PROJECT	ARTHY SOOCIATES-EMERALD RESIDENTIAL APRTMENT,COIMBATORE
CONSULTANT	THE DESIGN ONE
DESIGN OF SLAB	6.2X3m

DESIGN DATAS:

Length of the slab in x-direction, L_x = 3 m = 3000 mm

Length of the slab in x-direction, L_y = 6.2 m = 6200 mm

Thickness of the supporting wall, t = 200 mm = 0.2 m

Live load on the slab = 3 kN/m^2

Floor Finish = 1 kN/m^2

Grade of concrete, f_{ck} = 20 N/mm^2

Grade of steel, f_y = 500 N/mm^2

TYPE OF SLAB:

L_y/L_x = 2.0666667 >2

Hence the given slab is a one-way slab

LOAD CALCULATION:

Assume overall depth, D = 125 mm = 0.125 m

Unit weight of concrete = 25 kN/m^3

Self-weight of the slab = 3.125 kN/m^2

Live load on the slab = 2 kN/m^2

Floor Finish = 1 kN/m^2

Total load, W = 6.125 kN/m^2

Factored load, W_u = 9.1875 kN/m^2

EFFECTIVE SPAN:

$L_{eff.}$ = L_x+d

Effective cover to the slab, c' = 25 mm = 0.025 m

Effective Depth of the slab, d = 100 mm = 0.1 m

$L_{eff.}$ = 3.1 m

(OR)

$L_{eff.}$ = L_x+t = 3.2 m

whichever is smaller

Required $L_{eff.}$ = 3.1 m

BENDING MOMENT:

Bending Moment, M = $(W_u * L_{eff.}^2)/8$

= 11.036484 kNm

= 11036484 Nmm

CHECK FOR EFFECTIVE DEPTH:

$M_{u\ lim.}$ = $0.138 * f_{ck} * b * d^2$

Consider 1 m width of the slab

= 1000 mm width of the slab

Effective depth, d = 63.235482 mm

Slab Depth is correct

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AREA OF STEEL REINFORCEMENT:

M_u = $0.87*f_y*A_{st}*d*(1-((f_y*A_{st})/(b*d*f_{ck})))$

By Quadratic Equation A = $f_y/(b*d*f_{ck})$ = 0.0003

B = -1

C = $M_u/(0.87*f_y*d)$ = 253.71

$A_{st\,1}$ = $(-B+sqrt(B^2-4AC)/2B$ = 3727.8 mm^2

$A_{st\,2}$ = $(-B-sqrt(B^2-4AC)/2B$ = 272.24 mm^2

Required A_{st} = 272.24109 mm^2

Assume 10 mm dia bars

a_{st} = 78.5 mm^2

No. of bars = 3.4680393 Nos.

= 4 Nos.

Spacing = 250 mm

(OR)

= 3d = 190 mm

(OR)

= 300 mm

Whichever is smaller

Required Spacing = 150 mm

Provide 10 mm dia bars @ 150 mm c/c.

CRANK DISTANCE:

$L_{eff}/4$ = 0.775 m

= 775 m

DISTRIBUTORY REINFORCEMENT:

$A_{st\,D}$ = 0.12%*c/s area = 150 mm^2

Assume 8 mm dia bars

a_{st} = 50.24 mm^2

No. of bars = 2.9856688 Nos.

= 3 Nos.

Spacing = 333.33333 mm

= 334 mm

(OR)

= 5d

= 500 mm

(OR)

= 300 mm

Whichever is smaller

Required Spacing = 300 mm

Provide 8 mm dia bars @ 300 mm c/c.

CHECK FOR SHEAR STRESS:

Nominal shear stress, τ_v = $V_u/b*d$

V_u = $(W_u*L_{eff.})/2$

= 14.240625 kN

τ_v = 0.1424063 N/mm^2

$A_{st\ provided}$ = 314 mm^2

$(100*A_{st\ provided})/(b*d)$ = 0.314 %

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Interpolation from I.S. 456-2000

τ_c (for f_{ck}=25 N/mm^2)	(100*$A_{st\ provided}$)/(b*d)
0.36	0.25
0.39328	0.314
0.49	0.5

Permissible Shear stress, τ_c = 0.39328 N/mm^2

Hence safe agaist shear

CHECK FOR DEFLECTION:

From I.S. 456-2000, pg.no.: 38, fig. 4:

Steel Stress of service, f_s = $0.58*f_y*(A_{st\ required}/A_{st\ provided})$

= 251.43285 N/mm^2

p_t = 0.314 %

Modification factor, M = 1.4

Allowable deflection, $\delta_{all.}$ = $k_f*k_c*k_t*M$

kf = 1

kc = 1

kt = $L_{eff.}/d$ = 31

$\delta_{all.}$ = 43.4 mm

But actual deflection, $\delta_{act.}$ = L_x/D = 24 mm

Hene safe against deflection

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PROJECT	ARTHY ASSOCIATES- EMERALD RESIDENTIAL APARTMENT, COIMBATORE
CONSULTANT	THE DESIGN ONE
DESIGN OF FOOTING	COLUMN-C3

DESIGN DATAS:

Column transfer load, P =	1100 kN			
=	1100000 N			
Breadth of the column on footing, b =	230 mm	=	0.23 m	
Depth of the column on footing, D =	530 mm	=	0.53 m	
Grade of concrete, f_{ck} =	25 N/mm^2			
Grade of steel, f_y =	500 N/mm^2			
Safe Bearing Capacity of Soil, SBC =	230 kN/m^2			

SIZE OF FOOTING:

Factored Load, P_u =	1650 kN		
=	1650000 N		
Self-weight of footing =	110 kN		
Total load on footing =	1210 kN		
=	1210000 N		
Area of footing =	Total load on footing/SBC		
=	5.26086957 m^2		
Size of footing =	2.29365855 m		
	2.3 m		
Length of the footing =	2.3 m	=	2300 mm
Breadth of the footing =	2.3 m	=	2300 mm
Provided area of footing =	5.29 m^2		
Ultimate soil pressure, W_u =	311.909263 kN/m^2		

Hence, size of footing is correct.

BENDING MOMENT:

Cantilever portion, l' =	0.885 m		
Bending Moment =	$W_u l'^2/2$		
=	122.147566 kNm		
=	122147566 Nmm		

DEPTH OF FOOTING:

Assuming the given section as balanced section,

$M_{u\,lim}$ =	$0.138*f_{ck}*b*d^2$		
Keep length of the footing =	1000 mm		
Effective depth, d =	188.162408 mm		
Hence provide,d=	600 mm depth of footing		
	0.6 m depth of the footing		
Keep minimum cover, c' =	50 mm		
Overall depth, D =	650 mm		

AREA OF TENSILE REINFORCEMENT:

M_u =	$0.87*f_y*A_{st}*d*(1-((f_y*A_{st})/(b*d*f_{ck})))$		
By Quadratic Equation A =	$f_y/(b*d*f_{ck})$	=	3E-05
B =	-1		
C =	$M_u/(0.87*f_y*d)$	=	468
$A_{st\,1}$ =	$(-B+\sqrt{B^2-4AC})/2B$	=	29524 mm^2
$A_{st\,2}$ =	$(-B-\sqrt{B^2-4AC})/2B$	=	475.5 mm^2
Required A_{st} =	475.536159 mm^2		
Minimum A_{st} =	0.12%*gross area of footing		
=	720 mm^2		
Hence, adopt A_{st} =	720 mm^2		
Assume	10 mm dia bars		

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a_{st} = 78.5 mm^2

No. of bars = 6.05778547 Nos.

= 7 Nos.

Spacing = 142.857143 mm

Provide 10 mm dia bars @ 143 mm c/c.

$A_{st\ provided}$ = 549.5 mm^2

% of tensile reinforcement, p_t = $(100*A_{st\ provided})/(b*d)$

= 0.09158333 %

CHECK FOR ONE-WAY SHEAR:

Nominal shear stress, τ_v = V_u/bd

V_u = $W_u*((L/2)-(b/2)-d)$

= 88.8941399 kN

= 88894.1399 N

Nominal shear stress, τ_v = 0.1481569 N/mm^2

Permissible shear stress, τ_c' = $k*\tau_c$

k = 1

(From I.S. 456-2000, pg.no.: 72, clause 40.2.1)

Interpolation from I.S. 456-2000

τ_c (for f_{ck}=25 N/mm^2)	$(100*A_{st\ provided})/(b*d)$
0.28	0.15
0.233266667	0.092
0.36	0.25

τ_c = 0.23326667

Permissible Shear stress, τ_c' = 0.23326667 N/mm^2

Hence safe agaist one-way shear

CHECK FOR TWO-WAY SHEAR:

Nominal shear stress, τ_v = $V_{uB}/b_o d$

VuB = $W_u*(L_xL_y-(b+d)(D+d))$

= 1357.4603 kN

= 1357460.3 N

b_o = 2(b+d)+2(D+d)

= 3920 mm

Nominal shear stress, τ_v = 0.57715149 N/mm^2

Permissible Shear stress, τ_c' = $k_s*\tau_c$

τ_c = $0.25*sqrt(f_{ck})$

= 1.25 N/mm^2

k_s = $0.5+\beta_s$

β_s = (Shortdimension/Long dimension) of column

= 0.43396226

k_s = 0.93396226 <1

= 1

Permissible Shear stress, τ_c' = 1.25 N/mm^2

Hence safe against two-way shear

CHECK FOR DEVELOPMENT LENGTH:

From I.S. 456-2000, Clause 26.2.1 :

Development Length, L_d = $(\Phi\sigma_s)/(4\tau_{bd})$

= 566.40625 mm

Assume 60 mm side cover

Available Length = 1005 mm

Hence O.K.

CHECK FOR BEARING OR TRANSFER OF LOAD ON FOOTING:

Allowable stress, $f_{br\ all.}$ = $0.45*f_{ck}*sqrt(A_1/A_2)$

where A_1 = Area of footing

A_2 = Area of column on footing

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A_1 =	5290000	m^2
A_2 =	121900	m^2
sqrt(A_1/A_2) =	6.58758123	>2
Therefore, keep sqrt(A_1/A_2) =	2	
Allowable stress, $f_{br\ all.}$ =	22.5	N/mm^2
But actual stress, fbr act. =	13.535685	N/mm^2

Hence safe against bearing

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PROJECT	ARTHY ASSOCIATES- EMERALD RESIDENTIAL APARTMENT ,COIMBATORE
CONSULTANT	THE DESIGN ONE
DESIGN OF COLUMN	C3

DESIGN DATA:

Axial Load, P =	1250	kN
Breadth of the column, b =	200	mm
=	0.2	m
Depth of the column, D =	530	mm
=	0.53	m
Unsupported length of the column in x-x axis, L_x =	3	m
Unsupported length of the column in y-y axis, L_y =	3	m
Bending Moment about x-x axis, M_x =	14.85	kNm
Bending Moment about y-y axis, M_y =	20.83	kNm
Effective length factor =	0.85	
Grade of concrete, f_{ck} =	25	N/mm^2
Grade of steel, f_y =	500	N/mm^2

EFFECTIVE LENGTH:

Effective length of the column in x-x axis, L_{ex} =	2.55	m
=	2550	mm
Effective length of the column in y-y axis, L_{ey} =	2.55	m
=	2550	mm

CHECK FOR SLENDERNESS RATIO:

L_{ex}/D =	4.8113208	
	<12	

Hence, short column

L_{ey}/b =	12.75	
	>12	

Hence, long column

CHECK FOR MINIMUM ECCENTRIES:

$e_{ex, min.}$ =	$(L_{ex}/500)+(D/30)$	
=	22.766667	mm
OR	20	mm
$e_{ey, min.}$ =	$(L_{ey}/500)+(b/30)$	
=	11.766667	mm
OR	20	mm
Factored Bending moment in x-x axis, M_{ux} =	22.275	kNm
Factored Bending moment in y-y axis, M_{uy} =	31.245	kNm
Factored Load, P_u =	1875	kN
=	1875000	N
M_{ux}/P_u =	11.88	mm
	<20	mm
M_{uy}/P_u =	16.664	mm
	<20	mm

Hence, design for uniaxial moment

LONGITUDINAL REINFORCEMENT:

Hence, Factored Bending Moment, M_u =	31.245	kNm
=	31245000	Nmm

Governing Axis for Bending is y-y axis

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$M_u/f_{ck}bD^2$	=	0.0589528	
	=	0.059	
$P_u/f_{ck}bD$	=	0.7075472	
	=	0.71	
Effective cover, d'	=	50	mm
Therefore, d'/D	=	0.25	
	=	0.2	
From Interaction chart of SP 16			
p/f_{ck}	=	0.1	
p	=	2.5	%
0.8%<p<4%			
Hence O.K.			
A_s	=	2650	mm^2
Assume		20 mm dia bars	
No. of bars, N	=	8.4394904	
	=	10	Nos.
$A_{s\ provided}$	=	3140	mm^2
Actual p	=	2.9622642	%
0.8%<p<4%			
Hence O.K.			
Hence, provide	***10 #***	***20 mm dia***	
bars uniformly distributed on four sides			

LATERAL REINFORCEMENT:

Diameter of Lateral Ties:

Dia of Lateral Ties	=	5 mm	(OR)
Dia of Longitudinal rft./4	=	5 mm	
Whichever is lesser			
Therefore Dia of lateral ties	=	5 mm	
	=	8 mm	

Spacing:

Least lateral dimension of the column	=	200	mm
16*Dia of the longitudinal bar	=	320	mm
	=	300	mm
Whichever is lesser			
Required Spacing	=	200	mm
Therefore, adopt	***8 mm dia lateral ties @ 200 mm c/c***		

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PROJECT	ARTHY ASSOCIATES-EMERALD APRTMENT, COIMBATORE
CONSULTANT	THE DESIGN ONE
DESIGN OF BEAM	Beam -B105

DESIGN DATA:

Breadth of the beam, b	=	230	mm	=	0.23 m
Effective depth of the beam, d	=	480	mm	=	0.48 m
Width of the supports	=	200	mm	=	0.2 m
Grade of concrete, f_{ck}	=	20	N/mm^2		
Grade of steel, f_y	=	500	N/mm^2		
Moment of Resistance, M	=	97	kNm		
Uniformly Distributed Load, W	=	58.1	kN/m		
Length of the beam,L	=	5.1	m	=	5100 mm

PRELIMINARY DIMENSIONS OF THE BEAM:

Assume effective cover of the beam,c'	=	30	mm	=	0.03 m
Therefore,overall depth of the beam, D	=	510	mm	=	0.51 m
a)Effective Span, $L_{eff.}$	=	L+d	=	5.58 m	
b)Effective Span, $L_{eff.}$	=	L+W	=	5.3 m	
Therefore, adopt Effective Span, $L_{eff.}$	=	***5.3 m (minimum)***			

ULTIMATE MOMENTS:

$x_{u\,lim.}/d$	=	0.48		
Therefore, $x_{u\,lim.}$	=	230.4	mm	
M_u'	=	$0.36*f_{ck}*b*x_u*(d-0.42x_u)$		
	=	146219257	Nmm	
	=	146.219257	kNm	
Ultimate moment, M_u	=	1.5*M	=	145.5 kNm
M_u''	=	M_u-M_u'		
	=	-0.71925704	kNm	
	=	-719257.037	Nmm	

TENSILE REINFORCEMENT:

From I.S. 456:2000, pg.no. 96, Annex-G,section G-1.1 a)				
$x_{u\,lim.}/d$	=	$(0.87*f_y*A_{st})/(0.36*f_{ck}*b*d)$		
Therefore, $A_{st\,1}$	=	877.108966	mm^2	
M_u''	=	$f_{sc}*A_{st\,2}*(d-d')$		
where d'	=	distance from top fibre to the centre of compression reinforcement.		
	=	50	mm	

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Therefore, $A_{st\,2}$ = -3.84526617 mm^2

Total A_{st} = 873.263699 mm^2

Assume 16 mm dia bars.

a_{st} = 200.96 mm^2

Number of bars, n = 4.34546029 = 5 Nos.

Spacing = 46 mm

Therefore adopt 5 nos. of. 16 mm dia bars @ 40 mm c/c. in tension zone.

$A_{st\ provided}$ = 1607.68 mm^2

COMPRESSION REINFORCEMENT:

$A_{st\,2}$ = $(A_{sc}*f_{sc})/(0.87*f_y)$ (In pg.no. 56 in R.C.C. book)

ε_{sc} = $0.0035*((x_u-d')/x)$ By interpolating:

f_{sc} = 353.725 N/mm^2

A_{sc} = -4.7287887 mm^2

ε_{sc}	f_{sc}
0.00276	351.8
0.00298	353.725
0.0038	360.9

Assume 16 mm dia bars.

a_{st} = 200.96 mm^2

Number of bars, n = -0.02353099 = 1 Nos.

Spacing = 100 = 100 mm

Therefore adopt 2 nos. of 16 mm dia bars @ 100 mm c/c. in compression zone.

$A_{sc\ provided}$ = 401.92 mm^2

CHECK FOR SHEAR:

From I.S. 456:2000, pg.no. 73, Annex-B-5.1)

Nominal shear stress, τ_v = v_u/bd

where shear force ,v_u = 222.2325 kN

= 222232.5 N

τ_v = 2.01297554 N/mm^2

% of tensile reinforcement, p_t = 1.45623188

FROM IS 456:2000 pg.no.:73 table 19:

p_t	τ_c
1.5	0.75
1.456231884	0.74649855
2	0.79

Design shear stress, τ_c = 0.74649855 N/mm^2

$\tau_v > \tau_c$ **Hence not safe**

Hence provide shear reinforcement

v_{us} = $v_u-\tau_c bd$

= 139819.06 N

= 139.81906 kN

Use 2 legged 8 mm dia bars

A_{sv} = 100.48 mm^2

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s_v	=	$(0.87*f_y*Asv*d)/v_{us}$	
	=	150.052675	mm
	=	150	mm
s_v	>	0.75d	
	=	360	mm
Adopt		**2 legged**	**8 mm dia bars**
	@	**150 mm c/c**	**near the support &**
		350 mm c/c	**at mid-span.**

CHECK FOR DEFLECTION:

$(l/d)_{act}$	=	10.625	
$(l/d)_{max}$	=	$(l/d)_{basic}*k_t*k_c*k_f$	
p_t	=	1.45623188	
p_c	=	0.36405797	
Refering the diagram,			
k_t	=	0.93	
k_c	=	1.1	
k_f	=	1	
Therefore, $(l/d)_{max}$	=	20.46	
		Hence safe against deflection	

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