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 ma 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

1) **IN Q.NO-2(a) the span is not mention clearly weather it is effective or clear spa hence student may assume any of this two so accordingly two solutions have been provided which should be asses and marks should be given accordingly.**
2) **IN Q.NO-3-A-(b) in this problem unsupported length off column is not mention hence the checks for minimum eccentricity are not possible so if the students assume it and given the check for minimum eccentricity full credit should be given. In this model answer one sample calculation by assuming unsupported length have been given for reference. Also students may go for finding axial load directly without giving check for e_{min} in that case marks should be given.**
3) **IN Q.NO-5(a) in the given problem M_d is less than M_{lim} hence the section cannot be design as doubly reinforced and it is not possible to calculate A_{st2} and A_{SC}. So calculation up to Ast1 only given full credit.**
4) **IN Q.NO-5(b) for two alternate solutions are provided one by assuming given span as clear and other by effective.**
5) **IN Q.NO-6(e) in this problem the given value of factored load is not sufficient and giving negative value of Asc therefore though students calculated Asc full credits given to students**

Q .NO	SOLUTION	MARKS
Q1. A)	**Solve any THREE**	**12**
(a)	Solution: Limit state may be defined as "the acceptable limit for the safety and serviceability of the structure". Types: 1) Limit state of collapse. 2) Limit state of Serviceability.	02 01 01
(b)	Solution: 1) A normal section plane before bending remains plane before bending right upto collapse. 2) The maximum strain in concrete at the outermost compression fibre is taken as 0.0035 in bending. 3) Concrete under tension is ignored. Tension is to be taken by reinforcement only. 4) The distribution of compressive stress in concrete across the section is defined by idealized stress-strain curve of concrete. 5) Perfect bond exists between steel and concrete right upto collapse. 6) The design stress in steel reinforcement is obtained from the strain at reinforcement level using idealized stress-strain curve for the types of reinforcement used. 7) According to IS code, the maximum strain in steel in tension shall not be less than: $0.002 + fy/(1.15E_s)$ at collapse.	Any four 01 for each
(c)	1) Building with lateral load resisting system comprising I) A ductile moment resisting space frame or II) A dual system consisting of ductile moment resisting space frame and ductile flexural (shear) wall, qualify for low seismic induced forces. 2) A frame of continuous construction, comprising flexural member and column designed and detailed to accommodate reversible lateral displacements after the formation of plastic hinge.	**4M**
(d)	Advantages: 1) This provides a type of construction which is always free from cracks under full working load. Due to this reason, such type of construction is suitable where great danger of corrosion. 2) Sufficient horizontal compression due to prestress reduces principal tensile stress and shear resistance is developed without heavy reinforcement or large webs. 3) Less quantity of steel is used as high compressive strength concrete is used. 4) This type of construction saves material as well as dead loads and hence considerable saving in supports and foundation is also achieved. 5) Smaller sections are used hence can be used for large span. 6) The cost of shuttering and centring in large structure is reduced when prestressed precast elements are assembled. 7) Deflections of structures are reduced. 8) It saves time and money.	Any two 01 for each

Q .NO	SOLUTION	MARKS
	Disadvantages: 1) It requires high tensile steel and high strength concrete; hence it will become more expensive for small job. 2) Prestressing is tedious job and also requires skill supervision. 3) It does not increase ultimate strength of concrete. 4) It requires special equipments such as jacks, anchorages, high tensile cables etc. Different types of losses are occurs like losses due to creep in steel, friction, slip of anchorage etc.	Any two 01 for each
(e)	Solution: 1) Vertical stirrups 2) Bent up bars along with stirrups 3) Inclined stirrups	4M

Q .NO	SOLUTION	MARKS
B	Solve any one:	
a	A beam 300mm x 500mm effective size carries a factored B.M of 175 kN.m. If concrete M20 & steel grade fe 500 are used, find area of steel.	
	solution :-	
	Given $b = 300mm$, $d = 500mm$	
	$M_u = M_d = 175 kN.m$,	
	$f_{ck} = 20 N/mm^2$, $f_y = 500 N/mm^2$	
	(i) Find $M_{u,lim}$	
	$M_{u\,lim} = 0.133 f_{ck}\, b d^2$	1M
	$= 0.133 \times 20 \times 300 \times 500^2$	
	$= 199.5 \times 10^6$ N.mm	
	$M_{u\,lim} = 199.5$ kN.mm.	1 M
	(ii) since $M_d < M_{u\,lim}$, section is under-Reinforced.	2 M
	(iii) $A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 \times M_d}{f_{ck} b d^2}}\right] bd$	1M
	$= \frac{0.5 \times 20}{500}\left[1 - \sqrt{1 - \frac{4.6 \times 175 \times 10^6}{20 \times 300 \times 500^2}}\right] \times 300 \times 500$	
	$\boxed{A_{st} = 957.94\ mm^2}$	1 M

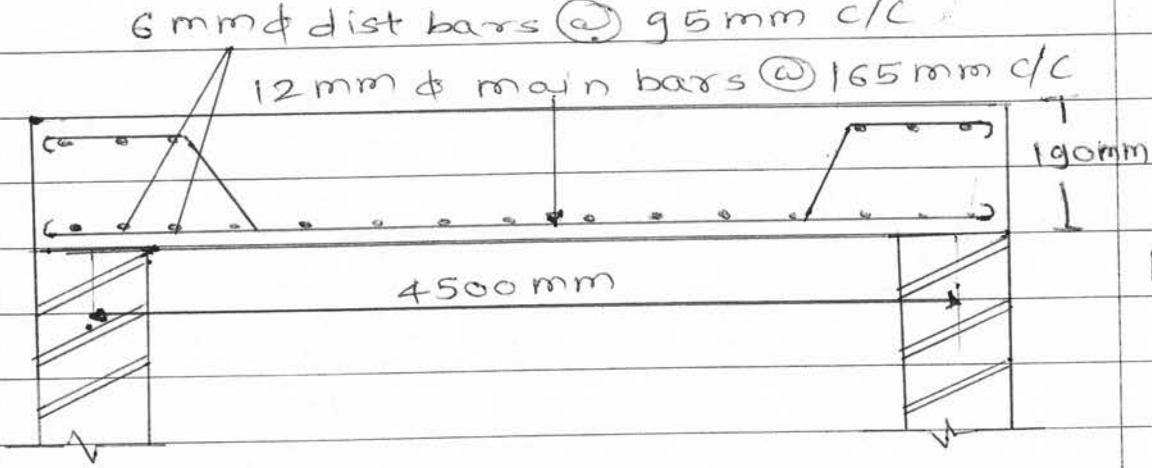
Q.NO	SOLUTION	MARKS
b	Find moment of resistance if steel provided is 4 bars of 16mm diameter in beam 300X600mm effective & concrete M20 & steel fe500 are used	
	solution ÷	
	Given ÷ $A_{st} = 4 \times \frac{\pi}{4} \times 16^2 = 804.24\ mm^2$	
	$b = 300mm$ & $d = 600\ mm$	
	~~fck = 300mm~~	
	$f_{ck} = 20\ N/mm^2,\ f_y = 500\ N/mm^2$	
	(i) To find actual N.A 'x_u'	
	$x_u = \frac{0.87\ f_y\ A_{st}}{0.36\ f_{ck}\ b}$	
	$= \frac{0.87 \times 500 \times 804.24}{0.36 \times 20 \times 300} = 161.96mm$	1 M
	(ii) To find $x_{u max}$	
	$x_{u max} = 0.46\ d = 0.46 \times 600$	
	$\boxed{x_{u max} = 276\ mm}$	1 M
	(iii) As $x_u < x_{u max}$	1 M
	∴ section is under reinforced section	1 M
	(iv) $M_u = 0.87\ f_y \cdot A_{st}\ (d - 0.42\ x_u)$	1 M
	$= 0.87 \times 500 \times 804.24\ (600 - 0.42 \times 161.96)$	
	$\boxed{M_u = 186.109\ kN \cdot m}$	1 M

Q .NO	SOLUTION	MARKS
2	Solve any TWO	
a	Design one way slab with following data, span = 4.5m, Live load = 4 kN/m² Floor Finish = 1 kN/m². Concrete M20 & steel Fe 415, M.F = 1.4. Sketch C/S of slab showing reinforcement details. (No checks)	
	In above problem it is not clearly mentioned weather, the span is clear or effective, hence student can solve this problem by two way Full Marks should be given accordingly	
	Solution - 1 . If Given span is effective span	
	$L = L_{eff} = 4.5\ m$	
	Live load = 4 kN/m²	
	Floor finish = 1 kN/m²	
	$f_{ck} = 20\ N/mm^2$, $f_y = 415\ N/mm^2$	
	M.F = 1.4	
	(i) To calculate Design constant	
	$x_{u\,max} = 0.48d = 0.48 \times d$	½ M
	(ii) To calculate slab thickness	
	$d = \frac{span}{20 \times M.F} = \frac{4500}{20 \times 1.4} = 160.7$	
	$d = 160.7\ mm$	

Q .NO	SOLUTION	MARKS
	overall depth, $D = d + dc + \phi/2$	
	assuming 12 ϕ main bars	
	$D = 160.7 + 20 + \frac{12}{2} = 186.7 \simeq 190mm$	
	$d_{avail} = D - dc - \phi/2$	
	$= 190 - 20 - \frac{12}{2} = 164$ mm	
	$\boxed{d_{avail} = 164mm}$	1 M
	(iii) Effective span	
	$L = L_{eff} = 4500mm$	½ M
	(iv) Load calculation	
	Dead load $= 25D = 25 \times 0.190 = 4.75$	
	Live load $= 4 \ kN/m^2$	
	Floor finish $= 1 \ kN/m^2$	
	Total load $W = D.L + L.L + F.F$	
	$= 4.75 + 4 + 1 = 9.75 \ kN/m^2$	
	Factored Load $= \gamma_F \times W$	
	$= 1.5 \times 9.75$	
	$W_d = 14.625 \ kN/m$	1 M
	(v) Factored Maximum B.M.	
	$M_d = \frac{W_d \times l_e^2}{8} = \frac{14.625 \times 4.5^2}{8}$	
	$M_d = 37.01 \ kN.m$	1 M

Q .NO	SOLUTION	MARKS
	(vi) Required effective depth	
	$M_d = M_{u\,lim} = 0.138 f_{ck} b d^2$ – Fe415	
	$37.01 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$	
	$d_{req} = 115.81$ mm < d provided	
	(vii) Area of main steel	
	$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6 \times M_d}{f_{ck} b d^2}}\right] bd$	
	$= \frac{0.5 \times 20}{415}\left[1-\sqrt{1-\frac{4.6 \times 37.01 \times 10^6}{20 \times 1000 \times 164^2}}\right] 1000 \times 164$	
	$A_{st} = 684.66 \text{ mm}^2$	1 M
	spacing of 12 mm ϕ bars	
	$S = \frac{a_{st}}{A_{st}} \times 1000$	
	$a_{st} = \pi/4 \times 12^2 = 113.09 \text{ mm}^2$	
	$S = \frac{113.09}{684.66} \times 1000 = 165.17 \approx 165$ mm	
	$S = 165$ mm c/c	
	Max spacing = 3d or 300 mm whichever is less	
	= 3 × 164 or 300	1 M
	= 300 mm	

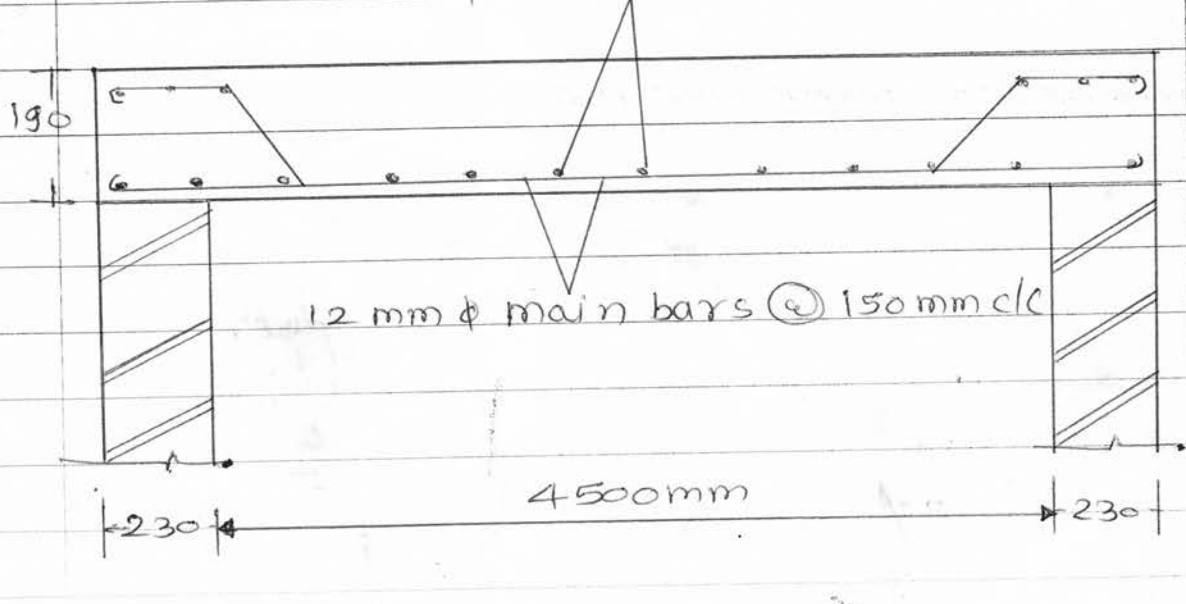
Q .NO	SOLUTION	MARKS
	provide 12 mm ϕ bars @ 165 mm c/c	
	Area of steel provided	
	$A_{stp} = \frac{1000 \times a_{st}}{s}$	
	$= \frac{1000 \times 113.09}{165}$	
	$A_{stp} = 685.39 \ mm^2$	
	(viii) Area & spacing for distribution steel	
	$A_{std} = \frac{0.15}{100} bD$ for fe 250	
	$= \frac{0.15}{100} \times 1000 \times 190 = 285 \ mm^2$	
	spacing of 6 mm ϕ m.s bars	
	$S_d = \frac{a_{st} \times b}{A_{std}} = \frac{\pi/4 \times 6^2 \times 1000}{285}$	
	$= 99.20$	
	$S_d \simeq 95$ mm c/c	
	Maximum spacing = 5d or 450 mm	
	$= 5 \times 164$ or 450	
	$S_{d\,max} = 450$ mm	
	provide 6 mm ϕ bar @ 95 mm c/c	1 M

Q .NO	SOLUTION	MARKS
	Detailing	
		1 M
	OR	
	Solution II – considering Given span as clear span.	
	l = 4.5 m = 4500 mm	
	Width of support = 230 mm	
	(i) Design constant	
	$x_{u}\ max = 0.48\ d$	½ M
	(ii) calculate slab thickness	
	$d = \frac{span}{20 \times MF} = \frac{4500}{20 \times 1.4} = 160.71$	
	consider 12 mm ϕ dia. of main bar	
	$D = d + dc + \phi/2 = 160.71 + 20 + \frac{12}{2} = 186.71$	
	D = 190 mm say	1 M

Q .NO	SOLUTION	MARKS
	$d_{avail} = D - d_c - \phi/2$	
	$d_{avail} = 190 - 20 - 6 = 164$ mm	
	(iii) Effective span	
	Take lesser of following	
	$l_{eff} = l + t = 4500 + 230 = 4730$ mm	
	$l_{eff} = l + d = 4500 + 164 = 4664$ mm	
	Take $l_{eff} = 4.664$ m	½ M
	(iv) load calculation	
	Dead load $= 25D = 25 \times 0.19 = 4.75$ kN/m^2	
	Live load $= 4$ kN/m^2	
	Floor Finish $= 1$ kN/m^2	
	Total load $= D.L + L.L + F.F = 4.75 + 4 + 1$	
	$W = 9.75$ kN	
	Factored load $= \gamma_f \times W$	1 M
	$W_d = 1.5 \times 9.75 = 14.625$ kN/m	
	(v) Factored Max B.M	
	$M_d = \dfrac{W_d\, l_e^2}{8} = \dfrac{14.625 \times 4.664^2}{8}$	
	$M_d = 39.76$ kN.m	1 M

Q .NO	SOLUTION	MARKS
	(vi) Required effective depth	
	$M_d = M_{ulim} = 0.138 f_{ck} b d^2$ – for fe415	
	$39.76 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$	
	$d_{req} = 120.02$ mm < $d_{provided} = 164$ mm	
	(vii) Area of main steel	
	$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 \times M_d}{f_{ck} b d^2}}\right] bd$	
	$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 39.76 \times 10^6}{20 \times 1000 \times 164^2}}\right] 1000 \times 164$	
	$A_{st} = 741.35$ mm^2	1 M
	spacing of 12 mm ϕ bars	
	$S = \frac{a_{st}}{A_{st}} \times 1000 = \frac{\pi/4 \times 12^2}{741.35} \times 1000$	
	$S = 152.55 \simeq 150$ mm say	
	$S_{max} = 3d$ or 300 mm whichever is less	
	$= 3 \times 164$ or 300 mm	1 M
	$= 300$ mm	
	$\therefore$ provide 12 mm ϕ bar @ 150 mm c/c	
	Area of steel provided	
	$A_{stp} = \frac{1000 \times a_{st}}{S} = \frac{1000 \times 113.09}{150}$	

$A_{stp} = 753.93$ mm^2

Q .NO	SOLUTION	MARKS
viii	Area & spacing of distribution steel	
	$A_{std} = \frac{0.15}{100} \times bD$ — for Fe 250	
	$= \frac{0.15}{100} \times 1000 \times 190 = 285 mm^2$	
	spacing of 6 mm ϕ MS bars	
	$sd = \frac{a_{st}}{A_{std}} \times 1000 = \frac{\pi/4 \times 6^2}{285} \times 1000$	
	$= 99.20 mm \simeq 95 mm$ c/c	
	Maximum spacing = 5d or 450 mm	
	= 5×164 or 450	
	$sd_{max} = 450 mm$	
	provide 6 mm ϕ bar @ 95 mm c/c	1 M
	6 mm ϕ bar (dist) @ 95 mm c/c	
		
	12 mm ϕ main bars @ 150 mm c/c	1 M
	4500 mm	
	230 230	

Q .NO	SOLUTION	MARKS
b	Design a reinforced concrete slab panel for 6.3 × 4.5 m simply supported on all the four sides. It has to carries live load of 4 kN/m^2 in addition to its dead load. Use M25 concrete Fe415 steel. Sketch the c/s of slab along shorter span showing steel details (No checks) use $\alpha_x = 0.062$ & $\alpha_y = 0.060$	
	Solution	
	(i) $\frac{Ly}{Lx} = \frac{6.3}{4.5} = 1.4 < 2$ Two way slab	$\frac{1}{2}$ M
	(ii) Design Constant	
	Mulim = $0.138\, f_{ck}\, bd^2$ – for Fe 415	$\frac{1}{2}$ M
	(iii) calculate slab thickness	
	as $lx > 3.5$ m & live load $> 3\ kN/m^2$	
	For Fe 415, MF = 1.4	
	$d = \frac{lx}{20 \times M.F} = \frac{4.5 \times 10^3}{20 \times 1.4} = 160.71$ mm	
	Assuming 10 mm φ main bars & nominal cover 20 mm	
	$D = d + dc + \phi/2 = 160.71 + 20 + \frac{10}{2}$	
	$D = 185.71 \approx 190$ mm say	1M

Q .NO	SOLUTION	MARKS
	$d_{avail} = 190 - 20 - \frac{10}{2} = 165\,mm$	
	(iv) Effective span	
	Take thickness of support 230mm	
	$l_e = l_x + t = 4.5 + 0.23 = 4.73\,m$	
	$l_e = l_x + d = 4.5 + 0.165 = 4.665\,m$	
	Take smaller of above two	
	$l_e = 4.665\,m$	1 M
	(v) load calculation	
	Dead load $= 25D = 25 \times 0.190 = 4.75\,kN/m^2$	
	Live load $= 4\,kN/m^2$	
	Floor Finish $= 1\,kN/m^2$ (Assume)	
	Total load $= 9.75\,kN/m$	
	Factored load $= \gamma_f \times W = 1.5 \times 9.75$	
	$W_d = 14.625\,kN/m$	1 M
	(vi) factored Max B.M	
	$M_{xd} = \alpha_x W_d \times l_{ex}^2 = 0.062 \times 14.625 \times 4.665^2$	
	$M_{xd} = 19.73\,kN.m$	½ M
	$M_{yd} = \alpha_y . W_d \times l_{ex}^2 = 0.060 \times 14.625 \times 4.665^2$	
	$M_{yd} = 19.096\,kN.m$	½ M

Q .NO	SOLUTION	MARKS
	(vii) Required effective depth of slab	
	$d_{req} = \sqrt{\dfrac{M_{xd}}{q\, f_{ck}\, b}}$	
	$q = 0.138$ for Fe 415	
	$d_{req} = \sqrt{\dfrac{19.73\times10^6}{0.138\times25\times1000}}$	
	$d_{req} = 75.62\,mm < d_{provided} = 165\,mm$	
	(viii) Area & spacing of steel	
	$A_{stx} = \dfrac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\dfrac{4.6 M_{xd}}{f_{ck}\, b d^2}}\right] bd$	
	$= \dfrac{0.5\times25}{415}\left[1-\sqrt{1-\dfrac{4.6\times19.73\times10^6}{25\times1000\times165^2}}\right]1000\times165$	
	$A_{stx} = 343.20\,mm^2$	$\frac{1}{2}$ M
	spacing of 10 mm ϕ bars	
	$S_x = \dfrac{a_{st}}{A_{stx}}\times1000 = \dfrac{\frac{\pi}{4}\times10^2}{343.20}\times1000$	
	$S_x = 228.84\,mm \simeq 225\,mm$ say c/c	$\frac{1}{2}$ M
	Now A_{sty}	
	$A_{sty} = \dfrac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\dfrac{4.6\times M_{yd}}{f_{ck}\, b d'^2}}\right] b\times d'$	
	$d' = d-\phi = 165-10 = 155\,mm$	

Q.NO	SOLUTION	MARKS

$$A_{sty} = \frac{0.5 \times 25}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 19.096 \times 10^6}{25 \times 1000 \times 155^2}}\right] 1000 \times 155$$

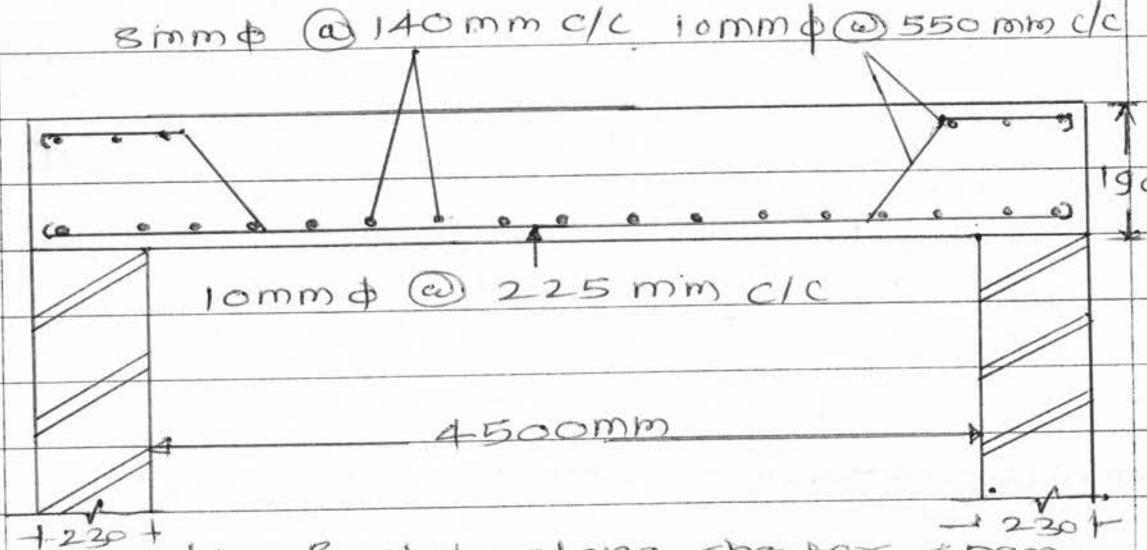
$A_{sty} = 354.88\ mm^2$ — $\frac{1}{2}$ M

spacing of 8mm φ dia bars

$$S_y = \frac{a_{st}}{A_{sty}} \times 1000 = \frac{\pi/4 \times 8^2 \times 1000}{354.88}$$

$S_y = 141.64$ mm say ≈ 140 mm c/c — $\frac{1}{2}$ M

c/s of slab showing R/F details along shorter span.

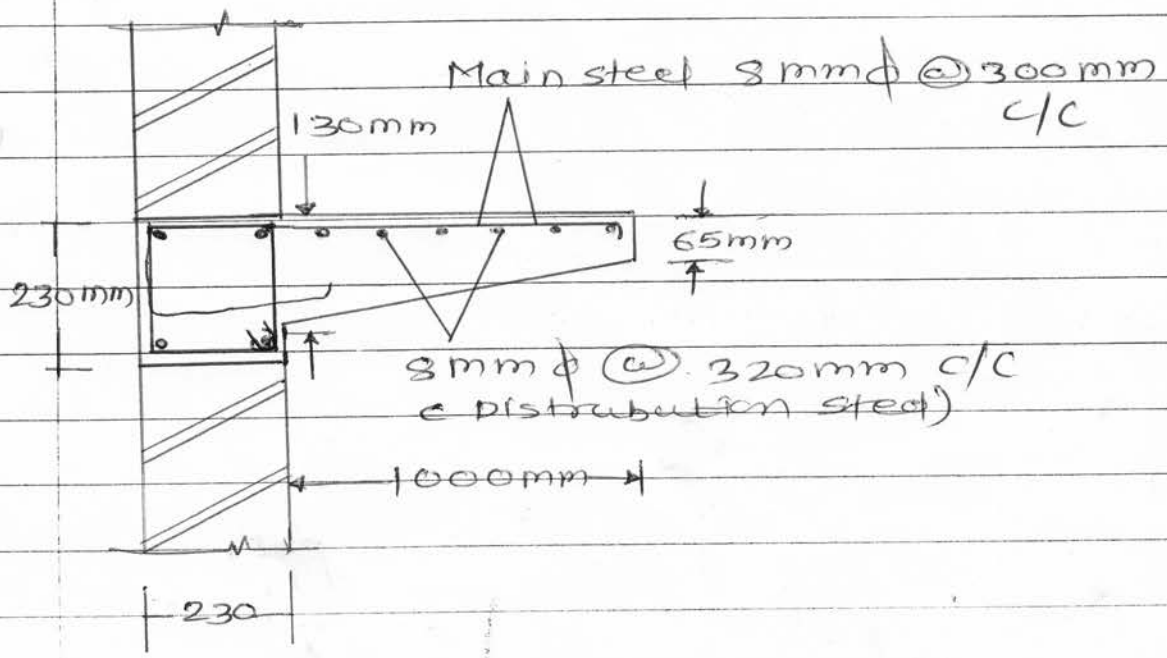


c/s of slab along shorter span — 1 M

Q .NO	SOLUTION	MARKS
C	Design cantilever chajja with following data:	
	span = 1.0 m, width = 1.5 m, L.L = 1.00 kN/m²	
	Floor finish = 0.5 kN/m² support lintel = 230 mm × 230 mm. concrete M20,	
	Fe 415 steel. sketch the c/s of chajja showing steel details (No checks)	
	solution:	
	(i) Design constant	
	$M_{u\,lim} = q \cdot f_{ck} \cdot bd^2 = 0.138 \times f_{ck}\, bd^2$	
	(ii) Estimation of slab thickness	
	$d = \dfrac{\text{span}}{7 \times M.F}$	
	Take M.F = 1.4 – for Fe 415	
	$d = \dfrac{1000}{7 \times 1.4} = 102.04 \simeq 105\,mm$	
	Assuming cover 20 mm & 10 mm ϕ bars	
	$D = d + d_c + \phi/2 = 105 + 20 + \frac{10}{2} = 130\,mm$	
	$D = 130\,mm$	1 M
	(iii) Effective span	
	for cantilever $l_e = l + \frac{d}{2} = 1000 + \frac{105}{2}$	
	$l_e = 1052.5\,mm \simeq 1053\,mm$	

Q .NO	SOLUTION	MARKS
	(iv) Load calculation	
	(i) Dead load $= 25D = 25 \times 0.130$	
	$D.L = 3.25$ kN/m	
	(ii) Live load $= 1 \times 1 \times 1 = 1$ kN/m	
	(iii) Floor finish $= 1 \times 1 \times 0.5 = 0.5$ kN/m	
	Total load $W = 3.25 + 1 + 0.5 = 4.75$	
	Factored load $= \gamma_f \times W$	
	$W_d = 4.75 \times 1.5 = 7.125$ kN/m	1M
	(v) Factored B.M	
	$M_d = \dfrac{w_d \times l_e^2}{2} = \dfrac{7.125 \times 1.053^2}{2}$	
	$M_d = 3.82$ kN·m	1M
	(vi) Required effective depth & overall thickness	
	$d_{req} = \sqrt{\dfrac{M_d}{q\, f_{ck} b}} = \sqrt{\dfrac{3.82 \times 10^6}{0.138 \times 20 \times 1000}}$	
	$d_{req} = 37.20 < d_{provided} = 105$ mm	

Q .NO	SOLUTION	MARKS
	(vii) Area of spacing of main steel	
	$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 \times M_d}{f_{ck}\, b d^2}}\right] bd$	
	$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 3.82 \times 10^6}{20 \times 1000 \times 105^2}}\right] 1000 \times 105$	
	$A_{st} = 102.90\ mm^2$	
	$A_{st\,min} = \frac{0.12}{100} \times bD$	
	$= \frac{0.12}{100} \times 1000 \times 130$	
	$= 156\ mm^2$	
	As $A_{st} < A_{st\,min}$ provide $A_{st} = 156\ mm^2$	1 M
	spacing of 8 mm ϕ bars	
	$S = \frac{a_{st}}{A_{st}} \times 1000 = \frac{\pi/4 \times 8^2 \times 1000}{156}$	
	$S = 322.21\ mm$	
	$S_{max} = 3d$ or $300\ mm$ whichever is smaller	
	$= 3 \times 105$ or $300\ mm$	
	$= 300\ mm$	
	hence provide 8 mm ϕ bar @ 300 mm c/c	1 M

Q .NO	SOLUTION	MARKS
	(viii) Area & spacing of distribution steel	
	$A_{std} = A_{st\,min} = 156\ mm^2$	1M
	if 8mm φ bars used as distribution steel $S = 322.21$ mm	
	say $S \approx 320$ mm	
	$S_{max} = 5d$ or 450 mm whichever is less	
	$= 5 \times 105$ or 450 mm	
	$= 450$ mm.	
	hence provide 8mm φ @ 320 mm c/c as a distribution steel	1 M
	(ix) c/s showing R/F details	
		1 M

Q .NO	SOLUTION	MARKS
Q 03.	Attempt ANY FOUR of following: (04 x 04 = 16)	16
a)	**Conditions of formation of flanged beam:** 1. The slab shall be cast integrally with the web, or the web and the slab shall be effectively bonded together in any other manner; and 2. If the main reinforcement of the slab is parallel to the beam, transverse reinforcement shall be provided such that reinforcement shall not be less than 60 percent of the main reinforcement at mid span of the slab.	2
	Effective flange width for L and T beam: For T beam: $b_f = \frac{L_0}{6} + b_w + 6D_f < Actual\ flange\ width\ (B)$	1
	For L beam: $b_f = \frac{L_0}{12} + b_w + 3D_f < Actual\ flange\ width\ (B)$ Where, $b_f = effective\ width\ of\ flange$ $L_0 = distance\ between\ points\ of\ zero\ moments$ $b_w = width\ of\ web$ $D_f = thickness\ of\ flange$	1
b)	**To find moment of resistance of T-beam:** **Data Given:** $D_f = 120$mm , $b_f = 1100$mm, $b_w = 275$mm, d = 450mm, $A_{sf} = 2400mm^2$ $f_{ck} = 25\ MPa,$ $f_y = 500MPa$ $X_{umax} = 0.46 \times 450 = 207mm$ **Assume,** $x_u < D_f$ $\therefore x_u = \frac{0.87 \times f_y \times A_{st}}{0.36 \times f_{ck} \times b_f}$	1
	$\therefore x_u = \frac{0.87 \times 500 \times 2400}{0.36 \times 25 \times 1100}$ $\therefore \boldsymbol{x_u = 105.45\ mm < D_f\ also < X_{umax}}$	½
	$\therefore \underline{Assumption\ is\ correct.}$ $M_{ur} = 0.36 f_{ck} b_f x_u (d - 0.42x_u)$ $\therefore M_{ur} = 0.36 \times 25 \times 1100 \times 105.45 \times (450 - 0.42 \times 105.45)$ $\therefore \boldsymbol{M_{ur} = 423.54 \times 10^6 Nmm = 423.54kNm}$	1
	OR	½
	$M_{ur} = 0.87 f_y A_{st} (d - 0.42x_u)$ $\therefore M_{ur} = 0.87 \times 500 \times 2400 \times (450 - 0.42 \times 105.45)$ $\therefore \boldsymbol{M_{ur} = 423.56 \times 10^6 Nmm = 423.56kNm}$	1

c)	**To find development length in tension and compression.** **Data Given:** $\emptyset = 16mm, \quad f_y = 415\ MPa,$ $\tau_{bd} = 1.2\ MPa for plain bars in tension$ Development length (L_d) in tension: $$L_d = \frac{0.87 \times f_y \times \emptyset}{4 \times \boldsymbol{\tau_{bd}} \times \mathbf{1.6}}$$ …(τ_{bd} has been increased by 60% due to HYSD bars.) $$\therefore L_d = \frac{0.87 \times 415 \times 16}{4 \times 1.2 \times 1.6} = \mathbf{752.19\ mm}$$ Development length (L_d) in Compression: $$L_d = \frac{0.87 \times f_y \times \emptyset}{4 \times \boldsymbol{\tau_{bd}} \times \mathbf{1.6 \times 1.25}}$$ ...(τ_{bd} Has been increased by 60% due to HYSD bars) and ... (τ_{bd} Has been further increased by 25% due to bars in compression.) $$L_d = \frac{0.87 \times 415 \times 16}{4 \times \mathbf{1.2 \times 1.6 \times 1.25}} = \mathbf{601.75mm}$$	1 1 1 1
d)	**Development Length:** Development length is the length required to develop the stress in the bar from zero to maximum by transfer of stress from concrete to steel. **Factors affecting development length:** 1. Grade of steel, 2. Grade of concrete, 3. Diameter of bar, 4. Design bond stress,	2 2
e)	**Design of column:** **Data Given:** Factored load P_u= 3000kN, Concrete M20 and Steel Fe415, unsupported length =3m, Assume p%= 1%. p= p%/100 = 1/100 = 0.01 $$\therefore P_u = \{0.4f_{ck} + (0.67f_y - 0.4f_{ck})p\}A_g$$ $$\therefore 3000 \times 10^3 = \{0.4 \times 20 + (0.67 \times 415 - 0.4 \times 20)0.01\}A_g$$ $$\therefore A_g = 280360.73mm^2$$ Side of square column = $\sqrt{280361} = 529.49 \approx 530mm$ Provide column of 550 mm x 550 mm Area provided $\boldsymbol{A_g = 550 \times 550 = 302500mm^2}$ Check for minimum eccentricity, $$e_{min} = \frac{3000}{500} + \frac{550}{30} = 24.33mm$$ 0.05h =0.05x550 =27.5mm Here, $e_{min} < 0.05h$, therefore the column can be designed as axially loaded $$\therefore P_u = [0.4f_{ck}A_g + (0.67f_y - 0.4f_{ck})A_{sc}]$$ $$\therefore 3000 \times 1000 = [0.4 \times 20 \times 302500 + (0.67 \times 415 - 0.4 \times 20)A_{sc}]$$ $$\therefore \boldsymbol{A_{sc} = 2147.75mm^2}$$	½ ½ ½ ½ ½

	Provide 8-20mm bars area provided =2513mm^2 **Diameter and Pitch of lateral ties:** Diameter of lateral ties >20/4 = 5mm and, Pitch < 550 < 16x20= 320 < 300mm **Provide 6mm lateral ties at 300mm $^c/_c$**	½ ½ ½
Q04(A)	**Attempt ANY THREE of following: (03 x 04 = 12)**	**12**
a)	**Methods of prestressing:** There are two major methods of prestressing 1. Pre-tensioning 2. Post-tensioning – (a) Internal prestressing , b) External prestressing) **Pre-tensioning:** In this system, the tendons (steel wires, cables, strands used to impart prestress to concrete) are first tensioned between rigid anchor blocks then concrete is subsequently placed and compacted to the required shape and size. After the concrete hardens, the tendons are released from the prestressing bed transferring the stress to concrete. ➢ This method has greater certainty about durability. ➢ No expenditure for making ducts and anchorages. ➢ Loss of prestress is more ➢ Rigid anchor blocks cannot be reused till the concrete hardens. **OR** **Post-tensioning:** This method of prestressing is classified into internal and external prestressing. In internal prestressing, the untensioned wires are placed in the duct of beam during its casting. After the casing and hardening of concrete the wires are stretched against the member itself. Once desired stretching is over they are anchored at the two ends. After which the space between wires and sheathing is filled with grout. In external prestressing system, wires are wound around the existing structure and prestressing operation is done to protect the existing damaged structure. In this case externally wound tensioned wires, cables are encased by dense concrete secured to the main concrete. ➢ Post tensioned beams are cheaper than pre tensioned beams for heavy loads. ➢ Losses of prestress less compared to post tensioned beams.	1 3
b)	**Load carrying capacity of column:** **(Note: In given problem unsupported length is not mentioned, and hence checks for minimum eccentricity are not possible without the value of unsupported length. So if students have assumed it and given the check for minimum eccentricity full credit should be given.)** **Data Given:** Column size 300 mm x 450 mm, Reinforcement 4-16mmØ& 4-12mmØ, Materials used = M20 and Fe415. $A_g = 300 \times 450 = \mathbf{135000mm^2}$ $A_{sc} = 4 \times \frac{\pi}{4} \times 16^2 + 4 \times \frac{\pi}{4} \times 12^2 = \mathbf{1256.64mm^2}$ $P_u = 0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) \times A_{sc}$ $P_u = 0.4 \times 20 \times 135000 + (0.67 \times 415 - 0.4 \times 20) \times 1256.64$ $\therefore \mathbf{P_u = 1419.35kN}$ **OR**	 1 1 1

	Here one sample calculation is given by assuming Unsupported length of column as 3m. $$A_g = 300 \times 450 = \mathbf{135000mm^2}$$ $$A_{sc} = 4 \times \frac{\pi}{4} \times 16^2 + 4 \times \frac{\pi}{4} \times 12^2 = \mathbf{1256.64mm^2}$$ $$e_{min} = \frac{L}{500} + \frac{h}{30} \text{ or } 20mm \text{ whichever is greater}$$ $$e_{min} = \frac{3000}{500} + \frac{450}{30} = 21mm \text{ or } 20mm \text{ whichever is greater}$$ $$\therefore \mathbf{e_{min} = 21mm}$$ Also, $e_{min} = 21mm < 0.05h = 0.05 \times 450 = 22.5mm$ $$P_u = 0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) \times A_{sc}$$ $$P_u = 0.4 \times 20 \times 135000 + (0.67 \times 415 - 0.4 \times 20) \times 1256.64$$ $$\therefore \mathbf{P_u = 1419.35kN}$$	1
c)	**Partial safety factors:** Since the safety of the structure depends on each of the two principal design factors (viz. load and material strengths) which are not dependent on each other, two different safety factors, one for load and other for material strength are used. Because each of two factors contributes partially to safety, they are termed as partial safety factors. **As per IS 456-2000 there are two partial safety factors:** 1. Partial safety factor for Load (γ_f) 2. Partial safety factor for Material Strength (γ_m) **Values recommended in IS456-2000 for concrete and steel.** **Material** — **Partial safety factors (γ_m)** Concrete — 1.5 Steel — 1.15	1 1 2
d)	**Situations in which a doubly reinforced beam is provided: (ANY FOUR)** 1. When the applied moment exceeds the moment resisting capacity of a singly reinforced section. 2. When the section of the beam is restricted due to the requirements of head room, appearance etc. 3. When the section of the beam is subjected to reversal of bending moment. 4. In continuous T beam where the portion of beam over middle support has to be designed as doubly reinforced. 5. When the beams are subjected to eccentric loading, shocks, or impact loading. 6. If the bending moment exists over a relatively short length of the beam only, then the cost of additional steel is small as compared to the cost of singly reinforced beam of constant section. 7. Compression steel is sometimes provided to reduce the deflection.	1 for each any four
(B)	Attempt <u>ANY ONE</u>:	**06**
a)	**Ultimate moment capacity of beam:** **Data Given:** b = 280mm, d = 510mm, d' =50mm, A_{st} =2455mm², A_{sc}= 402mm², f_{ck} =30MPa, f_y =415MPa, f_{sc} =353 MPa $$A_{st2} = \frac{f_{sc} A_{sc}}{0.87 f_y}$$ $$\therefore A_{st2} = \frac{353 \times 402}{0.87 \times 415} = \mathbf{393mm^2}$$ But, $A_{st1} = A_{st} - A_{st2}$ $$\therefore A_{st1} = 2455 - 393 \therefore \mathbf{A_{st1} = 2062mm^2}$$	1 ½ ½

	$M_u = 0.87 f_y A st_1 (d - 0.42 x_{umax}) + f_{sc} A_{sc} (d - d')$ $\therefore M_u = 0.87 \times 415 \times 2062 \times (510 - 0.42 \times 244.8) + 353 \times 402(510 - 50)$ $\therefore \mathbf{M_u = 368.42kNm}$ **OR** $M_u = 0.87 f_y A st_1 (d - 0.42 x_{umax}) + (f_{sc} - f_{cc}) A_{sc} (d - d')$ $M_u = 0.87 f_y A st_1 (d - 0.42 x_{umax}) + (f_{sc} - 0.446 f_{ck}) A_{sc} (d - d')$ $\therefore M_u = 0.87 \times 415 \times 2062 \times (510 - 0.42 \times 244.8) + (353 - 0.446 \times 30 \times 402(510 - 50)$ $\therefore \mathbf{M_u = 365.21kNm}$ **OR** $M_u = 0.36 f_{ck} b x_{umax} (d - 0.42 x_{umax}) + f_{sc} A_{sc} (d - d')$ $\therefore M_u = 0.36 \times 30 \times 280 \times 244.8 \times (510 - 0.42 \times 244.8) + 353 \times 402(510 - 50)$ $\therefore \mathbf{M_u = 366.7kNm}$ **OR** $M_u = 0.36 f_{ck} b x_{umax} (d - 0.42 x_{umax}) + (f_{sc} - f_{cc}) A_{sc} (d - d')$ $\therefore M_u = 0.36 \times 30 \times 280 \times 244.8 \times (510 - 0.42 \times 244.8) + (353 - 0.446 \times 30 \times 402(510 - 50)$ $\therefore \mathbf{M_u = 364.23kNm}$ **Note: Here students may calculate the value of Mu by either considering the value of fcc or neglecting the value of fcc, so full credit should be given accordingly.**	1 1
b)	**Data Given:** Beam size: 200mm x 300mm effective b= 200 mm, d= 300 mm, f_{ck}= 20 MPa, f_y=415 MPa, Mu=74 kNm, d'=30mm, σ_{sc}=353 MPa $x_{umax} = 0.48 \times 300 = \mathbf{144mm}$ $M_{urmax} = R_{umax} b d^2$ $\therefore M_{urmax} = 0.138 \times 20 \times 200 \times 300^2 = \mathbf{49.68kNm}$ $A_{st1} = \frac{M_{urmax}}{0.87 f_y (d - 0.42 x_{umax})}$ $\therefore A_{st1} = \frac{49.68 \times 10^6}{0.87 \times 415 \times (300 - 0.42 \times 144)}$ $\therefore \mathbf{A_{st1} = 574.48mm^2}$ Balanced moment, $M_{u2} = M_u - M_{u1}$ $= 74 - 49.68$ $\therefore \mathbf{M_{u2} = 24.32\ kNm}$ $A_{st2} = \frac{M_{u2}}{0.87 f_y (d - d')}$ $\therefore A_{st2} = \frac{24.32 \times 10^6}{0.87 \times 415 \times (300 - 30)}$ $\therefore \mathbf{A_{st2} = 249.48mm^2}$ Total area of tension steel, $A_{st} = A_{st1} + A_{st2}$ $\therefore A_{st} = 574.48 + 249.48$ $\therefore \mathbf{A_{st} = 823.96mm^2}$ Total area of compression steel, $A_{sc} = \frac{0.87 f_y A_{st2}}{f_{sc}}$ $\therefore A_{sc} = \frac{0.87 \times 415 \times 249.48}{353}$ $\therefore \mathbf{A_{sc} = 255.68mm^2}$	1 1 1 ½ 1 ½ 1

Q .NO	SOLUTION	MARKS
Q-5 (a)	given data	
	$b = 250\,mm \quad d = 500\,mm \quad d' = 50\,mm$	
	$f_{ck} = 200\ N/mm^2 \quad f_y = 500\ N/mm^2 \quad M_d = 165\ kN\text{-}m$	
	$\sigma_{sc} = 353\ N/mm^2$	01 M
	i) $x_{umax} = 0.46d$	01 M
	$= 0.46 \times 500$	
	$= 230\ mm$	½ M
	ii) $M_{ulim} = 0.133\ bd^2 f_{ck}$	01 M
	$= 0.133 \times 250 \times 500^2 \times 20$	01 M
	$M_{ulim} = 166.25 \times 10^6\ N\text{-}mm$	
	for M-20 & Fe500	
	$\therefore\ P_{tlim} = 0.76$	01 M
	solution-I: $Ast_1 = \dfrac{0.76 \times 250 \times 500}{100}$	
	$\boxed{Ast_1 = 950\ mm^2}$	
	$M_d < M_{ulim}$ --- Check	
	solution-II: $Ast = \dfrac{0.5 f_{ck}}{f_y}\left(1 - \sqrt{1 - \dfrac{4.6 M_d}{f_{ck} \cdot bd^2}}\right) bd$	
	$Ast = \dfrac{0.5 \times 20}{500}\left(1 - \sqrt{1 - \dfrac{4.6 \times 165 \times 10^6}{20 \times 250 \times 500^2}}\right) 250 \times 500$	
	$\boxed{Ast = 933\ mm^2}$	2M for Ast or Ast_1 (any one)
	Here given M_d is less than $M_{ulim} = M_{u1}$ hence above section will not be doubly reinforced Hence it is not possible to find the value Asc & Ast_2	01 M for $M_d < M_{lim}$ check

Q .NO	SOLUTION	MARKS
Q-5 (b)	given data	
	$b = 300\ mm$	
	$d = 1010\ mm$	
	$l = 7\ m$	
	$w = 45\ KN/m$	
	i) Factored load $(Wd) = 1.5 \times 45 = 67.5\ KN/m$	01M
	∴ ii) Factored shear force $V_u = \frac{Wd \times l}{2}$	½M
	$= \frac{67.5 \times 7}{2}$	
	$V_u = 236.25\ KN$	01½M
	$A_{st} = 6 \times \frac{\pi}{4} \times 22^2 = 2280.72\ mm^2$	
	∴ $p_t = \frac{A_{st}}{b \times d} \times 100$	½M
	$= \frac{2280.72}{300 \times 1010} \times 100 = 0.75\%$	01½M
	iii) calculate nominal shear stress $\tau_v = \frac{V_u}{b \times d}$	
	$\tau_v = \frac{236.25 \times 10^3}{300 \times 1010} = 0.78\ N/mm^2$	01M
	iv) check for τ_{cmax}	
	$\tau_{cmax} = 2.8\ N/mm^2$ for M20 concrete	
	∴ As $\tau_v < \tau_{cmax}$ ---- OK	½M

Q .NO	SOLUTION	MARKS
	v) Calculate shear strength of concrete τ_c for M20 grade concrete	
	$P_t = 0.75\%$	
	$\tau_c = 0.56\ N/mm^2$ $\tau_c = \tau_{uc}$	½ M
	As $\tau_v > \tau_c$	
	$V_{uc} = \tau_{uc} \times b \cdot d$	
	$= 0.56 \times 300 \times 1010$	
	$V_{uc} = 169.68$ KN	½ M
	As $V_u > V_{uc}$ hence shear reinforcement required to design for V_{usv}	½ M
	$V_{usv} = V_u - V_{uc}$	
	$= 236.25 - 169.68$	
	$V_{usv} = 66.57$ KN	½ M
	using 8 mm- 2-legged stirrups	
	$A_{sv} = 2 \times \pi \times 8^2/4 = 100 mm^2$	
	spacing $S = \frac{0.87 f_y A_{sv} \cdot d}{V_{usv}}$	½ M
	$= \frac{0.87 \times 415 \times 100 \times 1010}{66.57 \times 10^3} < 300$ or $0.75 \times 1010 = 757.5$ mm (whichever is less)	
	$= 547.78$ mm say 550 mm c/c	
	spacing of minimum stirrups $= \frac{0.87 \times 415 \times 100}{0.4 \times 300}$	½ M
	$= 300$ mm	
	Provide 8 mm- 2-legged stirrups 300 mm c/c	½ M

alternate solution No-2 for Q-5(b)

SUMMER – 15 EXAMINATION

Model Answer

Subject Code: 17604

Q .NO	SOLUTION	MARKS
Q-5 (b)	given data	
	$b = 300\,mm$, $d = 1010\,mm$, $l = 7\,m$ $w = 45\,KN/m$	
	$L_{eff} = 7.230\,m$ (assume wall thickness = 230 mm)	
	i) Factored load $(W_d) = 1.5 \times 45 = 67.5\,KN/m$	01M
	ii) Factored shear force $(V_u) = \frac{W_d \times L_{eff}}{2}$	
	$= \frac{67.5 \times 7.23}{2}$	
	$V_u = 244.012\,KN$	01M
	$A_{st} = 6 \times \frac{\pi}{4} \times 22^2 = 2280.72\,mm^2$	
	$\therefore p_t = \frac{A_{st}}{b \times d} \times 100$	
	$= \frac{2280.72}{300 \times 1010} \times 100 = 0.75\,\%$	01M
	iii) calculate nominal shear stress	
	$\tau_v = \frac{V_u}{b \times d}$	
	$\tau_v = \frac{244.012 \times 10^3}{300 \times 1010} = 0.8053\,N/mm^2$	01M
	iv) check for $\tau_{c\,max}$	
	$\tau_{c\,max} = 2.8\,N/mm^2$ for M-20 concrete	
	$\therefore$ As $\tau_v < \tau_{c\,max}$ - - - OK	½M

Q .NO	SOLUTION	MARKS
	v) calculate shear strength of concrete τ_c for M-20 grade concrete.	
	$p_t = 0.75\%$	
	$\tau_c = 0.56\ N/mm^2 \quad \tau_c = \tau_{uc}$	½M
	As $\tau_v < \tau_c$	
	$V_{uc} = \tau_{uc} \times b \cdot d$	
	$= 0.56 \times 300 \times 1010$	
	$V_{uc} = 169.68\ KN$	½M
	As $V_u > V_{uc}$ hence shear reinforcement required to design for V_{usv}	½M
	$V_{usv} = V_u - V_{uc}$	
	$= 244.012 - 169.68$	
	$V_{usv} = 74.332\ KN$	½M
	using 8mm - 2-legged stirrups	
	$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100\ mm^2$	
	spacing $(s) = \frac{0.87\ f_y\ A_{sv} \cdot d}{V_{usv}}$	½M
	$= \frac{0.87 \times 415 \times 100 \times 1010}{74.332 \times 10^3}$ < 300mm or $0.75 \times 1010 = 757$mm which ever is less	
	$= 490.58$ mm say's 500mm c/c	
	spacing of minimum stirrups $= \frac{0.87 \times 415 \times 100}{0.4 \times 300}$	½M
	$= 300$ mm	
	provide 8mm - 2legged stirrups 300mm c/c	½M

Q .NO	SOLUTION	MARKS
Q-5 (c)	given data	
	$b = 400$ mm	
	$D = 400$ mm	
	$SBC = 200\ KN/m^2$	
	$f_{ck} = 20\ N/mm^2$	
	$f_y = 415\ N/mm^2$	
	i> Load from column = 1000 KN	
	self weight of footing at 10% = 100 KN	
	Total load = 1100 KN	½ M
	ii> proportioning of base size	
	Area of footing $A_f = \frac{1100}{200} = 5.5\ m^2$	½ M
	$L_f = \frac{D-b}{2} + \sqrt{\frac{(D-b)^2}{4} + A_f}$	½ M
	$L_f = \frac{400-400}{2} + \sqrt{\frac{(400-400)^2}{4} + 5.5 \times 10^6}$	
	$L_f = 2345.20$ mm say 2350 mm	½ M
	iii> projection of the footing about x-axis	
	$C_x = \frac{2350 - 400}{2} = 975$ mm	½ M
	Breadth of the footing $B_f = b + 2C_x$	
	$B_f = 400 + 2 \times 975 = 2350$ mm	½ M
	provide footing of 2350mm X 2350 mm	
	Area of footing provided	
	$A_f = 2.350 \times 2.350 = 5.5225\ m^2$	½ M

Q .NO	SOLUTION	MARKS
	iv) upward soil reaction $k_{u} = \frac{P_u}{A_f}$	1/2 M
	$= \frac{1.5 \times 1100}{5.5}$	
	$= 300 \ KN/m^2$	01 M
	v) Depth of footing from bending moment consideration	
	$M_{ux} = k_u \cdot B_f \cdot C_x^2/2$	1/2 M
	$= \frac{300 \times 2.35 \times 0.975^2}{2}$	
	$= 335.095 \ KN\text{-}m$	1/2 M
	$M_{uy} = k_u \cdot L_f \times C_y^2/2$	1/2 M
	$= 300 \times 2.35 \times 0.975^2/2$	
	$= 335.095 \ KN\text{-}m$	1/2 M
	$d_x = \sqrt{\frac{M_{ux}}{R_{umax} B_f}} = \sqrt{\frac{335.09 \times 10^6}{(0.138 \times 20) \times 2350}} = 227.29 \ mm$	(01 M for dx or dy)
	Required effective depth for bending about y-axis	
	$d_y = \sqrt{\frac{M_{uy}}{R_{umax} \cdot L_f}} = \sqrt{\frac{335.09 \times 10^6}{(0.138 \times 20) \times 2350}} = 227.29 \ mm$	
	There hence adopt d required = 227.29 mm and provide 230 mm	
Note:	According IS-code Lf is 1.5 but if student have Assume other value calculation should check accordingly	

Q .NO	SOLUTION	MARKS
Q-6 (a)	given data	
	$b_f = 1500mm$	
	$b_w = 300mm$	
	$d = 700mm$	
	$D_f = 100 mm$	
	$A_{st} = 2500 mm^2$	
	$f_{ck} = 20 N/mm^2$	
	$Fe415 = f_y = 415 N/mm^2$	
	i) find x_u	
	$0.36 f_{ck} \cdot x_u \cdot b_f = 0.87 f_y \cdot A_{st}$	01/2 M
	$0.36 \times 20 \times x_u \times 1500 = 0.87 \times 415 \times 2500$	
	$10800 x_u = 902.625 \times 10^3$	
	$x_u = 83.576 mm$	01/2 M
	ii) find x_{umax}	
	$x_{max} = 0.479 d$	1/2 M
	$= 0.479 \times 700$	
	$= 335.3 mm$	
	as $x_u < x_{umax}$ section is under reinforced	1/2 M
	iii) find M_u	
	$M_u = T_u \times Z$	01 M
	$= 0.87 f_y A_{st} (d - 0.42 x_u)$	
	$= 0.87 \times 415 \times 2500 (700 - 0.42 \times 83.576)$	
	$M_u = 600.37 KN\text{-}m$	01 M

Q .NO	SOLUTION	MARKS
Q-6 (b)	i) under reinforced section	
	when the percentage steel in the section is less than critical percentage, the section is known as under-reinforced section.	02M
	ii) Balanced section	
	when the ratio of steel to concrete in a section is such that maximum strain in steel and maximum strain in concrete reach their maximum values simultaneously, the section is referred to as balanced-section or critical section.	02M
Q-6 (c)		
	1) Minimum percentage of tension steel	
	$P_t \% \nless \frac{85}{fy}$	02M
	ii) Maximum area of tension steel	
	$\ngtr 0.04\, bD$	02M
	where b = bredth of web or T Beam	
	D = Depth of Beam (overall depth)	
	(Reference : page-47, IS-456-2000 Cl. No 26.5.1.1)	

Q .NO	SOLUTION	MARKS
Q-6 (d)		
	i) Minimum area of steel ≮ 0.8 % of gross cross sectional area	01 M
	ii) Maximum area of steel ≯ 6% of gross cross-sectional area	01 M
	iii) Necessasity of lateral ties	(½ M for any TWO)
	a) To prevent the buckling of longitudinal reinforcement	
	b) To confine the concrete	
	c) To hold longitudinal reinforcement in position	
	d) To prevent longitudinal splitting of concrete	
	e) To resist diagonal tension due to transverse shear.	
	iv) lateral ties spacing	(½ M for any TWO)
	the pitch of the lateral ties shall not be more than the least of the following	
	a) Least lateral dimension of the column	
	b) 16 times the smallest diameter of the longitudinal reinforcing bar to be tied.	
	c) 300 mm	

Q .NO	SOLUTION	MARKS
Q-6		
e	d = 400mm	
	l_{eff} = 4.5m	
	factored load = 900 kN	
	M-20 = fck = 20 N/mm²	
	Fe500 = fy = 500 N/mm²	
	$P_u = 0.4\, f_{ck}\, A_g + (0.67\, f_y - 0.4\, f_{ck})\, A_{sc}$	01M
	$900\times10^3 = 0.4\times20\times\frac{\pi}{4}\times(400)^2 + (0.67\times500 - 0.4\times20)\,A_{sc}$	
	$900\times10^3 = 1.005\times10^6 + 327\,A_{sc}$	02M
	$\boxed{A_{sc} = -321.10\ mm^2}$	01M
	Full Mark's given to student as above steps	
	As Mention above student may use the direct formula for calculation of Asc with out giving check for minimum eccentricity. still if they have done the calculation full credit give to students.	