributed equally

Load in outstand leg = Load in conn leg

$$= \frac{430}{2} = 215\ KN$$



Lug Angle is to be designed to take a load of $1.2 \times 215 = 258\,kN$

Yielding : $T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{A_g \times 250}{1.1}$

$$\frac{258 \times 10^3 \times 1.1}{250} = A_g$$

$$A_g = 1135\ mm^2$$

Provide ISA $100 \times 100 \times 6mm$

Rupture : $T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 1008 \times 410}{1.25} = 297.56\,kN > 258$

$A_n = [100 + 100 - 10 - 22] \times 6 = 1008\ mm^2$

strength in single shear, $V_{dsb} = 46.40\ kN$

strength in bearing, $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 K_b d t f_u}{\gamma_{mb}}$

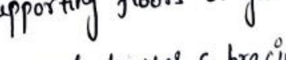
$$= \frac{2.5 \times 0.45 \times 20 \times 6 \times 410}{1.25}$$

$$= 44.28\ kN$$

$\therefore$ Bolt value $= 44.28\ kN$

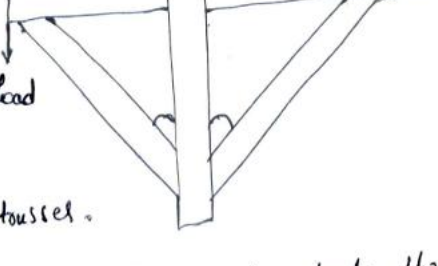
No. of bolts $= \frac{258}{44.28} = 5.82 \simeq 6$

★★★ **Design of Compression Members or Struts** : A comp member is a structural member which is ~~connected~~ subjected to 2 equal, opposite compressive forces applied at its ends.



Column : It is a vertical compressing member supporting floors or ~~gutt~~ girders.

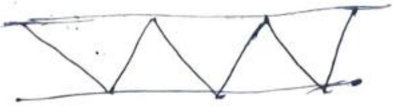
Strut : It is a compression member used in the roof trusses & bracing. It maybe continuous or discontinuous.



Continuous Strut Member : It is a compression member which is continous over a no. of joints.

Eg : Top chord members in the roof trusses.

Discontinuous Strut Member : It is a comp member which extends b/w 2 adjacent joints.



The tendency of a comp member to buckle is usually measured by its slenderness ratio.

$$\boxed{\lambda = \frac{KL}{r_{min}}}$$

Where ; KL - Eff. Length of member [pg-45, Table-11]

r_{min} - Radius of gyration about minor axis - $\sqrt{\frac{I}{A}}$ → MOI

Design of Axial Loaded Compression Member : pg. 34 ; Section - 7

Design Comp strength of member, $P_d = A_e f_{cd}$

Where ; A_e - Eff. section Area

f_{cd} = Design Comp stress = $\dfrac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$

$\phi = 0.5 [1 + \alpha [\lambda - 0.2] + \lambda^2]$

Design of single angle strut : pg - 47, 48 ; Cl. 7.5

Angle strut ; $\lambda_e = \sqrt{K_1 + K_2 \lambda_{vv}^2 + K_3 \lambda_\phi^2}$

① A single angle member is discontinuous ISA 110 x 110 x 10mm with single bolted connection is 2.5m long. Calculate the design compression strength of the section. f_y = 250 Mpa.

Sol: Given : ISA 110x110x10 mm

1 Bolt connection

f_y = 250 Mpa

L = 2.5m = 2500m

Use f_e = 410 N/mm²

P_d = ?

$P_d = A_e \cdot f_{cd}$ pg - 34

A_e = 2106 mm² [pg - 9]

$f_{cd} = \dfrac{f_y/\gamma_{m0}}{\phi[\phi^2 - \lambda^2]^{0.5}}$ pg - 34

$\lambda = \sqrt{K_1 + K_2 \lambda_{vv}^2 + K_3 \lambda_\phi^2}$

pg - 48

$$\lambda_{vv} = \frac{\dfrac{l}{r_{vv}}}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}}$$ pg - 48

From steel tables
pg - 10

$r_{vv} = 2.14\ cm = 21.4\,mm$

$r_{min} = 21.4\,mm$

$$= \frac{\dfrac{2500}{21.4}}{\left[\dfrac{250}{f_y}\right]^{0.5}\sqrt{\dfrac{\pi^2 \times 2\times10^5}{250}}}$$

$\lambda_{vv} = 1.314$

$$\lambda_{\phi} = \frac{\dfrac{b_1 + b_2}{2t}}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}}$$ pg - 48

$$= \frac{\dfrac{110+110}{10}}{\left[\dfrac{250}{250}\right]^{0.5}\sqrt{\dfrac{\pi^2 \times 2\times10^5}{250}}}$$

$\lambda_{\phi} = \cancel{0.247}\ 0.123$

From Table-10, pg-35

Assume No. of bolts @ each end connection = 1, Take $\alpha = 0.49$

Gusset member fixity = Hinged

[Buckling class - c]

$K_1 = 1.25$; $K_2 = 0.5$; $K_3 = 60$

$$\lambda = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_{\phi}^2}$$

$$= \sqrt{1.25 + 0.5(1.314)^2 + 60(\cancel{0.247}\ 0.123)^2}$$

$\boxed{\lambda = 1.744}$

$\phi = 0.5\left[1 + \alpha[\lambda - 0.2] + \lambda^2\right]$

$= 0.5\left[1 + 0.49[1.74 - 0.2] + 1.74^2\right]$

$= \cancel{3.761}\ 2.391$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \frac{250/1.1}{2.391 + [2.391^2 - 1.744^2]^{0.5}}$$

$\boxed{f_{cd} = 56.38\ N/mm^2}$

$P_d = A_e \times f_{cd} = 2106 \times 56.38$

$\boxed{P_d = 118.736\ KN/mm^2}$

$\therefore P_d = 118.74\ KN/mm^2$

5) A single angle discontinuous member ISA 100×100×10mm with two bolted connection is 3m long. Calc the design compression strength of the angle.

Sol: $\lambda_{vv} = \dfrac{\frac{l}{r_{vv}}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$ $\qquad r_{vv} = 19.4\,mm$ $\qquad \lambda_{\phi} = \dfrac{\frac{b_1+b_2}{2t}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$

$$= \dfrac{\frac{3000}{19.4}}{\left[\frac{250}{250}\right]^{0.5}\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}} \qquad\qquad = \dfrac{\frac{100+100}{2\times 10}}{1\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}}$$

$$= 1.740 \qquad\qquad = 0.112$$

No. of bolts = 2 ; Gusset Member fixity = Hinged

$K_1 = 0.70$, $K_2 = 0.60$, $K_3 = 5$

$$\lambda = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_{\phi}^2}$$

$$= \sqrt{0.7 + 0.6(1.74)^2 + 5(0.112)^2}$$

$$= 1.606$$

From Table -10, $\alpha = 0.49$ [Buckling class - c] pg-35

$$\phi = 0.5\left[1+\alpha(\lambda - 0.2) + \lambda^2\right]$$

$$= 0.5\left[1 + 0.49(1.606 - 0.2) + 1.606^2\right]$$

$$= 2.134$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \frac{250/1.1}{2.134 + [2.134^2 - 1.606^2]^{0.5}} = 64.21\ N/mm^2$$

$$P_d = A_e \times f_{cd}$$

$$= 1903 \times 64.21$$

$$= 122.19\ KN/mm^2$$

Type	Section	Effective Length: In place of gusset	Effective Length: ⊥ to gusset
① Cont angles top or bottom chord of truss	Single or double angle	0.7L to 1.0L	1.0L
② Discontinuous	Single angle connected with 1 bolt	1.0L	1.0L
	Single angle connected with > 1 bolt or equivalent weld	0.85 L	1.0L
③ Discontinuos	Double angle placed on either side of the gusset plate	0.70L to 0.85L	1.0 L
	Double angle placed on same side of the gusset plate	0.70L to 0.85L	1.0L

Design of Double Angle Struts :

⇒ There are too ways 2 calc fcd : 1. Using formulae
2. Using Tables - 9a, b, c, d.
[pg:- 40, 41, 42, 43]

① Calc the strength of a discontinuous strut of length 3.5 m, the strut consists of 2 unequal angles 150 x 75 x10mm. fy = 250 Mpa with long legs connected & placed. Use 10mm thick gusset plate.
a. On either sides of gusset plate b. On the same side of a gusset plate.

Sol: Given : L = 3.5 m = 3500mm

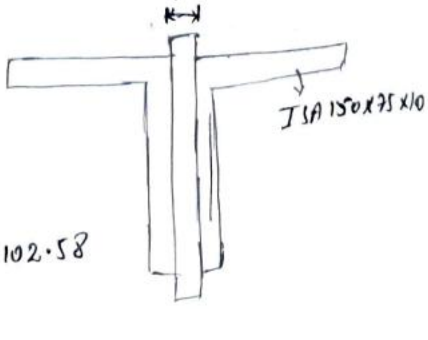
2 ISA 150 x 75 x 10 mm

$f_y = 250$ Mpa

Gusset plate t = 10 mm

Net Area, Ae = ~~2x2156~~ 4312 mm² [pg - 25 ; Steel Tables]

a. On either sides of gusset plate :



$KL = 0.85\,L = 0.85 \times 3500 = 2975\,mm$

$r_{min} = 2.9\,cm = 29.0\,mm$

Slenderness Ratio $= \frac{KL}{r_{min}} = \frac{2975}{29} = 102.58$

from IS 800 : 2007 ; Table 9(c), pg- 43

KL/r	fy
100	107
102.58	? y
110	94.6

$$y = \cancel{\frac{(x-x_1)(y_1-y_2)}{(x_2-x_1)}} \qquad y_1 + \frac{y_2-y_1}{x_2-x_1}(x-x_1)$$

$$= \cancel{\frac{(102.58-100)(107-94.6)}{(110-100)}}$$

$$f_{cd} = 103.80\ N/mm^2$$

$P_d = A_e \times f_{cd} = 4312 \times 103.80 = 447.58\ KN/mm^2$

b. On same side of the gusset plate :

$r_{vv} = 1.59\ cm = 15.9\ mm$ [Tb-4 , pg-13 from ST]

$$\frac{KL}{r_{min}} = \frac{2975}{15.9} = 187.10$$

KL/r	fy
180	43.6
187.10	-
190	39.7

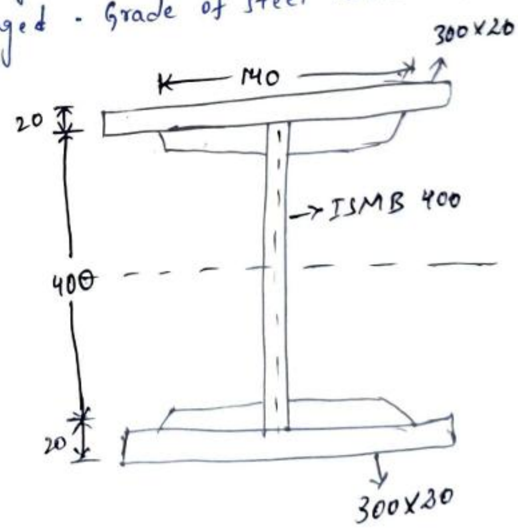
$$y = y_1 + \frac{y_2-y_1}{x_2-x_1}(x-x_1)$$

$$= 40.83\ N/mm^2$$

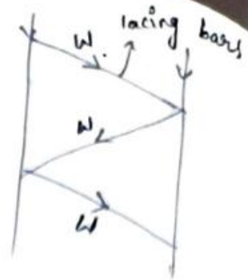
$$f_{cd} = 40.83\ N/mm^2$$

$P_d = 4312 \times 40.83 = 176.06\ KN/mm^2$

(2) Determine the load carrying capacity of the column section as shown in fig. if its actual length is 4.5 m.. Its one end may be assumed fixed & other end hinged. Grade of steel is Fe 415 (i.e., E = 250 Mpa)



Design procedure for Lacing Columns :



* Assuming f_{cd} b/w 110 - 150 Mpa
2. Calc area required . $P_d = A_e \times f_{cd}$
3. Selection of suitable section
4. select buckling class - C for any builtup section .
5. Find out slenderness ratio ; $\lambda = \frac{KL}{r} \times 1.05$

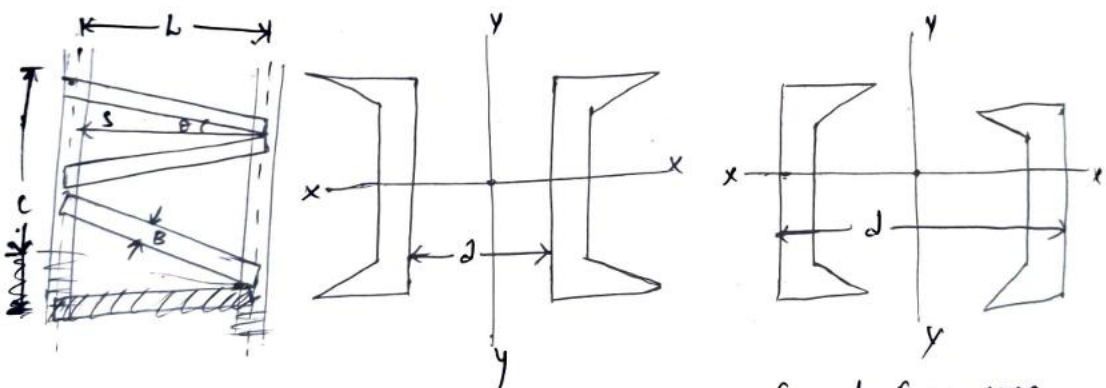
The eff. slenderness ratio of laced column should be ↑ed by 5%. to take care of shear effect.

6. Using λ, calc f_{cd} by interpolation use table - 9c of IS 800:2007, pg-42
7. Design of comp strength , $P_d = A_e \cdot f_{cd} > P$
8. Design of lacing bar procedure :

a. Clear spacing b/w channels (s)

if $s = d + 2g$ is bact to back connection

if $s = d - 2g$ is face to face conn



Back to Back conn

Face to Face conn

* Using parallel axis theorem $I_{yy} = I_{xx}$

If back to back $\Rightarrow 2\left[I_{yy} + A\left[\frac{d}{2} + C_{yy}\right]^2 \right] = 2 I_{xx}$

If face to face $\Rightarrow 2\left[I_{yy} + A\left[\frac{d}{2} - C_{yy}\right]^2 \right] = 2 I_{xx}$

b. Inclination Angle, $\theta = 40 - 70°$ (mostly 45°)

c. Vertical spancing, $C \Rightarrow \tan\theta = \frac{\frac{C}{2}}{S}$

$C = 2S\tan\theta$

d. Length of lacing Bar (L) : $L\cos\theta = S \Rightarrow L = \frac{S}{\cos\theta}$

e. Thickness of lacing Bar (T) : $T = L/40$ for single lacing

$T = L/60$ for double lacing

f. Width of lacing Bar (B) : $B = 3D \rightarrow$ Dia of bolt [16, 18, 20, 22, 24]

g. Area of lacing bar, $A = B \times T$

h. Radius of gyration of lacing bar, $R = T/\sqrt{12}$

i. Calc $\lambda = \frac{L}{R} > 145$ using λ calc fcd by interpolation, use table - 9(c).

j. Calc design comp stg of lacing bar. $P_L = A_e \times f_{cd}$.

$P_L > V \rightarrow$ Transverse shear i.e., resisted by lacing system.

$V = \frac{V_t}{2} \rightarrow 2.5\%$ of P_d.

① Design a built-up column 10m long to carry factored axial load of 1080KN. The column is restrained in position but not in direction at both the ends. Design the column with 2 channels placed back to back. Provide single lacing system with bolted conn. Assume Fe410 & bolt grade 4.6.

Sol: Given : Length of column, L = 10m = 10000mm

Factored load P = 1080KN

Both ends are hinged.

$KL = 1.0L = 1 \times 10000 = 10000mm$

Fe 410 $\Rightarrow f_u = 410$ Mpa, $f_y = 250$Mpa, $f_{ub} = 400$ Mpa

Bolt grade 4.6

1. Assume f_{cd} = 150 Mpa

2. Area required, $A_e = \frac{P_d}{f_{cd}} = \frac{1080 \times 10^3}{150} = 7200 mm^2$

for each channel, $A_e = \frac{7200}{2} = 3600 mm^2$

Note : For column, 1st preference = ISHB

2nd " = ISMB for beam

for channel, 1st " = ISMC

2nd " = ISLC

Use ISMC ~~200~~ 300

$A = \cancel{38.67}\ 45.64\ cm^2$

$A_e = \cancel{38.67}\ 45.64 \times 2 = \cancel{77.34}\ 91.28\ cm^2$ (provided)

Selection of suitable section :

$C_{yy} = \cancel{2.30 cm}\ 2.36\ cm$

$g = \cancel{45 mm}\ 50\ mm$

$I_{xx} = \cancel{3816.8 cm^4}\ 6362.6\ cm^4$

$I_{yy} = \cancel{219.1 cm^4}\ 310.8\ cm^4$

r_{xx} ~~min~~ $= \cancel{2.38 cm}\ 11.81\ cm$

Buckling class - C

Slenderness Ratio , $\lambda = \frac{KL}{r_{min}} \times 1.05 = \frac{10000}{\cancel{23.8}\ 118.1} \times 1.05 = \cancel{441.17}\ 88.90$

KL/r	f_y
80	136
88.9	-
90	121

$y = y_1 + \frac{y_2 - y_1}{x_2 - x_1}(x - x_1)$

$f_{cd} = 122.65\ N/mm^2$

$P_d = A_e \times f_{cd} = 9128 \times 122.65 = 1119.5\ KN/mm^2$

$P_d > P \Rightarrow$ Hence OK.

Design of lacing bar procedure :

clear spacing : B-B

$s = d + 2g = 183.10 + 2(50) = 283.1\ mm$

Use parallel axis theorem

$I_{yy} = I_{xx}$

$2\left[I_{yy} + A\left(\frac{d}{2} + C_{yy}\right)^2\right] = 2 I_{xx}$

$2\left[310.8 \times 10^4 + 45.64 \left[\frac{d}{2} + \cancel{23.6}\ 2.36\right]^2\right] = 2 \times 6362.6 \times 10^4$

$d = 183.10\ mm$

Inclination Angle, $\theta = 45°$.

Vertical spacing, $c = 2s \tan\theta$

$= 2 \times 283.1 \tan 45°$

$c = 566.2$ mm

Length of lacing bar, $L = \frac{s}{\cos\theta} = \frac{283.1}{\cos 45°}$

$L = 400.36$ mm

Thickness of lacing bar, $T = L/40 = \frac{400.36}{40}$ [for single

$T = 10$ mm

$\therefore T \simeq 12$ mm

Width of lacing bar, $B = 3D = 3 \times 16 = 48$ mm [Assume $D = 16$ mm]

$B \simeq 50$ mm

Area of lacing bar, $A = B \times T = 50 \times 12 = 600 \text{ mm}^2$

Radius of gyration of lacing bar, $R = \frac{T}{\sqrt{12}} = \frac{12}{\sqrt{12}} = 3.46$ mm

$\lambda = \frac{L}{R} = \frac{400.36}{3.46} = 115.71$

From Table - 9(c) of IS 800 : 2007

λ	f_{cd}
110	94.6
115.71	-
120	83.7

$y = y_1 + \frac{y_2 - y_1}{x_2 - x_1}(x - x_1)$

$y = 88.37$

$f_{cd} = 88.38 \text{ N/mm}^2$

$P_L = A \times f_{cd} = 600 \times 88.38 = 53.03$ KN

$P_L < V$

$V = \frac{V_t}{2} = \frac{2.5\% \, (1119.5)}{2} = 13.99 \simeq 14$

H.W

(2) Design a built-up column with 2 channels back to back of length 10m to carry a factored load of 750 KN. The column is restrained in position but not in direction of both ends.

Laterally supported beam

UNIT :03

1

Design of beams:

The beams are structural members are frequently used to carry the loads that are transverse to their longitudinal axis. They transfer the load primarily by bending and shear. The flexural members have different names given as;

* Purling a member carrying load from sheeting of roof truss. It is a roof beam usually supported by trusses.

* Grid :- The members carrying load from side sheeting. It is the horizontal wall beam used to support wall covering on the side of

* Joist : They closely spaced beams supporting the floor and roof of building. It is the light member carrying load from floor in a building.

* Girder : A large beam supporting a joist. Mostly the member carrying the heavy loads.

* Lintel : The member carrying load over window or door opening.

* Spandrel : peripheral member in building or the beam around the outside perimeter of the floor that support the extension walls and outside edge of the floor.

In code IS:800-2007 has recommended design procedure for members subjected to flexural, mostly in four categories;

1. Laterally supported beam
2. Laterally unsupported beam
3. Plate girders
4. Beam under bi-axial bending

Types of beam sections:

The angles and T-section are inherently weak in bending while the channel only being used for light rolled Structure.. The rolled steel I-section generally preferred as a beam.

2

Condition to qualify as a laterally restrained beam

1. It should not laterally buckle.
2. None of its element should buckle until a desired limited state is achieved.
3. Limit state of serviceability must be satisfied.
4. Member should behave in accordance with a expected performance of the system.

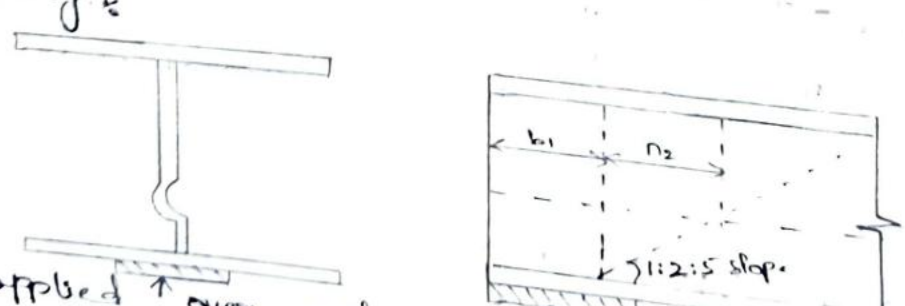
Web buckling:

The web in a rolled steel section behave like a column when placed under concentrated load. The web is thin & is subjected to buckling. Web buckling occurs when the intensity of vertical compressive stress near the centre of section becomes greater than the critical buckling stress. The web buckling strength.

$$P_{wb} = (b_1 + n_1) t_w \cdot f_{cd}$$

Effective length of the web $kt = 0.7d$

Web crippling:



A load applied over a short length of beam can cause failure, due to crushing due to high compressive stress in the web of the beam below the load or above the reaction is called web crippling or web crushing.

The strength of the web crippling $P_{wc} = (b_1 + n_2) t_w \cdot \frac{f_y}{\ldots}$

$t_f = 16$ mm.

$t_w = 8.9$ mm

where, n_1

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

f_{yw} - yield stress of the web.

Design procedure for laterally supported beams:

- From code pg: 52; cl: 8.2.0

1. A trial section is selected and findout the classification of the section.
2. Check for the shear strength
3. Check for bending strength
4. Check for deflection
5. Check for web - bearing.

Problems:-

(1) Determine the design bending strength of ISFB 350 @ 495 N/m. Consider the beam to be laterally supported. Assume the steel of grade Fe410.

Sol: Given data:

Step: a) properties of the section:

(From steel table:) ISFB 350 @ 495 N/m

Depth of section $(h) = 350mm$

Width of flange $(b) = 165mm$

Thickness of flange $(t_f) = 11.4mm$

Thickness of web $(t_w) = 7.4mm$

Radius of root $(R_1) = 16.0mm$

$\therefore$ Depth of web $(d_w) = h - 2(t_f + R_1)$

$= 350 - 2(11.4 + 16)$

$d_w = 295.2 mm$

moment of

$I_{zz} = 13158.3 \times 10^4 mm^4$

$I_{yy} = 631.9 \times 10^4 mm^4$

(Steel table) (From code Annex-1.1 pg: 188)

Plastic section modulus $Z_{px} = 851.1 \times 10^3 mm^3$

Elastic section modulus $Z_{ex} = 751.9 \times 10^3 mm^3$

2 m

3. Design of Beams :

PF ⑥

① Design a laterally simply supported beam of 4m for concentrated load of 400KN at midspan. The load is transformed through base plate of 200mm length of support design. A check for deflection using ISMB 400 section.

1. Given Data :

Sol: • Span of SSB , = 4m

• Support Wall Width = 200mm = 0.2m

• Eff. Length , $L_{eff} = \frac{0.2}{2} + 4 + \frac{0.2}{2} = 4.2\,m$

• Service Load = 400KN

Factored Load, W = 400 × 1.5 = 600 KN.

2. Calc Max. Factored BM :

$$BM = \frac{Wl}{4} = \frac{600 \times 4.2}{4} = 630\ KNm.$$

• Factored shear force :

$$V = \frac{W}{2} = \frac{600}{2} = 300\ KN$$

3. Calc Modulus of plasticity : Assume Fe 410

$$Z_p = \frac{M}{f_y} \times \gamma_{m0}$$

$$= \frac{630 \times 10^6\ Nmm}{250\ N/mm^2} \times 1.1$$

$$Z_p = 2.77\ mm^3$$

4. Properties of ISMB 400 : · pg - 138 of IS 800:2007

A_e = 7846 mm²

I_{xx} = ~~20390.8~~ 20458.4 cm⁴

I_{yy} = ~~834~~ 422.1 cm⁴

r_{xx} = 16.15 cm

r_{yy} = 2.82 cm

b = 140mm

Z_{xx} = 1022.9 cm³

t_f = 16 mm

t_w = 8.9 mm

Z_{yy} = 88.9 cm³

h = 400mm

5. Elastic Section Classification:

$$\varepsilon = \sqrt{\frac{f_y}{250}} = \sqrt{\frac{250}{250}} = 1$$

Ratio of width to thickness of flange:

$$\frac{b}{t_f} = \frac{140}{16} = 8.75 < 9.4\varepsilon$$

6. check for shear capacity:

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

$$= \frac{250}{\sqrt{3}\times 1.1} \times 400 \times 8.9$$

$$\boxed{V_d = 467.12\ KN}$$

Check for low shear or high shear:

$0.6\,V_d = 0.6 \times 467.12 = 280.27$ KN

$0.6V_d < V$ – High

$0.6V_d > V$ – Low

$\therefore 0.6V_d < V \Rightarrow 280.27 < 300$

Hence it is high shear

7. check for Bending shear:

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} \le \frac{1.2 Z_e f_y}{\gamma_{m0}}$$

Pg-53

$$= \frac{1 \times 1176.18 \times 250}{1.1} < \frac{1.2 \times 1020 \times 250}{1.1}$$

$$M_d = 267.12 < 278.18$$

Hence the section ISMB 400 is safe & economical.

$\therefore M_d = 267.12$ KNm

8. Check for deflection:

$$\delta_{max} = \frac{WL^3}{48 E I_{xx}} = \frac{600 \times 4000^3}{48 \times 2\times10^5 \times 204.58\times10^6} = \cancel{0.019}\ 19.55\,mm$$

Permissible limit, $\delta = \frac{L}{300} = \frac{4000}{300} = 13.33$ mm

it is safe.

② Design a laterally supported beam. Eff. span 6m for grade of steel Fe410 from BM 150 KNm, SF 210KN, use ISLB 350 @ 495 KN/m

Sol: Given : span = 6m

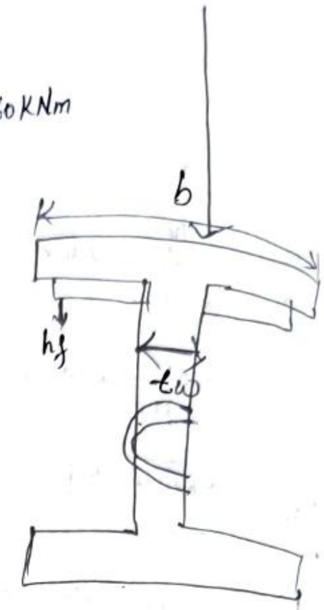
M_u = 150 KNm

V_u = 210 KN

ISLB 350 @ 495 KN/m

$$Z_p = \frac{M}{f_y} \times \gamma_{m0} = \frac{150 \times 10^3}{250} \times 1.1 = 660 \text{ KNm}$$

Properties of section :



A_e = 6301 mm^2

h = 350 mm

b = 165 mm

t_f = 11.4 mm

t_w = 7.4 mm

r_{zz} = 14.45 cm

r_{yy} = 3.17 cm

h_1 = 288.3 mm

h_2 = 30.85 mm

Z_{ez} = 751.9 cm^3
Z_{pz} = 851.11 cm^3 } IS 800 : 2007.

Depth of Web, $d = h - 2(t_f + r_{zz})$

$= 350 - 2(11.4 + 144.5)$

$= 38.2$ mm

The outstand of flange element, $h_f = \frac{b_f}{2} = \frac{165}{2} = 82.5$ mm

$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.23 < 9.4\varepsilon$ [pg-18 of IS 800 : 2007]

Check for shear :

Design of shear strength, $V_d = \frac{f_y}{\sqrt{3} \times \gamma_{m0}} \times h \times t_w$

$= \frac{250}{\sqrt{3} \times 1.1} \times 350 \times 7.4$

$= 339.82 \simeq 340 \text{ KN} > V_u$

Hence OK.

check for low or high shear :

$0.6\,V_d = 0.6 \times 340 = 204$ KN

$V_u > 0.6\,V_d \rightarrow 210 > 204$

It is off high shear

check for design bending strength : [pg-53, Cl 8.2.1.2]

$M_d = \dfrac{\beta_b Z_p f_y}{\gamma_{m0}} = \dfrac{1 \times 851.11 \times 250}{1.1} = 193.43$ KNm

$M_{dv} = M_d - \beta(M_d - M_{fd})$ [pg-70, cl 9.2.2.2] $y_w = \dfrac{y}{4}$

$M_{fd} = Z_{fd} \cdot \dfrac{f_y}{\gamma_{m0}}$

$Z_{fd} = Z_{pz} - A_w \times y_w$

$= 851.11 \times 10^3 - (350 \times 7.4) \times \dfrac{350}{4}$

$= 624.48 \times 10^3\ mm^3$

$= 624.48 \times 10^3 \times \dfrac{250}{1.1}$

$= 141.92$ KNm.

$\beta = \left[\dfrac{2V}{V_d} - 1\right]^2$

$= \left[\dfrac{2 \times 210}{340} - 1\right]^2$

$= 0.055$

$M_{dv} = 193.43 - 0.055(193.43 - 141.92)$

$= 190.6$ KN $< \dfrac{1.2 \times 751.9 \times 10^3 \times 250}{1.1}$

$= 190.6 < 205.06$ KNm

Hence OK & it is safe.

check for buckling :

Assume stiffness bearing length $b = 100$ mm

{pg-66, cl 8.7.1.5}

$A_b = B \times t_w$

$= (b + x)\, t_w$ $x = \dfrac{h}{2}$

$= (100 + 175) \times 7.4$

$A_b = 2035\ mm^2$

[pg-58, Tb-15] $KL = 0.7L = 0.7 \times 288.3 = 201.8$ ~~26.24~~ mm

$\lambda = \dfrac{KL}{r} = \dfrac{201.8}{2.136} = 94.47$

KL/r	f_y
90	121
94.47	?
100	107

$$y = y_1 + \frac{y_2 - y_1}{x_2 - x_1}(x - x_1).$$

$$= 114.74\ N/mm^2$$

$$f_{cd} = 114.74\ N/mm^2$$

Capacity of web section $= f_{wd}$

$$= A_b \times f_{cd}$$

$$= 2035 \times 114.74$$

$$= 233.49\ KN$$

(3) Design a laterally unsupported beam for the following data:
Eff. span = 4m, Max BM = 550 KNm, Max SF = 200 KN,
Grade of steel = Fe410, ISMB 550 @ 1037 KN/m

Sol: ~~Let us try~~ Given Data: $L_{eff} = 4m$

$M_u = 550 KNm$

$V_u = 200 KN$

$Fe410 \Rightarrow f_y = 250 Mpa$

$$Z_p = \frac{M_u}{f_y} \times \gamma_{mo} = \frac{550 \times 10^6}{250} \times 1.1 = 2.42\ KNm$$

Properties of section: ISMB 550 @ 1037 KN/m

$A_e = 13211\ mm^2$

$b_f = 190$ mm

$t_f = 19.3$ mm

$t_w = 11.2$ mm

$r_z = 221.6$ mm

$Z_{ez} = 2359.8\ cm^3$

$Z_{pz} = 2711.98\ cm^3$

$h_1 = 467.5$ mm

$h_2 = 41.25$ mm

$h/D = 550$ mm

$r_y = 37.3$ mm

Depth of web, $d = h - 2(t_f + r_z)$

$\cancel{= 550 - 2(19.3 + 221.6)}$

$\cancel{= 484.6\,mm\ \ 68.2\,mm}$

$d = h - 2h_2$

$= 550 - 2(41.25)$

$= 467.5$ mm

Outstand of flange element, $b_f = \frac{b_f}{2} = \frac{190}{2} = 95\ mm$

$\frac{b}{t_f} = \frac{95}{19.3} = 4.92 < 9.4\varepsilon$ [pg 18 of IS 800:2007]

Check for shear:

$V_d = \frac{f_y}{\sqrt{3} \times \gamma_{m0}} \times h \times t_w = \frac{250}{\sqrt{3} \times 1.1} \times 550 \times 11.2 = 808.29\ kN$

$V_d > V_u \Rightarrow 808.29 > 200$

Hence OK

Check for low or high shear:

$0.6\ V_d = 0.6 \times 808.29 = 484.97\ kN$

$V_u < 0.6\ V_d \Rightarrow 200 < 484.97$

It is off low shear.

[pg-58 Tb-15] $KL = 1.00L = 1 \times 4 = 4m$

$\frac{h}{t_f} = \frac{550}{19.3} = 28.49\ mm$

$\lambda = \frac{KL}{r} = \frac{4000}{37.3} = 107.24$

KL/r	h/tf
100	277.09
107.24	–
110	238.04

KL/r = 100

	h/tf	
x_1 25	291.4	y_1
x 28.49	–	y
x_2 30	270.9	y_2

$y = y_1 + \frac{y_2 - y_1}{x_2 - x_1}(x - x_1)$

$\boxed{y = 277.09}$

KL/r = 110	h/tf
25	251.8
28.49	–
30	232.1

$\boxed{y = 238.04}$

107.24:

$y = 248.81$

$\boxed{\therefore f_{cr,b} = 248.81\ N/mm^2}$

pg-55 Tb-13(a)

$f_{cr,b}$	f_y
250	159.3
248.81	–
200	134.1

$\boxed{f_{bd} = 151.86\ N/mm^2}$

Plate Girders:

76 ①

Introduction: A plate Girder is defined as a beam built-up beam used to carry heavy loads, over long spans, It comprises of plate and angle and posses high flexural strength.

Plate Girder used in bridges and buildings.

Components of Plate Girders

The various Components of Plate Girder are as follows.

2M ★★

1) web Plate
2) Flange plate
3) Flange Angles
4) stiffness
5) Stiffness
6) splices.

1) Web plates:-

* web plate is vertical plate which having a thickness of 8mm to 15mm
* The depth of the web plate separately on different factors such as loads, clearness of head group.
* The depth of web plate may ranges from 1/8 - 1/12 of the span
* The depth may vary from 1000mm - 300mm for small spans to long spans respectively
* The plate may be unstiffened or stiffened.
* The clear depth to thickness of web less than 85 then web is unstiffened.
* The clear depth and thickness of web greater than 85 then web is stiffened

2) Flango plates :- (v)

* Normally horizontal flango plates are provided and the Plates are fixed to the at standing legs of flange angles.
* The flango plate which may vary from $\frac{1}{30}$ to $\frac{1}{40}$ of span
* The flango plate width may be selected equal to $2hf$ where;

hf: length of out standing leg of thick flange angle.

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3) Flange angles:-

Normally 2 flange angles are provided at the top & bottom of Girder respectively.

This flango angles Expanded from one End of the Girdes to other End.

This angles have a thickness of 10mm to 25mm. Flango angles may Equal (or) unequal angles.

The unequal angles are used to connect the shorter legs and the web. where are as Equal angle are commonly used.

The area of flango angles must be at least $\frac{1}{3}$th of the flange area

4) Stiffness:-

The web is protected against to buckling by providing the Stiffness

The thin and deep web plate is subjected to vertical & diagonal buckling.

The web may be stiffened by providing vertical & longitudinal Stiffness

5) Splices:

web splices are connecting plates used to connect to 2 web plates

Flango & its splice are the connecting elements used to connect 2 flango angles

Flango plate splices are the connecting plates moment curvature relationship ... be compute

a fully yielding section, the bending stress distribution ... should ... here the ... equired.

78 (3)

Design procedure for plate Girders.

01. Calculate the load of plate girders.

02. Calculate the max. Bending moment

03. Calculate the shear force

04. Calculate the economical depth of plate girder

$$d = \left[\frac{M.K}{f_y}\right]^{1/2}$$

05. Calculate the webthickness ;

$$t_w = \left[\frac{M}{f_y}\right]^{1/2}$$

06 provide a web plate of obtain size

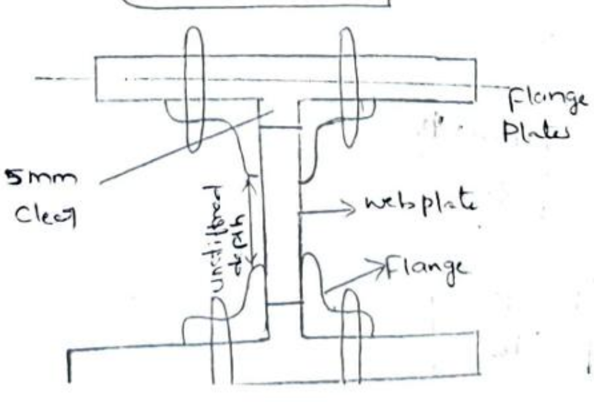
07 Compute the flange area ;

$$\boxed{A_F = \frac{M.\gamma_{mo}}{f_y.d}}$$

08. Calculate thickness and width of flange by using the "AF".

09. Calculate the moment capacity of girder

$$\boxed{M_c = \frac{Z.f_y}{\gamma_{mo}}}$$



10. Calculate the shear capacity of web

11. Calculate the bearing stiffness, intermediate stiffness

12. Check for bearing and intermediate stiffness

13. connections;

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<u>Self wt of Girder:</u>

$$W = \frac{WL}{200} \text{ kN/m} \Rightarrow \text{Factored load}$$

economical depth:

$$M = f_y \cdot b_f \cdot t_f \cdot d$$

$$\frac{M}{f_y \times d} = b_f \cdot t_f$$

$$A = \frac{M}{f_y \times d} \Rightarrow A = 2 b_f t_f + d \cdot t_w$$

$$= 2 \times \frac{M}{f_y \times d} + d \cdot t_w$$

Let $\frac{d}{t_w} = k$

$$\boxed{A = \frac{2M}{f_y \cdot d} + \frac{1}{k} \cdot d^2}$$

Differentiate the above eqn;

$$0 = \frac{2M}{f_y d^2} + \frac{1}{k} \cdot 2d$$

$$\frac{\cancel{2}M}{f_y \cdot d^2} = \frac{1}{k} \cancel{2}d \Rightarrow$$

$$\frac{Mk}{f_y} = d^3$$

$$d = \left(\frac{M \cdot k}{f_y}\right)^{1/3}$$

where; $\boxed{k = \frac{d}{t_w}}$

(4)

* If $\frac{d}{t_w} \leq 67\varepsilon \rightarrow$ ordinary Beam.

From IS:800 [cl: 8.6.1.1] pg: 63.

a) when transverse stiffness are not provided;

$$\frac{d}{t_w} \leq 200\varepsilon \quad ; \frac{d}{t_w} \leq 70\varepsilon$$

(80)

b) when transverse stiffness are provided;

$\Rightarrow$ when $3d \geq c \geq d$

$$\frac{d}{t_w} \leq 200\varepsilon$$

$\Rightarrow$ when $0.75d \geq c \geq d$

$$\frac{c}{t_w} \leq 200 t_w$$

$\Rightarrow$ when $c < d$

$$\frac{d}{t_w} \leq 270\varepsilon$$

$\Rightarrow$ $c \geq 3d$ the web should be designed as unstiffness

web thickness based on compression flange buckling requirement:

[IS: 800-2007 - clause 8.6.1.2]

a) when transverse stiffness are not provided

$$\frac{d}{t_w} \leq 345 t_f^2$$

b) when transverse stiffness are provided

$\Rightarrow$ $c \geq 1.5d$

$$\frac{d}{t_w} \leq 345 t_f^2$$

$\Rightarrow$ $c < 1.5d$; $d/t_w \leq 345 t_f^2$

* $k = \frac{d}{t_w} \leq 67 \rightarrow$ ordinary beam with end bearing stiffness only

* $k = 67$-200 ; with out intermediate

$k = 100$-110 ;

$k = 270$: longitudinal stiffness is needed.

K $345

(6)

$\frac{D}{L} = \frac{1}{15}$ to $\frac{1}{25}$ for girder in buildings

$\frac{D}{L} = \frac{1}{12}$ to $\frac{1}{18}$ for highway bridges

$\frac{D}{L} = \frac{1}{10}$ to $\frac{1}{15}$ for railway bridge.

Size of flanges :

(81)

$$M = \frac{A_F \times f_y \times d}{\gamma_{mo}}$$

Semi-compact section

$b_f \times t_f = A_F$

$13.6\,\epsilon\, \cancel{\times} t_f^2 = A_F$

$$\boxed{b_f = \frac{A_F}{t_F}}$$

$\frac{b}{t_f} = 13.6\,\epsilon$

$b_f = 13.6\,\epsilon\, t_f$

$b_f \times t_f = 13.6\,\epsilon\, t_f^2$

Shear buckling resistance of web : [Cl: 8.4.2]

$\frac{d}{t_w} > 67\,\epsilon$, web without stiffeness

$\frac{d}{t_w} > 67\,\epsilon \sqrt{\frac{K_v}{5.35}}$ with stiffness

* Nominal shear strength $V_n = V_{cr}$ can be calculated from;

a) Simple post-critical method [cl: 8.4.2.2(a)

b) Tension field method cl: 8.4.2.2(b)]

a) Simple post-critical method :

$$V_{cr} = A_v \cdot \tau_b$$

b) Tension field method :

$$\geq 1.0$$

$$V_n = V_{tf}$$

End panel design : [Cl: 8.5]

Anchor force [Cl: 8.5.3]

Design of connection b/w flange and web plate :

Shear stress @ the junction

$$q_w = \frac{V_n \bar{y}}{b I_z}$$

shear force per unit length ;

$$= \frac{V a \bar{y}}{I_z}$$

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If of throat thickness 't' provided on the strength of shop weld per unit length.

$$= \frac{2 t f_w}{\sqrt{3} \cdot \gamma_{mo}} \rightarrow 1.25$$

$$\left[I_z = \frac{b_f \cdot d^3}{12} \right]$$

$$\frac{V a \bar{y}}{I_z} = \frac{2 t f_w}{\sqrt{3} \gamma_{mo}}$$

$$\Rightarrow 't' = ?$$

Size of weld ; $s = \frac{t}{0.7}$

Design of Bearing stiffness : [Cl ; 8.7.12]

Then stiffness should be checked

a) Buckling resistance [Cl ; 8.7.15]

$$\lambda = \frac{KL}{r} = \frac{0.7 \times L}{r} = \frac{0.7 \times L}{\sqrt{I/A}}$$

$\lambda = ?$
$f_{cd} = ?$

Buckling resistance $= f_{cd} \times A >$ Applied load.

b) Bearing capacity [Cl 8.7.5.2]

$$F_{psd} = \frac{A_q \cdot f_y \cdot v}{0.8 \gamma_{mo}} \geq F_x \rightarrow S.F$$

C. Torsional resistance ; [Cl : 8.7.9]

$Is \geq 0.34 \alpha_s D^3 T_{cf}$ (8)

weld for end stiffness : [Cl : 8.7.2.ic]

$$\frac{tw^2}{5bs}$$

(83)

$\Rightarrow$ 8.7.3.1 clause ; $b = b_1 + n_1$

$$I = \frac{d\,tw^3}{12}$$

$A = d\,tw$

$$r = \sqrt{I/A} = \sqrt{\frac{d\,tw^3}{12\,d\,tw}} = \sqrt{\frac{tw^2}{12}} = \frac{t}{\sqrt{12}}$$

$$\lambda = \frac{0.7 \times d}{r} = 0.7 \times \sqrt{12}\,\frac{d}{t} = 2.42\,\frac{d}{t}$$

$\lambda = ?$

$f_{cd} = ?$

Buckling resistance $= b\,t\,f_{cd}$

Design of Intermediate stiffness : [clause . 8.7.12]

Minimum MOI

Cl.(8.7.2.5)

$c/d \geq \sqrt{2}$ $\quad Is \geq 0.75\,d\,tw^3$

$c/d \leq \sqrt{2}$ $\quad Is = \dfrac{1.5\,d^3 tw^3}{c^2}$

$$F_{qr} = \frac{V - V_{cr}}{\gamma_{mo}} \leq F_{qd}$$

$$\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \leq 1$$

If $f_d < F_x$, then $(F_q - F_x) = 0$

Connection of Intermediate stiffness to web : [Cl : 8.7.2.6]

$$\frac{tw^2}{5bs}$$

① (22)

Design of purlins

Purlin's is a member which rest between roof trusses and supports roof sheeting. I-sections channels, angles, cold formed C or Z-sections are commonly used as purlin's. The purlins are spaced on main rafter of trusses such that the sheets overlap on them. For A.C sheets the overlaps of at least 150mm is to be ensured. Hence one can see in most of the building purlins are spaced at 1.35 m to 1.40 m. when A.C sheets are of length 1.65 m.

Design procedure;

01) Resolve the factored forces parallel to and ⊥ar to sheeting.

02) Determine moments and shear forces about z-z and y-y axis (M_z, F_z, M_y and F_y).

03) To account for biaxial bending, the required value of section value of section modulus about z-z axis may be taken as;

$$Z_{pz} = \frac{M_z}{f_y}\gamma_{mo} + 2.5\frac{d}{b}\cdot\frac{M_y}{f_y}\gamma_{mo}$$

where;

γ_{mo} is partial safety factor for materials = 1.1

d is the depth of trail section and,

b is the breadth of trail section.

The second term in the expression above make an approximate relation b/w Z_{pz} and Z_{py}. since d and b values will be known only after a section is selected, first trail section is selected and Z_p required is found. It it is sufficient proceed further otherwise try another section.

04) check the section for shear capacity, for which the following expressions can be used;

$$V_{dz} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times A_{vz}$$

$$V_{dy} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} A_{vy}$$

where; $A_{vz} = ht_w$

$A_{vy} = 2b_f t_f$

05) Compute ...

$$M_{dz} = \frac{Z_{pz} f_y}{\gamma_{mo}} \leq 1.2 Z_{ez} \times \frac{f_y}{\gamma_{mo}}$$

23 10

$$M_{dy} = \frac{Z_{py} \cdot f_y}{\gamma_{mo}} \leq 1.5 Z_{ey} \cdot \frac{f_y}{\gamma_{mo}}$$

06) The design should satisfy the following interaction formulae;

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

07) Apply check for deflection.

08) Check for wind section (suction).

problems :-

① symmetric trusses of span 20m and height 5m are spaced at 4.5m centre to centre. Design channel sections purlin's to be placed at 1.4m distances to resist the following loads.

Wt of sheeting including bolts = 171 N/m²

Live load = 0.4 kN/m²

wind load = 1.2 kN/m², suction

Sol: Given data;

Height of truss = 5m

span of truss = 20m

$\tan\theta = \frac{5}{10}$

$\therefore \theta = 6.565$

Design for DL + LL:

Self weight of purlins = 125 N/m² (assumed)

DL for sheeting = 171 N/m²

∴ Total dead load = 171 + 125

= 296 N/m²

= 0.296 kN/m²

Live load = 0.4 kN/m²

∴ Factored DL + LL is;

= 1.5 (0.296 + 0.4)

$= 1044 \times 1.4 = 1.46$ kN/m.

[vertically downward]

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$\therefore$ Load normal to sheeting;

$= 1.46 \cos\theta$

$= 1.46 \times \cos(26.565)$

$= 1.306$ kN/m

Load in direction parallel to sheeting: $1.46 \sin\theta$

$= 0.653$ kN/m.

Bending moment are :

$M_z = 1.306 \times \frac{4.5^2}{8} = 3.306$ kN-m $\left[\text{i.e } \frac{wl^2}{8}\right]$

$M_y = 0.653 \times \frac{4.5^2}{8} = 1.653$ kN-m

Shear force :

$F_z = 1.306 \times \frac{4.5}{2} = 2.939$ kN $\left[\because \frac{wl}{2}\right]$

$F_y = 0.653 \times \frac{4.5}{2} = 1.469$ kN

Try ISMC 100 section ;.

$d = h_1 + 2r_1$, $b = 50$ mm

$= 64 + 2\times 9$

$= 82$ mm

$Z_{pz(req)} = \frac{M_z}{f_y} \times \gamma_{m0} + 2.5 \times \frac{d}{b} \times \frac{M_y}{f_y} \times \gamma_{m0}$

$= \frac{3.306\times 10^6}{250} \times 1.1 + 2.5 \times \frac{76}{50} \times \frac{1.653\times 10^6}{250} \times 1.1$

$= 42.185 \times 10^3$ mm.

$\therefore Z_{pz}$ of ISMC 100 is 43.825×10^3 mm^3. Hence adequate (steel table).

Check for shear :

$V_{dz} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{m0}} \times h \times t_w$

$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 100 \times 4.7$

$= 61671$ N $> F_z$

$= 61.67$ kN > 2.939 .

Design of angle purlins.

As per IS 800-2007, angle purlins should be designed for biaxial bending. However IS.800-1984 and present British code permit design of angle by purlins by assuming that load normal to sheeting is resisted by purlins and load parallel to sheeting is resisted by sheeting, provided the following conditions are fullfilled:

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a) Roof slope is less than 30°

b) width of angle leg llar to sheeting $\geq \frac{L}{45}$

c) width of angle llel to sheeting $\geq \frac{L}{60}$

In the above situations, bending moment, about z-z axis should be taken as $\frac{W_z l}{10}$, where W_z is total load in the direction normal to sheeting and 'L' is the spacing of trusses.

Problems:

① Design angle purlin for the following data by simplified method.

spacing of trusses = 3.5m

Spacing of Purlins = 1.6m

wt of A.C sheets including laps and fixtures = 0.205 kN/m²

Live load = 0.6 kN/m²

Wind load = 1 kN/m² (suction)

Inclination of main rafter of truss = 21°

Sol: Trail section:

$$\text{Min depth} = \frac{L}{45} = \frac{3500}{45} = 78 \text{ mm}$$

$$\text{min width} = \frac{L}{60} = \frac{3500}{60} = 58.3\text{mm}$$

Let us try ISA 9060, 6mm thick.

Dead Load:

wt of Ac sheets with overlap & fixtures

$$= 0.205 \times 1.6$$

$$= 0.328 \text{ kN/m}$$

Plate Girders

(1)

5. Plastic Analysis & Design

Plastic analysis is a method of structural analysis used to determine the ultimate load carrying capacity of structures by allowing the material to yield & distribute internal forces ~~beyon~~ before collapse.

* Steel has unique physical property called ductility.
* Because of which it is able to observe large deformations beyond the elastic limit without fracture.
* During the process of yielding, structural steel deforms under a constant & uniform stress known as yield stress.
* This property of steel known as ductility is utilized in plastic analysis & design.

Plastic Moment Capacity: It is the max. moment of a section can resist when the entire c/s has yield.

$M_p = f_y \times Z_p$ → Plastic section modulus

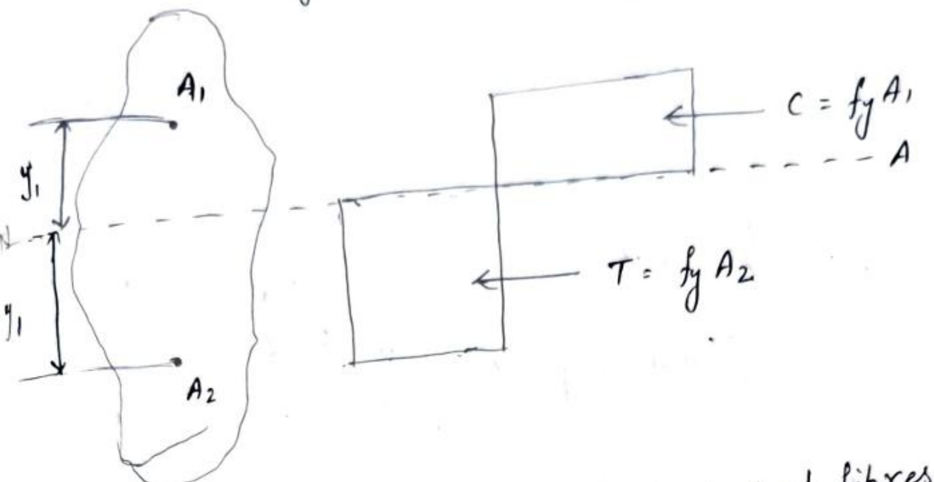
M_p ↙ Plastic Moment

f_y ↓ Yield stress of material

Assumptions:

* ~~...~~ Instability of the structure will not occur to the attainment of the ultimate load.
* The connections provided fully continuity such that the fully plastic moment can be transmitted

* The loading is proportional, i.e., all the loads increases in a fixed proportion to one another.
* The plastic moment capacity of any section may be compute with reference to the moment curvature relationship
* Consider a fully yielding section, the bending stress distribution will be rectangular as shown in fig.



Let The nature of BM be sagging such that fibres above NA are in compression, those below NA are in tension.

By the equilibrium conditions

$\therefore$ ~~Eq~~ $C = T$

$$\cancel{f_y} A_1 = \cancel{f_y} A_2$$

$$\boxed{\therefore A_1 = A_2 = \frac{A}{2}}$$

Where A_1 is the area of ~~ab~~ section above NA
A_2 " " " " below "
A " " " " cross " of the beam

Imp → $\therefore$ The NA of the classified section is sometimes called equal area axis.

The two forces tensile & compression from a couple which resists the plastic moment.

$$M_p = f_y A_1\bar{y}_1 + f_y A_2\bar{y}_2$$
$$= f_y (A_1\bar{y}_1 + A_2\bar{y}_2)$$

$$M_p = f_y \left[\frac{A}{2}\bar{y}_1 + \frac{A}{2}\bar{y}_2 \right]$$

$$= f_y \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

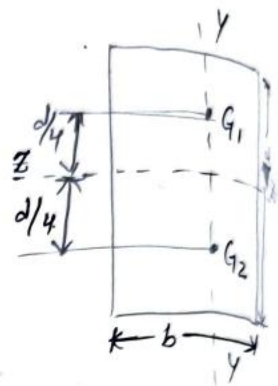
$$\boxed{\therefore M_p = f_y Z_p}$$

Where Z_p = plastic modulus of section = $\frac{A}{2}(\bar{y}_1 + \bar{y}_2)$

$\bar{y}_1, \bar{y}_2$ = Distance of COG of the area above & below the NA respectively.

6M

For a rectangular section of fig



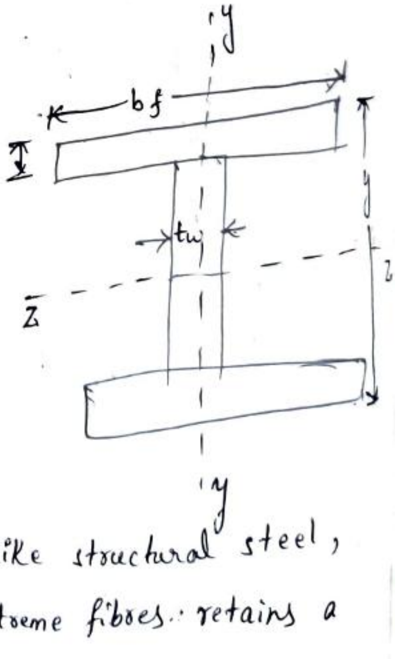
$$Z_{pz} = \frac{bd}{2}\left[\frac{d}{4} + \frac{d}{4}\right]$$

$$\boxed{Z_{pz} = \frac{bd^2}{4}}$$ (about major axis Z-Z)

$$Z_{py} = \frac{bd}{2}\left[\frac{b}{4} + \frac{b}{4}\right]$$

$$\boxed{Z_{py} = \frac{db^2}{4}}$$ (about minor axis y-y)

~~from~~ for a Rolled I-section:



$$Z_{pz} = b_f t_f (h - t_f) + t_w\left(\frac{h}{2} - t_f\right)^2$$

(about major axis)

$$Z_{py} = t_f b_f^2 + \frac{t_w^2}{4}(h - 2t_f)$$

(about minor axis)

Shape factor: For a ductile material like structural steel, a member reaching yield at the extreme fibres retains a reserve strength that varies with the shape factor.

$$S = \frac{M_p}{M_y} = \frac{\cancel{f_y} Z_p}{\cancel{f_y} Z_e} = \frac{Z_p}{Z_e}$$

where ; Z_p , Z_e = Plastic & Elastic section modulus.

For a rectangular section, the elastic modulus

$$Z_e = \frac{\frac{bd^3}{12}}{d/2} = \frac{bd^2}{6}$$

$$Z_p = \frac{bd^2}{4}$$

From shape factor, $S = \frac{Z_p}{Z_e} = \frac{\frac{b\cancel{d^2}}{4}}{\frac{b\cancel{d^2}}{6}} = \frac{3}{2} = 1.5$

Steps in Plastic Design :

1 : Determine degree of static indeterminacy
2 : Calculate no. of plastic hinges required.
3 : Assume possible hinge location
4 : Use virtual method or equilibrium method
5 : Calculate collapse load .

3. The spacing of restraints to member lengths not containing a plastic hinge should satisfy the recommendations for lateral buckling strength of beams. Where the restraints are placed at the limiting distance L_{m}, no further checks are required.

Collapse load of standard cases

Concentrated Load at Centre

Simply Supported Beam	*Fixed Beam*

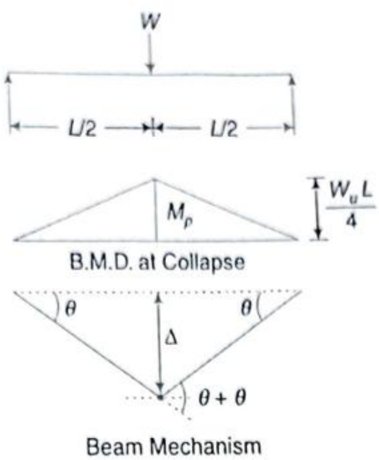


Fig. 3.17

Static method

$$\frac{W_u L}{4} = M_p$$

or $$W_u = \frac{4M_p}{L}$$

Kinematic method

$$W_u\left(\frac{L}{2}\right)\theta = M_p(\theta + \theta)$$

or $$W_u = \frac{4M_p}{L}$$

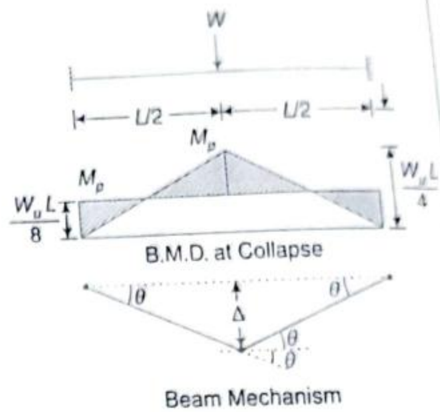


Fig. 3.18

Static method

$$\frac{W_u L}{4} = 2M_p$$

or $$W_u = \frac{8M_p}{L}$$

Kinematic method

$$W_u\left(\frac{L}{2}\right)\theta = M_p\theta + M_p(\theta + \theta) + M_p\theta$$

or $$W_u = \frac{8M_p}{L}$$

Eccentric Load

Simply Supported Beam	*Fixed Beam*

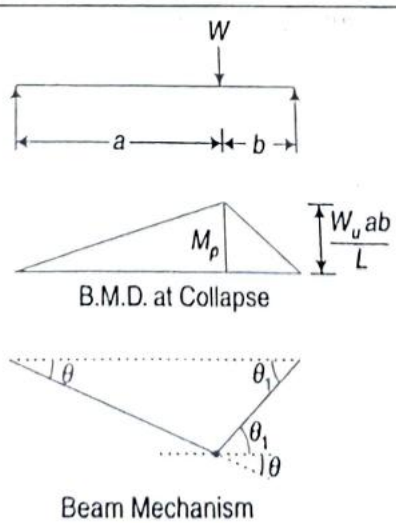


Fig. 3.19

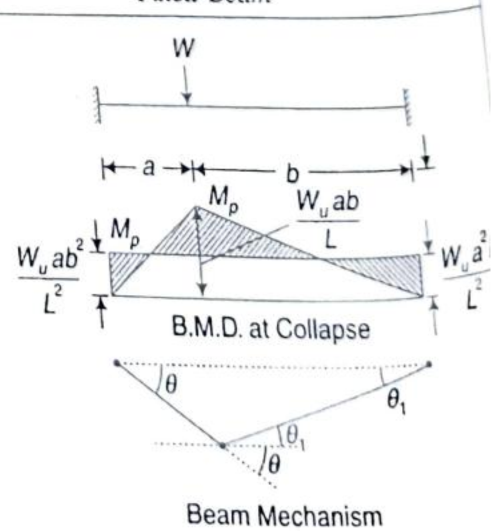


Fig. 3.20

Static method

$$\frac{W_u ab}{L} = M_p$$

or $$W_u = \frac{M_p L}{ab}$$

Kinematic method

$$W_u a\theta = M_p(\theta + \theta_1)$$

$$= M_p\left(\theta + \frac{a}{b}\theta\right)$$

or $$W_u a\theta = M_p\,\theta\left(1 + \frac{a}{b}\right)$$

$$= M_p\,\theta\left(\frac{b + a}{b}\right)$$

$$W_u a\theta = M_p\,\theta\left(\frac{L}{b}\right)$$

or $$W_u = M_p\,\frac{L}{ab}$$

Static method

$$\frac{W_u ab}{L} = 2M_p$$

or $$W_u = \frac{2M_p L}{ab}$$

Kinematic method

$$W_u a\theta = M_p\theta + M_p(\theta + \theta_1) + M_p\theta_1$$

or $$W_u a\theta = M_p\theta + M_p\left(\theta + \frac{a}{b}\theta\right) + M_p\,\frac{a}{b}\theta$$

$$= 2M_p\theta\left(\frac{a + b}{b}\right)$$

$$= 2M_p\theta\frac{L}{b}$$

or $$W_u = \frac{2L}{ab}M_p$$

or $$W_u = \frac{2M_p L}{ab}$$

Uniformly Load Distributed

Simply Supported Beam

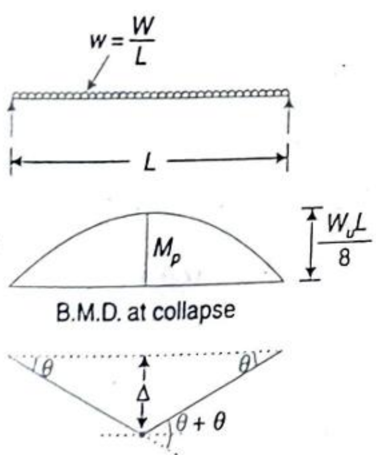


Fig. 3.21

Static method

$$\frac{W_u L}{8} = M_p$$

$$W_u = \frac{8M_p}{L}$$

Kinematic method

$$W_u\,\frac{1}{2}\left(\frac{L}{2}\theta\right) = M_p(\theta + \theta)$$

or $$W_u = \frac{8M_p}{L}$$

Fixed Beam

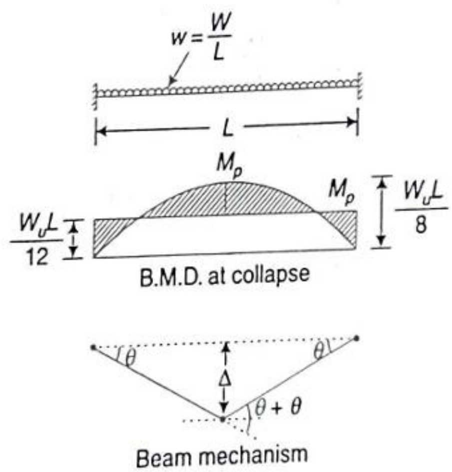


Fig. 3.22

Static method

$$\frac{W_u L}{8} = 2M_p$$

$$W_u = \frac{16M_p}{L}$$

Kinematic method

$$W_u\frac{1}{2}\left(\frac{L}{2}\theta\right) = M_p\theta + M_p(\theta + \theta) + M_p\theta$$

or $$W_u = \frac{16M_p}{L}$$

Concentrated Load

Propped Cantilever	*Propped Cantilever*

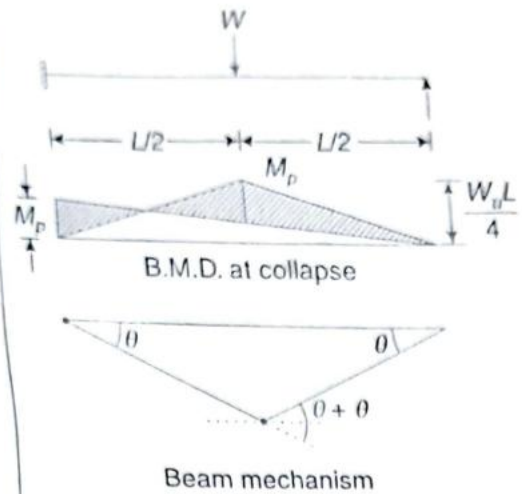


Fig. 3.23

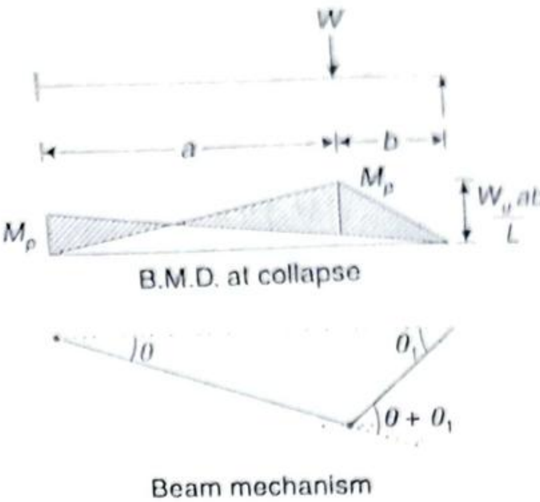


Fig. 3.24

Static method (Fig. 3.23)

$$\frac{W_u L}{4} = M_p + M_p \frac{L/2}{L}$$

or $$W_u = \frac{6M_p}{L}$$

Kinematic method

$$W_u \frac{L}{2}\theta = M_p\theta + M_p\,(\theta + \theta)$$

or $$W_u = \frac{6M_p}{L}$$

Static method (Fig. 3.24)

$$\frac{W_u ab}{L} =$$

or $$W_u = \frac{L + b}{ab} M_p$$

Kinematic method

$$W_u a\theta = M_p\theta + M_p(\theta + \theta_1)$$

or $$= M_p\theta + M_p\left(\theta + \frac{a}{b}\theta\right)$$

or $$= \frac{b + (a + b)}{b} M_p\theta$$

or $$W_u = \frac{L + b}{ab} M_p$$

Solved Examples

Example 3.1 Find the shape factors for the following sections:

(a) Square of side a with its diagonal parallel to the zz-axis.

(b) A diamond section with unequal diagonals, the shorter being b and longer h; the shorter diagonal placed parallel to the zz-axis.

(c) Hollow tube section of external diameter D and internal diameter d.

(d) Triangular section of base b and height h.

(e) Tee-section as shown in Fig. Ex. 3.1 (e).

Solution

(a) Refer Fig. Ex. 3.1(a).

$$AC = \sqrt{a^2 + a^2} = \sqrt{2}a$$

$$BE = \frac{BD}{2} = \frac{AC}{2} = \frac{\sqrt{2}a}{2} = \frac{a}{\sqrt{2}}$$

Moment of inertia about zz-axis,

$$I_z = \frac{2\sqrt{2}}{12}\, a \left(\frac{a}{\sqrt{2}}\right)^3 = \frac{a^4}{12}$$

Elastic section modulus,

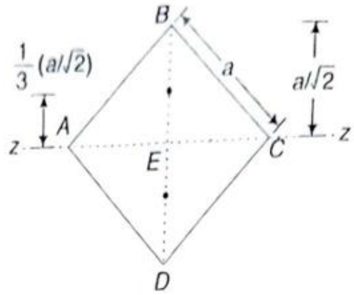
$$Z_{ez} = \frac{a^4/12}{a/\sqrt{2}} = \frac{a^3}{6\sqrt{2}}$$

Plastic section modulus,

$$Z_{pz} = \frac{A}{2}\,(\bar{y}_1 + \bar{y}_2)$$

$$= \frac{a^2}{2} \times \left(\frac{a}{3\sqrt{2}} + \frac{a}{3\sqrt{2}}\right) = \frac{a^3}{3\sqrt{2}}$$

Shape factor $S = \dfrac{Z_{pz}}{Z_{ez}} = \dfrac{a^3/3\sqrt{2}}{a^3/6\sqrt{2}} = 2$



(a)

Fig. Ex. 3.1

(b) (Ref. Fig. Ex. 3.1(b))

Moment of inertia about zz-axis,

$$I_z = 2 \times \frac{1}{12}\, b \left(\frac{h}{2}\right)^3 = \frac{bh^3}{48}$$

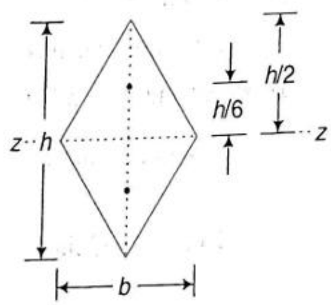
Elastic section modulus,

$$Z_{ez} = \frac{bh^3/48}{h/2} = \frac{bh^2}{24}$$

Plastic section modulus,

$$Z_{pz} = \frac{A}{2}\,(\bar{y}_1 + \bar{y}_2) = \frac{1}{2} \times \left(\frac{bh}{2}\right) \times \left(\frac{h}{6} + \frac{h}{6}\right) = \frac{bh^2}{12}$$

Shape factor $S = \dfrac{bh^2/12}{bh^2/24} = 2$



(b)

Fig. Ex. 3.1

(c) Refer Fig. Ex. 3.1(c)

Moment of inertia about zz-axis,

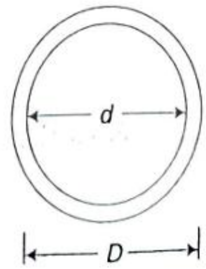
$$I_z = \frac{\pi}{64}\,(D^4 - d^4)$$

Elastic section modulus,

$$Z_{ez} = \frac{\pi}{64} \times \frac{(D^4 - d^4)}{D/2} = \frac{\pi}{32} \times \frac{D^4 - d^4}{D}$$

Plastic section modulus, $Z_{pz} = \dfrac{A}{2}\,(\bar{y}_1 + \bar{y}_2)$

$$A = \frac{\pi}{4}(D^2 - d^2)$$



(c)

Fig. Ex. 3.1

Solution

(a) Refer Fig. Ex. 3.1(a).

$$AC = \sqrt{a^2 + a^2} = \sqrt{2}a$$

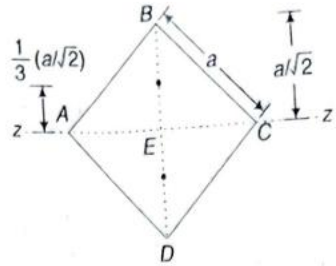
$$BE = \frac{BD}{2} = \frac{AC}{2} = \frac{\sqrt{2}a}{2} = \frac{a}{\sqrt{2}}$$

Moment of inertia about zz-axis,

$$I_z = \frac{2\sqrt{2}}{12} a \left(\frac{a}{\sqrt{2}}\right)^3 = \frac{a^4}{12}$$

Elastic section modulus,

$$Z_{ez} = \frac{a^4/12}{a/\sqrt{2}} = \frac{a^3}{6\sqrt{2}}$$



(a)
Fig. Ex. 3.1

Plastic section modulus,

$$Z_{pz} = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

$$= \frac{a^2}{2} \times \left(\frac{a}{3\sqrt{2}} + \frac{a}{3\sqrt{2}}\right) = \frac{a^3}{3\sqrt{2}}$$

$$\text{Shape factor } S = \frac{Z_{pz}}{Z_{ez}} = \frac{a^3/3\sqrt{2}}{a^3/6\sqrt{2}} = 2$$

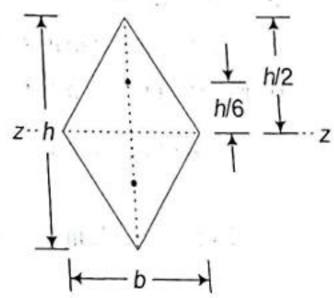
(b) (Ref. Fig. Ex. 3.1(b))

Moment of inertia about zz-axis,

$$I_z = 2 \times \frac{1}{12} b \left(\frac{h}{2}\right)^3 = \frac{bh^3}{48}$$

Elastic section modulus,

$$Z_{ez} = \frac{bh^3/48}{h/2} = \frac{bh^2}{24}$$



(b)
Fig. Ex. 3.1

Plastic section modulus,

$$Z_{pz} = \frac{A}{2}(\bar{y}_1 + \bar{y}_2) = \frac{1}{2} \times \left(\frac{bh}{2}\right) \times \left(\frac{h}{6} + \frac{h}{6}\right) = \frac{bh^2}{12}$$

$$\text{Shape factor } S = \frac{bh^2/12}{bh^2/24} = 2$$

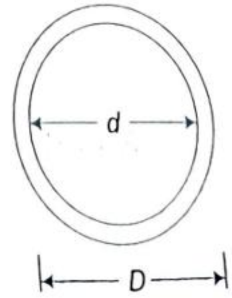
(c) Refer Fig. Ex. 3.1(c)

Moment of inertia about zz-axis,

$$I_z = \frac{\pi}{64}(D^4 - d^4)$$

Elastic section modulus,

$$Z_{ez} = \frac{\pi}{64} \times \frac{(D^4 - d^4)}{D/2} = \frac{\pi}{32} \times \frac{D^4 - d^4}{D}$$



(c)
Fig. Ex. 3.1

Plastic section modulus, $Z_{pz} = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$

$$A = \frac{\pi}{4}(D^2 - d^2)$$

$$\bar{y}_1 = \bar{y}_2 = \frac{\frac{1}{2} \times \left[\frac{\pi D^2}{4}\left(\frac{2}{3} \times \frac{D}{\pi}\right) - \frac{\pi d^2}{4}\left(\frac{2}{3} \times \frac{d}{\pi}\right)\right]}{\frac{1}{2} \times \left[\frac{\pi}{4}(D^2 - d^2)\right]} = \frac{2}{3\pi} \times \frac{(D^3 - d^3)}{(D^2 - d^2)}$$

$$Z_{pz} = \frac{1}{2} \times \frac{\pi}{4}(D^2 - d^2)\left[2 \times \frac{2}{3\pi} \times \frac{(D^3 - d^3)}{(D^2 - d^2)}\right] = \frac{1}{6}(D^3 - d^3)$$

$$\text{Shape factor} = \frac{Z_{pz}}{Z_{ez}} = \frac{\frac{1}{6}(D^3 - d^3)}{\frac{\pi}{32}\left(\frac{D^4 - d^4}{D}\right)} = \frac{32}{6\pi} \times \frac{D^3\left(1 - \frac{d^3}{D^3}\right)}{D^3\left(1 - \frac{d^4}{D^4}\right)} = \frac{32}{6\pi} \times \frac{\left(1 - \frac{d^3}{D^3}\right)}{\left(1 - \frac{d^4}{D^4}\right)}$$

Let $k = \frac{d}{D}$

$$\text{Shape factor} = \frac{32}{6\pi} \times \frac{(1 - k^3)}{(1 - k^4)} = 1.7\frac{(1 - k^3)}{(1 - k^4)}$$

when $D \approx d$, $k = 3/4$

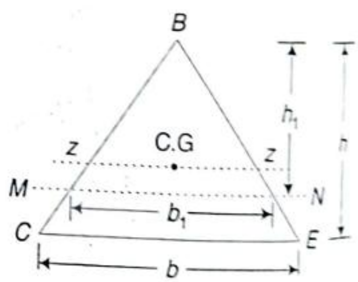
Shape factor = 1.27

As a special case, when $d = 0$, i.e., for a circular plate, $k = 1$ shape factor = 1.7

(d) Refer Fig. Ex. 3.1(d).

Moment of inertia, $I_z = \frac{bh^3}{36}$

Section modulus, $Z_{ez} = \frac{bh^3/36}{(2/3)h} = \frac{bh^2}{24}$



(d)

Fig. Ex. 3.1

Plastic section modulus, $Z_{pz} = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$

Let MN be the equal area axis; then

$$\frac{1}{2} b_1 h_1 = \frac{1}{2} \times \left(\frac{bh}{2}\right) \Rightarrow b_1 h_1 \times \left(\frac{bh}{2}\right) \quad (1)$$

$$\frac{h_1}{b_1} = \frac{h}{b} \quad (2) \quad \text{(From similar } \Delta^s \text{ } BMN \text{ and } BCE)$$

From Eqs. (1) and (2),

$$h_1 = \frac{h}{\sqrt{2}}$$

and

$$b_1 = b/\sqrt{2}$$

$$A = \frac{bh}{2}$$

$$\bar{y}_1 = \frac{h_1}{3} = \frac{h}{3\sqrt{2}}$$

$$\bar{y}_2 = \left(\frac{h - h_1}{3}\right) \times \left(\frac{b_1 + 2b}{b_1 + b}\right)$$

$$= \left(\frac{h - \frac{h}{\sqrt{2}}}{3}\right) \times \left(\frac{2b + \frac{b}{\sqrt{2}}}{b + \frac{b}{\sqrt{2}}}\right) = \frac{h(\sqrt{2} - 1)}{3\sqrt{2}} \times \frac{(2\sqrt{2} + 1)}{(\sqrt{2} + 1)}$$

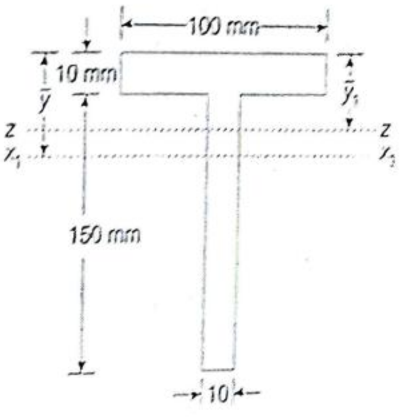
$$= \frac{0.4643h}{3}$$

$$Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2) = \frac{1}{2} \times \frac{bh}{2} \times \left[\frac{h}{3} \times \frac{1}{\sqrt{2}} + \frac{h}{3} \times 0.4643\right]$$

$$= \frac{bh^2}{12} \times [0.7072 + 0.4643] = 0.0976\ bh^2$$

$$\text{Shape factor } S = \frac{Z_{pz}}{Z_{ez}} = \frac{0.0976\ bh^2}{bh^2/24} = 2.343$$

(e) Refer Fig. 3.1(e).



(e)

Fig. Ex. 3.1

Let the distance of the neutral axis from the top be $\bar{y}$.

$$\bar{y} = \frac{100 \times 10 \times 10/2 + 150 \times 10 \times (75 + 10)}{100 \times 10 + 150 \times 10} = 53 \text{ mm}$$

Moment of inertia about zz-axis,

$$I_z = \frac{100 \times 10^3}{12} + 100 \times 10 \times (53 - 5)^2 + \frac{10 \times 150^3}{12} + 10 \times 150 \times (85 - 53)^2$$

$$= 666.0833 \times 10^4 \text{ mm}^4$$

$$\text{Section modulus, } Z_{ez} = \frac{I_z}{y_{max}} = \frac{666.0833 \times 10^4}{(160 - 53)} = 62.250 \times 10^3 \text{ mm}^3$$

Let x_1-x_1 be the equal area axis at a distance y_1 from top.

Total area of section, $A = 100 \times 10 + 150 \times 10 = 2500$ mm^2

Then,

$$100 \times 10 + (y_1 - 10) \times 10 = 2500/2$$

$$\Rightarrow \qquad y_1 = 35 \text{ mm}$$

$$Z_{pz} = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

$\bar{y}_1$ = the distance of CG of the area above the equal area axis

$$= \frac{100 \times 10 \times (35 - 10/2) + 10 \times (35 - 10) \times (35 - 10)/2}{100 \times 10 + 10 \times (35 - 10)} = 26.5 \text{ mm}$$

$\bar{y}_2$ = the distance of CG of the area below the equal area axis

$$= \frac{(150 + 10) - 35}{2} = 62.5 \text{ mm}$$

Plastic section modulus, $Z_{pz} = \dfrac{2500}{2} \times (26.5 + 62.5) = 111250 \text{ mm}^3$

$$\text{Shape factor} = \frac{111250}{62.250 \times 10^3} = 1.7871$$

Example 3.2 Determine the plastic section modulus Z_{pz} and Z_{py} for ISMB 225@ 306.07 N/m and verify the result from the value given in Appendix IX.

Solution From *IS Handbook No. 1* the properties of ISMB 225 @ 306.07 N/m are:

Area of the section, $A = 3972 \text{ mm}^2$ (Area includes fillets)

Overall depth, $d = 225$ mm

Width of flange, $b = 110$ mm

Thickness of flange, $t_f = 11.8$ mm

Thickness of web, $t_w = 6.5$ mm

From Appendix IX:

Plastic section modulus, $Z_{pz} = 348.27 \times 10^3 \text{ mm}^3$

Figure Ex. 3.2(b) shows the idealised cross section with rectangular plates of I-section of Fig. Ex. 3.2(a).

Cross sectional area $= 2 \times (110 \times 11.8) + (225 - 2 \times 11.8) \times 6.5 = 3905.1 \text{ mm}^2$

(a) From Eq. (12); Refer to [Fig. Ex. 3.2(b)].

$$Z_{pz} = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

$$\bar{y}_1 = \bar{y}_2 = \frac{\left(\frac{225}{2} - 11.8\right) \times 6.5 \times \left(\frac{225}{4} - \frac{11.8}{2}\right) + 11.8 \times 110 \times \left(100.7 + \frac{11.8}{2}\right)}{\left(\frac{225}{2} - 11.8\right) \times 6.5 + 11.8 \times 110} = 87.74 \text{ mm}$$

$$Z_{pz} = \frac{3905.1}{2} \times (87.74 + 87.74) = 342.64 \times 10^3 \text{ mm}^3$$

Alternatively, refer to Fig. Ex. 3.2(c);

$\bar{y}_1 = 201.4/4 = 50.35$ mm, $\bar{y}_2 = 225/2 - 11.0/2 = 106.6$ mm

(d) From Eq. (18),

$$z_{pw} = \frac{t_f b_f^2}{2} + \frac{t_w^2(d - 2t_f)}{4}$$

$$= \frac{11.8 \times 110^2}{2} + \frac{6.5^2 \times (225 - 2 \times 11.8)}{4} = 73.52 \times 10^3 \text{ mm}^3$$

Example 3.3 Find out the collapse load for the cantilever shown in Fig. Ex. 3.3.

Solution The beam shown in Fig. Ex. 3.3 is of a non-uniform cross section. A plastic hinge can form at the point where the section changes. A plastic hinge can also form at the fixed support.

Case I Plastic hinge at the point, where the cross-section changes.

$$\text{External work done} = \text{Load} \times \text{Deflection}$$
$$= 2W_u \times \Delta$$
$$= 2W_u \frac{L}{2}\theta$$
$$\text{Internal work done} = \text{Moment} \times \text{Rotation}$$
$$= M_p\theta$$

By the principle of virtual work,

$$\text{External work done} = \text{Internal work done}$$

$$\therefore \quad 2W_u \frac{L}{2}\theta = M_p\theta$$

or $$W_u = \frac{M_p}{L}$$

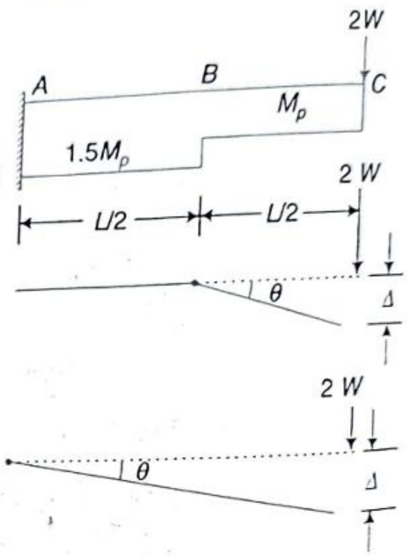


Fig. Ex. 3.3

Case II Plastic hinge at the support.

$$\text{External work done} = \text{Load} \times \text{Deflection}$$
$$= 2W_u \times \Delta = 2W_u L\theta$$
$$\text{Internal work done} = 1.5M_p\theta$$

By the principle of virtual work,

$$\text{External work done} = \text{Internal work done}$$
$$2W_u L\theta = 1.5 M_p\theta$$

or $$W_u = 0.75 \frac{M_p}{L}$$

Therefore, the collapse load for the beam is $0.75 M_p/L$.

Note

In case I, the plastic hinge forms at B. The value of the plastic moment M_p will reach prior to 1.5 M_p and the hinge will form in the limb BC. Hence, in calculation the value of plastic moment M_p is considered.

Example 3.4 A fixed ended beam is subjected to load W at 1/3rd span as shown in Fig. Ex. 3.4. Estimate the collapse load.

Solution To convert the beam into mechanism three plastic hinges are required. There can be two cases.

9

Case I Two plastic hinges at the supports, and one plastic hinge at the point where cross section changes. One plastic hinge will form at B in the limb BC and here the plastic moment will be M_p.

$$\text{External work done} = \text{Load} \times \text{Deflection}$$
$$= W_u \frac{L}{3}\theta$$

$$\text{Internal work done} = \text{Moment} \times \text{Rotation}$$
$$= 2M_p\theta + M_p(\theta + \theta) + M_p\theta$$
$$= 5M_p\theta$$

By the principle of virtual work,

External work done = Internal work done

$$W_u\frac{L}{3}\theta = 5M_p\theta$$

or $$W_u = 15\frac{M_p}{L}$$

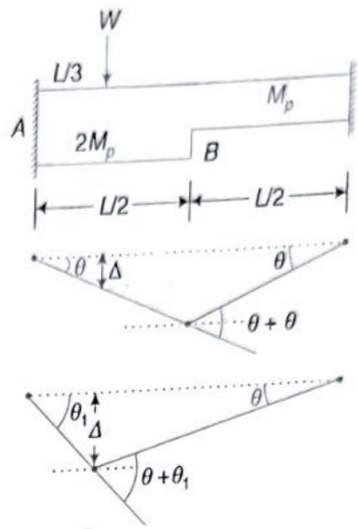


Fig. Ex. 3.4

Case II Two plastic hinges can develop at the supports and one below the concentrated load.

$$\Delta = \frac{L}{3}\theta_1 = \frac{2}{3}L\theta \qquad \text{or} \qquad \theta_1 = 2\theta$$

$$\text{External work done} = \text{Load} \times \text{Deflection}$$
$$= W_u\frac{L}{3}\theta_1 = W_u\frac{L}{3}\,2\theta = \frac{2}{3}W_u\,L\theta$$

$$\text{Internal work done} = \text{Moment} \times \text{Rotation}$$
$$= 2M_p\theta_1 + 2M_p(\theta + \theta_1) + M_p\theta$$
$$= 2M_p \times 2\theta + 2M_p(\theta + 2\theta) + M_p\theta = 11M_p\theta$$

By the principle of virtual work,

External work done = Internal work done

$$\frac{2}{3}W_uL\theta = 11M_p\theta$$

or $$W_u = 16.5\frac{M_p}{L}$$

The collapse load will be the minimum of the above two values of ultimate loads W_u. Therefore, the value of the collapse load for the above beam is $15\frac{M_p}{L}$.

Example 3.5 Find out the collapse load for a propped cantilever subjected to a uniformly distributed load w/unit length, as shown in Fig. Ex. 3.5(a).

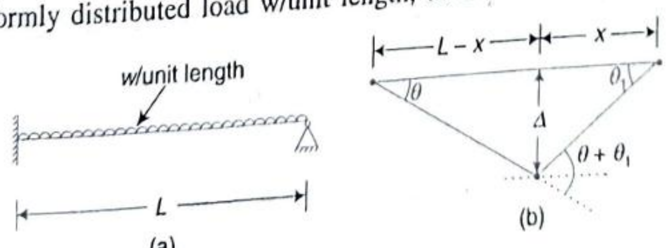
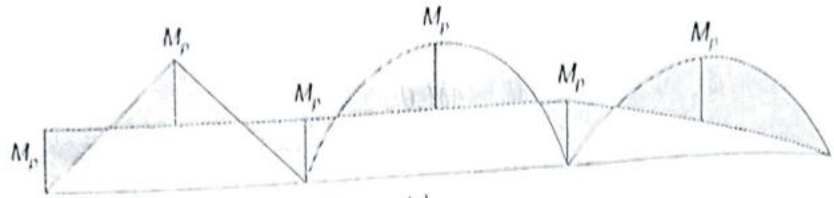


Fig. Ex. 3.5

Solution For a propped cantilever subjected to uniformly distributed load the plastic hinge does not form at the centre of span but lies off the centre and towards the propped end. Therefore, it becomes essential to first locate the position of the plastic hinge. Let us assume that it forms at a distance x from the propped end.

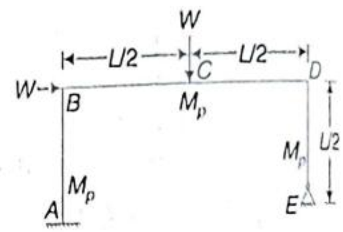
The collapse load is the least of the three ultimate loads. Hence, the collapse load $W_u = \dfrac{8M_p}{L}$. The plastic moment diagram is shown in Fig. Ex. 3.10(c).



(c)

Fig. Ex. 3.10

Example 3.11 Find out the collapse load for a portal frame shown in Fig. Ex. 3.11(a). The beams and columns are of same cross section. Also draw the plastic moment diagram.



(a)

Fig. Ex. 3.11

Solution In the case of portal frames, the first step is to find out the number of independent mechanisms and then to form all the possible combined mechanisms.

Possible location for plastic hinges are A, B, C and D.

Number of possible plastic hinges,

$N = 4$

Degree of redundancy, $r = 5 - 3 = 2$

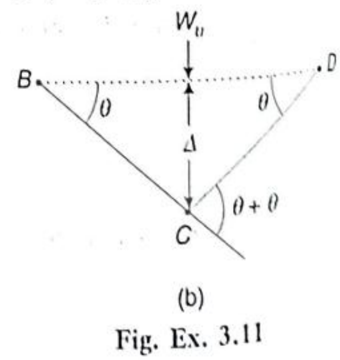
Number of possible independent mechanisms, $n = N - r$

$= 4 - 2 = 2$

The two independent mechanisms are

1. Beam mechanism
2. Sway mechanism

These two independent mechanisms can form a combined mechanism also.



(b)

Fig. Ex. 3.11

Beam mechanism The ends B and D of the beam BD are fixed joints. Therefore, the beam BD act as a fixed ended beam. The possible location of plastic hinges are B, D and C (below the concentrated load) as shown in Fig. Ex. 3.11(b).

$$\text{External work done} = \text{Load} \times \text{Deflection}$$

$$= W_u \times \frac{L}{2}\theta$$

$$\text{Internal work done} = M_p\theta + M_p(\theta + \theta) + M_p\theta$$

$$= 4M_p\theta$$

By principle of virtual work,

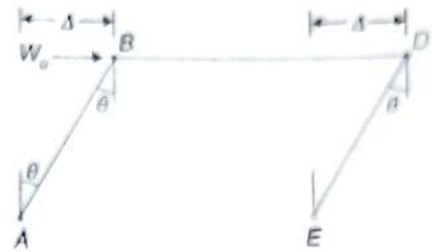
External work done = Internal work done

$$W_u \frac{L}{2}\theta = 4M_p\theta$$

or $$W_u = 8\frac{M_p}{L}$$

Sway mechanism The columns AB and ED are of the same length. These deflect by the same amount Δ, hence the rotations at the ends are also same, i.e. θ.

Possible locations of plastic hinges are A, B and D (E being a mechanical hinge, a plastic hinge will not develop) as shown in Fig. Ex. 3.11(c).



(c)

Fig. Ex. 3.11

External work done = Load × Deflection = $W_u \times 0.5L\theta$

Internal work done = Moment × Rotation

$$= M_p\theta + M_p\theta + M_p\theta = 3M_p\theta$$

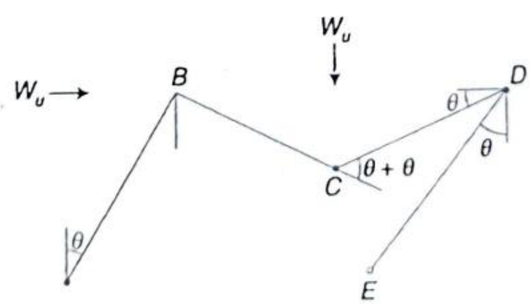
By the principle of virtual work,

External work done = Internal work done

$$W_u \times 0.5L\theta = 3M_p\theta$$

or $$W_u = \frac{6M_p}{L}$$

Combined mechanism The possible locations of plastic hinges are at A, C and D. A plastic hinge does not develop at B because as the link BC of the beam rotates clockwise, link BA also rotates clockwise by the same amount, therefore there is no change of angle at joint B and the two links still make an angle of 90°. The moment at B, in the beam link BC, is hogging while that in column AB is sagging. Therefore, the value of the moment never reaches the plastic moment and the plastic hinge will not develop. On the other hand, at D the beam link DC rotates anti-clockwise and the column DE rotates clockwise. The net change in the angle between the two links is $(\theta + \theta)$ and a plastic hinge may, therefore, develop at D. Refer Fig. Ex. 3.11(d).



(d)

Fig. Ex. 3.11

External work done = Σ(Load × Deflection)

$$= W_u \times 0.5L\theta + W_u \times \frac{L}{2}\theta$$

12

$$\text{Internal work done} = \Sigma(\text{Moment} \times \text{Rotation})$$
$$= M_p\theta + M_p(\theta + \theta) + M_p(\theta + \theta) = 5M_p\theta$$

By the principle of virtual work,

$$\text{External work done} = \text{Internal work done}$$
$$W_u L\theta = 5M_p\theta$$

or
$$W_u = 5\frac{M_p}{L}$$

As the collapse load is being worked out by the kinematic theorem, it is the least of the three ultimate loads corresponding to the various mechanisms.

$$W_u = \frac{5M_p}{L}$$

Plastic moment diagram The moment diagram is drawn on the tension side (See Fig. 3.11(e)). E is a mechanical hinge so the moment value is zero. D and C are the points where fully plastic moments are developed so ordinates, equal to M_p, can be drawn. At B, the value of moment is less than the plastic moment and is not known. To calculate the value of the moment at B, the value of the elastic moment in terms of the plastic moment at C is calculated. The moment is assumed to vary linearly and by this the value of moment at B is found. The process is illustrated below. The value of moment at A is also fully plastic moment M_p.

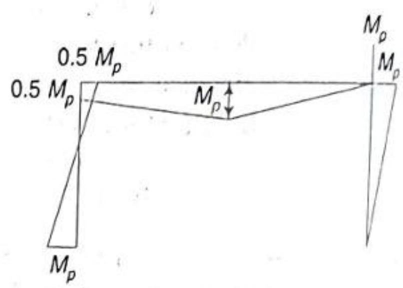


Fig. Ex. 3.11

For the beam BD,

$$M = \frac{WL}{4}$$
$$= 5\frac{M_p}{L} \times \frac{L}{4}$$
$$= \frac{5}{4}M_p = 1.25\ M_p$$

Thus, the ordinate is plotted as measured from the bottom of the plastic moment ordinate at C. From D to C (its span) the fall in plastic moment is of $0.75M_p$. For next half span, i.e. from C to B, it falls further by $0.75M_p$. Therefore, the net ordinate of the plastic moment for the beam BD is $0.5M_p$. For equilibrium, the plastic moment in column at B will also be $0.5M_p$.

Example 3.12 Compute the collapse load for the portal frame as shown in Fig. Ex. 3.12(a).

Solution The possible locations of the plastic hinges are at A, B, C, D and E.

Number of possible plastic hinges, $N = 5$

Introduction to Plastic Analysis

Plastic analysis is a method of structural analysis used to determine the ultimate load-carrying capacity of structures by allowing the material to yield and redistribute internal forces before collapse.

In traditional elastic analysis, stresses are limited to the elastic range. However, in plastic analysis, structures are allowed to reach the plastic stage, where sections can undergo large rotations without increase in bending moment.

This method is widely used in the design of steel structures because steel possesses high ductility and can sustain large plastic deformations.

Objectives of Plastic Analysis

- Determine the collapse load of a structure.
- Utilize the full strength of materials.
- Achieve economical design.
- Allow moment redistribution in indeterminate structures.

Theory of Plastic Analysis

The theory of plastic analysis is based on the plastic behavior of materials and the formation of plastic hinges in structural members.

When a beam is subjected to increasing load:

1. Initially, stresses are within elastic range.
2. When the stress reaches the yield stress, yielding begins.
3. As the load increases further, more portions of the section yield.
4. When the entire cross-section yields, the section reaches plastic moment capacity (Mp).
5. At this stage, a plastic hinge forms.

A plastic hinge allows rotation without increase in bending moment.

Plastic Moment Capacity

Plastic moment is the **maximum moment a section can resist** when the entire cross-section has yielded.

$$M_p = f_y \times Z_p$$

Where:

- M_p= Plastic moment
- f_y= Yield stress of material
- Z_p= Plastic section modulus

Plastic Hinge

A **plastic hinge** is formed when the bending moment at a section reaches the plastic moment.

Characteristics:

- Allows rotation
- Moment remains constant
- Not a real hinge but behaves like one

Collapse Mechanism

A structure collapses when sufficient plastic hinges form to convert the structure into a **mechanism**.

$$\text{Number of plastic hinges required} = r + 1$$

Where

r= degree of static indeterminacy.

Example:

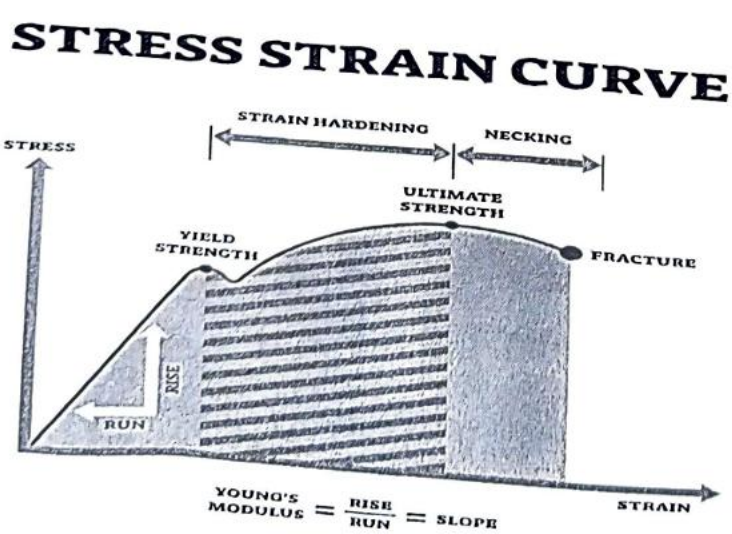
Structure	Indeterminacy	Hinges Required
Simply supported beam	0	1
Fixed beam	2	3
Portal frame	3	4

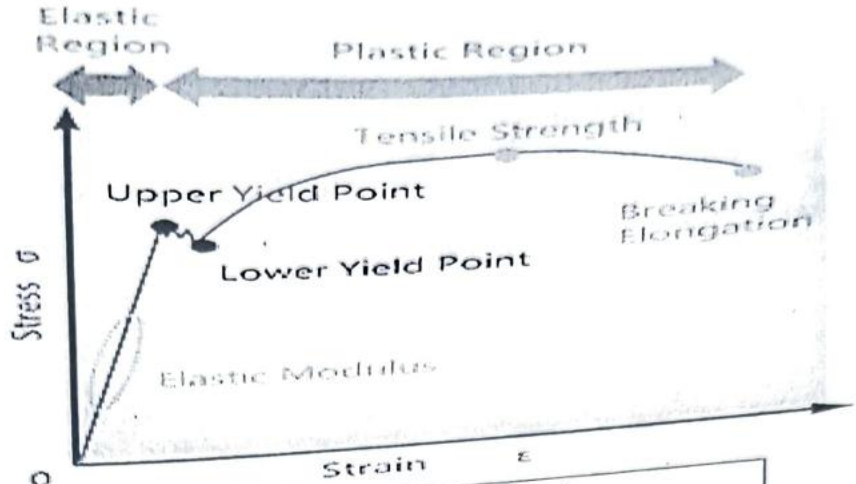
Assumptions of Plastic Theory

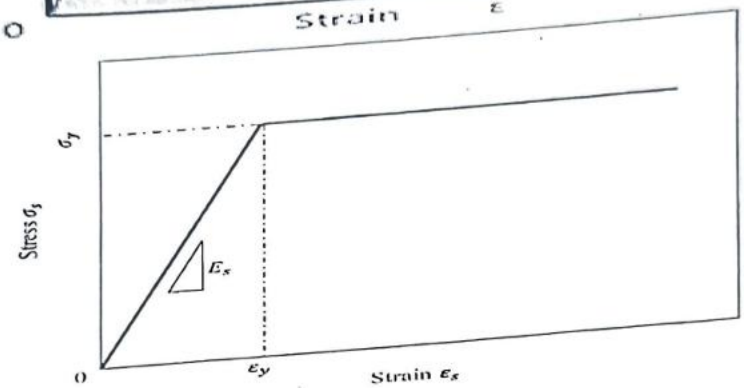
The theory of plastic analysis is based on the following assumptions:

1. Material is homogeneous and isotropic.
2. Material follows elastic–perfectly plastic behavior.
3. Plane sections remain plane after bending.
4. Shear deformation is neglected.
5. Sections must be compact to prevent local buckling.
6. Sufficient ductility exists so that plastic hinges can rotate.
7. Strain hardening is neglected.

Stress–Strain Behaviour of Structural Steel







The stress–strain curve of steel consists of several stages:

1. Elastic Region

- Stress is proportional to strain.
- Obeys Hooke's law.

2. Yield Point

- Stress remains constant while strain increases.
- Start of plastic deformation.

3. Plastic Region

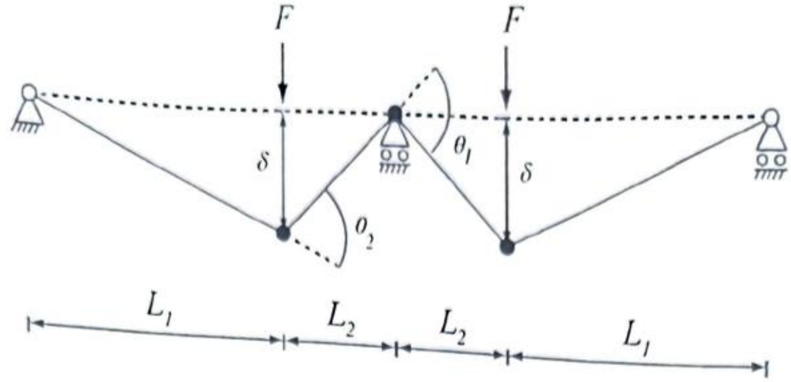
- Large deformation occurs.
- Material can rotate without increase in stress.

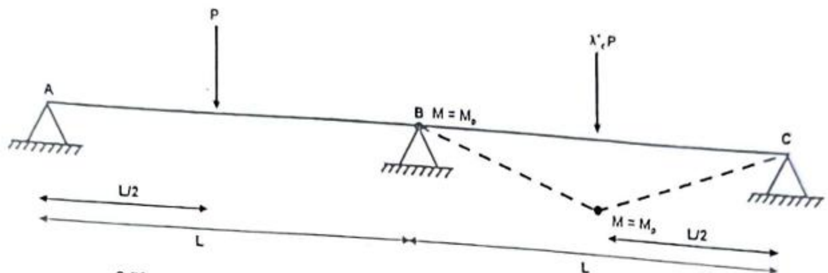
4. Strain Hardening Region

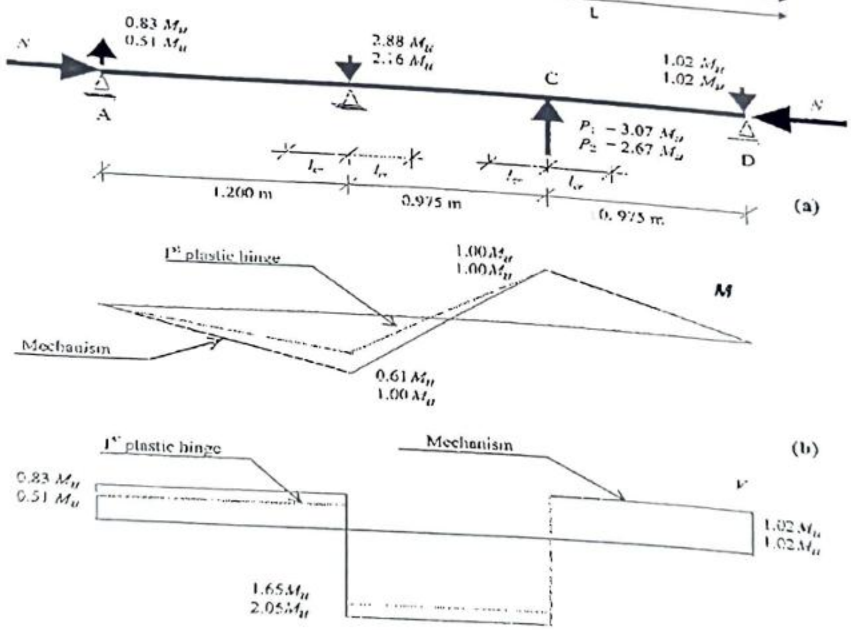
- Stress increases again after large strain.
- Usually ignored in plastic analysis.

For simplification, the curve is idealized as **elastic–perfectly plastic**.

Design of Continuous Beams Using Plastic Design







In continuous beams, plastic analysis is used to determine the collapse load by identifying the locations of plastic hinges.

Steps in Plastic Design

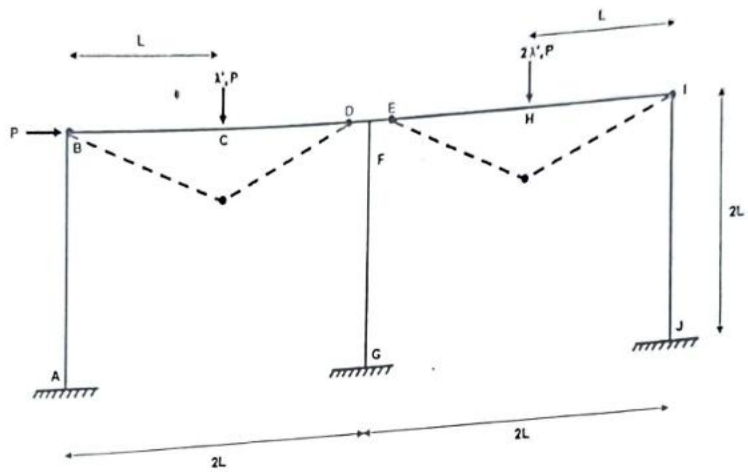
1. Determine degree of static indeterminacy.
2. Calculate number of plastic hinges required.
3. Assume possible hinge locations.

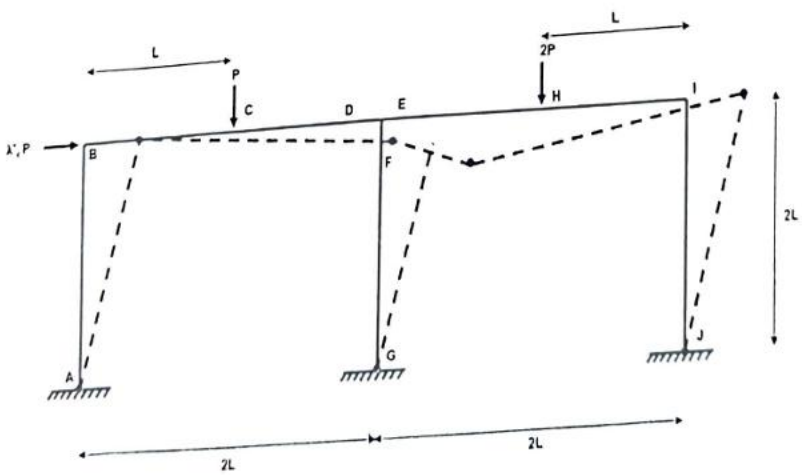
4. Use virtual work method or equilibrium method.
5. Calculate collapse load.

Typical hinge locations:

- Midspan
- Interior supports
- Fixed supports

Design of Portal Frames Using Plastic Design





Portal frames can fail through different collapse mechanisms.

Types of Mechanisms

1. Beam Mechanism

- Hinges form in the beam only.

2. Sway Mechanism

- Hinges form in the columns.
- Occurs under lateral loads like wind.

3. Combined Mechanism

- Hinges form in both beams and columns.

For a single bay portal frame:

- Degree of indeterminacy = 3
- Plastic hinges required = 4

Advantages of Plastic Design

- Economical design
- Utilizes full strength of material
- Realistic prediction of collapse load
- Allows redistribution of bending moments

Limitations of Plastic Design

- Applicable mainly to ductile materials
- Large deflections occur before collapse
- Not suitable for brittle materials
- Requires careful detailing to prevent local buckling