Important Instruction to Examiners:-

1) The answers should be examined by key words & not as word to word as given in the model answers scheme.

2) The model answers & answers written by the candidate may vary but the examiner may try to access the understanding level of the candidate.

3) The language errors such as grammatical, spelling errors should not be given more importance.

4) While assessing figures, examiners, may give credit for principle components indicated in the figure.

5) The figures drawn by candidate & model answer may vary. The examiner may give credit for any equivalent figure drawn.

5) Credit may be given step wise for numerical problems. In some cases, the assumed contact values may vary and there may be some difference in the candidate's answers and model answer.

6) In case of some questions credit may be given by judgment on part of examiner of relevant answer based on candidates understanding.

7) For programming language papers, credit may be given to any other programme based on equivalent concept.

Important notes to examiner

Q .NO	SOLUTION	MARKS
Q.1 a)		
i)	List various limit states and define any one.	
Ans:-	* Various limit states are:-	
	1) Limit state of collapse	(01M)
	a) Axial load	
	b) Flexure or Bending	
	c) shear	
	d) Torsion	
	2) Limit state of serviceability	(01M)
	a) Deflection	
	b) Cracking	
	1) Limit state of collapse :-	
	The resistance to bending, shear, torsion and axial loads at every section shall not be less than appropriate value at that section produced by probable most unfavourable combination of loads on structure using appropriate partial safety factors.	(02M) Any ONE explaination

Q .NO	SOLUTION	MARKS
	2) Limit state of serviceability :-	
	a) Deflection :-	
	1) Final deflection due to loads including effect of Temp., creep, shrinkage and measured from supports of all horizontal member should not normally exceed @ $\frac{span}{250}$.	
	2) The deflection including effect of Temp., creep & shrinkage occuring after erection of partitions and application of finishes should not normally exceed $\frac{span}{350}$ or 20 mm which is less.	
	3) Beams & slabs, vertical deflection limits may generally assumed to be satisfied provided that span to depth ratios are not greater than values obtained as below:	

i) For span upto 10m	$\frac{L}{d}$ Ratio
a) Cantilever	7
b) Simply support	20
c) Continuous	26
ii) For span above 10m	$\frac{L}{d} \times \frac{10}{span}$

Q .NO	SOLUTION	MARKS
ii)	Define magnitude of earthquake and intensity of earthquake.	
Ans:-	* Magnitude of Earthquake:-	
	"Magnitude is defined as it is measure of amount of energy released during an earthquake." Magnitude is a number that characterizes relative size of an earthquake.	(02M)
	* Intensity of Earthquake:-	
	The intensity of an earthquake at a perticular locality indicates the violence of earth motion produced there by the earthquake it is determined from reported effects of the tremor on human beings, furniture, buildings, geological structure, etc, Many places, including Hong Kong, have adopted the modified Mercallis scale (MMS) which classifies earthquake effects into twelve grades	(02M)

Q .NO	SOLUTION	MARKS
iii)	State 4 assumptions made in theory of bending of singly R/F section.	
Ans:-	* Assumptions in theory of bending of singly Reinforced section:-	
	1) Plane sections normal to axis remain plane after bending.	
	2) The max. strain in concrete at outer most compression fibre is 0.0035 in bending.	
	3) The tensile strength of concrete is ignored.	
	4) Stresses in R/F are derived from stress-strain curve for the type of steel used.	
	5) There is perfect bond betn steel and concrete right up to failure of RC section.	
	6) Max. strain in tension reinforcement in section at failure shall not less than $\epsilon_{su} = \frac{f_y}{1.15 E_s} + 0.002$	(1M for each Write Any (four))

Q .NO	SOLUTION	MARKS
iv)	List 4 losses in prestressing and explain any one of them.	
Ans:-	* Losses in prestressing :-	
	1) Loss of prestress due to Friction:- 2) Loss of prestress due to Anchorage slip 3) Loss of prestress due to Creep:- 4) Loss of prestress due to Shrinkage	(1/2 M) each write ANY (four)
	1) Loss of prestress due to Friction:-	
	Friction generated at interface of concrete and steel during the stretching of a curved tendon in post-tensioned member, leads to drop in prestress along member from end. Friction is generated due to curvature of tendon & vertical component of prestressing force.	(02M) for ANY ONE Explaination
	2) Loss of prestress due to Anchorage Slip:- In a post tensioned member, when prestress is transferred to concrete, the wedges slip through a little distance before they get properly seated in conical space	

Q .NO	SOLUTION	MARKS
	There is loss of prestress due to consequent reduction in length of tendon.	
	3] Loss of prestress due to Creep :-	
	Creep of concrete is defined as increse in deformation with time under constant load. Due to creep of concrete, the prestress in the tendon is reduced with time. The creep is due to sustained loads. Since, prestress may vary along the length of member, an avg. value of prestress can be considered.	
	4] Loss of prestress due to shrinkage of concrete :-	
	shrinkage of concrete is defined as contraction due to loss of moisture. Due to shrinkage of concrete, the prestress in tendon is reduced with time.	

Q .NO	SOLUTION	MARKS
v)	Why contribution of bent up bar is restricted to 50% in shear Resistance.	
	→ Contribution of bent up bar is restricted to 50% in shear resistance is because the of following reasons	
	i) Bent up bars tend to cause longitudinal cracking or crushing of concrete at bent point, as the bar tries to straighten due to tension	
	ii) They are unable to prevent pressing down of longitudinal reinforcement & consequent Spitting of concrete	
	iii) They fail to confine the concrete.	
	iv) Bent up bars alone (without stirrups) are not effective in preventing shear failure	(04M)

Q .NO	SOLUTION	MARKS
Q.1 b)		
i)	A R.C. section 250x450 mm effective is Reinforcement with 4-16 mm ϕ bars of Fe415 on tension side. If M20 concrete is used, calculate ultimate moment of resistance the beam can offer.	
Ans:-	* Given:-	
	$b = 250$ mm	
	$d = 450$ mm	
	$F_{ck} = 20\ N/mm^2$	
	$F_y = 415\ N/mm^2$	
	$A_{st} = (\pi/4 \times 16^2) \times 4 = 804.25\ mm^2$	
	Step 1 :-	
	Find $x_{u\,max}$:-	
	$x_{u\,max} = 0.48\,d$ --- for Fe415	(1/2 M)
	$= 0.48 \times 450$	
	$\boxed{x_{u\,max} = 216\ mm}$	(1/2 M)
	$x_u = \dfrac{0.87 \times F_y \times A_{st}}{0.36 \times F_{ck} \times b}$	
	$= \dfrac{0.87 \times 415 \times 804.25}{0.36 \times 20 \times 250}$	
	$= 161.31$ mm	
	$x_u < x_{u\,max}$	
	Hence beam is under Reinforced.	(02M)

Q .NO	SOLUTION	MARKS
	Step 3:-	
	Moment of Resistance (Mu):-	
	$Mu = 0.87\, fy\, Ast\, (d - 0.42\, xu)$	(1 M)
	$= 0.87 \times 415 \times 804.25\, (450 - 0.42 \times 161.31)$	(1 M)
	$\boxed{Mu = 110.99 \text{ KN-m}}$	(1 M)
ii)	Calculate ultimate moment of resistance & steel required for a beam 230 x 400 mm eff. if M25 & Fe500 are used.	
Soln:-	* Given:-	
	$b = 230$ mm	
	$d = 400$ mm	
	$fck = 25\ N/mm^2$	
	$fy = 500\ N/mm^2$	(01 M)
	Step 1:	
	Moment of Resistance:-	
	$Mu = 0.133\, fck\, b\, d^2$	(01 M)
	$= 0.133 \times 25 \times 230 \times 400^2$	
	$\boxed{Mu = 122.36 \times 10^6 \text{ N-mm}}$	(01 M)

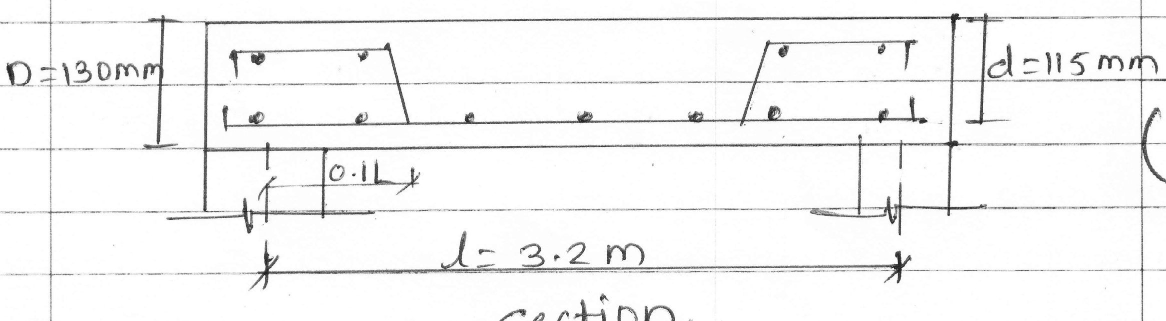
Q .NO	SOLUTION	MARKS
	Area of steel Ast.	
	$x_{umax} = 0.456 \times 400 = 182.4$.	(01M)
	$A_{st\,max} = \dfrac{0.36 \times f_{ck} \times b \times x_{umax}}{0.87 f_y}$	(01M)
	$A_{st\,max} = \dfrac{0.36 \times 25 \times 230 \times 182.4}{0.87 \times 520} = 874\,mm^2$	(01M)

Q .NO	SOLUTION	MARKS
Q. 2		
a)	Design simply supported RCC slab over a passage of eff span = 3.2m using M25 & Fe415. Assume imposed load including floor finish is 3 KN/m² and M.F. = 1.4.	
Ans:-	* Given:-	
	$l = 3.2 m$	
	$F_{ck} = 25 N/mm^2$	
	$F_y = 415 N/mm^2$	
	$w = 3 KN/m^2$	
	M.F. = 1.4	
	Step 1:-	
	Thickness of slab :-	
	$d_{assumed} = \frac{span}{20 \times M.F.}$	(½M)
	$= \frac{3200}{20 \times 1.4} = 114.28 mm \approx 115 mm$ say	
	Proved 15mm cover.	
	D = 115 + 15 = 130mm	(½M)
	Step 2:-	
	Consider loading for 1m strip	

Q .NO	SOLUTION	MARKS
	i) self wt. $= 0.13 \times 1 \times 25 = 3.25$ KN/m	
	ii) Superimposed load $= 3$ KN/m²	
	Total load (w) $= 6.25$ KN/m	(½M)
	∴ Factored Load $= 1.5 \times 6.25$	
	$(w_u) = 9.375$ KN/m	(½M)
	step 3:-	
	Factored B.M (Md):-	
	$\therefore Md = \frac{W_u l^2}{8} = \frac{9.375 \times 3.2^2}{8}$	
	$= 12$ KN-m	(½M)
	step 4:-	
	Calculate depth required:-	
	For Fe415,	
	$Md = 0.138 f_{ck} b d^2$	(½M)
	$\therefore 12 \times 10^6 = 0.138 \times 25 \times 1000 \times d^2$	
	$\boxed{\therefore d = 58.97 \text{ mm}}$	(½M)
	dreq. < d ass.	
	Hence safe for bending.	

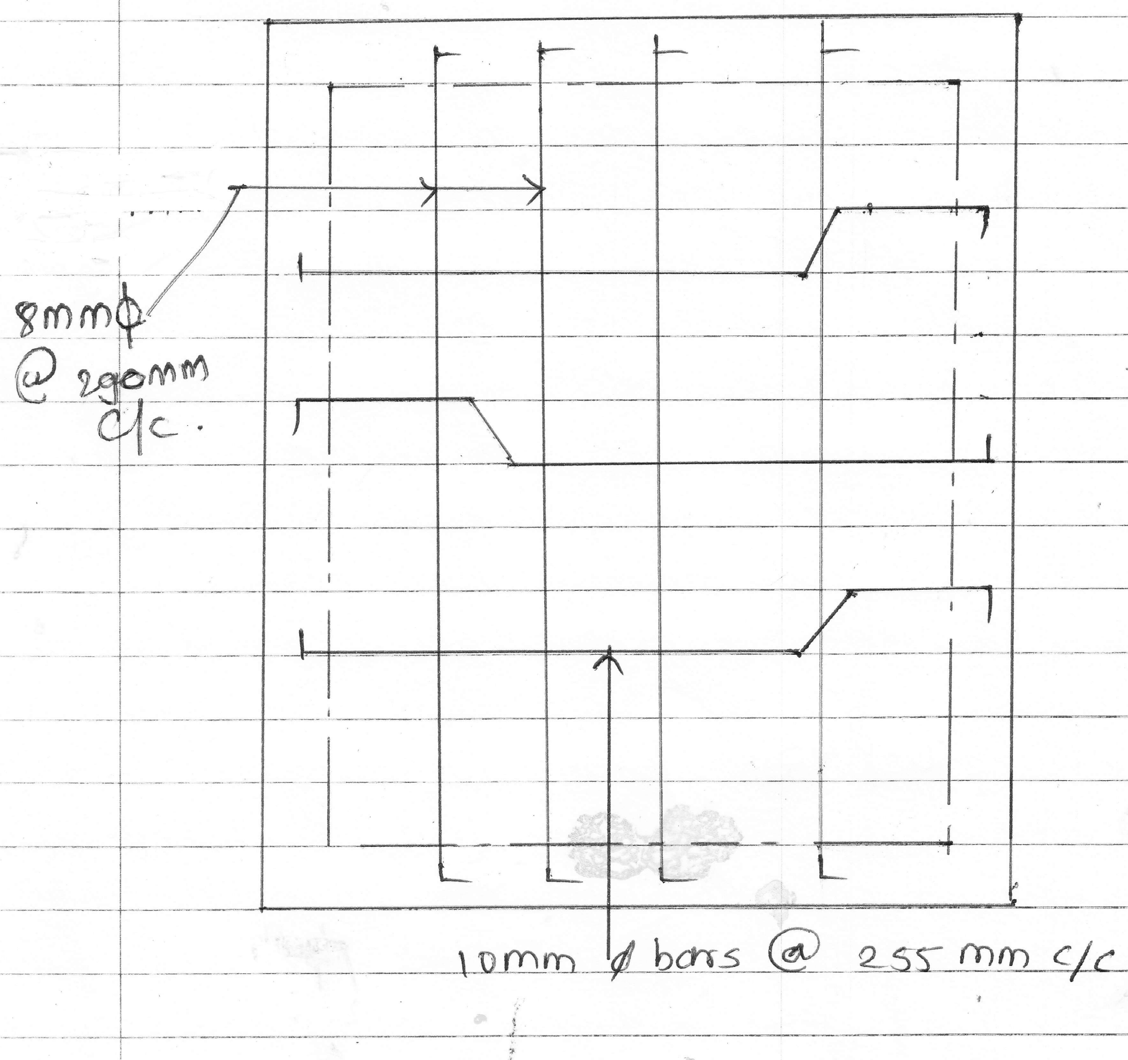
Q .NO	SOLUTION	MARKS
	step 5:-	
	* Area of main steel (Ast):-	
	$Ast = \frac{0.5\,fck}{fy}\left[1-\sqrt{1-\frac{4.6\,Mu}{fck\,bd^2}}\right] b \times d$	(½M)
	$= \frac{0.5 \times 25}{415}\left[1-\sqrt{1-\frac{4.6 \times 12 \times 10^6}{25 \times 1000 \times 115^2}}\right] \times 1000 \times 115$	(½M)
	$\boxed{Ast = 302.35\ mm^2}$	(½M)
	Provide 10mm ϕ bars,	
	spacing $(s) = \frac{\pi/4 \times 10^2}{302.35} \times 1000$	(½M)
	$s = 259.76\ mm \approx 255\ mm$	
	$s \ngtr 3d$ or $300\ mm$	(½M)
	Hence safe.	
	* Area of Distribution steel:-	
	$Astd = \frac{0.15}{100} \times 1000 \times 130$	(½M)
	$\boxed{Astd = 195\ mm^2}$	
	Provide 8mm ϕ bars	
	spacing $(s) = \frac{\pi/4 \times 8^2}{195} \times 1000$	(½M)
	$257.64 \cong 255\ mm$	

Q .NO	SOLUTION	MARKS
	$S \not> 5d$ or 450 mm Hence safe.	



section.

(1/2 M)



Plan

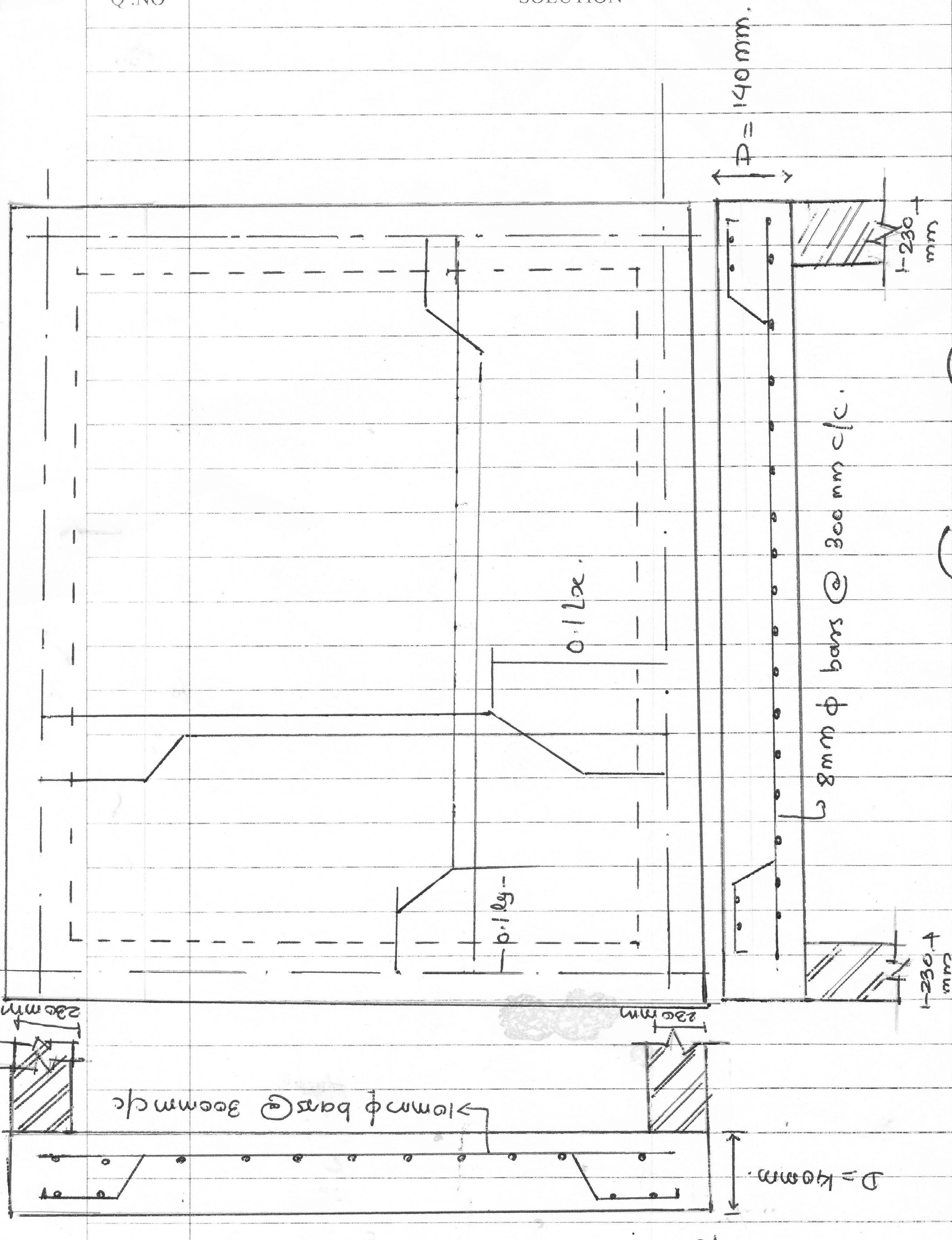
(1/2 M)

Q .NO	SOLUTION	MARKS
b)	Design a simply supported slab of 4x6 m. The slab is subjected to L.L. = 3.5 kN/m² & F.F = 1 kN/m². Use M25 & Fe500. Assume M.F. = 1.6 $\alpha_x = 0.104$ & $\alpha_y = 0.046$.	
Ans:-	$\frac{l_y}{l_x} = \frac{6}{4} = 1.5 < 2$	
	Slab is Two-way	(1/2 M)
	Step 1:- Thickness of slab,	
	$d_{ass.} = \frac{l_x}{20 \times M.F.}$	(1/2 M)
	$= \frac{4000}{20 \times 1.6}$	
	$\boxed{d_{ass.} = 125 \text{ mm}}$	
	Assume cover 15 mm D = 125 + 15 = 140 mm	(1/2 M)
	Step 2:- Loading for 1m strip	
	1) Self wt = 0.14 X 1X25 = 3.5 kN/m	
	2) Live Load = 3.5 kN/m	
	3) Floor Finish = 1 kN/m	
	Total = 8 kN/m.	(01M)

Q .NO	SOLUTION	MARKS
	Factored load $(w) = 1.5 \times 8$	
	$= 12$ kN/m	½M
	Step 3	
	Factored Moment:-	
	* B.M. along shorter span:-	
	$M_x = w \times l_x^2 \times \alpha_x$	
	$= 12 \times 0.104 \times 4^2$	
	$= 19.96$ kN-m	½M
	* BM along longer span:-	
	$M_y = w \times \cancel{l_x}^2 \times \alpha_y$	
	$= 12 \times 0.046 \times 4^2$	
	$= 8.83$ kN-m	½M
	Step 4	
	Depth required.	
	For Fe500,	
	$M_d = 0.133\, f_{ck}\, b d^2$	½M
	$\therefore 19.96 \times 10^6 = 0.133 \times 25 \times 1000 \times d^2$	
	$d = 77.47$ mm	½M
	$d_{req.} < d_{assu.}$	
	$\therefore$ safe for Bending.	

Q .NO	SOLUTION	MARKS
	Step 5:-	
	Area of main steel along shorter span:-	
	$* \; Ast = \frac{0.5 \times 25}{500}\left[1-\sqrt{1-\frac{4.6 \times 19.96 \times 10^6}{1000 \times 125^2 \times 25}}\right] 1000 \times 125$	
	$= 391.82 \; mm^2$	(1/2M)
	Using 10 mm ϕ bars,	
	Spacing $(s) = \frac{\pi/4 \times 10^2}{391.82} \times 1000$	
	$= 200.44 \approx 200 \; mm$	
	$s \ngtr 3d$ or 300 mm	(1/2M)
	$\therefore$ safe.	
	Area of ~~Distribution~~ steel: along longer span:-	
	$Ast = \frac{0.5 \times 25}{500}\left[1-\sqrt{1-\frac{4.6 \times 8.83 \times 10^6}{1000 \times 125^2 \times 25}}\right] 1000 \times 125$	
	$= 166.93 \; mm^2$	(1/2M)
	Using 8 mm ϕ bars	
	Spacing $(s) = \frac{\pi/4 \times 8^2}{166.93} \times 1000$	
	$= 301 \; mm \approx 300 \; mm$	
	$s \ngtr 3d$ or 300 mm	(1/2M)

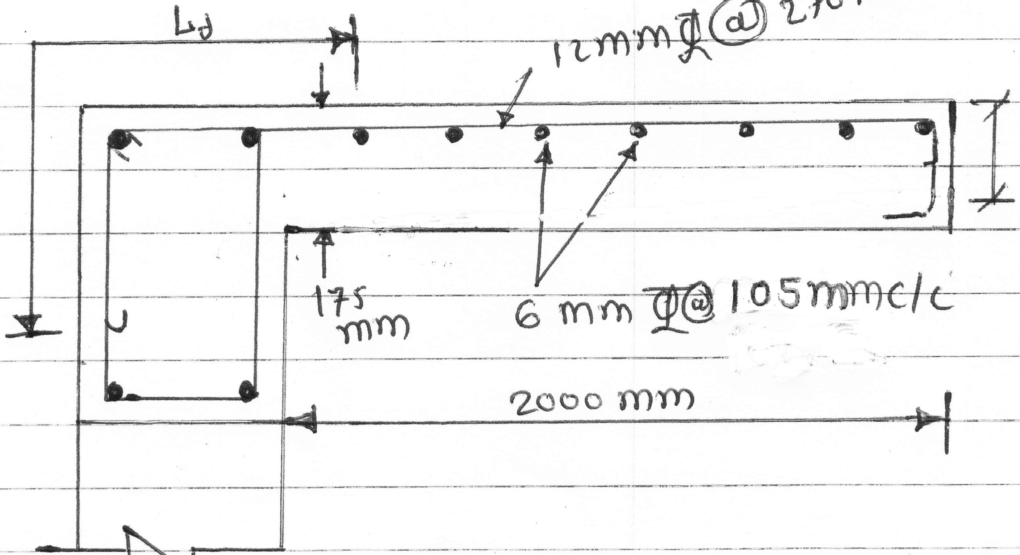
$\therefore$ safe.

Q .NO	SOLUTION	MARKS
		($\frac{1}{2}$M for plan) ($\frac{1}{2}$M for section)

Two-way slab Detailing

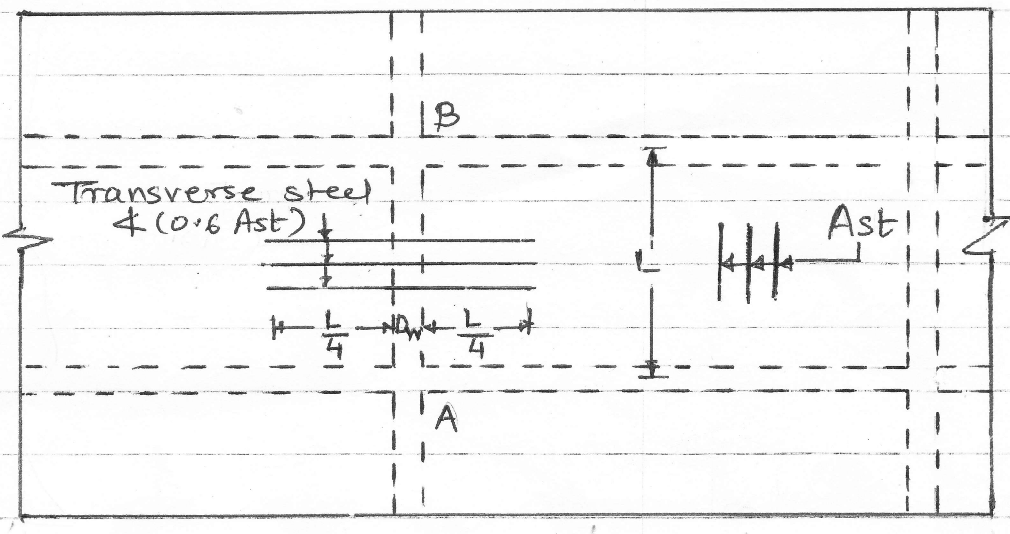
Q .NO	SOLUTION	MARKS
c]	Design a cantilever slab of span 2m using M20 & Fe415 having udl = 2.5 KN/m² M.F. = 1.8	
Ans:-	* Given:-	
	$l = 2m$	
	$f_{ck} = 20\ N/mm^2$	
	$f_y = 415\ N/mm^2$	
	Step 1:-	
	Slab Thickness:-	
	$d = \frac{span}{7 \times M.F.} = \frac{2000}{7 \times 1.8}$	(½M)
	$= 158.73\ mm \simeq 160\ mm$	
	$D = 160 + 15\ mm$	
	$D = 175\ mm$	(½M)
	$\therefore\ l = 2000 + \frac{160}{2} = 2080\ mm$	
	Step 2 Load calculation:-	
	1) Self wt. $= 0.175 \times 25 \times 1 = 4.375$	
	2) L.L. $= 2.5\ KN/m$	
	Total $= 6.875\ KN/m$	
	Factored load $= 1.5 \times 6.875$	
	$= 10.312\ KN/m$	(2M)

Q .NO	SOLUTION	MARKS
	step 3 :- Factored B.M:-	
	$Md = \frac{Wl^2}{2} = \frac{10.312 \times 2.08^2}{2}$	
	$= 22.30$ KN-m	(½M)
	step 4)	
	Depth required	
	$Md = Mu_{max}$	
	$22.30 \times 10^6 = 0.138\, f_{ck}\, b\, d^2$	(½M)
	$22.30 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$	
	$d = 89.88 < d_{ass.}$	
	Hence safe	(½M)
	step 5 Area of main steel:-	
	$Ast = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 22.30 \times 10^6}{20 \times 1000 \times 160^2}}\right] \times 1000 \times 160$	(½M)
	$= 407.78\ mm^2$	
	Using 12 mm ϕ bars	
	$S = \frac{\pi/4 \times 12^2}{407.78} \times 1000$	(½M)
	$= 277.34$ mm say 270 mm	
	$S \not> 3d$ or 300 mm	
	Hence safe.	

Q .NO	SOLUTION	MARKS
	* Area of distribution steel:-	
	$A_{std} = \frac{0.15}{100} \times 1000 \times 175$	(½M)
	$= 262.5\ mm^2$	
	Using 6 mm ∅ bars	
	$S = \frac{\pi/4 \times 6^2}{262.5} \times 1000$	(½M)
	$= 107.70$ mm say 105 mm c/c	
	$S \not> 5d$ or 450 mm	(½M)
	Hence safe.	
		(01M)

Q .NO	SOLUTION	MARKS
Q3 a)	Given	
	Clear span of beam = 6.20 m	
	width of support = 300 mm	
	Spacing of beam = 3 m c/c	
	width of Web (b_w) = 250 mm	
	slab thikness (D_f) = 120 mm.	
	i) Effective span of T-beam = clear span + width of support	
	$L_o = 6.20 + 0.30$	
	$L_o = 6.50$ m.	(1M)
	ii) Effective flange width of T-beam b_f	
	$b_f = \left(\frac{L_o}{6} + b_w + 6D_f\right)$	(2M)
	$= \left(\frac{6500}{6} + 250 + 6 \times 120\right)$	
	$b_f = 2053.33$ mm $\not>$ Centre to Centre distance of beam (3m)	(1M)
	$\therefore b_f = 2053.33$ mm.	
Q3 b)	IS code recommends the following two provisions for beam spanning parallel to slab to act as T-beam.	
	i) Transverse reinforcement (perpendicular to beam) is required to be provided at the top in flange portion for a length	$1\frac{1}{2}$M

Q .NO	SOLUTION	MARKS
	equal to L/4 on each side of beam, where L = span of slab.	
	ii> Transverse reinforcement > 60% of main steel of slab. (i.e. Transverse reinforcement > 0.60 Ast)	($1\frac{1}{2}$M)



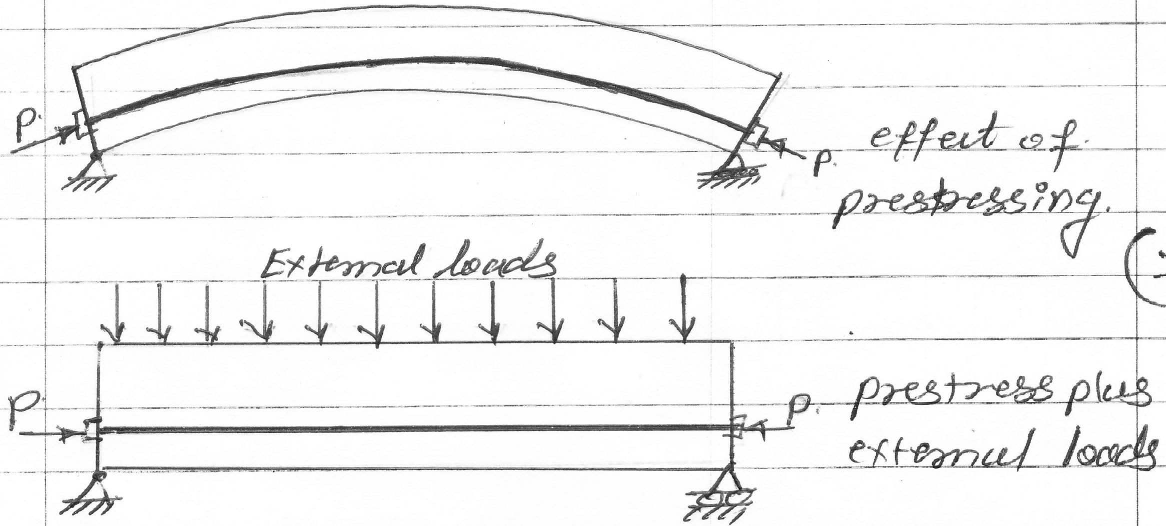
(1M)

Q .NO	SOLUTION	MARKS
Q3c>		
→	When the shear resisted by concrete is less half ultimate concrete strength (i.e. Vuc/2) only nominal shear reinforcement having minimum diameter at the maximum spacing should be provided to hold the longitudinal bars in position.	(2M)
Or →	Also when Vu < Vuc then provide min. shear r/f	
	$\frac{A_{sv}}{b \cdot S_v} \geq \frac{0.4}{0.87 f_y}$ or $S_v \leq \frac{0.87 f_y A_{sv}}{0.4 b}$	(1M)

Vu = Total shear Vuc = shear resisted by concrete

Q .NO	SOLUTION	MARKS
	where,	
	A_{sv} = Total c/s area of stirrup legs effective in shear	
	S_v = spacing of stirrups $\leq$ (0.75d or 300mm)	(1M)
	b = width of member	
	f_y = characteristic strength of stirrup reinforcement.	
Q3d)	Given data	
	diameter $\phi = 16mm$, Fe = 415 MPa,	
	$\tau_{bd} = 1.4$ MPa for plain bar in tension.	
	Development length $L_d = \frac{0.87 f_y \cdot \phi}{4 \tau_{bd}}$	(2M)
	$= \frac{0.87 \times 415 \times 16}{4 \times (1.6 \times 1.4) \times 1.25}$	
	$L_d = 515.78$ mm	(2M)
Q3e)	Given data	
	c/s of Column = 400 x 400 mm	
	A_{sc} = 8 – 16mmϕ, M20 & Fe 500	
	i) $A_{sc} = 8 \times \frac{\pi}{4}(16)^2 = 1608.49\ mm^2$	($\frac{1}{2}$M)
	$A_c = A_g - A_{sc} = (400 \times 400 - 1608.49)$	
	$A_c = 158.39 \times 10^3\ mm^2$	($\frac{1}{2}$M)

Q .NO	SOLUTION	MARKS
	Ultimate load carrying capacity of Column	
	$P_u = 0.4 f_{ck} A_c + 0.67 f_y \cdot A_{sc}$	(1M)
	$P_u = 0.4 \times 20 \times 158.39 \times 10^3 + 0.67 \times 500 \times 1608.49$	
	$P_u = 1805.96$ KN	(1M)
	$\therefore$ Safe Load $P = \frac{P_u}{1.5} = \frac{1805.96}{1.5}$	
	$\therefore P = 1203.97$ KN	(1M)
Q4 a) i)		
→	Basic principle of prestressing. In prestressed Concrete the Compressive stresses are induced in the Concrete section before the member is subjected to bending due to external loads. The magnetude of Compressive force leading to distribution of stresses produced in the section due to external loads are eliminated or reduced to permissible value. The Compressive force applied to the section is known as prestressing force.	(3M)
	OR	
	Prestressed Concrete is basically Concrete in which internal stresses of a suitable magnetude and distribution are introduced ~~to~~ so that the stresses resulting from external loads Counteracted to desired degree.	(3M)

Q .NO	SOLUTION	MARKS
		(1M)
Q4 a) ii)	→ Functions of Lateral ties in the Column.	
	1) To prevent buckling of longitudinal bars	
	2) To prevent longitudinal splitting of Concrete	(1 Mark each for any four)
	3) To resist diagonal tension due to transverse shear	
	4) To Confine the Concrete.	
	5) To hold the longitudinal reinforcement in position	
	6) To prevent or delay sudden Collapse and impart necessary ductility to the members.	

Q .NO	SOLUTION	MARKS
Q4 a)	iii)	
→	* Balanced section	
	If stresses in steel & concrete reach to its maximum value at one and the same time, such type of section is called balanced or critical or economical section.	(1M)
	$x_u = x_{umax}$.	
	* Over reinforced section	
	If more steel than that required for balanced section is used, the section is over reinforced. In such type of section compressive stress in concrete reaches its maximum permissible value while steel is not fully stressed to maximum permissible stress and the beam will fail initially due to overstressed in concrete.	(1M)
	$x_u > x_{umax}$.	
	* Under reinforced section	
	If steel less than that required for balanced section is used the section is under reinforced. In such section steel reaches its permissible value of stress first, while the stress produce in concrete is less than its permissible value.	(1M)
	$x_u < x_{umax}$	
	* In practice generally under reinforced & ballanced sections are preferred.	(1M)

Q .NO	SOLUTION	MARKS
Q4 a)	iv)	
→	Definition:- A R.C.C. beam in which the reinforcements are provided on both tension and Compression side is known as doubly reinforced beam/section.	(2M)
	* Situations	
	<1> When the applied moment exceeds the moment resisting capacity of a singly reinforced section.	(1M each for any two)
	<2> When the section of the beam is restricted due to the requirements of head room, appearance etc.	
	<3> Compression steel is provided sometimes to reduce the deflection and also to increase the rotation capacity.	
	<4> When the sections are subjected to reversal of bending moment.	
	<5> In Continuous T-beams where the portion of beam over middle support has to be designed as doubly reinforced.	
	<6> When the beams are subjected to eccentric loading, shocks, impact load.	

Q .NO	SOLUTION	MARKS
Q.4		
b (i)	Given:-	
	$b = 250\ mm$	
	$D = 450\ mm$ = $\therefore d = 450 - 40 = 410\ mm$	
	$A_{st} = 1250\ mm^2$	
	$A_{sc} = 240\ mm^2$	
	$d_c = 40\ mm$	
	$f_{ck} = 20\ N/mm^2$	
	$f_y = 250\ N/mm^2$	
	Sol^n:-	
	Assume $f_{sc} = 0.87 f_y$	
	$x_u = \dfrac{0.87 f_y A_{st} - f_{sc} A_{sc}}{0.36 f_{ck} \cdot b}$	(1/2 M)
	$= \dfrac{0.87 \times 250 \times (1250 - 240)}{0.36 \times 20 \times 250}$	
	$= 109.42\ mm$	(01 M)
	for fe 250, $x_{u\,max} = 0.53 d = 0.53 \times 410$	
	$= 217\ mm > x_u$	(1/2 M)
	$\therefore$ The section is under-reinforced.	
	check for f_{sc}, $f_{sc} = 700(1 - d_c/x_u)$	(1/2 M)

Q .NO	SOLUTION	MARKS
	$= 700\left(1 - 40/109.42\right)$	
	$= 444.11\ N/mm^2 > 0.87 f_y\ \cancel{\leq}\ 217.5\ N/mm^2$	(½M)
	$\therefore$ Take $f_{sc} = 217.5\ N/mm^2$	
	$\therefore M_{ur} = 0.36 f_{ck} b x_u (d - 0.42 x_u) + f_{sc} \cdot A_{sc} (d - d_c)$	(01M)
	$= [0.36 \times 20 \times 250 \times 109.42 (410 - 0.42 \times 109.42) + 217.5 \times 240 (410 - 40)] 10^{-6}$	(01M)
	$= 91.01\ KN \cdot m$	(01)

Q.NO	SOLUTION	MARKS
Q 4.		
b (ii)	Given:-	
	$f_{ck} = 25\ N/mm^2$	
	$f_y = 415$	
	$b = 300$	
	$d = 500mm$	
	$M_u = 350\ KNm.$	
	$d_c = 50\ mm$	
	$f_{sc} = 353\ MPa.$	
	Sol^n To find Asc and Ast.	
	for M:25 $R_{umax} = 3.45$	
	$P_{t\,max} = 1.2\%$ — fe 415.	
	$K_{umax} = 0.48$	
	or	
	$R_{umax} = 0.36 \times 25 \times 0.48 \times (1 - 0.42 \times 0.48)$	(½M)
	$R_{umax} = 3.449\ N/mm^2$	(½M)
	$M_{urmax} = R_{umax} \cdot bd^2$	(½M)
	$= 3.449 \times 300 \times 500^2$	
	$= 258.6\ KNm < 350\ KN \cdot m$	(½M)
	Hence design as doubly r/f.	
	$M_{u2} = M_u - M_{urmax}$	(½M)
	$= 350 - 258.6 = 91.325\ KN \cdot m$	(½M)
	Tension steel	
	$A_{st1} = \dfrac{M_{urmax}}{0.87 f_y (d - 0.42\, x_{umax})}$	(½M)

Q .NO	SOLUTION	MARKS
	$x_{u max} = 0.53 \times 500$	
	$x_{u max} = 265\, mm$	(½M)
	$Ast1 = \dfrac{258.6 \times 10^6}{353 \times (500 - 0.42 \times 265)}$	
	$Ast1 = 1884.68\ mm^2$	(½M)
	$Ast2 = \dfrac{Mu2}{0.87 fy (d - dc)}$ ... Assume $dc = 40mm$	
	$Ast2 = \dfrac{91.325 \times 10^6}{353 \times (500 - 40)}$	
	$Ast2 = 562\ mm^2$	(½M)
	$Ast = 1884.68 + 562 = 2447.09\ mm^2$. ok	
	$Asc = \dfrac{0.87 \times 415 \times 562}{353} = 574\ mm^2$	½M
	Summary	
	$Ast1 = 1884.68 mm^2$	
	$Ast2 = 562\ mm^2$	
	$Asc = 574 mm^2$	(½M)

Q.NO 5	SOLUTION	MARKS
(a)	Effective span = 6.0 m	
	working live load = 40 kN/m.	
	overall depth of beam is restriced to 600 mm	
	width = 300 mm	
	Cover = 50 mm assume	
(a)	Stresses	
	$f_{ck} = 20\ N/mm^2$	
	$f_y = 415\ N/mm^2$	($\frac{1}{2}$M)
	load factor = 1.5 for dead loads and live loads	
(b)	loads	
	Total working load = 40 kN/m × 1.5	
	= 60 kN/m.	(1M)
(c)	moments and shear force	
	$M_u = (60 \times 6^2 \times 0.125) = 270\ kN.m$	
	$V_u = (0.5 \times 60 \times 8) = 240\ kN.m$.	(1M)
(d)	Limiting moments of resistance	
	$M_{u\,lim} = 0.138 f_{ck} b d^2$	(½M)
	$M_{u\,lim} = 0.138 \times 20 \times 300 \times 550^2$	
	$= 250.47\ kN.m < M_u$	
	Hence design as doubly r/f section.	(1½M)

Q .NO	SOLUTION	MARKS
	$f_{sc} = 354$ already given	
	$Ast1 = \frac{Mulim}{0.87 \times fy (d - 0.42 xumax)}$	(01/2 M)
	$Ast1 = \frac{250.47 \times 10^6}{0.87 \times 415 (550 - 0.42 \times 0.48 \times 550)}$	
	$Ast1 = 1579.8\ mm^2$	01M
	$Ast2 = \frac{Mu - mulim}{0.87 \times fy (d - dc)}$	(01/2 M)
	$Ast2 = \frac{270 - 250.47 \times 10^6}{0.87 \times 415 (550 - 50)} = 108.18\ mm^2$	01M
	$Asc = \frac{0.87 fy Ast2}{fsc}$	1/2M
	$Asc = \frac{0.87 \times 415 \times 108.18}{354}$	
	$Asc = 110.33\ mm^2$	(1/2M)
	Total $Ast = 1579.8 + 108.18 = 1687.98\ mm^2$	

Q .NO	SOLUTION	MARKS
	Provide 5 bars of 25 mm ϕ	
	Ast provided = 2454.36 > 1829 Hence ok	(1/2 M)

Q .NO	SOLUTION	MARKS
(Q.5)		
(b)	Design shear r/f in form of vertical stirrups	
	Data given :-	
	Beam section = 300 x 600 mm	
	Ultimate shearforce = V_u = 300 kN.	
	τ_{uc} = 0.65 N/mm^2	
	f_{ck} = 25 N/mm^2.	
	f_y = 415 N/mm^2.	
	Step I] Shear resisted by concrete	
	$V_{uc} = \tau_{uc} \times b \times d$.	(1M)
	$V_{uc} = 0.65 \times 300 \times 600$	
	V_{uc} = 117 kN.	(1M)
	Step 2] As V_u (300 kN) > V_{uc} (117 kN)	
	Calculation of shear force (V_{us}) for which shear r/f is to be designed.	
	$V_{us} = V_u - V_{uc}$	
	= 300 - 117	
	V_{us} = 183 kN	(1M)

Q .NO	SOLUTION	MARKS

Step 3] Calculation of Spacing of Stirrups.

Given diameter of stirrups is 10mm.

$$S = \frac{0.87 \times f_y \times A_{sv} \times d}{V_{us}}$$ (1M)

$$A_{sv} = \left(\frac{\pi}{4} \times 10^2\right) \times 2 = 157.0\ mm^2$$

$$S = \frac{0.87 \times 415 \times 157.0 \times 600}{183 \times 10^3}$$

Spacing = 185.85 mm = 185mm. (1M)

Step 4] Check spacing of minimum stirrups.

$$\text{Spacing}, S = \frac{0.87 f_y \times A_{sv}}{0.4 \times b}$$ (1M)

$$= \frac{0.87 \times 415 \times 157.0}{0.4 \times 300}$$

Spacing = 472 > 450mm or 300mm (01M)

∴ Hence provide minimum spacing for stirrups as 300mm.

∴ Provide 2 legged 10 mm φ bars as Vertical stirrups and spacing of 185mm c/c. (01M)

Q .NO	SOLUTION	MARKS
(Q.5)		
(C)	Calculate size, depth and Ast required for a square footing.	
	Given data :-	
	f_{ck} :- 25 N/mm²	
	f_y :- 415 N/mm²	
	b :- 400mm	
	d :- 400 mm	
	P = 1200 KN.	
	S.B.C = 350 KN/m²	
Solⁿ :-	Load from column = 1200 KN.	
	Self wt of footing = 10% of 1200 = 120 kN	
	Total load. 1320 KN.	(1m)
	Area of footing required $= \frac{1320}{350} = 3.77\ m^2$	
	Say Area of footing as 3.8 m²	
	Side $= \sqrt{3.8} = 1.949 \approx 1.95m$	(1m)
	∴ Provide size of footing as 1.95m × 1.95m.	
	Note :- Students can also take size of ~~si~~ footing upto 2.0 x 2.0 m.	

Q .NO	SOLUTION	MARKS
	Depth from bending moment considerations,	
	$M_{ux} = M_{uy} = 473 \times 1.95 \times 0.775^2$	
	$= 554.41 \text{ KN·m}$	(1M)
	$d = \sqrt{\dfrac{554.41 \times 10^6}{0.138 \times 25 \times 1950}} = 287.07 \approx 290 \text{mm}$	(1M)
	$D = 290 + 70 = 360 \text{mm}$	
	$\boxed{D = 360 \text{ mm}}$	(1M)
	Area of steel	
	$A_{st} = 0.5 \times \dfrac{f_{ck}}{f_y} \left[1 - \sqrt{1 - \dfrac{4.6 \times 554.41 \times 10^6}{25 \times 1950 \times 290^2}}\right] \times 1950 \times 290$	(2M)
	where $f_{ck} = 25$	
	$f_y = 415$	
	$\boxed{A_{st} = 6561.42 \text{ mm}^2}$	(1M)

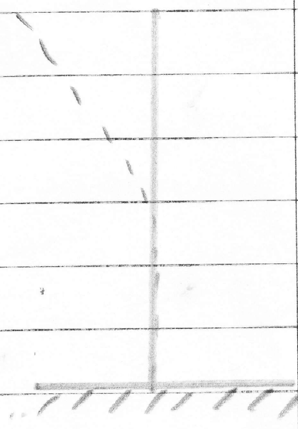
Q .NO	SOLUTION	MARKS
(Q.6)		
(a)	To calculate ultimate moment of resistance of T-beam	
	Data given :-	
	flange width $= b_f = 1100 mm$	
	flange depth $= D_f = 120 mm$	
	depth of beam $= 500 mm$ (d).	
	$b_w = 250 mm$	
	$f_y = 415 N/mm^2$	
	$f_{ck} = 25 N/mm^2$	
	$A_{st} = 4 \times \frac{\pi}{4} \times 20^2 = 1256.64 mm^2$	
	$x_{u max} = 0.48 \times d = 0.48 \times 500 = 240 mm$.	(1m)
	Assume $x_u < D_f$.	
	$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 \times 415 \times 1256.64}{0.36 \times 25 \times 1100}$	(1m)
	$x_u = 45.82 < D_f = 120 mm$ also $< x_{u max} = 240 mm$.	
	Our assumption is correct	(1m)
	$M_{ur} = 0.87 f_y A_{st} (d - 0.42 x_u)$	
	$M_{ur} = 0.87 \times 415 \times 1256.64 (500 - 0.42 \times 45.82)$	(1m)

$M_{ur} = 218.11 kN.m$

Q .NO	SOLUTION	MARKS
	<u>Or</u>	
	$M_{ur} = 0.36 f_{ck}\, b_f\, x_u (d - 0.42 x_u)$	
	$M_{ur} = 0.36 \times 25 \times 1100 \times (45.82)(500 - 0.42 \times 45.82)$	
	$M_{ur} = 218.07\ kN.m$	

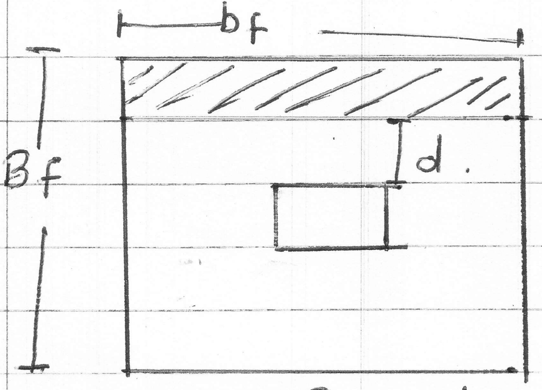
Q .NO	SOLUTION	MARKS
Q.6(b)		
Ans:-		
	Over reinforced section is a section in which steel is more than the required for balance section. In this case the compressive stresses in concrete at the extreme fibre reaches its maximum permissible value first. The steel is not stressed to its maximum permissible tensile stress. Therefore the beam will initially fail due to overstressing in concrete.	(2m)
	In limit state design under reinforced and balanced section are preferred as they will clearly warn about failure of the member in the form of large deflections associated with well distributed cracks before the final failure. On the other side over r/f. sections does not give us any prior warning before failure and such sections suddenly fails due to crushing of concrete, hence O/R sections are disallowed in LSM	(2m)

Q .NO	SOLUTION	MARKS
Q.6 c)		
	Expression for effective width of flange for T and L-Beams.	
	Effective width of flange should in no case be greater than the breadth of the web plus half the sum of the clear distances to the adjacent beams on either side.	
	T-beams $b_f = [(L_0/6) + b_w + 6D_f]$ — (1)	(2m)
	L-beams $b_f = [(L_0/12) + b_w + 3D_f]$ — (2)	
	Where b_f = effective width of flange L_0 = distance between points of zero moments b_w = breadth of the web D_f = Thickness of flange b = actual width of flange.	(2m)

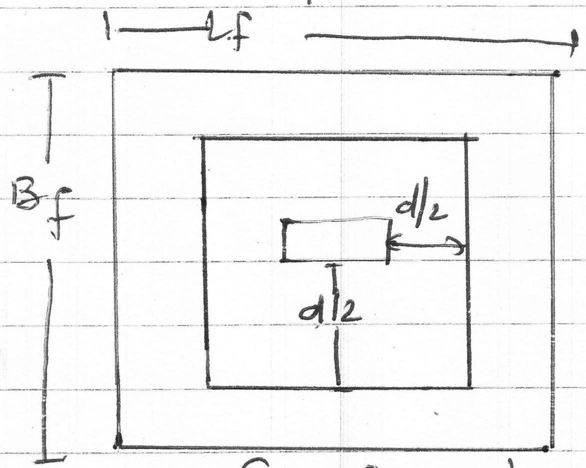
Q .NO	SOLUTION	MARKS
(Q. 6)		
(d)	Commonly used support conditions for columns and their effective lengths are as follows	
(a)	Effectively held in position and restrained against rotation at both ends $l_{eff} = 0.65L$ 	(1M each)
(b)	Effectively held in position at both ends, but no restrained rotation $l_{eff} = 1.00L$ 	
(c)	Effectively held in position at both ends, restrained against rotation at one end only $l_{eff} = 0.80L$ 	
(d)	Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end $l_{eff} = 2.0L$ 	

Q.NO	SOLUTION	MARKS

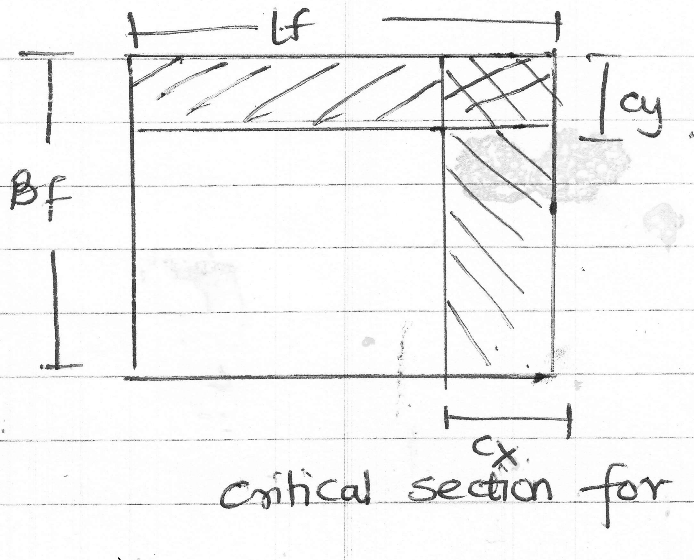
Q.6 e



One way shear (01M)



Two way shear. (01M)



(02M)

critical section for bending moments.