VERSION-A

SCHEME OF VALUATION

(Scoring Indicators)

Revision: 2015 **Course code: 6013**

Course title: Structural design II

Qn no.	Scoring Indicator	Spilt up score	Sub total	Total
	PART A (Max. mark : 10)			
1				
1.	a) Yeild stress (f_y), b) ultimate stress (f_u) (c) percentage elongation (d) Notch tougness (Any two)	1+1	2	
2.	It is the centre to centre spacing of the bolt in a row, measured along the direction of load.	2	2	
3.	(a) Yeilding of cross section. (b) Rupture (c) Block shear (Any two)	1+1	2.	
4.	A 'beam' is a structural member in which load acts perpendicular to the axis of member.	2	2.	
5.	~~(i) Principal rafter (ii) bottom chord (iii) Tie (iv) strut (v) purlin (vi) rafter (vii) Eves etc (Any 4)~~	~~4×½~~	~~2.~~	~~10.~~
	1) Dead load. (2) Live load (3) wind load (Any 2)	1+1	2	10.

Qn.	PART B	Split	Sub T.	Total
II	(Max. mark : 30, Answer any five question $5 \times 6 = 30$)			
1.	Advantage of welded connection. 1. Joints are more stiff 2. Better appearence. 3. Less weight 4. Less noise pollution. 5. ~~H Speed of~~ High speed of fabrication 6. No maintanance required hence economical 7. possible to achive 100% efficiency. 8. Alterations can be easily done (Any 6)	1x6	6.	
2.	Given: $f_u = 410\ N/mm^2$. $f_{ub} = 400\ N/mm^2$. $d = 16\ mm$, $\gamma_{mb} = 1.25$ Sol: $A_{nb} = 0.78 \times \frac{\pi}{4} (16)^2 = 156.82\ mm^2$ — 1 For lap joint In single shear $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ — 1 $V_{nsb} = \frac{f_u}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$ — 1 For single shear $n_s = 0$, $n_n = 1$. — 1 $V_{nsb} = \frac{410}{\sqrt{3}} \times (1 \times 156.82) = 44.58\ KN$ — 1 $\therefore V_{dsb} = \frac{44.58}{1.25} = 35.66\ KN$ —	1	6.	
3	(a) Design strength in Yielding of cross section $T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$, f_y – yield stress A_g – gross area γ_{mo} – Partial safety factor	2 (1+1)		
Qn no.	Scoring Indicator	Spilt up	Sub total	Total

Qn.	PART B	Split	Sub T.	Total
	b) Design str: due to rupture $T_{dn} = \frac{0.9\,A_n f_u}{\gamma_{m1}}$ →	2		
	$A_n = \left[b - n d_h + \sum \frac{p_{si}^2}{4 g_i}\right] t$ $A_n = 0.78 \frac{\pi}{4} d^2$ (for ISO thread) —	2	6	
4.	(a) Actual length: It is the centre to centre distance of compression member between the restrained ends. —	2		
	(b) Effective length: The eff length of a compression member is the length of an equivalent member with hinged ends having the same effect of buckling. —	2		
	(c) Slenderness ratio: Defined as the ratio of effective length to corresponding radius of gyration of the section $Sl.\ ratio = \frac{l_e}{r} = \frac{KL}{r}$ where $l_e = KL$ - effective length —	2	6	
5.	Given: $V = 100\,KN$ $f_y = 250\,MPa$ properties of ISWB 400 @ 667 N/m $h = 400\,mm$. $t_w = 8.60\,mm$. (Steel table) —	1		
	Factored SF $V_u = 1.50 \times 100 = 150\,KN$ —	1		
	Condition: $(0.60\,V_d > V_u)$			
Qn no.	Scoring Indicator	Spilt up	Sub total	Total

Qn.	PART B	Split	Sub T.	Total
	$V_d = \dfrac{A_v f_y}{\sqrt{3}\,\gamma_{mo}}$	1		
	$A_v = (h \times t_w) = 400 \times 8 = 3200\ mm^2$			
	$V_d = \dfrac{3200 \times 250}{\sqrt{3} \times 1.10} = 381.86\ KN$	2		
	$\therefore\ 0.60\ V_d = 0.60 \times 381.86 = 229.1\ KN$			
	$229.10\ KN > 150\ KN$ hence safe	2	6.	
6.	(a) Plate girders (Situations). 1. Larger column free halls are required in the lower floor of a multistorey building 2. In work shops, where girders are required to carry ~~ca~~ crane beams. 3. In road and railway bridges.	3.		
	(b). 1. web plate 2. Flang angle 3. Bearing stiffener 4. Vertical stiffners. 5. Horizontal stiffeners 6. web splice 7. Flange splice (Any 6 ~~7~~ $6 \times \frac{1}{2}$)	3.	6.	
7.	Effective height: The effective height of wall (h) to be used for calculating the slenderness ratio, is the function of the actual height (H) of the wall and the condition of lateral support. 'h' varies from 0.75 H to 1.50 H depending on the lateral support condition. (The value adopted as per IS 1905-1987)	3.		
Qn no.	Scoring Indicator	Spilt up	Sub total	Total

Qn.	PART B	Split	Sub T.	Total
	Eff. length: The eff. length of the wall (l) to be used in the calculation of slenderness ratio depends upon the length of wall from or between centres of cross wall, piers or buttresses (L) and the support conditions. The eff. length varies from 0.80L to 2.0 L depending upon support conditions (IS 1905-Part 3)	3	6	
	(Any 5x6)			30
	PART.C (Max. mark: 60).			
	UNIT-I			
III	(a). Advantages: 1. speedy construction. 2. Dismantling and reuse of steel structures will be possible 3. te occupy less space. 4. Modification of sectional properties can be easily carry out. 5. Less self weight. 6. Assured quality and high durability (Any four)	4.		
	~~(b)~~ dis advantages: 1. Suspect to corrosion 2. Maintanance cost is high. 3. costly.	3	7	
	(b) Given: Size of weld S = 6mm. length of weld L_w = 330 mm. ~~fy~~ ultimate stress f_u = 410 N/mm^2			
Qn no.	Scoring Indicator	Spilt up	Sub total	Total

Qn.	~~PART B~~	Split	Sub T.	Total
	Sol:			
	Throat thickness 't' = 0.70×6 = 4.2mm			
	Design stress $f_{wd} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$ $\left[\gamma_{mw} = 1.25 \text{ shop weld}\right]$	1		
	$f_{wd} = \frac{410}{\sqrt{3} \times 1.25} = 151.85\ N/mm^2$	2		
	Design strength $= (L_w.t)\,f_{wd}$.	1		
	$= 330 \times 4.2 \times 151.85 = 210.46\,KN$	2		
	Safe load = $\frac{\text{Ultimate load}}{\text{Factor of Safety}}$			
	∴ Safe load = $\frac{210.46}{1.50} = 140.30\,KN$	2	8	15.
	OR.			
IV				
	a) The aim of design is to decide shape, size and connection details of the member so that the structure being designed will perform satisfactorily during its intended life. With an appropriate degree of safety the structure should	2		
	(a) sustain all loads expected on it.			
	(b) sustain all deformations during and after construction.			
	(c) should have adequate durability.			
	(d) should have adequate resistance to mis use and fire			
	(e) should ~~have~~ be stable and have alternate load paths to prevent overall collapse under accidental loading	4		
	The design philosophies used are			
	(1) working stress method (2) Ultimate load Design (3) Limit State Design.	1	7.	
Qn no.	Scoring Indicator	Spilt up	Sub total	Total

Qn. **PART B** | Split | Sub T. | Total

b) Given:

$f_u = 410\ N/mm^2,\ t = 20\ mm$

$b = 250\ mm^2$

No. of bolt in single shear $n_n = 3$.

Grade 4.60 bolt – dia = 20 mm

$\therefore d_o = 22\ mm$.

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

Solution:

Net area of plate

$$A_n = \left[b - n\, d_o\right] t = \left[250 - 3\times 22\right] 20$$

$$= 3680\ mm^2$$

(i) Design str. of plate

$$T_{dn} = \frac{0.90 f_u A_n}{\gamma_{m1}} = \frac{0.90\times 410\times 3680}{1.25}$$

$$= 1086.33\ KN$$ — 2

(ii) D. strength of bolt in shear.

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

$$= \frac{400}{\sqrt{3}}\left(3\times 245\right) = 169.74\ KN \quad \cancel{\left[A_{nb} = 0.78\times\frac{\pi\times 20^2}{4}\right]}$$

$$\left(A_{nb} = 0.78\times\pi\times 20^2/4\right)$$

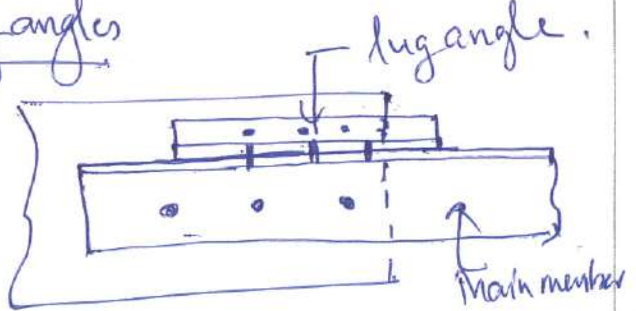
~~(iii) Des strength of bolt in bearing.~~

$$\cancel{V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}\quad V_{npb} =}$$

$\therefore$ Design strength in shear $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$

$$= \frac{169.74}{1.25} = 135.79\ KN$$ — 3

Qn no.	Scoring Indicator	Spilt up	Sub total	Total

Qn.	PART B	Split	Sub T.	Total
	iii) Design strength in bearing of bolt $V_{npb} = 2.50\, k_b . d . t . f_u$ where k_b least of the following a) $\frac{e}{3d_0} = \frac{50}{3\times22} = 0.75$. $\left[e = \frac{(250-3\times50)}{2}\right]$ b) $\frac{p}{3d_0} = \frac{50}{3\times22} = 0.75$. c) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$ d) 1.0. $\therefore k_b = 0.75$ $\therefore V_{npb} = 2.50\times0.75\times20\times20\times410$ $= 307.50$ KN. $\therefore$ Design strength $= \frac{V_{npb}}{\gamma_{mb}} = \frac{307.50}{1.25}$ $= \underline{246 \text{ KN}}$	2		
	Ans: Design strength joint is the least of the above three i.e. $\underline{135.79 \text{ KN}}$ (Ans).	1	8.	15.
	UNIT – II			
V	a) Use of lug angles  	1		
Qn no.	Scoring Indicator	Spilt up	Sub total	Total

Qn.	~~PART B~~	Split	Sub T.	Total
	Length of the end connections of a heavily loaded tension members may be reduced by using lug angles. By using lug angles there will be saving in gusset plate, but it is upset by additional fasteners and angles required.	2		
	IS specifications: (Cl: 10.12.1 to 12.6) 1. Leg angle connecting leg of a channel shaped member shall as far as possible, be disposed symmetrically with respect to the section of member. 2. In the case of angle member, the lug angle shall be capable of developing a strength of not less than 20% in excess of forces in the outstanding leg of the member 3. In case of an channel member shall be capable of develop a strength not less than 10% in excess of forces in the outstanding leg of the member. 4. In no case shall fewer than two bolts, rivetts or equivalent welds be used for attaching lug angles to gusset plate or supporting member. 5. The effective connection to the lug angle shall as far as possible terminate at the end of the member connected 6. where lug angles are used to connect an angle member, the whole area of the member shall be taken as effective not withstanding the requirement of section of this standard. (Any four) →	~~5~~ 4	7.	
Qn no.	Scoring Indicator	Spilt up	Sub total	Total

Qn.		score split	Sub T	Total.
	PART C			
	b) Given: ISA 90x60x6 mm.			
	$f_y = 250\ N/mm^2$, $f_u = 410\ N/mm^2$			
	Length of weld $L_w = 200$ mm.			
	Solution: Gross area $A_g = 865\ mm^2$.			
	Area of connected leg $A_{nc} = (90 - 6/2)6$ $= \underline{522}\ mm^2$.			
	Area of outstand leg $A_{go} = (60 - 6/2) \times 6$ $= 342\ mm^2$.	2		
	(i) Strength due to yielding			
	$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{865 \times 250}{1.10} = \underline{\underline{196.60 KN}}$	2		
	(ii) Strength due to rupture: at			
	$T_{dn} = \frac{0.90 f_u A_{nc}}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$ — i			
	where $\beta = 1.40 - 0.076 \times \left(\frac{w}{t}\right) \times \frac{f_y}{f_u} \times \frac{b_s}{L_w}$.			
	here $w = 90$ mm (~~welding~~)			
	$b_s = w = 90$ mm (welding)			
	$\therefore \beta = 1.40 - 0.076 \times \left(\frac{90}{6}\right) \times \frac{250}{410} \times \frac{90}{200}$	1		
	$= 1.087$			
	$\therefore T_{dn} = \frac{(0.90 \times 410 \times 522)}{1.25} + \frac{(1.087 \times 342 \times 250)}{1.10}$	2	8	15
	$= \underline{\underline{207.77\ KN}}$			

Qn.		Score split	Sub T	Total

PART C

VI a) Given: $P_u = \cancel{100}\ 200$ KN.

$l = 3$ m.

$f_y = 250\ N/mm^2$ $\therefore kl = 0.70 l$.

Soln. $kl = 2.10$ m.

$f_{cd} = 90\ N/mm^2$ (for angle sections). 2

$P_u = A \cdot f_{cd}$

$\therefore A = \frac{P_u}{f_{cd}} = \frac{200 \times 10^3}{90} = \underline{\underline{2222.22\ mm^2}}$ 3

Provide a section ISA $125 \times 95 \times \cancel{10}\ 12$ mm which has area more than calculated area

ISA $125 \times 95 \times 12$ mm $A_g = \underline{\underline{2498\ mm^2}}$ 2 7.

b) Procedure of determination of design strength of single angle tension member.

i) Design str. due to yielding of gross section

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

where A_g = gross area of given angle section. (From steel table)

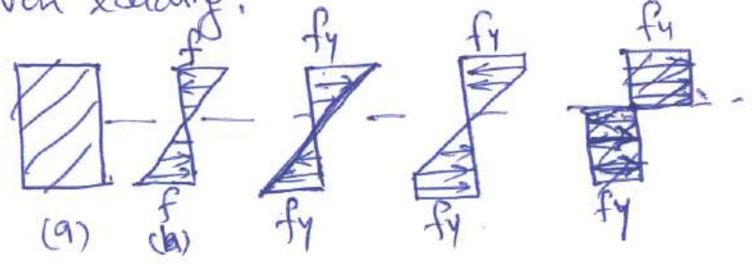
f_y = yield stress

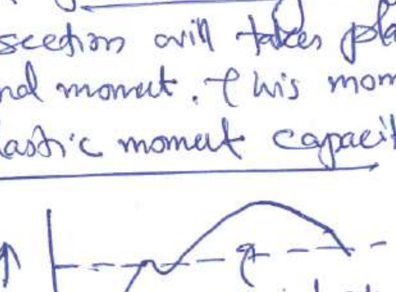
γ_{mo} = partial safety factor = 1.10 2

ii) Design strength due to rupture.

$T_{dn} = \frac{0.90 A_{nc} f_u}{\gamma_{m1}} + \beta \frac{A_{go} f_y}{\gamma_{mo}}$

where $\beta = 1.40 - 0.076 \left(\frac{W}{t}\right) \left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)$

Qn.		score split	Sub T	Total
	PART C			
	where lw = length of outstanding leg.			
	$b_s = a_s$ = shear leg width.			
	t = thickness of angle			
	L_c = length of weld along load direction.			
	$A_{nc} = \left[L - t/2\right] t$, $A_{go} = \left[b - t/2\right] t$.	3		
	iii) Design strength due to block shear:			
	$T_{db1} = \left[\frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.90 A_{tn} f_u}{\gamma_{m1}}\right]$			
	or			
	$T_{db2} = \left[\frac{0.90 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{mo}}\right]$			
	where A_{vg} = gross area in shear.			
	$A_{vg} = L_w (2 \times S)$			
	$A_{vn} = L_w (2S)$			
	A_{tg} = gross area in tension.			
	$A_{tg} = A_{tn} = L \times 2S$			
	S = Size of weld.	3	8	15.
	UNIT - III			
VII	a) Plastic moment carrying capacity of a beam section;			
	consider a SS beam where BM is max for given loading.			
		2.		

Qn.		Score Split	Sub T	Total
	PART C			
	As per this analysis, It is assumed that after yield point is reached strain goes on yielding without resisting any additional load. Hence according to theory of plastic analysis highly stressed fibre once yield is not capable of resisting any moment. But interior fibres are not yet yielded and hence additional loads are resisted by unyielded portion of ~~these~~ the section. As the load is gradually increased one by one fibre reach yield stress and stop resist additional load. fig (d) shows partially yielded case. However resistance to load ~~continuous~~ continues till all fibres are yielded as shown fig (e). This condition is formation of plastic hinge. After this stage rotation at section will takes place without resisting additional moment. This moment capacity is called plastic moment capacity 'Mp' of the section.	4		
		1	7.	
	b). Given: ISMB 350 @ 524 N/m. $f_y = 250 \ N/mm^2$. Properties: $h = 350$ mm. $b_f = 140$ mm. $t_f = 14.20$ mm. $t_w = 8.10$ mm. Solution: depth of web $d = h - 2t_f$ $= 350 - 2\times14.20$ $= 321.60$ mm.	2		

Qn.		Score split	Sub T	Total

PART C

$Z_{xx} = 778.90 \times 10^3 \ mm^3$.

$$M_d = \cancel{\frac{B_b Z_p f_y}{\gamma_{mo}}} \quad \frac{\beta_b Z_p f_y}{\gamma_{mo}}$$

Assuming plastic section. $\beta_b = 1$

$$S = \frac{Z_p}{Z_e} = 1.14 \text{ for Symmetrical I section.}$$

$$Z_p = 1.14 \times Z_{xx}$$
$$= 1.14 \times 778.90 \times 10^3 =$$
$$= \underline{887.946 \times 10^3 \ mm^3} \qquad 3$$

$$\therefore M_d = \frac{\beta_b \cdot Z_p \cdot f_y}{\gamma_{mo}} = \frac{1 \times 887.946 \times 10^3 \times 250}{1.10}$$

$$= \underline{\underline{201.80 \ KNm}}. \qquad 3 \qquad 8. \qquad 15.$$

OR

VIII

a) Classification of beam sections:

i) Class-1 (Plastic) Cross section:
These sections can develop plastic hinges and have rotation capacity required for failure of the structure by formation of plastic mechanism (width to ii) thickness ratio less than that specified in Table 7.1 of IS 800.

ii) Class-2 (Compact): Such sections can develop plastic moment of resistance, but have ~~find~~ inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling.

iii) Class-3- (Semi-compact): In which extreme fibre in compression can reach yield stress, but can't develop plastic

Qn.		score split	Sub T	Total

PART C

moment of resistance, due to local buckling.

iv) class-4 (slender): The cross section of element of which buckle locally even before reaching yield stress belongs to this category. (7).

b) Given: ISMB 500 @ 869 N/m.

Span $l = 6.50\ m$.

$f_y = 250\ N/mm^2$

Solution:

Properties $A = 11074\ mm^2$

$h = 500\ mm$.

$b_f = 180\ mm$.

$t_f = 17.20\ mm$,

$t_w = 10.20\ mm$, 2

$I_{xx} = 45218.3 \times 10^4\ mm^4$

$Z_{xx} = Z_e = \cancel{1808\times}\ 1808.7 \times 10^3\ mm^3$.

$$Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

where $\bar{y}_1 = \bar{y}_2 = \dfrac{A_1y_1 + A_2y_2}{A_1 + A_2}$

$\bar{y}_1 = \bar{y}_2 = 187.14\ mm$.

$$Z_p = \frac{11074}{2}(2 \times 187.14)$$

$Z_p = 2.072 \times 10^6\ mm^3$. 2

bending consideration:

$$Z_p = \frac{M_u \gamma_{mo}}{f_y}$$

$\therefore M_u = \dfrac{2.072 \times 10^6 \times 250}{1.10} = 470.90 \times 10^6\ Nmm$. 2

Qn.		score split	Sub T	Total
	PART C			
	$Mu = \frac{Wul^2}{8} = 470.90 \times 10^6$ Nmm			
	$\therefore Wu = 89.16$ N/mm.			
	$\therefore Wb = \frac{Wu}{1.50} = \frac{89.16}{1.50} = \underline{59.44 N/mm}$	2	8	15.
	UNIT - IV			
IX a)	Component parts of a roof truss. 1. Principal rafter. 2. Bottom chord. 3. Ties. 4. Struts. 5. Purline. 6. Rafters. 7. Ridge line 8. Eaves. 9. Panel points 10. Roof covering (Any 7)	7.	7.	~~15~~.
b)	Given: Load = ~~9.80~~ 10 KN/m. eff: length = 3.30 m. eff: height = 3.0 m. Solution: Assuming a wall thickness 200 mm eff: height < eff. length $\therefore$ Slenderness ratio = $\frac{\text{Eff. height}}{\text{Eff. Thickness}}$ $= \frac{3000}{200} = 15$.	2		
	Assuming cement mortar (1:5) and brick with comp. strength 5.0 N/mm2 the basic comp. stress from table (8) of IS 1905			

Qn.		Score split	Sub T	Total
	PART C			
	$f_b = 0.50\ N/mm^2$.			
	Net area of wall $= A = \cancel{3.30 \times 2}$ 3300×200 $= 660000\ mm^2$			
	$= 0.66\ m^2 > 0.20\ m^2$ hence no reduction factor needed $\therefore k_a = 1$.	3		
	Permissible comp. stress $f_c = k_a . k_s . k_p \times f_b$			
	$= 1 \times 0.84 \times 1.20 \times 0.50$			
	$= 0.504\ N/mm^2$.			
	$\therefore$ ~~Actual~~ comp. stress $f = \frac{P}{A} = \frac{10 \times 10^3}{0.20 \times 10^4}$			
	$= \frac{10000}{1000 \times 200} = 0.05\ N/mm^2$	2		
	$0.05\ N/mm^2 < 0.50\ N/mm^2$			
	Hence provide 200 mm thick wall with bricks of comp. strength $5.0 N/mm^2$ in cm 1:5.	1.	8.	15
	OR			
X a)	Design considerations:			
	Masonry structures gain stability from the support offered by cross walls, floors, roofs and other elements such as piers and buttresses. Eccentric loading is to be avoided by providing adequate bearing of floor slab or roof slab on the wall.			
	The strength of wall is measured in terms of its resistance to the combination of self wt,			

Qn.		score split	Sub T	Total

PART C

super imposed loads and lateral pressure.
The stability of wall is indicated by resistance to overturning by lateral force and buckling caused by excessive slenderness ratio.

The strength of masonry wall depends on the strength of masonry unit and strength of mortar.
The quality of workmanship and method of bonding also ~~eff~~ have effect on strength.
The thickness of 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.
The design is one basis of ~~for~~ permissible comp: stress. (7).

b)

Solution:

From wind zone Map $V_b = 47$ m/sec

Risk coefficient $k_1 = 1.0$ (Referring table for Gen: buildings).

$k_2 = 0.88$ if $h = 10$ m
$= 0.94$ if $h = 15$ m.

$\therefore$ for $h = 12$ m. (Referring table 875)
$k_2 = 0.904$ — 2

Topographic factor $k_3 = 1 + Cs$.
where $C = \frac{Z}{L} = 0$.
$\therefore k_3 = 1$.
Design wind speed $V_z = k_1 k_2 k_3 V_b$ — 2

Qn.		Score split	Sub T	Total
	PART C			
	$= 1.0 \times 0.904 \times 1.0 \times 47.$			
	$= 42.488 \ m/sec.$	2		
	Hence basic wind pressure			
	$P_z = 0.60 V_z^2 = 0.60 \times 42.488^2$			
	$= 1083 \ N/m^2$ or $\underline{1.083 \ kN/m^2}$	2	8.	15.