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SCHEME OF EVALUATION
(Scoring Indicators)

Revison: 2015
Course Title: Structural Design II
Course Code: 6013

Qn No	Scoring Indicators	Split Score	Total Score
	PART A		
I **1)**	Mechanical Properties – Yield stress, ultimate stress, percentage of elongation, Notch toughness ***Any two Mechanical properties***		2
2)	**Gauge of the bolt** It is the centre to centre distance between two consecutive bolts measured in a direction perpendicular to the applied forces/stresses		2
3)	The net area is the gross area of a tensile member taken perpendicular to the load, minus area for holes.		2
4)	Shape factor is the ratio of plastic moment M_p to yield moment M_y $S = \frac{M_P}{M_y} = \frac{Z_P f_y}{Z_e f_y} = \frac{Z_P}{Z_e}$		2
5)	a) Dead load b) Live load or imposed load c) Wind load d) Earthquake load e) Erection load f) Snow load ***Any four***	0.5 0.5 0.5 0.5	2
	PART B		
II **1)**	Advantages of welded joints • As no holes are required for welding, the structural members are more effective in taking load • The overall weight of structural steel required is reduced by the use of welded joints • Welded joints are economical as less labour and material are required for a joint • Welded connections look better than the usually bulky bolted joints • The speed of fabrication is higher with welding process	**3**	

	• Any shape of joint can be made with ease • The welding process requires less space than the bolting process • Complete rigid joints can be provided with the welding process • No noise is produced in the welding process as in the riveting process *Any 3 points*		
	Disdvantages of welded joints • Skilled labour and electricity are required for welding • Internal stresses and warping are produced due to uneven heating and cooling • Welded joints are more brittle and therefore their fatigue strength is less than the members joined • Defects like internal air pockets, slag inclusion and incomplete penetration are difficult to detect *Any 3 points*	**3**	**6**
2)	Welds are classified as a) According to their position • Flat • Horizontal • Vertical • Overhead	**2**	
	b) According to the type of weld • Butt weld / groove weld • Fillet weld • Slot weld • Plug weld	**2**	**6**
	c) According to type of joint • Butt joint • Lap joint • Tee joint • Edge joint • Corner joint	**2**	

3) The effective length L_e for different end conditions in terms of actual length (L) are listed in the following table

Support Conditions	**Effective length , L_e**		
a)Both ends hinged/ pinned	$L_e = L$	**1.5**	
b) one end hinged and other end fixed	$L_e = L/\sqrt{2}$	**1.5**	**6**
c) Both ends fixed	$L_e = L/2$	**1.5**	
d) one end fixed and other end free	$L_e = 2\ L$	**1.5**	

All four conditions and corresponding effective length

4)

Uses of lug angles

Lug angle is small piece of angle used to connect outstand legs of the members to the gusset plate.

3

- The purpose of lug angle is to reduce the length of connection to the gusset plate and to reduce shear lag effect.
- If lug angle is used then the unconnected length of main angle behave like a connected leg and entire cross section area of the angle become effective in resisting tension. So if lug angle is used, then efficiency of the tension member increases because it reduces shear lag effect.
- If lug angle is used the resultant reaction at bolt location 1 and 2 pass through CG of cross section. Since action and reaction pass through CG of angle, stress and strain distribution are uniform hence no shear lag

Relevant points – 3 marks

Specifications

The India Standard IS 800 specifies the following for the design of lug angles:

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	1. Lug angles connecting a channel-shaped member should as far as possible, be disposed symmetrically with respect to the section of the member. 2. In the case of angle members, the lug angles and their connections to the gusset or any other supporting member should be capable of developing a strength not less than 20% in excess of the force in the outstanding leg of the angle and the attachment of the lug angle member should be capable of developing a strength 40% in excess of that force. 3. In the case of channel sections, the lug angles and their connections to the gusset or any other supporting member should be capable of developing a strength of not less than 10% in excess of the force not accounted for by the direct connection of the member, and the attachment of the lug angles to the member should be capable of developing a strength 20% in excess of that force. 4. In no case should fewer than two bolts or rivets be used for attaching the lug angle, to the gusset or another supporting member. 5. The effective connection of the lug angle should, as far as possible, terminate at the end of the member connected and the fastening of the lug angle to the member should preferably start in advance of the direct connection of the member to the gusset or other supporting member. 6. Where lug angles are used to connect an angle member, the whole area of the member should be taken as effective, i.e. A(net) = Gross Area – deduction for holes ***Any three specifications – 3 marks***	**3**	**6**
5)	**A). Plastic Section** • The plastic section is capable of developing plastic moment and plastic hinges with sufficient rotation capacity *without local buckling*. • These sections can take load until a failure mechanism, which is beyond the plastic moment, by redistribution of moments.	**1.5**	
	B.Compact Section • These sections can develop the plastic moment but the plastic	**1.5**	

	hinge rotation capacity is inadequate because of local buckling. Hence, it fails even before developing a plastic mechanism. • There is *no redistribution of moments* here. • The width to thickness ratio of plate elements is greater than that of Plastic sections.		
	C. Semi-Compact Section • The extreme fibre stress attains yield stress but cannot develop plastic moment. • The maximum moment it can attain is *Yield Moment*. • The member fails by local buckling even before it forms a mechanism. • The width to thickness ratio of plate elements is greater than that of Compact sections.	**1.5**	
	4. Slender Section • Even the extreme fibre does not reach yield stress in these sections. The width to thickness ratio is very high that the elements buckle locally even before reaching yield stress. • These sections *can not attain* the yield moment.	**1.5**	**6**
6)	**Stress reduction factor** • This factor as given in table 9, takes into consideration the slenderness ratio of the element and also the eccentricity of loading	**2**	
	Shape Modification factor • This factor takes into consideration , the shape of the unit , that is height to width ratio (as laid) and is given in table 10 • This factor is applicable for units of crushing strength upto 15 N/mm^2	**2**	**6**

	Area Reduction factor	2	
	• This factor takes into consideration smallness of the sectional area of the element and is applicable when the sectional area of the element is less than 0.2 m^2 • The factor , k =0.7 + 1.5 A , A being the area of section in m^2		
7)	Components of a roof truss		6
	a) Principal Rafters / Top Chord • Top chord members of a roof truss are called principal rafters • They support the roof covering through purlins • They are compression members may be subjected to shear and bending moment if the purlins are not placed at nodal points	1	
	b) Main tie / Bottom Chord • It is usually in tension and takes compression if reversal of loads occur due to wind load	1	
	c) Purlins • These are horizontal beam members that run parallel to the ridge and connect the trusses along the length of the ridge. Purlins are subjected to vertical loads due to dead and live loads and to loads normal to roof covering due to wind pressure. Therefore, purlins are subjected to biaxial bending.	1	
	d) Ridge line • Top line of the roof truss is called the ridge line **e) Eaves** • Bottom edge of the roof surface is called eaves	1	
	f) Roof coverings • Corrugated sheets of galvanized iron or asbestos cement are commonly used		
	g) Sag tie • A sag-tie is a member provided to support a long horizontal member of the bottom chord of a truss (normally in tension) to reduce the excessive deflection under its own self weight. • The forces in various members are either compressive or	1	

	tensile **Any six major components with functions (one or two points)**	**1**	
III **a)**	**PART C** **Advantages of steel structures over RCC Structures** • Speedy construction is possible in case of steel structures • Dismantling and reuse of steel structures will be possible • Modifications of sectional properties can be easily carried out in steel • Self weight of steel members is less comparec to RCC • Due to its high density , steel is completely non porous • Steel is dimensionally more durable than concrete • Steel will not warp, split, shrink or crack when exposed to the elements • Steel structures are more effective at withstanding earthquakes		**5**
b)	***Any five relevant points*** ***Solution*** Diameter of bolt , d=20 mm Diameter of hole , d_o= 20+2 = 22 mm For bolts of grade 4.6	**2**	
	f_{ub}= 4x100 = 400 MPa f_{yb}= 0.6 x 400 = 240 MPa Thickness of connecting plate = 10 mm For Fe 410 grade steel , f_u=410 MPa, γ_{mb} = 1.25 Minimum pitch , p = 2.5 x 20 = 50 mm Minimum end distance , e = 1.5 x 22= 33 mm	**1**	
	n=Design load (P)/Design strength of one bolt (V_{db}) Design load (P) = 1.5 x 375 = 562.5 kN	**1**	

	Lap Joint - **Design strength in shearing** The bolts will be in single shear Design shear strength of bolt $V_{dsb} = \frac{\frac{f_{ub}}{\sqrt{3}}(n_n A_{nb}+n_s A_{sb})}{\gamma_{mb}}$	1	
	Here $n_n = 1$, $n_s = 0$ (single shear assuming shear plane to be in threaded portion) $A_{sb} = (\pi/4)\ d^2 = 3.14 * \{20^2/4\} = 314.16\ mm^2$	1	
	$A_{nb} = 0.78\ A_{sb} = 0.78 \text{ x } 201.06 = 245.04\ mm^2$ $V_{dsb} = \frac{\frac{f_{ub}}{\sqrt{3}}(A_{nb})}{\gamma_{mb}} = \frac{\frac{400}{\sqrt{3}}(314.16)}{1.25} * 10^{-3}$ = 58.04kN	1	10
	- **Design strength in Bearing**	1	
	Design bearing strength , $V_{dpb} = 2.5 \text{ x } k_b \text{ x } d_b \text{ x } t \text{ x } f_u$ k_b is the least of the following - $\frac{e}{3d_o} = \frac{33}{3*22} = 0.50$ - $\frac{p}{3d_o} - 0.25 = \frac{50}{3*22} - 0.25 = 0.507$ - $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$ - 1	1	
	Select k_b= 0.50 $V_{dpb} =$ (2.5 x k_b x d_b x t x f_u)/ γ_{mb} = (2.5 x 0.50 x 20 x 16 x 410)/ 1.25 = **128 kN**	1	
	Design strength of bolt V_{db} = 58.04 kN **Number of bolts required , n = P/ V_{db} = 562.5 / 58.04 = 10 Nos**		

IV **a)**	**Pitch (p)** It is the distance between centres of two consecutive bolts measured along parallel to the direction of force or stress in a member	**2**	**5**
	End distance It is the distance from centre of bolt hole to the nearest edge of member or cover plate in the direction of stress or force	**1.5**	
	Edge distance It is the distance from centre of bolt hole to the nearest edge of member or cover plate at right angles to the direction of stress	**1.5**	
b)	ISA 100 x 75 x 10 mm $\frac{f_y}{\gamma_{mo}}$ =permissible stress = 150 N/mm^2 A =Area = 1650 mm^2	**1**	
	f_u= 410 N/mm^2 C_{xx} = 31.9 mm **1) Size of weld** S = T – 1.5 S = 10-1.5= 8.5 mm Adopt 8 mm	**1**	
	2) Throat thickness t = 0.7 x size of weld	**1**	

$t = 0.7 \times 8 = 5.6$ mm

3) **Design strength**

Design stress (f_{wd}) = $\dfrac{f_u}{\gamma_{mw}\sqrt{3}}$ **10**

γ_{mw} =1.25for shop weld

(f_{wd}) = $\dfrac{410}{1.25\sqrt{3}} = 189.37$ N/mm^2 **1**

Design strength = $L_w\, t\, f_{wd}$

= $L_w \times 5.6 \times (189.37)$

4) **Factored full design strength**

Full design strength = gross area x permissible stress **10**

Factored full design strength = $\dfrac{1.5 A_g f_y}{\gamma_{mo}}$ = FOS x full design strength **2**

Factored full design strength = $\dfrac{1.5 A_g f_y}{\gamma_{mo}}$

$= 1.5 * 1650 * 150$

$= 371250$ N

5. **length of weld (L_w)**

For length of weld

Design strength can be equated to factored full design strength

$$L_w\, t\, f_{wd} = \frac{1.5 A_g f_y}{\gamma_{mo}}$$

2

L_w x 5.6 x 189.37 = 371250

L_w = 350.07 mm

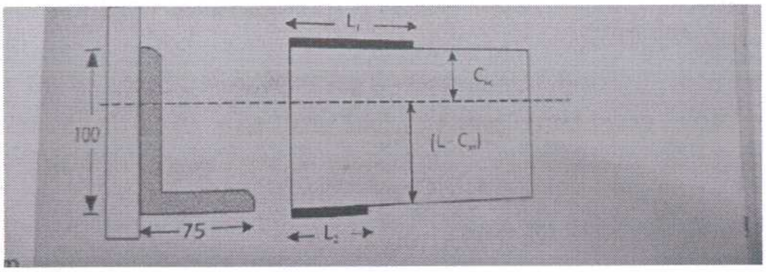
6) length of weld in top and bottom portion

$L_w = L_1 + L_2$

L_1 = length of weld in top portion

L_2 = length of weld in bottom portion

2



L= length of connected leg = 100 mm

	C_{xx} = centroid = 31.9 mm		6
	To make centre of gravity of weld to coincide with that of angle		
	$L_1 \times C_{xx} = L2 (L - C_{xx})$		
	$L_1 \times 31.9 = L2 (100 - 31.9)$	2	
	$L_1 = 2.134 L_2$		
	$L_w = L_1 + L_2 = 350.07$	2	
	$2.134 L_2 + L_2 = 350.07$		
	L_2 = **111.67 mm**		
	$L_1 = L_w - L_2 = 350.07 - 111.67$ = **238.39 mm**	2	
V a)	- **Gross area** : The total cross-sectional area of a tensile member taken perpendicular to the load, where no holes are provided		
	- **Net area:** the net area is the gross area of a tensile member taken perpendicular to the load, minus area for holes		
	- **Net effective area:** The effective area will be considered if there is a structural steel element that is subjected to tensile force, for which load is not transmitted to all parts		
	- As an example of the effective area, an angle acted upon by tension force, for which a bolted connection only exists in one leg, while the other leg has no bolts, hence the effective area will be less than the net area for that angle.	1	

b)	**Given data** Area of ISA 90x60x 6 mm = 8.65 mm^2 = 865 mm^2 Area of two angles = 2 x 865 = 1730 mm^2 Area of connected leg, Anc =2 x(90 – 6/2) 6 = 1044 mm2 Area of outstanding leg, Ago = 2 x(60-6/2) 6 = 684 mm2 - **Design strength due to yielding** $T_{dg} = \frac{A_g * f_y}{\gamma_{mo}} = \frac{1730*250}{1.1} = 393181.812 = \mathbf{393.182kN}$ (clause 6.2 IS 800, Page 32)	1	
	- **Design strength due to rupture of critical section,** $T_{dn} = \frac{0.9 * A_{nc} * f_u}{\gamma_{ml}} + \frac{\beta * A_{go} * f_y}{\gamma_{mo}}$ $\beta = 1.4 - 0.076 * (w/t) * (\frac{f_y}{f_u}) * (\frac{b}{L_c})$ w = 90 mm, t = 6 mm , $f_y = 250\ MPa$ $f_u = 410\ MPa$, b = 90 mm, L_c = 200 mm $\beta = 1.4 - 0.076 * (90/6) * (\frac{250}{410}) * (\frac{90}{200}) = 1.087$ $\gamma_{mo} = 1.1, \gamma_{ml} = 1.25$ A_{nc} =1044 mm^2 , A_{go}= 684 mm^2	3	9

$$T_{dn} = \frac{0.9 * A_{nc} * f_u}{\gamma_{ml}} + \frac{\beta * A_{go} * f_y}{\gamma_{mo}}$$

$$T_{dn} = \frac{0.9 * 1044 * 410}{1.25} + \frac{1.087 * 684 * 250}{1.1}$$

= 308188.8 + 168979.0 = 477167.8909 N = **477. 167 kN**

- **<u>Block Shear strength</u>**

Area under shear , $A_{vg} = A_{vn} = 2 \text{ x}200\ (4+4) = 3200 \text{ mm}^2$

Area under tension at failure 3

$A_{tg} = A_{tn} = 2 \text{ x } 90 \text{ x}4 = 720 \text{ mm}^2$

- Clause 6.4.1 IS 800, Page No : 33)

$$T_{db} = \frac{A_{avg} * f_y}{\sqrt{3}\,\gamma_{mo}} + \frac{0.9 * A_{tn} * f_u}{\gamma_{ml}}$$

$$T_{db} = \frac{3200*250}{\sqrt{3} * 1.25} + \frac{0.9* 720*410}{1.25} = 632435 = 632 \text{ Kn}$$ 1

-

$$T_{db} = \frac{0.9 * A_{vn} * f_u}{\gamma_{ml} * \sqrt{3}} + \frac{A_{tg} * f_y}{\gamma_{mo}}$$

$$T_{db} = \frac{0.9 * 3200 * 410}{1.25 * \sqrt{3}} + \frac{720 * 250}{1.1}$$

= **709024.52 N = 709.02kN**

Least of (i) and (ii) , Block shear strength = 632 Kn

	Strength of tension member = least of I , II , III = **393.182 kN**	2	
VI a)	**Battens** • Compression members can be intermediate horizontal connecting plates or angle connecting two or four elements of a column. These horizontal; connecting plates are called battens • The battens shall be placed opposite to each other at each end of member and at point where the member is stayed in it length and as far as practicable be proportioned uniformly throughout		6
	Requirements • Spacing of batten C, from centre-to-centre of end fastening should be such that the slenderness ratio of the lesser main component, $\left(\frac{C}{r_{e,\min}}\right) \ngtr 50$ or 0.7 times the slenderness ratio of the compression member as a whole about X – X axis (parallel to battens) whichever is less. • Effective depth of battens, d shall be taken as distance between end rivets or end welds. $d > \frac{3}{4}a$ for intermediate batten d > a, for end batten d > 2b , for any batten where d = effective depth of batten a = centroidal distance of members b = width of members in the plane of battens.	4	
	• Thickness of battens, $t > \left(\frac{l_b}{50}\right)$ Where, $l_b =$ distance between the innermost connecting line of rivets or welds • Battens should be designed to carry bending moment and	1	

	shear arising from a transverse shear, $V = \frac{2.5}{100} P$		
b)			
	Axial load / Working load= P = 150kN	**1**	
	$l = 2.4$ m , $t_g = 12$ mm		
	- **Factored load** , P_u = FOS (Factor of safety) x P		
	= 1.5 x 150= 225kN		
	For single angle , load = 112.5 kN		
	- **Design compressive stress (f_{cd})**		**9**
	Assume design compressive stress as 90 N/mm^2 for angle sections		
	$f_{cd} = 90$ N/mm^2		
	- **Area** can be calculated as		
	Stress = load /area	**2**	
	Area = load / stress		
	$A_{cal} = P_u / f_{cd} = (112.5 \times 10^3) /90$		
	$A_{cal} = 1250$ mm^2		
	- **Providing of section**		
	For safety purposes , area can be increased by 20%		
	Increased area = A_{cal} + 20% of A_{cal} = 1.2 A_{cal} = 1500 mm^2		

Provide a section which has area very near to increased area

Provide I SA 80 x 80 x 10 mm

Area = 1505 mm^2

1

$r_{xx} = r_{yy}$ =24.1 mm

Iyy = 87.7 x 10^4 mm^4

c_{yy} = 23.4 mm

- **Effective length**

Assume two bolt connection

kl = 0.85 l = 0.85 x 2.4 = 2040 m

- **Slenderness ratio**

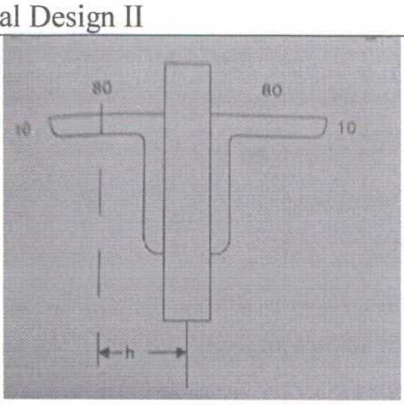
For single angle least radius of gyration is r_{yy} but for double angle least radius of gyration is miniumum of r_{yy} and r_{yyd}

r_{yyd} = radius of gyration for double angle

2

$$\text{Ryyd} = \sqrt{\frac{I_{yy\,d}}{2\,A}}$$

$I_{yy\,d}$ =moment of inertia for double angle



9

$h = c_{yy} + \frac{t_g}{2} = 23.4 + 12/2 = 29.4$

From parallel axis theorem

$$I_{yy\,d} = 2\left[I_{yy} + Ah^2\right]$$

$$I_{yy\,d} = 2[87.7 * 10^4 + 1505 * 29.4^2]$$

$I_{yy\,d} = 435.57 \text{ x } 10^4 \text{ mm}^4$

$r_{yyd} = \sqrt{\frac{I_{yyd}}{2\,A}} = \sqrt{\frac{435.57 * 10^4}{2 * 1505}}$

$r_{yyd} = 38.04$ mm

$r_{min} = 24.1$ mm

S.R = kl /r $_{min}$ = 2040/24.1 = 84.64

2

Design compressive stress (f_{cd})

Angles belong to buckling class (C) refer table 9(c) of IS 800: 2007

84.64, it is in between 80 and 90 requires, $f_y = 250$ N/mm^2

84.64= 80+ 4.64

	90 ------121		
	80-----136	**2**	
	10------ - 15		
	1------- - 1.5		
	For 4.64		
	4.64 x 1 --------4.64 x(-1.5)		
	4.64 ------ 6.96	**1**	
	fcd = 136 + (-6.96) = 129.04 N/mm^2		
	Design compressive strength (P_d)	**1**	
	$P_d = A_{pro}$ x f_{cd} = 1505 x 129.04 =194.20 Kn		
	For double angle , P_d = 2 x 194.20 = 388.41 kN		
	$P_d > P_u$	**1**	
	Hence safe		
VII a)	**Laterally supported beam**		
	• A beam may be assumed to be adequately supported at the supports provided the compression flange has full lateral restrain and nominal torsional restraint at support		
	• It is mainly subjected to bending and shear		
	Laterally unsupported beam		

	• If the compression flange of beam is not laterally supported, lateral buckling of compression flange occurs		
	• The compression flange of the beam tends to buckle laterally		
	• The lateral buckling of compression flange is accompanied by twisting and torsional buckling will occur if compression flange is unsupported		
	• Subjected to bending , web buckling, web bearing and shear	**1**	
	• It will reduce the load carrying capacity of the beam		
		1	
		1	
b)			
	Given data		
	ISWB 450@ 794 N/m	**1**	**9**
	Maximum shear force, V =120 kN	**1**	
	$f_y = 250$ N/mm^2		
		2	
	h = 450 mm		
	t_w =9.2 mm		
	Factored shear force = 1.5 x 120 = 180 kN	**2**	
	0.6 $V_d > V_u$		
	V_u = 180 kN		

V_d = Design shear = $= \dfrac{A_V f_y}{\gamma_{mo}\sqrt{3}}$

A_v = shear area = h x t_w

$$V_d = \frac{A_V f_y}{\gamma_{mo}\sqrt{3}} = \frac{450*9.2*250}{1.1*\sqrt{3}} = 569.75 \text{ Kn}$$

2

0.6 V_d = 0.6 x 569.75 =341.85 Kn

0.6 $V_d > V_u$

Hence the beam is safe against shear

2

VIII a)

The components of typical plate girder are as follows :

- **Web**
- The deep central vertical plate is called as a web in plate girder.
- It separates the two flange plates by a required distance.
- Web is responsible to resist shear developed in the plate girder

6

Flanges

- Flanges or flange plates are horizontal elements of plate girder which are provided at the top and bottom and they are separated by the web.
- The main purpose of flange plates is to resist the bending moment acting on the girder.
- The top flange resists the bending moment by developing compression and the bottom flange resists the tensile force. They should be provided with a required width and thickness to offer good resistance against

Stiffeners

Stiffeners are classified into two types :

2

	• **Vertical Stiffeners** ➢ Vertical stiffeners are provided at right angles to the flanges and they are also called as transverse stiffeners • **Horizontal Stiffeners** ➢ Horizontal stiffeners are provided in parallel to the flange plates. They are also called as longitudinal stiffeners. ➢ These stiffeners will improve the buckling strength of the web portion. ➢ Horizontal stiffeners are either continuous or discontinuous	1	
		1	
b)	Given data		
	W= working load = 15 kN/m	2	
	l= span = 6 m		
	$f_y = 250 \text{ N/mm}^2$	1	9
	Factored load / Ultimate load w_u		
	FOS = Ultimate load (w_u) / Working load (w)		
	w_u = FOS x w = 1.5 x 15 = 22.5 kN/m	1	
	Shear force (V_u)		
	$V_u = w_u l/ 2= (22.5 \times 6) / 2 =$ **67.5 Kn**	1	
	Check for shear		
	$0.6\ V_d > V_u$		

	$V_u = 67.5$ kN	1	
	V_d = Design shear strength= $= \frac{A_V f_y}{\gamma_{mo}\sqrt{3}}$	1	
	A_v = shear area = h x t_w, $t_w = 7.5$ mm $V_d = \frac{A_V f_y}{\gamma_{mo}\sqrt{3}} = \frac{300*7.5*250}{1.1*\sqrt{3}} = 295.23$kN 60 % of design shear, $0.6\,V_d = 0.6 \times 295.23 = 177.14$ kN $0.6\,V_d > V_u$ Hence the beam is safe against shear	3	6
IX a)	**Effective height of masonry wall** - Is used for calculating the slenderness ratio - It is the function of actual height of wall (H) and condition of lateral support - The value of effective height varies from 0.75 H to 1.5 H depending lateral support conditions - The effective height values given in table (4), IS 1905	3	
	Effective length of masonry wall - Used for calculation of slenderness ratio - Depends upon length of wall from or between centres of cross walls, piers or buttresses and the support condition	1	

	- The effective length varies from 0.8L to 2.0 L depending upon support conditions - The effective length of wall given in table (5) , IS 1905	1	
b)			
	Let us assume a masonry wall with thickness 200 mm	1	
	Effective height is less than effective length	1	
	Slenderness ratio= 2800/200 = 14		
	Assume cement mortar 1:5 , M1 grade and brick with compressive strength 5 N/mm^2		
	Basic compressive stress in masonry from table (8)	1	
	f_b = 0.5 N/mm^2		9
	Area of wall = 3.6 x 2 = 0.72 m^2> 0.2 m^2	2	
	Hence k_a = 1		
	Stress reduction factor with zero eccentricity , **k_s = 0.78**	2	
	Assume height to width ratio of unit 1.0		
	The shape modification factor of brick of crushing strength 5 N/mm^2		
	k_p=1.2		
	Permissible compressive stress, $f_c=k_a * k_s * k_p * f_b$		
	= 1 * 0.78 * 1.2 * 0.5 =0.488		

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	N/mm^2	1	9
	Actual compressive stress, $f = P/A = 12000/(1000 * 200) = 0.06 N/mm^2$ the actual compressive stress is less than the permissible compressive stress, hence wall is safe	1	
	Hence provide 200 mm thick brick wall with brick of compressive strength 5 N/mm^2 with M1 grade cement mortar		
X a)	From wind zone map of the country (IS 875 Part 3) I) Basic wind speed in Lucknow is $V_b = 47$ m/s (Appendix A page 53) II) Risk coefficient , k $k_1 = 1$, for general buildings (Table 1 , IS 875 PART 3)	2	
	III) Terrain , height and structure size factor k_2 since shed is in industrial area, it may be considered belonging to category 3 Greatest dimension = 40 m (It belongs to class B structure) $k_2 = 0.88$, h = 10 m $k_2 = ?$ h = 12 m $k_2 = 0.94$ h = 15 m	1	
	$\frac{0.94 - k_2}{0.94 - 0.88} = \frac{15 - 12}{15 - 10}$	2	

	$$\frac{0.94 - k_2}{0.06} = \frac{3}{5}$$		
	$k_2 = 0.94 - 0.036 = 0.904$	2	
	IV) Topography factor, k_3		
	$k_3 = 1 + cs$		
	In Lucknow, ground near shed may be assumed plain		
	Slope = 0		
	c= z/l =0, $k_3 = 1$	2	
	V) Design wind speed ,		
	$V_z = k_1 * k_2 * k_3 * V_b = 1 * 0.904 * 1 * 47 = 42.488$ m/s		
	VI) Basic wind pressure		
	$P_z = 0.6\ V_z^2 = 0.6 * 42.488 * 42.488 =$ **1083 N/m² = 1.083 kN/m²**		
	(as per Clause 5.4 IS 875 PART 3, Page No: 12)		
b)			
	Angle purlins		
	Trial section		9
	Minimum depth = L/45 = 4000/45 = 89	2	
	Minimum width = L/60 =4000/60 = 66.7 mm		
	Let try ISA 100 x 75 x 6 mm	1	
	Dead load		

	Weight of AC sheets with overlap and fixtures = 0.205 kN/m^2		
	= 1.6 x 0.205 = 0.328 kN/m		
	Live load = 0.6 x 1.6 = 0.96 kN/m	1	
	Total vertical downward load = 0.328+ 0.96 =1.288 kN/m		
	Factored (DL+LL) = 1.5 x (1.288) cos 21 = 1.804 kN/m		
	Factored (DL+ LL) normal to sheeting = 1.5 (0.328 -1) = -1.008 kN/m		
	Therefore , Live load + Dead load is critical	1	
	$M_z = wL^2/10$ = (1.804 x 4 x 4)/ 10 = 2.886 kNm		
	For ISA 100 X 75 , 6mm thick		
	$\frac{b}{t_f} = \frac{75}{6} = 12.5\ between\ 9.4\ and\ 15.7$	1	
	$\frac{d}{t_w} = \frac{100}{6} = 16.67 < 84$	1	
	Hence it belongs to class 3 (Semi compact section)		
	For such section		
	$M_d = (\beta_b Z_P f_y)/\gamma_{mo}$		
	For ISA 100 x 75 x 6 mm, $\beta_b = 1$, $f_y = 250\ N/mm2$		
	$Z_c = 14.4 \times 10^3\ mm^3$		
	$M_d = \frac{14.4*10^3*250}{1.1}$ =3.272 x 10^6 N mm		

	$=3.272\ kNm > M_z$ Hence ISA 100 x 75 x 6 mm , is suitable as a purlin		
		3	
		3	