WINTER-19 EXAMINATION

Subject Name: DESIGN OF STEEL & RCC STRUCTURES **Subject Code : 22502**

Model Answer

Important Instructions to examiners:

1) The answers should be examined by key words and not as word-to-word as given in the model answer scheme.
2) The model answer and the answer written by candidate may vary but the examiner may try to assess the understanding level of the candidate.
3) The language errors such as grammatical, spelling errors should not be given more Importance (Not applicable for subject English and Communication Skills.
4) While assessing figures, examiner may give credit for principal components indicated in the figure. The figures drawn by candidate and model answer may vary. The examiner may give credit for any equivalent figure drawn.
5) Credits may be given step wise for numerical problems. In some cases, the assumed constant values
6) may vary and there may be some difference in the candidate's answers and model answer
7) In case of some questions credit may be given by judgement on part of examiner of relevant answer
8) For programming language papers, credit may be given to any other program based on equivalent concept.

QN		Attempt any FIVE of the following	Marking scheme	10 M
Q1	a)	**Enlist the components and corresponding functions of steel water tank** The components of a steel water tank are:- The side wall plates Bottom wall plates, bracings in case of rectangular tanks Elevated steel tank consists of, Ring beam, Staircase The primary functions of a steel water tank is storage of water	1 1	2
	b)	**Define bolt value and pitch** Pitch : it's the centre to centre distance between the bolts in the direction of force. Bolt value: it's the least value of the shear strength and bearing strength of a bolt	1 1	2
	c)	**State the values of partial safety factors for material strength of concrete and steel for limit state of collapse** The values of partial factor of safety for steel and concrete. 1) Partial factor of safety for steel = 1.15 2) Partial factor of safety for concrete = 1.5	1 1	2

	d)	**Write the expression for minimum shear reinforcement giving the meaning of the terms involved** Sv = $\frac{0.87 fyAsv}{0.4b}$ Sv – Spacing of stirrups fy – Characteristic strength of steel, Asv – Area of Stirrup bar, b – width of the beam	1 1	2
	e)	**Define the aspect ratio in case of slab and state its importance.** The ratio of Ly/Lx is know as aspect ratio of the slab, where Ly is longer side and Lx is the shorter side of the slab. The importance of this is that, if the ratio of Ly/Lx is greater than 2, then the should be designed as a one way slab and if the ratio is less than 2 it should be designed as a two way slab.	1 1	2
	f)	**Write the two IS specifications for longitudinal reinforcement of an axially loaded short column** IS specifications for longitudinal reinforcement of an axially loaded short column: i) Minimum diameter of bar in column = 12 mm ii) Minimum number of bars in square/rectangle column = 4 Nos iii) Minimum number of bars in circular column = 6 Nos iv) Cover of the column = 40 mm v) Minimum and maximum steel in column vi) Max % of steel = 6 % of gross cross-sectional area of column vii) Min % of steel = 0.8 % of gross cross-sectional area of column	1 M any2	2
	g)	**Enlist two loads to be considered as per IS 875 -1987 while designing steel structure** Loads to be considered as per IS 875 -1987 while designing steel structures 1.DEAD LOAD ----IS 875-PART-1 -1987 2. LIVE LOAD -- IS 875-PART-2 -1987 3. WIND LOAD --- IS 875-PART-3 -1987 4. SNOW LOAD --- IS 875-PART-4 -1987	1M any2	2
Q2		**Attempt Any THREE of the following**		12 M
	a)	**Explain the limit state of serviceability applicable to steel structures.** The acceptable limit for safety and serviceability of the structure before failure occurs is called as Limit state. To assure the serviceability of the	2	4 M

		structure throughout its lifetime, it is related to the satisfactory performance of the structure at working load. The following limit state of serviceability is considered: 1)Deflection and deformation 2) Durability 3) crack due to fatigue 4) Fire	2	
	b)	**In steel constructions bolts of grade 4.6 are generally used. What do you mean by grade 4.6?** In bolts of grade 4-6, The number 4 is 1/100Th of nominal ultimate stress of bolt fub = 4 x 100 = 400 N/mm^2 and yield stress fyb is 0.6 x 400 = 240N/mm^2	2 2	**4 M**
	c)	**Define over reinforced sections and state two reasons due to which they are avoided.** When xu > xmax or pt > pt lim The section is called a over reinforced section, It is avoided due to the following reasons: **1)** In over reinforced section, percentage of steel is more than critical percentage, Due to this, the concrete crushes and reaches its ultimate stress before steel reaches its yield point . In this case, the beam will fail initially due to overstress in the concrete, suddenly without giving any warning by way of large deformations and cracks as it does in the case of under reinforced section. So, there is a huge loss of life and property. **2)**The moment of resistance of the section does not increase more than that of balanced section even if the steel is increased as compared to balanced section	2 2	**4 M**
	d)	**Diameter of steel bar is 20 mm. Use Fe415 steel and design bond stress is 1.2 MPa. For plain bars in tension. Find development length in tension and compression.** d= 20mm fy= 415N/mm^2 τ bd= 1.2N/mm^2 Development Length is given by $Ld = (\frac{0.87 f y Ø)}{4 \tau bd}$ **For deformed bars,** τbd = 1.2*1.6 **In tension** $Ld = (\frac{0.87*415*20)}{4*1.6*1.2}$ = 940.23 mm	1 1 1	**4 M**

		In compression τbd = 1.6*1.25* τbd Ld =($\frac{0.87*415*20)}{4*1.6*1.25*1.2}$ = 752.19 mm	1	
Q3		**Attempt any TWO of the following**		12M
	a)	**Design the lap joint for plates 100x10mm and 80x10mm thick connected to transmit 120kN factored load using single row of 18mm dia bolts of 4.6 grade and plates of 415 grades.** For bolts of grade 4.6 fub = 400 N/mm^2 For Fe 415 grade steel fu = 415 (d) Dia of bolts = 18mm Dia of bolt hole = 18+2 = 20mm. Gross Area of bolt = $\frac{\pi}{4}x18^2 = 254.47mm^2$ Net area of bolts = 0.78*Gross Area =0.78x254.47 = 198.49mm^2 **Single shear strength of bolt** Vdsb = $\frac{Vnsb}{\gamma_{m1}}$ Vnsb = $\frac{fub}{\sqrt{3}}$(nn.Anb + ns Asb) = $\frac{400}{\sqrt{3}}$(1x198.49+0) = 45838.33N Vdsb = $\frac{45838.33}{1.25} =$ 36670.67N Vdsb = 36.67 kN **Strength of bolts in bearing** Vdpb = $\frac{Vnsb}{\gamma_{m1}}$ Vnpb = 2.5kb.d.t.fu Kb is least of the below $\left[\frac{e}{3d_o}, \frac{P}{3d_o} - 0.25, \frac{fub}{fu}, 1.0\right]$	1 1 1	6M

Assuming e = 1.5do

= 1.5x20 = 30mm

P = 2.5d = 45 mm

$\frac{30}{3x20} = 0.5$, $\left(\frac{45}{3x20} - 0.25\right) = 0.5$, $\frac{400}{415} = 0.964, 1.0$

The least of the above is 0.5 hence kb is 0.5

Vnpb = 2.5x0.5x18x10x415

= 93375 N

$Vdpb = \frac{Vnpb}{1.25} = \frac{93375}{1.25}$ = 74700N 74.70kN

Bolt valve is the least of the above Vdsb & Vdpb **1**

= 36.67 kN

No. of bolts = $\frac{120}{36.67} = 3.27 \approx 4\ No's$ **1**

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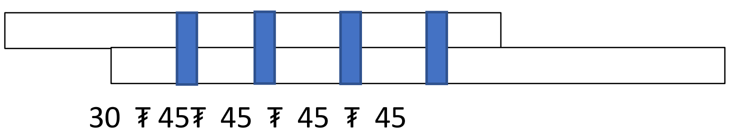
Arrange the bolts as shown below in one row. **1**



Alternatively

Single shear strength of bolt

Vdsb = $\frac{Vnsb}{\gamma_{m1}}$ **1**

Vnsb = $\frac{fub}{\sqrt{3}}$(nn.Anb + ns Asb)

= $\frac{400}{\sqrt{3}}$(1x198.49+0)

= 45838.33N

Vdsb = $\frac{45838.33}{1.25} =$ 36670.67N

Vdsb = 36.67 kN

Therefore no of bolts: $\frac{120}{36.67} = 3.27 \approx 4\ No's$ **1**

Equating this to tensile strength of plate per pitch length

Tdn=$(0.9 * fu(p - do) * t)/\gamma_{m1}$ = $\frac{0.9x415(p-20)x10}{1.25}$ = 120000 **1**

p = 60.16mm ~ 65 mm check if greater than minimum pitch which is 2.5x18 =45 mm hence ok

Consider **e** as 35 mm **1**

Strength of bolts in bearing

Vdpb = $\frac{Vnsb}{\gamma_{m1}}$

Vnpb = 2.5kb.d.t.fu **1**

Kb is least of the below

$\left[\frac{e}{3d_o}, \frac{P}{3d_o} - 0.25, \frac{fub}{fu}, 1.0\right]$

$\frac{35}{3x20} = 0.583$, $\left(\frac{65}{3x20} - 0.25\right) = 0.833$, $\frac{400}{415} = 0.964$, 1.0

Hence the value of kb is 0.583

Vnpb = 2.5x0.583x18x10x415

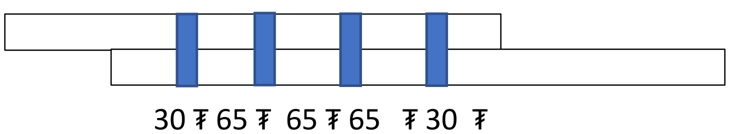
= 108875.25 N

$$Vdpb = \frac{Vnpb}{1.25} = \frac{108875.25}{1.25}$$

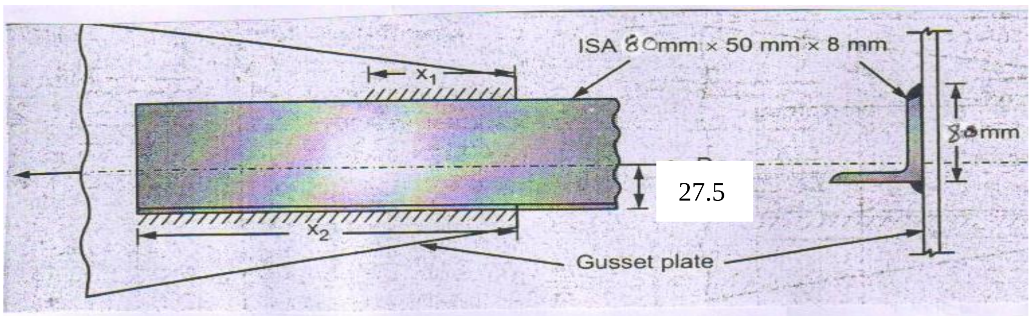
= 87100.2N

= 87.1 kN

Therefore, the bolt value is 36.6kN **1**



	b)	**Design a suitable fillet welded connection for ISA 80x50x8 with its longer leg connected to gusset plate of thickness 8mm. The angle is subjected to factored load of 270kN. Cxx = 27.5mm. Assume weld applied to all 3 edges and shop weld. Take fy = 250 MPa & fu = 410 MPa.** Given Pu = 270 kN Minimum size of weld = 3 mm Maximum size of weld = $\frac{3}{4}(8) = 6mm$ Provide a weld size of 4 mm. **Note: - Students may assume any other size of weld between 3 mm to 6 mm. The answer will vary examiner needs to check as per the size of weld considered by the student.** **# Design stress of shop weld.** fwd = $\frac{fu}{(\sqrt{3}) * \gamma_{m1}} = 189.37$ N/mm^2 γm1 = 1.25 Throat thickness (t) = 0.7 x 4 = 2.8 mm. **# Weld length required** Pdw = fwd * L * t 270 x 10^3 = 189.37 * L * 2.8 L = 509.21 mm. ~510mm **Alternatively** **#Strength of weld per mm** Pdw = fwd * 1 * t Pq x 189.37 x 1 x 2.8 Pq = 530.24 N Length of weld required = $\frac{270*10^3}{530.24}$ = 509.21 $\approx$ 510 mm	1 1 1 1 or 1	**6M**

		As per the requirement the welding is done on 3 sides X1 + X2 + 80 = 510 X1 + X2 = 430  Taking moment @ bottom weld X1(530.24)*80+80x530.24x40=270x10^3 x27.5 X1 = 135.04 $\approx 140\ mm$ X2 = 294.96 $\approx 295\ mm$	1 1	
	c)	**A RC section 250mm x 450mm effective in reinforced with 4 no – 16 mm dia bars of Fe 415 on tension side only. If M20 concrete in used, calculate ultimate moment of resistance the beam can offer.** Size of beam = 250 x 450 Given Ast = 4[$\frac{\pi}{4} x16^2$] = 804.25 mm^2 **Check it the beam is under reinforced** Xu = $\frac{0.87 fyAst}{0.36 fck\ b}$ Xu = $\frac{0.87x415x804.25}{0.36x20x250}$ = 161.32 mm **Check Xu max** Xumax = 0.48d = 0.48x450 = 216mm Since Xu < Xumax section is under reinforced MR = 0.87 fy Ast (d – 0.42Xu) = 0.87x 415 x 804.25 (450 – 0.42 x 161.32) = 110994360 N-mm MR = 110.99 kN-m	1 1 1 1 1 1	6M

Q4		**Attempt any TWO of the following**		**12M**
	a)	**Calculate depth and area of steel at mid span of a simply supported beam over a clear span 6m. The beam is carrying all-inclusive load 20 kN/M. Assume 300mm bearings. Use M20 & Fe500 Assume b = $\frac{1}{2}d$** Beam load = 20 kN/m Spam of beam = 6m. fck = 20 N/mm^2 fy = 500 N/mm^2 Effective span Le = $6+\frac{0.3}{2}+\frac{0.3}{2}=6.3m.$	1	
		BM for simply supported beam = $\frac{wl^2}{8}=\frac{20*6.3^2}{8}=99.23kNM$ Factored BM = 1.5x99.23 = 148.84 kNM	1	
		Equate Factored BM to Mulim to calculate b & d. Mulim = 0.133*fck*b*d^2 As b = $\frac{1}{2}d$ **Calculate effective depth d and width b** Mulim = 0.133 fck $\frac{d^3}{2}$ 148.84x10^6 = 0.133x20$\frac{d^3}{2}$ = 481.89mm ≈ 490 mm.	1	
		take effective cover = 40 , therefore Overall Depth D = d+d^1 = 490+40 = 530mm. As b = $\frac{d}{2}$ b = $\frac{490}{2}$ = 245 mm. **Area of steel** Ast = $\frac{0.5xfck}{fy}[1-\sqrt{1-\frac{4.6Mulim}{fckbd^2}}]$ bd	1	

		$= \frac{0.5x20}{500}\left[1 - \sqrt{1 - \frac{4.6x148.84x10^6}{20x245x490^2}}\right]$ 245x490 = 848.6mm^2 No. of bars to be provided if dia of bars is 20mm. Area of one bar = $\frac{\pi}{4}x20^2$ = 314.16 mm^2 No. of bars = $\frac{Ast}{Area\ of\ one\ bar} = \frac{848.6}{314.16} = 2.7 \approx$ 3 No's **Alternatively,** Ast can also be determined as Pt,lim = $\frac{Ast}{bd} * 100$ Pt,lim for Fe500 = 0.038*20 = 0.76% Ast = $\frac{0.76*bx*d}{100} = \frac{0.76*245*490}{100} =$ 912.38mm^2 No. of bars to be provided if dia of bars is 20mm. Area of one bar = $\frac{\pi}{4}x20^2$ = 314.16 mm^2 $\frac{912.38}{314.16} = 2.9 \approx$ 3 No's.	**1** **1** **Or** **1** **1**	
	b)	**A simply supported beam of span 5m carries a working udl of intensity 40 kN/m. Size of beam 350 x 500mm (effective). It is reinforced with 4 bars 20mm diameter. Design 8mm diameter 2 legged stirrups if one 20 mm diameter bar is bent up. Take τ_c = 0.5 ^{2}N/mm $\tau_{cmax = 2.8}$ N/mm^2 Use M20 grade & Fe415 steel.** Span = 5mm w = 40kN/m Size of beam = 350 x 500 mm effective τ_c = 0.5 ^{2}N/mm $\tau_{cmax = 2.8}$ N/mm^2 **#Calculation of shear force** Factored load = 1.5x40 = 60 kN/m	**1**	**6M**

Factored shear force (Vu) = $\frac{wd*l}{2} = \frac{60*5}{2} = 150kN$

 # Calculate Shear stress τ_v **1**

$\tau_v = \frac{Vu}{bd} = \frac{150x10^3}{350x500} = 0.857 \text{N/mm}^2$ $< \tau_{c\,max}$ hence OK

τ_c value is 0.5

Check if shear reinforcement is required

As $\tau_v > \tau_c$ Shear reinforcement is required **1**

Calculation of Balance shear

Vus = Vu - τ_c x bd

= $150x10^3$-0.5x350x500 = 62500 N.

Area of bent up bar = $[\frac{\pi}{4} x20^2]$ x 1 = 314.16mm^2

Assuming 45^0 bend

 # Shear resisted by Bent-up bar **1**

Vusb = shear resisted by bent-up bar

= 0.87 x fy x Asb x Sin45

= 0.87 x 415 x 314.16 x Sin45 = 80205.33 N

Vusb > $\frac{Vu}{2}$ hence OK.

 # Shear Resisted by vertical shear stirrups will be **1**

Vusv = $\frac{Vu}{2} = 31.25\ kN$

For two legged vertical stirrups Asv = $\frac{2*\pi 8^2}{4}$ = 100.53 mm^2

S = $\frac{0.87 * fy * Asv * d}{\text{Vusv}}$

= $\frac{0.87x415x100.53x500}{31.25x103}$ = 580.74 mm

Maximum spacing is given as 300 mm

0.75d = 0.75x500=375 mm

580.74 mm **1**

Least value of the above is 8 mm stirrups @ 300 mm c/c

Alternatively

Check for minimum stirrups if capable of taking the shear force resisted by minimum stirrups **OR**

Vusv (min) = 0.4 x b x D = 70 kN >Vusv

Minimum stirrups are sufficient

Asv = $\frac{2*\pi*8^2}{4}$ = 100.53 mm^2

		Sv = $\frac{0.87*fy*Asv}{0.4*b} = \frac{0.87*415*100.53}{0.4*350} = 259.25$ ≈ 250 mm Provide 8mm dia stirrups @250mm $^c/_c$	1	
	c)	**State the various forms of shear reinforcements.** **i) Vertical Shear Stirrups.** Vus = $\frac{0.87*fy*Asv*d}{Sv}$ 1) Vus – balance shear 2) Fy – Grade of steel/characteristic strength of stirrup 3) Asv – Area of shear stirrups with in a distance Sv 4) d – effective depth of beam 5) Sv – spacing of shear stirrups along the length of members **ii) Inclined Shear stirrups** Vus = $\frac{0.87*fy*Asv*d}{Sv}(Sin\ \alpha + Cos\ \alpha)$ 1) α – angle between the inclined stirrup and the axis of member not less than 45^0 **iii) Bent up bars.** Vus = 0.87fy Asb.Sin α 1) Asb – area of bentup bar 2) Vus - balance shear 3) Fy - Grade of steel/characteristic strength of stirrup 4) α - angle between the inclined stirrup and the axis of member not less than 45^0	2 2 2	6M
Q5		**Attempt any TWO of the following**		12M
		Design a one way slab with the following data, span = 3 m, live load = 4 *kN/m2* floor finish = 1 *kN/m2.* Concrete M20 andFe415 steel. Take M.F. as 1.4. (No check required). Design a one way Lx = 3m LL = 4KN / m^2 FF = 1KN / m^2 fck = 20N / mm^2 , fy = 415 N/ mm^2 , MF = 1.4 Slab Thickness d= $\frac{Span}{20 \times MF}$		6M

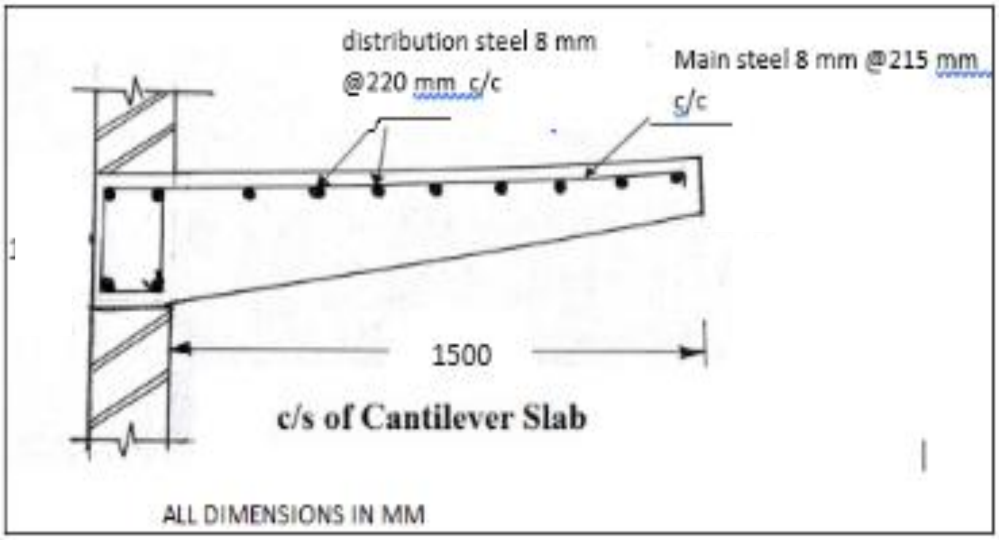
		$= \frac{3000}{20 \times 1.4} = 107.14$ mm	1	
		Assuming 10 mm Ø main bars and nominal cover of 20 mm **[Note: some student may assume 15mm cover]** D = d + cover + Ø /2 = 107.14 + 20 +10/2 = 132.14mm Take D = 135mm davail = D – cover - Ø /2 135 – 20 – 10/2 d = 110mm Effective span le= l + d = 3000 + 110 = 3110mm le= 3.11m **#Calculation of Loads** Dl = 0.135 x 25 = 3.375 KN/m^2 LL = 4 KN/m^2 FF = 1KN / m^2 ——————————— W = 8.375 KN m^2 The load is to be considered for one meter 8.375 * 1 =8.375 Factored Load (wd) = 8.375 x 1.5 = 12.563 KN/ m	1	
		# Factored max BM Md = wd x le^2 / 8 $Md = \frac{12.563\ x\ 3.11^2}{8}$ = 15.19 kN-m	1	
		#Check for required depth d= SQRT(15.19 x 10^6/0.138 x 20 x 1000) d = 74.19mm davail = 110mm > dreqd -: ok Provide D = 135mm davail 110mm	1	
		#Area of main steel and its spacing $Ast = 0.5\ fck/fy \left[1 - \sqrt{1 - \frac{4.6\ Md}{fckbd^2}}\right] x\ bd$ $=0.5 \times 20/415 \left[1 - \sqrt{1 - \frac{4.6 * 15.19 * 10^6}{20 * 1000\ x * 110^2}}\right] * 1000 * 110$ = 415.18mm^2	1	

		Spacing of main reinforcement Assuming 10 mm bar. AØ = ∏ x 10^2/4 = 78.5mm2 Spacing = 1000 x AØ/Ast = 1000 x 78.5\415.18 = 189.17mm =180mm c/c Check for spacing = 3d or 300mm = 330 or 300mm Provide 10 mm bar at 180mm c/c	Spacing of main reinforcement Assuming 8 mm bar A^{ϕ} = ∏ x 8^2/4 = 50.26mm^2 = Spacing = 1000AØ/Ast = 1000 x 50.26/415.18 = 121.06mm = 120 mm c/c Check for spacing = 3d or 300mm = 330 or 300mm Provide 8mm bar at 120mm c/c	1
		#<u>Area and spacing of distribution steel</u> Asd = 0.12/100 x bD =0.12/100 x 1000 x 135 = 162mm2 Spacing 8mm bars A^{ϕ} = 50.26mm^2 Sd = 1000 x AØ/Astd 1000 x 50.26/ 162 = 310mm Check for spacing minimum of 1) 5d = 5 x 110 = 550mm 2) 450mm 3) 310mm Provide distribution steel of 8mm at 310mm c/c		1
	b)	**Design a reinforced concrete slab panel for 6.3 x 4.5 m simply supported on all the four sides. It has to carry a live load of 4 kN/m2 in addition to its dead load. Use M25concrete Fe 415 steel. (No checks) Use αx = 0.062 & αy = 0.060.** Ly = 6.3mm Lx= 4.5m Ly/Lx = 6.3/4.5 = 1.4< 2 : Two way slab Note: student may assume different MF, to be checked as per the values taken Assume MF = 1.4 d= Span/20 x 1.4 = 160.71mm Assume clear over of 20mm Assume 10mm diameter of bar D = d + cover +$^{\phi}$/2 = 160.71 + 20 +10/2 = 185.71 = 190mm davail 190-20-10/2 = 165mm **#<u>Calculation of effective span</u>**		6M

		Lex = 4500+165 =4665mm **#Calculation of Loads** Self weight of slab 0.19 x 25 = 4.75KN/m^2 Live Load = 4KN/m^2 w = 8.75KN/m^2 Factored Load (wd) = 8.75 x 1.5 = 13.125kN/m^2 Consider the loads for 1 m strip hence the loads will be 13.125kN/m **#Bending Moment Calculation** Mx = αx* wd* Lex^2 =0.062 * 131.25 * 4.6652 = 17.71KNm My= αy *wd *Lex^2 0.060 *13.125* 4.6652 = 17.14KNm **#Check for Depth** Effective depth of slab dreqd = $\sqrt{\frac{M}{qfckb}}$ = $\sqrt{\frac{17.71*106}{0.138*25*1000}}$ = 71.65mm davail = 165mm > dreqd , Hence OK **Area & spacing of steel** **Astx** = 0.5 fck/fy $[1-\sqrt{1-\frac{4.6\ x\ 17.71\ x\ 10^6}{25\ x\ 1000\ x\ 165^2}}]$ 1000 x 165 = 306.91 mm^2 Ast min =0.12/100 x 1000 x 190 = 228mm^2 Astx > Astmin OK Spacing using 8mm bars (AØ = 50.26mm^2) Sx = AØb/Astx = 50.26 x 1000/306.91 = 163.76mm =160mm **Check for spacing** S should be min of 3d= 3 x 165 = 495mm , 300 mm, S = 160mm Provide 8mm ᶲ bars or 160mm c/c **Asty** = 0.5fck/fy [1- $\sqrt{1-\frac{4.6My}{fckbd^2}}]bd$ =0.5 x 25/415 $\left[1-\sqrt{1-\frac{4.6\ x\ 17.14\ x\ 10^6}{25\ x\ 1000\ x\ 165^2}}\right]x\ 1000\ x\ 165$ = 296.71 mm^2 Ast min = 228mm^2	**1** **1** **1** **1** **1** **1**	

		Asty > Astmain : OK **Alternatively** D= 165 – 8 = 157 **Asty** = 0.5fck/fy [1- $\sqrt{1-\frac{4.6Md}{fckbd^2}}]bd$ =0.5 x 25/415 $\left[1-\sqrt{1-\frac{4.6*17.14*10^6}{25*1000*157^2}}\right]*\ 1000*\ 157$ = 312.87 mm^2 **Check for spacing** Sx = AØb/Asty = 50.26 x 1000/296.71 = 169.39mm =165mm Spacing should be min of 3d = 495mm, 300, 160 min S = 160mm Provide 8mm at 165mm c/c	1	
	c)	**Design a cantilever chajja with following data: Span = l.50 m, width = 2.0 m, 1.1. = 1.5 kN/m2. Floor finish = 0.5 kN/m2, support lintel = 230 x 300 mm concrete M20, Fe 415 steel, sketch the c/s of chajja. Showing steel details.** Lx = 1.5m LL = 1.5KN / m^2 FF = 05KN / m^2 fck = 20N / mm^2 fy = 415 N/ mm^2 d = span /(7 x MF) Assuming MF = 1.4 **# Estimation of depth** d= 1500/(7 x 1.4) = 153.06mm Assuming 20mm cover and 10mm bars D = 153.06+20+10/2 = 178.06 = 180mm davail = 180 - 20-10/2 = 155mm Effective Span le= l + d/2 = 1500+155/2 = 1577.5mm =1.58m **#Load Calculation & BM** DL = 1x1 x 0.19 x 25 = 4.75KN/m^2 LL = 1.5KN/m^2 FF = 0.5KN/m^2 Total Load w= 6.75KN/m^2 Factored Load wd = 6.75 x 1.5 = 10.13KN/m^2 **#Factored BM** Md= $\frac{wd \times le^2}{2}$	1 1	6M

		= (10.13 x 1.58^2)/2 = 12.64KN-m **# Required Depth** 0.138 fckbd^2 = Md dreq = $\sqrt{\frac{12.64*10^6}{0.138*20*1000}}$ = 67.67mm davial = 155mm > dreqd :-OK **Area and spacing of main steel** Ast = 0.5fck/fy $\left[1-\sqrt{1-\frac{4.36\ Mu}{qfckbd^2}}\right]bd$ =0.5 x 20/415 $\left[1-\sqrt{1-\frac{4.6\ x\ 12.64\ x\ 10^6}{20\ x\ 1000\ x\ 155^2}}\right]x\ 1000\ x\ 155$ = 233.26mm^2 Astmin =0.12/100 x b x D = 0.12/100 x 1000 x 190 = 228mm^2 Ast > Astmin :- OK Spacing using 8mm bars (AØ = 50.26mm^2) Sx = AØb/Ast = 1000 x 50.26 / 233.26 = 215.47mm2 = 215mm Check for spacing S is minimum of 3d = 3 x 155 = 465mm , 300 mm , S = 215mm Hence provide 8mm at 215mm c/c **#Area of spacing of distribution steel** Astd = 0.12/100 x bD = 0.12/100 x 1000 x 190 = 228mm^2 Using 8mm bar ((AØ = 50.26mm2) Spacing = 1000 x AØ/ Astd = 1000 x 50.26/228 = 220.438mm = 220mm Spacing Should be minimum of 5d = 5 x 155 =775mm, 450 mm, 220mm Provide 8mm at 220mm c/c	**1** **1** **1**	



		(figure above)	1	
Q6		**Attempt any TWO of the following**		12M
	a)	**Design a square column to carry an axial load of 1500 kN. The unsupported length of the column is 3.5 m. Use M20 concrete & 1% Fe500 steel for longitudinal reinforcement. Use MS bar for lateral ties. Apply the check for minimum eccentricity.**		
		P = 1500 kN Factored load = 1500 x 1.5 = 2250kN Lo = 3.5 m. fck = 20 N/mm^2 fy = 500 N/mm^2 As Asc is 1% of gross area , Asc = 0.01 Ag Ac = Ag – Asc = Ag – 0.01 Ag = 0.99 Ag Pu = 0.4 fck. Ac + 0.67 fy Asc 2250 x 10^3 = 0.4 x 20 x 0.99 Ag + 0.67 x 500 x 0.01 Ag Ag = 199645.08 mm^2	1	
		#Size of Footing $D = \sqrt{Ag}$ = 446.82 ~ = 450mm	1	

		#Check D should not be less than 0.12 L = 0.12 x 3500 = 420 mm – ok D should not be less than 400 mm – ok **#Check for minimum eccentrically** $e_{min} = \frac{L}{500} + \frac{D}{30}$ $= \frac{3500}{500} + \frac{450}{30}$ = 22 mm e_{max} = 0.05 x D = 0.05 x 450 = 22.5 mm $e_{min} < e_{max}$ – ok **#Main steel Calculation** Asc = 0.01 Ag = 0.01 (450^2) = 2025 mm^2 Provide 8 bars of 20 mm diameter **#Transverse Steel** a) Diameter ØT= 1/4 x ØL or 6mm whichever is greater = (1/4) 20 or 6mm = 5mm or 6mm Use 6mm diameter mild steel reinforcement b) Spacing__ Min of the following 1) D = 450mm 2) 16 × Ø L = 16× 20 = 320mm 3) 300mm Use 6mm ϕ links at 300 mm c/c	1 1 1 1	
	b)	**Design a circular column to carry an axial load of 1500 kN. using MS Lateral ties. Use M25 concrete and Fe415 steel. The unsupported length of column is 3.75 m.** P = 1500 KN Factored load = 1500 × 1.5 =2250KN fck =25 N/mm^2 fy=415 N/ mm^2 Lo=3.75 m **Assume 1 % of Ag.** **Note: Students may assume different % of steel, to be checked based on student assumption.**	 1	6M

		Asc = 0.0 1 A g		
		Ac = Ag – A S C		
		= 0.99 Ag		
		Pu = 0.4 fck.Ac + 0.67 fy ASC		
		2250x103 = 0. 4 × 25 × 0.99 A g + 0.67 × 415 × 0.01 Ag		
		A g = 177 437.8 mm^2		
		#Diameter of column		
		$D = \sqrt{\frac{4\,x\,177437.8}{3.14}}$	**1**	
		= 475.43 mm D = 480 mm		
		D should not be less than 0.12× L		
		0.12 × 3 750 = 450mm -ok		
		D should not be less than 400 mm -ok		
		D=480mm		
		#Check for min eccentricity		
		e_{min} = Lo/500 + D/30	**1**	
		= 3750/500 + 480/30		
		=23.5mm		
		e_{max} = 0.05 x D		
		= 0.05 x 480 = 24mm		
		emin < emax :- OK		
		#Check for slenderness ratio		
		L/D = 3750/480 = 7.8 < 12 :- short column		
		Asc = 0.01 x (π x 480^2/4)	**1**	
		= 1808.64 mm^2		
		Assume 20mm dia of bar		
		Provide 6 bars of 20mm		
		Transverse steel		
		Diameter = ϕt = ¼ x ϕ_L max or 6mm whichever is greater		
		a) = ¼ x 20 or 6mm	**1**	
		5mm or 6mm		
		ϕt = 6mm		
		a) Spacing		
		Min of the following		
		1) D = 480mm	**1**	
		2) 16 ϕ_L = 16 x 20 = 320mm		
		3) 300mm		
		Use 6mm ϕ links at 300mm c/c		

	c)	**Design on R.C. column footing with following data. Size of column = 400 rnm x 400 mm. Safe bearing capacity of soil = 200 kN/m2. Load on column = 1400 kN. Concrete M20 and steel Fe 415 is used. Calculate depth of footing from B.M. Criteria. No shear check is required.** Column = 400mm x 400mm SBC = 200KN/m^2 P = 1400KN Factored Load = 1400 x 1.5 = 2100KN fck = 20N/mm2 fy = 415N/mm^2 Ultimate Bearing Capacity = 2 x SBC = 400KN/m^2 **Size of footing** Wf = 2100KN Af = 1.05 x Wf/SBC = 1.05 x 2100/400 = 5.51m^2 L = B = $\sqrt{Af}$ = $\sqrt{5.51}$ = 2.35m Adopt footing of size 2.35m x 2.35m **#Calculation of Upward soil pressure (q)** q = Wf/(L x B) = 2100 / (2.35 x 2.35)= 380.27KN/m^2 **# Calculation of Depth from flexure** Lx = Ly= (2.35-0.4)/2 = 0.975 m M = q*Lx^2/2 Mx= My = 380.27* 0.975*0.975/2 = 180.75 KN m **# Check for depth required** d-reqd = $\sqrt{[\frac{M}{qfckb}]}$ $= \sqrt{\frac{180.75\ x\ 10^6}{0.138\ x\ 20\ x\ 1000}}$ = 255.91mm = 260mm D = 260 + 50 = 310mm **#Calculation of Steel** $\mathbf{Astx} = \mathbf{Asty} = 0.5\ \text{fck/fy}\left[1 - \sqrt{1 - \frac{4.6\ Md}{fckbd^2}}\right] *\ bd$ = 0.5 x 20 / 415 $\left[1 - \sqrt{1 - \frac{4.6 * 180.75 * 10^6}{20 * 1000 * 260^2}}\right] * 1000 * 260$ = 2377.58 mm^2	1 1 1 1 1	6M

		Using 20mm bar ($A\phi$ = 314mm^2) Spacing = 1000 $A\phi$/ Ast = 1000 x 314.8/2377.58 = 132.40mm =120mm Provide 20mm at 120mm **c/c**	**1**	