Qn. No.	Scoring Indicators	Split score	Total score
I 1.	The methods of analysis are a) Elastic analysis b) Plastic analysis c) Advanced analysis d) Dynamic analysis	2	
2.	It is the distance between the two consecutive bolts of adjacent rows and is measured at right angles to the direction of load.	2	
3.	The area of cross section of the member after deducting the area of the holes in that section is called the net area of the member .	2	
4.	Shape factor is the ratio of plastic moment (Mp) to yield moment (My) or Shape factor is the ratio of plastic section modulus to elastic section modulus .	2	
5.	The slenderness ratio of a wall is the ratio of its effective height to the effective thickness or effective length to effective thickness which ever is less.	2	10
II. 1.	Advantages of welded connection Lighter structures due to absence of connecting plates More quicker process due to the absence of making holes Efficiency is 100% Noise produced is less No mismatching of holes Provide rigid joints More pleasing appearance	6 × 1	6
2.	Physical properties of structural steel irrespective of its grade may be taken as: a) Unit mass of steel, p = 7850 kg/m^3 b)) Modulus of elasticity, E = 2.0 x 10^5 N/mm2 (MPa) c)Poisson ratio, p = 0.3 d)Modulus of rigidity, G = 0.769 x 10^5 N/mm2 (MPa) e)Co-efficient of thermal expansion α_t= 12 x $10^{-6}/^0c$	3	
	Mechanical properties of structural steel The principal mechanical properties of the structural steel important in design are yield stress, fy; the tensile or ultimate stress, fu; the maximum percent elongation on a standard gauge length and notch toughness Effective length of compression member for different end conditions	3	6

3. Effective length of compression member for different end conditions

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation	Rotation	Translation	Rotation		
(1)	(2)	(3)	(4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0L
Free	Restrained	Free	Restrained		
Restrained	Free	Restrained	Free		1.0L
Restrained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Restrained		0.65L

6 × 1

6

4.	f_y = 300N/mm^2 Υ_{m0} = 1.1 ISA 90 x 90 x 6mm A_g = 1047 mm^2 T_{dg} = A_g x f_y/ Υ_{m0} = 1047 x 300 / 1.1 =285545.45N = 285.54kN	2 2 2	6
5.	Class 1 - Plastic sections Cross sections which can develop plastic hinges and have rotation capacity required for failure of structure by formation of plastic mechanism. Class 11 – Compact sections Cross sections which can develop plastic moment of resistance but have inadequate plastic hinge rotation capacity for formation of plastic mechanism due to local buckling. Class 111 – Semi compact sections Cross sections in which the extreme fibre in compression can reach yield stress but cannot develop the plastic moment of resistance due to local buckling. Class 1V – Slender sections Cross sections in which the elements buckle locally even before reaching yield stress .	1·5x4	6
6.	The component parts of a roof truss are 1. Principal rafters 2. Top chord members 3. Bottom chord members 4. Ties 5. Struts 6. Purlins 7. Ridge piece 8. Roof covering	6x1	6
7.	Masonry structures gain stability from the support offered bu cross walls, floors roofs and other elements such as piers and buttresses. Load bearing walls are structurally more efficient when the load is uniformly distributed and the structure is planned such that the eccentricity of the loading on the member is as small as possible. Eccentric loading is to be avoided by providing adequate bearing of floor or roof slab on the wall. Adequate stiffness in slab is to be provided and fixity at supports is to be avoided. The strength of a wall is measured in terms of its resistance to the combination of self weight, super imposed load and lateral pressure. The stability of a wall is indicated by its resistance to over tuning by lateral forces and buckling caused by excessive slenderness ratio. The strength of a masonry wall depends upon the strength of masonry wall depends upon the strength of masonry		

	unit and strength of mortar. The quality of work manship and the method of bonding also have a bearing on the strength of masonry wall. In a masonry wall the strength of the mortar is general shall not be greater than that of the masonry unit. There is a optimum brick wall strength with a certain strength of mortar used. Therefore there is no advantage of using a stronger mortar (i.e. rich mix). Use of rich mortars results in unnecessary rigidity of the structure with tendency to crack. Normally stronger mortars (i.e. rich mix) used with stronger bricks and weaker mortars with weaker bricks develop maximum strength. In masonry structures due to differential movement between various parts of the structure or due to shrinkage or thermal movements cracks are likely to occur. In such cases use of weaker mortar (i.e. lean mix) leads to distribution of cracks throughout the brick work in joints and development of cracks through the bricks does not occur. Therefore if is necessary that richer mixes are avoided in brick work. The mix proportions and strength of commonly used mortars for masonry may be obtained from. Table 1 of IS : 1905 – 1987 (code of practice for structural use of unreinforced masonry third revision). The thickness of a load bearing wall should be such that it is sufficient at all points to ensure the resulting stresses due to the loading for which the wall is designed are with in the prescribed limits for that particular type of wall. The thickness of masonry wall is to be designed on the basis of permissible compressive stress. When a masonry wall is subjected to axial load or load at small eccentricity it acts as a column. The strength of the wall depends on slenderness ratio. The slenderness ratio of a wall is a function of (a) height of the wall (b) length of the wall (c) thickness of the wall and (d) support conditions.	6	6
III a)	Riveted connection Bolted connection Welded connection	1 x 3	3

b)

Solution For Fe 410 grade of steel: f_u = 410 MPa

For bolts of grade 4.6: f_{ub} = 400 MPa

γ_{mb} = partial safety factor for the material of bolt = 1.25

A_{nb} = net tensile stress area of 20 mm diameter bolt = 245 mm^2 (From Table 5.1)

(a) The bolts will be in single shear and bearing.

Diameter of bolt, d = 20 mm

The strength of bolt in single shear,

$$V_{dsb} = A_{nb}\frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} = 245 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3} = 45.26 \text{ kN}$$ 2

The strength of bolt in bearing,

$$V_{dpb} = 2.5\ k_b dt\ \frac{f_u}{\gamma_{mb}}$$ 1

From Table 5.3, for a 20 mm diameter bolt, d_o = 22 mm and e = 33 mm.

Assume pitch = 50 mm

k_b is least of $\frac{e}{3d_o} = \frac{33}{3 \times 22} = 0.5$; $\frac{p}{3d_o} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.5$; $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$; and 1.0. 2

Hence, k_b = 0.5.

(The value of end distance, e has been assumed to be for rolled edge.)

$$V_{dpb} = 2.5 \times 0.5 \times 20 \times 12 \times \frac{410}{1.25} = 98.4 \text{ kN}$$ 2

The strength of the bolt will be minimum of the strength in shear and bearing and is 45.26 kN. 1

(b) The bolt will be in single shear and bearing. The thickness to be considered for bearing will be the least of aggregate thickness of cover plates and the minimum thickness of the main plates jointed. Hence, t = 10 mm. 1

The strength of bolt in single shear = 45.26 kN (as found in (a))

The strength of bolt in bearing,

$$V_{dpb} = 2.5\ k_b dt\ \frac{f_u}{\gamma_{mb}} = 2.5 \times 0.5 \times 20 \times 10 \times \frac{410}{1.25} \times 10^{-3} = 82.0 \text{ kN}$$ 2

The strength of the bolt will be minimum of the strength in shear and bearing and is 45.26 kN. 1 12

IV a)

Advantages

It has high strength per unit mass.

Has assured quality and high durability

High speed of construction

Can be strengthened at any later time

It can be easily dismantled and transported to other sites. 1x5 5 b)

Solution

The properties of ISA 65 x 65 x 6 are

A_g = 744 mm2

C_{zz}.= 18.1 mm

Tensile capacity of the member = A_g x f_y/ Υ_{m0} = 744 × 250/1.1 = 169.1 kN 2

The force resisted by the weld at the lower side of the angle

P_1= 169.1 x (65- 18.1)/65 = 122.01 kN 2

Force to be resisted by the upper side of the angle 2

P_2 =169.1 x 18.1/65= 47.09 kN

Assuming a weld size of 4 mm (>3 mm),

Effective throat thickness of the weld t_t= 0.7 x 4 = 2.8 mm

Design strength of the weld Pdw= Lw x t_t x fu/ /3 x Υ_{mw} N/mm

ie; 169.1 x 10^3 = Lw x 2.8 x 410/ /3 x 1.25

Lw = 319.05 say 319mm But Lw = Lw1 + Lw2

Pdw1 = P1 = 122.01kN = Lw1 x 2.8 x 410/ /3 x 1.25

Lw1 = 230.1mm 2

Hence provide 230mm 2 10

Lw2 = 319 -230 = 89mm

Provide 4mm weld of Lw1 = 230mm and Lw2 = 89mm as shown in figure .

V a) To achieve maximum value for minimum radius of gyration without increasing the area of the section, lacings and battens are provided. Rolled steel flats and angles are used for lacing. The object of providing lacings is to keep the main members of the column away from principal ones. In doing so , the lacings are subjected to shear forces due to horizontal forces on columns. 5 5

b) From the consideration of yield strength , gross area = 250 x 1000/(250/1.1) = 1100mm^2

Try ISA 10075, 8 mm which has gross area A_g = 1336 mm^2.

Number of bolts required:

d = 20 mm ∴ d_0 = 22 mm

Use gusset plate of thickness 10 mm.

Strength of one bolt in single shear

$$= \frac{400}{\sqrt{3}} \frac{\left(0+\frac{\pi}{4}\times 20^2\times 0.78\right)}{1.25} = 45272 \text{ N}$$

Adopting edge distance e = 40 mm; pitch p = 60 mm,

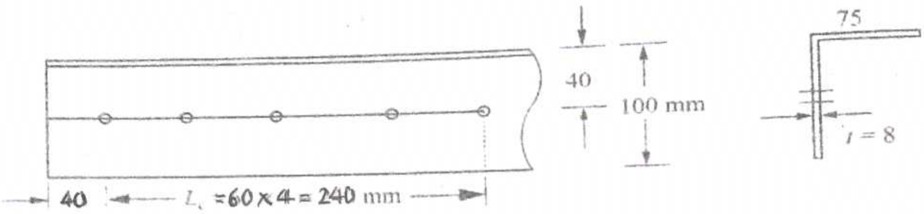
K_b is smaller of $\frac{40}{3\times 22}, \frac{60}{3\times 22} - 0.25, \frac{400}{410}, 1.0$ i.e., $K_b = 0.606$.

$$\therefore \quad V_{dpb} = \frac{1}{1.25} \times 2.5 \times 0.606 \times 20 \times 8 \times 400 = 77568 \text{ N}$$

$\therefore$ Bolt value = 45272 N. 3

Number of bolts required $= \frac{225\times 1000}{45272} \approx 5$

Provide the bolts as shown in Fig.



Checking the design:

(a) Strength against yielding:

$$= \frac{A_g f_y}{\gamma_{mo}} = \frac{1336\times 250}{1.1} = 303636 \text{ N} > 225000 \text{ N}$$ O.K. 2

(b) Strength of plate in rupture:

Area of connected leg $A_{nc} = \left(100 - 22 - \frac{8}{2}\right) \times 8 = 592 \text{ mm}^2$

Area of outstanding leg $A_{go} = \left(75 - \frac{8}{2}\right) \times 8 = 568 \text{ mm}^2$

$$\beta = 1.4 - 0.076 \frac{75}{8} \times \frac{250}{410} \times \frac{(75+40-8)}{(240)}$$ 2

$= 1.206$

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

$$= \frac{0.9 \times 410 \times 592}{1.25} + 1.206 \times \frac{568 \times 250}{1.1}$$

$$= 330442\ N > 225000\ N$$

O.K.

(c) **Strength against block shear failure**

$A_{vg} = (40 + 60 \times 4)\ 8 = 2240\ mm^2$ $\quad A_{vn} = (40 + 60 \times 4 - 4.5 \times 22) \times 8 = 1448\ mm^2$

$A_{tg} = (100 - 40) \times 8 = 480\ mm^2$ $\quad A_{tn} = (100 - 40 - 0.5 \times 22) \times 8 = 392\ mm^2$

Strength against block shear failure is smaller of

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

$$= \frac{2240 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 392 \times 410}{1.25} = 409642\ N$$

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

$$= \frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \frac{480 \times 250}{1.1}$$

$$= 355879\ N$$

$$\therefore \quad T_{db} = 355879\ N > 225000\ N$$

O.K 3

Hence safe.

VI a) Battens are plates or any other rolled sections used to connect the main components of compression members. Battens should be placed opposite to each other on the two parallel faces of the compression member and should be spaced and proportioned uniformly through out. The Number of battens should be such that the member is divided into atleast three bays within its actual length. 5

b)

Assuming fcd = 90 N/mm^2 for angle sections

A = 175 x 10^3/90 = 1944.44mm^2

Try ISA 130 x 130 x 8mm 2

Area = 2022 mm^2 KL = 0.85L = 0.85 x 3000 = 2550mm

rmin = rvv = 25.5mm 2

KL / rmin = 2550/ 25.5 = 100 2

From table 10 page 44. For angle sections buckling class is C

For KL / rmin = 100 and fy = 250 1

fcd = 107 N/ mm^2

Pu = A. fcd

= 2022 x 107

= 2,16,354N

= 216.35kN > 180kN Safe 2 10

Provide ISA 130 x 130 x 8mm for the strut. 1

VII a) A beam in which the compression flange is laterally supported by flooring is known as laterally supported beam. It is mainly subjected to bending and shear. 3

A beam in which the compression flange of beam is not laterally supported is known as laterally unsupported beam. It is mainly subjected to bending, shear, web buckling and web bearing. 3 6

b)

Solution:

$W = 24\frac{\text{kN}}{\text{m}}$ (Including self weight)

$l = 6\text{ m} = 6000\text{ mm}$

$f_y = 250\text{ N/mm}^2$ 2

ISMB 600 @ 1226 N/m, $I_{xx} = 91813 \times 10^4\text{ mm}^4$

Check for deflection condition ($\delta_{max} < \delta_{per}$) 1

$W = 24\text{ kN/m}$

δ_{max} for with SS beam udl 2

$$= \frac{5Wl^4}{384EI} = \frac{5\times24\times6000^4}{384\times2\times10^5\times91813\times10^4}$$

$\delta_{max} = 2.20\text{ mm}$ 2

δ_{per}: SS beam $\delta_{per} = \frac{l}{240}$ 2

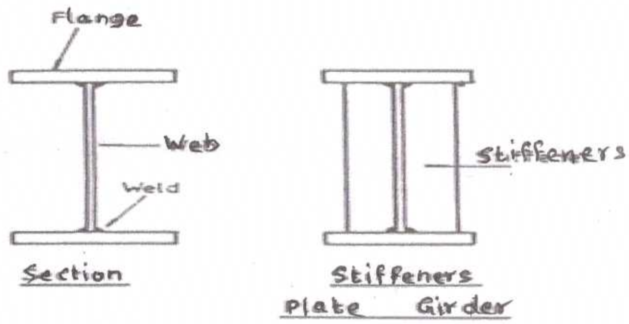
$\delta_{per} = \frac{6000}{240} = 25\text{ mm}$

$\delta_{max} < \delta_{per}$

$2.20 < 25$ 2 9

Hence the beam is safe against deflection.

VIII a)



2

Web plate

Flange plate

Stiffeners - Horizontal stiffeners, Intermediate stiffeners, End stiffeners

Web plates are provided to keep flange plates at required distances and to resist the shear in beam.

Flanges are provided to resist bending moment acting on the beams by developing compressive force in one flange and tensile force in another flange. 4 6

Stiffeners are provided to safe guard the web against local buckling failure.

b)

Given data:

Shear check condition ($0.6\,V_d > V_u$)

Imposed load $= W_1 = 20$ kN/m $=$ live load

dead load $=$ self weight $= 995$ N/m

$= 0.995$ kN/m $= W_2$

Total load $=$ dead load $+$ live load

$W = W_1 + W_2 = 20 + 0.995$

$= 20.995$ kN/m 1

Factored load (or) ultimate load $= 1.5 \times$ working load

$W_u = 1.5 \times W$

$W_u = 1.5 \times 20.995$

$= 31.49$ kN/m, $l = 6$m

$\therefore\ V_u = \dfrac{W_u l}{2} = \dfrac{31.49 \times 6}{2}$

$V_u = 94.47$ kN

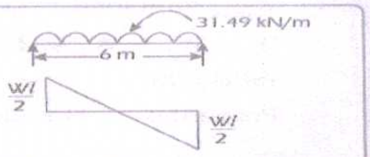


FIGURE 3.14:

$$V_d = \text{design shear} = \frac{A_v f_y}{\sqrt{3}\, r_{mo}}$$ 2

ISLB 600 $\quad A_v =$ shear area $= 600 \times 10.5$ 1

Properties $\quad V_d = \dfrac{(h \times t_w)\, f_y}{\sqrt{3} \times r_{mo}} = \dfrac{600 \times 10.5 \times 250}{\sqrt{3} \times 1.1}$ 2

$h = 600$ mm $V_d = 826.66$ kN

$t_w = 10.5$ mm

60% of $V_d = 0.6\ V_d = 0.6 \times 826.66 = 496$ kN

$0.6\ V_d > V_u$

496 kN > 94.47 kN 2

The beam is safe in shear 1 9

IX a) The effective height of the wall to be used for calculating the slenderness ratio is the function of the actual height of the wall and the condition of lateral support. The value of effective height varies from 0.75 to 1.5 times the actual height depending on the lateral support condition. 3

The effective length of the wall to be used in the calculation of slenderness ratio depends upon the length of wall from or between centres of cross wall, piers, etc and the support condition. 3 6

b) Let the thickness of the wall be 200mm. Since effective height is less than the effectivelength, the slenderness ratio is to be based on height.

$$\text{Slenderness ratio} = \frac{\text{Effective height}}{\text{Effective thickness}} = 2400/200 = 12$$ 2

Assuming C.M 1:5 and brick with compressive strength of 5 N/mm^2

The basic compressive stress in masonry from table (8) of IS 1905 -1987 is $f_b = 0.5\ N/mm^2$

Area of wall = $A = 3.5 \times 0.2 = 0.70 m^2 > 0.20 m^2$ 2

Hence no size reduction factoris needed ie; $K_a = 1$

From table (9) for slenderness ratio of 12 and zero eccentricity, the stress reduction factor $k_s = 0.84$ 1

Assuming height to width ratio of unit as 1.0. the shape modification factor for brick of crushing strength of 5. N/mm^2 is $k_p = 1.2$ (Table 10 of IS : 1905 – 1987).

$\therefore$ Permissible compressive stress $= f_c = k_a \cdot k_s \cdot k_p \times f_b$

$= 1.0 \times 0.84 \times 1.2 \times 0.5$

$= 0.504\ N/mm^2$.

Actual compressive stress $= f = \frac{P}{A} = \frac{9800}{1000 \times 200}$ 2

$= 0.049\ N/mm^2$.

As the actual compressive stress is less than the permissible compressive stress the wall is safe.

Hence provide 200 mm thick brick wall with bricks of compressive length of 5.0 N/mm^2 and use M1 grade mortar. 2 9

X a) The main loads on trusses are

Dead loads

Imposed loads

Wind loads

Earth quake loads — 1×6 — 6

Erection load

Snow load

b) ***Wind load :*** This is to be obtained as per IS : 875 (part 3) - 1987. for calculating the wind load basic wind pressure and design wind pressure are to be used as defined in the code. As per the revised code IS : 875 (part 3) - 1987 the country is divided into six zones corresponding to basic wind speeds of 33, 39, 44, 47, 50 and 55 m/sec at a mean height of 10 m above ground level. The basic wind speed is designated by (V_b). From the basic wind speed the design wind speed (V_z) is to be calculated from

$V_z = k_1 \cdot k_2 \cdot k_3 \cdot V_b$

when V_z = design wind speed at any height Z in m/sec.

K_1 = probability factor (risk coefficient)

K_2 = terrain, height and structure size factor

K_3 = topography factor. — 3

The coefficients K_1 & K_2 are to be obtained from table (1) & table (2) respectively of IS : 875 (part 3) - 1987. The topography factor K_3 may be taken as 1.0 for sites where the upwind slope is less than 3^0. For slopes greater than 3^0 the valve of K_3 is in the range of 1.0 to 1.36.

For the region knowing the basic wind speed (V_b) and the coefficient K_1, K_2 & K_3, the design wind speed at any height is calculated as $V_z = K_1 . K_2 . K_3 . V_b$.

The design wind pressure : The design wind pressure is to be calculated from $p_z = 0.6\ V_z^2$ where p_z = design wind pressure in N/m² at height 'z'. — 3

Wind load on individual members : The wind load on structural elements such as roofs and walls may be calculated as

$F = (C_{pe} - C_{pi})\ A.\ p_d$

where C_{pe} = External pressure coefficients — 3 — 9

C_{pi} = Internal pressure coefficients

A = Surface area of structural element or cladding

p_d = Design wind pressure.

The external pressure coefficients (C_{pe}) depend on building height to width ratio and are given in table 5 of IS : 875 (part 3) - 1987.

The internal pressure coefficients (C_{pi}) are to be taken as per clause 6.2.31 of IS : 875 (part 3) - 1987. They depend upon the degree of permeability of the building.

Snow load : The snow load may be taken as 25 N/m² per millimeter depth of snow. In places where snowfall is expected a structure should be designed for the actual snow load or live load which ever is more.

Erection load : While designing a roof truss provision should be made to take up the stresses likely to be developed in the truss members due to roofing material or erection machinery being kept on them.

TED(15) 6013 structured design