

STABILITY CERTIFICATE

(To be furnished while filing application for registration of building)

The detailed engineering and structure design of the proposed building comprising of S+5 floors at **R.S.F.NO. 517/1B, SUB-DIVISION R.S.F. NO. 517/1B2, SITE NO. 10, 11, 12, 13, 14, OLD WARD NO. 46, NEW WARD NO. 49, ERODE " C " VILLAGE (KASIPALAYAM), AT BLUE LEAF GARDEN (PHASE-II), IRANIYAN 2nd STREET (ANNA NAGAR), ERODE CITY MUNICIPAL CORPORATION, ERODE TALUK & DISTRICT** has been done by me/us based on the report of geotechnical investigation (soil test) done by **V R GEOTECHNICAL ENGINEERS** Report No. **Ref: 751/Soil-2022-23** Dated **27.07.2022** and considering the functional requirement of the building.

2. It is certified that among other factors, the proposed building has been designed to resist earthquake. I/We have checked various parameters and found that the proposed building would be safe.

3. I/We further certify that:

(i) The minimum grade of concrete is **M25**

(ii) The design and analysis has been done using the code of practice for plain and reinforced concrete as per IS 456, design loads as per IS Codes No **875 Pt.1 & Pt.2** and design of earthquake resistant structure as per IS Code No.1893.2002.

4. The building will be safe and sound when used for the purpose for which it is designed.

Encl.: Structural Design Calculation




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BLUE LEAF GRANGE APARTMENT ERODE. DESIGN CALCULATIONS

INTRODUCTION:

The structure is a “Six– Storied Building” consisting of Silt + 5 upper floors proposed to be constructed for “BLUE LEAF” in R.S.F.NO. 517/1B, SUB-DIVISION R.S.F. NO. 517/1B2, SITE NO. 10, 11, 12, 13, 14, OLD WARD NO. 46, NEW WARD NO. 49, ERODE " C " VILLAGE (KASIPALAYAM), AT BLUE LEAF GARDEN (PHASE-II), IRANIYAN 2nd STREET (ANNA NAGAR), ERODE CITY MUNICIPAL CORPORATION, ERODE TALUK & DISTRICT.

It is designed as Reinforced concrete framed structure supported on footings and columns. Footings are placed at 10'-0" depth below ground level and the **SBC of the soil is taken 310 kN/Sq.m**

MATERIALS USED

Following material are generally used in the design of the various components of the structure.

1. M25 & M30 grade concrete for all concrete parts of the structure.
2. Fe 500 & Fe550D grade Reinforcement steel for all Reinforced concrete members.

CODES FOLLOWED:

Following codes and specifications has been used in the design of the members of the structure.

1. IS:456 - 2000 Plain and Reinforced Concrete
2. SP: 16 Design Aids for Reinforced Concrete to IS: 456
3. IS: 875 Part1,2,4,5(1987) and Part 3(2015) Design Loads
4. IS: 1893 – 2016 Criteria for Earthquake Resistant Design of Structures
5. IS: 13920 – 2016 Ductile Detailing for Reinforced Concrete Structures

Subjected to Seismic forces.




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DESIGN DATA:

1. No. of floors	=	Stilt + 5 upper floors
2. Height of floors	=	3.05 m
3. SBC of soil @ 10'-0"	=	310.0 kN/m ²
4. D.L. of 150 thk slab	=	3.75 kN/m ²
5. D.L. of 140 thk slab	=	3.5 kN/m ²
6. D.L. due to flooring	=	1.00 kN/m ²
7. D.L. of 230 thk walls per metre length	=	13.50 kN/m
8. D.L. of 115 thk walls per metre length	=	6.00 kN/m
9. L.L on rooms	=	2.00 kN/m ²
10. Design mix of Columns, Footings, Slabs, Beams	=	M25 & M30

ANALYSIS OF STRUCTURE:

The structure is analyzed by using Etabs. All members are Manually designed as Reinforced Concrete members.

Shear Walls are designed by using Etabs.

Independent Footing has been provided for columns.

DESIGN OF COLUMNS:

Material – M25; Fe550D

LOAD ON COLUMNS:

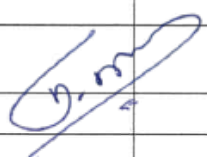
R1	=	3500 KN
R2	=	2690 KN
R3	=	2130 KN
R4	=	2002 KN
R5	=	1440 KN
R6	=	1000 KN
R7	=	800 KN

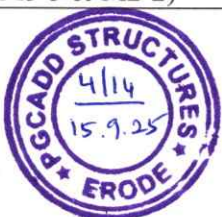



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COLUMN DESIGN:

DESIGN OF COLUMN								
LOAD(P)	=	3550	KN					
Fact. Load	=	5325						
f_{ck}	=	25	N/Sq .mm					
f_y	=	550	N/Sq .mm					
Unsupported length of column	=	3.5	m					
Column end conditions	=	Effectively held in position and restrained against rotation						
Recommended value of effective length	=	0.65L	=	2.275	m			
Assumed area of steel	=	0.80%	=	0.08A _g				
Size of column Assumed	=	300	x	760				
Provided Area of concrete	=	228000						
Check for Slenderness ratio								
Effective length of column	=	2.275	m					
Slenderness ratio	=	l/b	=	7.58	<	12		
		l/d	=	2.99	<	12		
So the column will be designed as short column								
Check for eccentricity								
Min. Eccentricity	=	$(L/500) + ((b/30))$			=	17.00	mm	
	=				=	32.33	mm	
	=	20	mm		(As per IS 456-2000 cl 25.4)			
Eccentricity taken as (e)	=	32.33	mm					
Moment calculations								
Moment due to eccentricity vs As per Analysis	=	$(P \cdot e)$	=	114783333.3	N.mm	=	230	KN .M
Reinforcement calculations								
Longitudinal reinforcement								
$P/(f_{ck} \cdot b \cdot d)$	=	0.9342						
$M/(f_{ck} \cdot b \cdot d^2) - (Max \text{ of } M3-3 \text{ \& } M2-2)$	=	0.053						


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d'	=	56.000	mm	(Assumed as	32	mm	dia bars to be used)	
d'/d	=	0.2						
For these values p/fck from the chart 38 of IS hand book SP-16 - As	=	0.14	fck	=	3.5	%		
Min. Reinforcement for column	=	0.8	%					
Area of steel taken as	=	3.5	%					
Area of steel reqd.	=	7980	Sq.m		8493.724			
Provide	32	mm dia bars	6					
and	32	mm dia bars	6					
Ast. Provided	=	9646.08	Sq.m	>	8493.724	HENC EOK		
Transverse reinforcement								
Max. Length of the Longitudinal reinforcement direction =					760			
No. of rods placing in one side =					2			
Spacing of Longitudinal bars other than corner =					226.6667	>75		
Provide	8	mm dia double stirrups @			150	mm C/C		
Check for Min spacing								
150	<	300	mm	(Least lateral dimension)				
150	<	512	mm	(16 times to the smallest dia of long. bar)				
150	<	300	mm			Hence O.K		
Column Design Result								
Size of Column	=	300	x	760				
Longitudinal Reinforcement	32	mm dia bars	6	nos. and				
	32	mm dia bars	6	nos.				
Transverse reinforcement	8	mm dia double stirrups @			150	mm C/C		



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COLUMN SCHEDULE:

COLUMN NAME	AXIAL LOAD(KN)	COLUMN SIZE	REINFORCEMENT	STIRRUPS
R1	3550	300 x 760	6Y32 + 6Y32	Y8 @ 150mm c/c
R2	2690	230 x 760	14Y25	Y8 @ 150mm c/c
R3	2130	230 x 680	10Y25	Y8 @ 150mm c/c
R4	2002	230 x 600	10Y20	Y8 @ 150mm c/c
R5	1440	230 x 460	4Y25 + 8Y20	Y8 @ 150mm c/c
R6	1000	230 x 610	10Y20	Y8 @ 150mm c/c
R7	800	230 x 460	4Y20 + 4Y16	Y8 @ 150mm c/c

SHEAR WALL SCHEDULE:

SHEAR WALL NAME	AXIAL LOAD(KN)	SHEAR WALL SIZE	REINFORCEMENT	STIRRUPS
SW1	2860	230 x 1220	8Y25 + 16Y20	Y8 @ 150mm c/c



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FOOTING DESIGN:

Design of Footing										
(A) INPUT DATA										
Safe Bearing Capacity of Soil, SBC			310.00	KN/m ²	Concrete, F _{ck}		25.00	MPa		
Cross- sectional width of column, b			300.00	mm	Steel, F _y		550.00	MPa		
Cross- sectional depth of column, D			760.00	mm	Clear Cover, CC		50.00	mm		
Axial Load (Service), P			3550.00	KN	Dia. of Steel, d _b		12.00	mm		
Footing wieght by (%) P	5.00	%	177.50	KN	Gross Load,		3727.50	KN		
(B) FOOTING SIZE										
Area of Footing Required, A _f		12.02	m ²	Provided, A _f		12.08	m ²			
Length of footing required, L _f		3.71	m	Provided, L _f		3.96	m	C _{LL}	1.601	
Width of footing required, B _f		3.25	m	Provided, B _f		3.05	m	C _{BB}	1.374	
Top projection from column face, K		801	mm	Depth at edge of footing, E _d				914	mm	
Upword soil pressure,		308.63	m ²	Upword Soil Pressure < SBC , Hence OK						
(C) Depth Required For Flexure										
Moment along L-L axis, M _{uL}		1808.89	KN-m	Moment along B-B axis, M _{uB}				1731.56	KN -m	
Resisting width in concern		1901.20		Resisting width in concern				2361.20	mm	
Depth required along length, d _{req}		598.07	mm	Depth required along width,d _{req}				525.06	mm	
Provide the depth of footing, D		914	mm	Available effective depth, d _{avai}				858.40	mm	
Actual Self weight of footing,		276.09	KN	Weight by % of loading (P),				7.78	%	
(D) Design for Flexural Reinforcement										
Direction	A _{st} Required (mm ²)	A _{st} Minim un (mm ²)	Dia. of Bar (mm)	Sapcing (mm)		Rebar Provided				
				Require d	Provided					
Along L _f	4704.48	3344.51	20	203.54	150.00	T20 @ 150 mm				
Along B _f	4430.87	4347.86	20	280.94	150.00	T20 @ 150 mm				
A _{st} finally provided along L _f - L _f		6383.73	mm ²	Pt (%)	0.367	1-way shear is needed to check along L-L				
A _{st} finally provided along B _f - B _f		8298.85	mm ²	Pt (%)	0.384					
(E) Check for 1-Way Shear										
Note: Critical Section is at d = 858.4 mm from column face										
Check along Length (L _f) of Footing					Check along Width (B _f) of Footing					



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Where, $\beta = (0.8 f_{ck}) / (6.89 P_t)$ or 1 whichever is greater,							$\tau_c = 0.85 \sqrt{(0.8 f_{ck}) (\sqrt{1 + 5\beta} - 1)}$		
(%) R/F available	Pt =	0.35	%	(%) R/F available	Pt =	0.37			
Therefore,	$\beta =$	8.39	-	Therefore,	$\beta =$	7.92			
Critical Shear Capacity, $T_{cc} =$		0.48	MPa	Critical Shear Capacity, $T_c =$		0.43	MPa		
Shear Force along length, $V_u =$		1048.15	KN	Shear Force along width, $V_u =$		945.82	KN		
Critical Section Moment, $M_u =$		389.28	KN-m	Critical Section Moment, $M_u =$		243.83	KN-m		
				Critical section b' & d'		858.40	mm		
At critical section, b'		2016.80	858.40			2476.80			
Nominal Shear, $\tau_v =$		0.42	MPa	Nominal Shear, τ_v		0.39	MPa		
Provided Depth is Safe For One Way Shear				Provided Depth is Safe For One Way Shear					
(F) Check for 2-Way Shear				Note: Critical Section is at d/2 = 429.2 mm					
Effective depth at proposed section, d/2 from the face of column, d'					858.40		mm		
Shear Force on Shaded Area, $V_u =$		4723.33	KN	Perimeter length of Shear Area, =		5553.60	mm		
							0.991 MPa		
$\tau_v = \frac{V_u}{(\text{Perimeter length} \times \text{Effective depth})}$									
IS 456-2000, Pg.No.58, Cl.31.6.3, Shear Stress @ Critical Section,							$T_c \times K_s$		
$K_s = 0.5 + \beta_c$		Where, $\beta_c =$		0.39	Therefore,	K_s	=	0.89 or 1.00	
				1.118	MPa	Provided Depth is Safe For Two Way Shear			
$\tau_c = K_s \times 0.25 \times \sqrt{f_{ck}}$									

FOOTING SCHEDULE:

F.No	FOOTING SIZE						Bottom Mat	
	Step-1	Depth(D1)	Step-2	Depth(D2)	Step-3	Depth(D3)	Rebar(X)	Rebar(Y)
F1	3.96 x 3.05	910	Refer section on F1-F1				Y20 @ 150 C/C	Y20 @ 150 C/C
F2	3.35 x 2.74	835	Refer section on F2-F2				Y16 @ 125 C/C	Y16 @ 125 C/C
F3	2.89 x 2.89	460	2.14 x 2.14	460	1.37 x 1.37	460	Y16 @ 150 C/C	Y16 @ 150 C/C
F4	2.36 x 2.36	300	1.75 x 1.75	300	1.00 x 1.00	300	Y12 @ 125 C/C	Y12 @ 125 C/C
F5	2.21 x 2.21	460	1.52 x 1.52	460	---	---	Y12 @ 150 C/C	Y12 @ 150 C/C
F6	1.98 x 1.98	460	1.06 x 1.06	460	---	---	Y12 @ 150 C/C	Y12 @ 150 C/C
F7	1.68 x 1.68	380	0.91 x 0.91	380	---	---	Y12 @ 150 C/C	Y12 @ 150 C/C



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MAXIMUM MOMENT ON BEAMS:

End of the span = 239 kNm

Middle of the span = 387 kNm

MAXIMUM SHEAR ON BEAM:

End of the span = 210 kN

Middle of the span = 15 kN

BEAM DESIGN:

Beam Design										
Properties of Beam						Forces				
Fck	=	30	N/m ²				at top LHS	at bottom Mid Span	at top RHS	Units
Fy	=	550	N/m ²							
Length	=	8.2296	m			Moment	239	387	100	Kn.m
Size	=	228.6	X	685.8	mm	Shear	210	1	166	Kn
DESIGN CALCULATIONS										
FACT.	at top LHS	at bottom Mid Span	at top RHS	Units	Mu Lim for Balanced section					
MOMENT	358.5	580.5	150	Kn.m	Mu Lim = (0.36*Xu max/d)*(1-(0.42*Xu max/d))*b*d ² *fck					
SHEAR	315	1.5	249	Kn	ML	=	355.88			Kn.m
Xu max/d	0.48	0.48	0.48							
MF>Mu limit, so the beam is designed as Doubly reinforced beam										
MF - ML	at top LHS	at bottom Mid Span	at top RHS	Units	Dia. Of Bars to be used					
					at top LHS		at bottom Mid Span		at top RHS	
	2.62	224.62	0.00	Kn.m	I Layer	II layer	I Layer	II layer	I Layer	II layer
Effective depth d=	613.3	613.3	613.3	mm	Tensile zone reinforcement					
					25	25	25	25	25	25
d'=	47.5	47.5	47.5	mm	Comp. zone reinforcement					
d' / d	0.0774	0.07745	0.07745		25		25		25	
					at top LHS		at bottom Mid Span		at top RHS	
Design stress in compression reinforcement (fsc)				700*(1-(d'/Xu)) =			(Xu limited to Xumax)			



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	587.05	587.05	587.05	N/m m ²
Allowable stress in compression steel (0.8*0.87fy)	382.8	382.8	382.8	N/m m ²
Design stress in comp. reinforcement taken (fsc)	382.8	382.8	382.8	N/m m ²
Strain in compression steel	0.0029	0.0029	0.0029	Max. strain

Tensile Steel Calculations

Ast for singly reinforced beam (Ast1)	(Mu=0.87*fy*Ast*d - 0.87* fy ² * Ast ² / b*fck)			
	at top LHS	at bottom Mid Span	at top RHS	
(from the eqn. IS 456 Annex.G 1.01 b)	1511.42	1511.42	550.809	mm ²

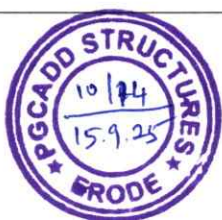
Compression Steel Calculations

Area of comp steel (Asc)	(Mu- Mulim) / (fsc*(d- d'))			
(from the eqn. IS 456 Annex.G 1.2)	12.08119	1037.0675 93	0	mm ²
Area of comp steel (Asc) for max. stress	12.55002	1077.3120 07	0	mm ²
Ast 2 = (Asc*fsc) / (0.87fy)	at bottom LHS	at Top Mid Span	at bottom RHS	
	10.04001	861.84960 52	0	mm ²

Shear Calculations

	LHS	MIDSPA N	RHS	
Percentage of tensile steel=	1.56	1.56	1.56	%
Tc from Table 19 of IS 456 : 2000	0.770	0.791	0.770	N/mm ²
Tv=	2.246784	0.0106989 72	1.77602 9	N/mm ²
Maximum Allowed Shear stress Tc max =		3.4		
Shear strength of concrete (Tc*b*d)	108.00	110.90996 18	108.004 8	KN
Strength of shear reinforcement reqd. (Vus)	207.00	-109.41	141.00	KN
Dia. Of shear reinforcement	10	10	10	
Shear reinforcement = Sv= (0.87*fy*Asv*d) / Vus	222.5842	300	326.776 2	mm C/C
Min. shear reinforcement	821.5715	821.57152 23	821.571 5	mm C/C
Max spacing (0.75*d)	459.975	459.975	459.975	mm C/C
	300	300	300	mm C/C

Check for development length



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							1.3 * M1/V+	L0	>=Ld	L0=	613.3
Ld=(0.87*fy / (4*tbd))* Dia. Of Bar=							1246.0937 5	mm	<	2082. 025	mm
RESULTS											
Fck	=	30	Fy	=	550	Beam Size	=	228.6	x	685.8	
SUP P.	Steel Provided						Min.Reqd spacing for stirrups	Tensile steel		Comp. steel	
	Straight		Curtail		Stirrups			Prov.	Reqd.	Prov.	Req d.
	Di a.	Nos.	Dia.	Nos.	Dia.	Spacing					
LHS	25	3	25	2	10	150	223	2453	1521	1472	13
MID SPA	25	3	25	2	10	150	300	2453	2373	1472	1077
RHS	25	3	25	2	10	150	300	2453	551	1472	0
Check for Deflection											
Stress Developed in Steel, F _s							=	308.62	N/Sq.mm		
Maximum Provided % of reinforcements, P _t							=	1.56	%		
Modification Factor, K _t (M/F)							=	0.75	-		
Maximum Allowed Deflection Ratio, δR _{Allowed}							=	19.40	-		
Actual Deflection Ratio, δR _{Actual}							=	13.42	-		
DESIGN IS SAFE IN DEFLECTION											

ROOF BEAM SCHEDULE:

BEAM No	BEAM SIZE	REINFORCEMENT			STIRRUPS
		Bottom	Top	Extra	
RB1	230 X 685	3-Y25 + 2-Y25	3-Y25	2-Y25	Y10 @ 150 C/C
RB2	230 X 609	3-Y25 + 2-Y25	3-Y25	2-Y25	Y10 @ 150 C/C
RB3	230 X 609	2-Y25 + 2-Y25	2-Y25	2-Y25	Y10 @ 150 C/C
RB4	230 x 530	3-Y25 + 2-Y25	3-Y25	2-Y25	Y10 @ 150 C/C
RB5	230 X 609	2-Y25 + 2-Y25	2-Y25	2-Y25	Y10 @ 150 C/C
RB6	230 X 609	2-Y20 + 2-Y20	3-Y20	2-Y20	Y10 @ 150 C/C
RB7	230 X 609	2-Y20 + 2-Y20	2-Y20	2-Y20	Y10 @ 150 C/C
RB8	230 x 530	2-Y20 + 2-Y20	2-Y20	2-Y20	Y10 @ 150 C/C
RB10	230 x 460	2-Y20 + 2-Y20	2-Y20	2-Y20	Y8 @ 150 C/C
CB1	230 x Slab Tk	3-Y12	3-Y12	---	Y8 @ 150 C/C




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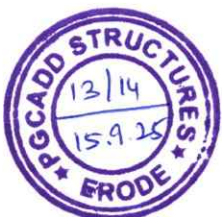
ROOF SLAB DESIGN:

Slab Design						
Size of the slab in 'm'	=	5.03	x	5.03		
Support width	=	0.23	m			
Slab tk.(Span/25)	=	201.168	mm			
Adopt eff. Depth	=	125	mm			
Overall Depth	=	150	mm			
Grade of concrete	=	M 20		20	N/sq.mm	
Grade of steel	=	Fe 500		500	N/sq.mm	
Loads						
Dead				Live	Fact.	
Dead	Floor finish	Total			Load(1.5D.I+1.5I.I)	
Kn/Sq.m	Kn/Sq.m	Kn/Sq.m		Kn/Sq.m	(w) Kn/Sq.m	
3.75	1.5	5.25		2	10.875	
Design Calculations						
From IS 456:2000, Anx.D2.1						
Shorter span Bending Moment		Mx=	ax*w*lx ²	ax,ay are co efficients		
Shorter span Bending Moment		My=	ay*w*lx ²			
Effective Shorter span		Lx=	5.26	m		
Effective Longer span		Ly=	5.26	m		
		Ly/Lx=	1.00	<	2	
So the Slab is designed as Two way slab						
Edge Condition =		1	Interior panels			
(-)ax=	0.032		(-)ay=	0.032	Refer IS 456:2000 Table 26	
ax=	0.024		ay=	0.024		
Midspan moments		Support Moments				
Mx=	7.22	Kn.m	(-)Mx	9.63	Mx max=	9.63
My=	7.22	Kn.m	(-)My	9.63	My max=	9.63
Check for Depth :						
Mmax=		0.138*fck*bd ²				
d=		Sqrt(M/(0.138*fck*b))				
d reqd=		59.0547				
		408	mm	<	125	
Area of steel Calculations						
Ast for Slab			(Mu=0.87*fy*Ast*d - 0.87* fy ² * Ast ² / b*fck)			
			Shorter span		Longer Span	
			Reqd	Provide d	Reqd	Provide d
			171.1596	523.3333	171.15965	523.3333
(from the eqn. IS 456 Annex.G 1.01 b)			56	33	6	33
			At support		At mid span	



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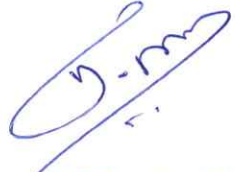
		Dia of rod	Spacing	% of ast	Dia of rod	Spacing	% of ast
Shorter span		10	150	0.42	10	150	0.42
Longer span		10	150	0.42	10	150	0.42
Torsion reinforcement							
Area of steel		Dia of rod	Spacing	Length in mm	To be Provide @ corners of the slab in both directions & both faces		
346		10	150	1051.84			
Check for shear							
	% of Ast=	0.42		$\zeta_c \text{ max} =$	1.4	N/mm ²	
	$\zeta_v =$	V/b*d	0.18	N/mm ²	Refer IS 456 cl 40.1		
	$\zeta_c =$		0.44	N/mm ²	Refer IS 456 Table 19		
	ζ_v	<	ζ_c	< $\zeta_c \text{ max}$	DESIGN IS SAFE		
Check for Crack width							
ϵ_1 =Strain@lvl considered		$\epsilon_2 = 1.5b(D-x)(a-x)/3A_sE_s(d-x)$			$W_{cr} = \{3 \cdot a_{cr} \cdot \epsilon_m\} / \{1 + [2(a_{cr} - c_{min}) / (D-x)]\}$		
	acr	X	fsb	$\epsilon_1 = f_{sb} / E_s$	ϵ_2	$\epsilon_m (\epsilon_1 - \epsilon_2)$	Wcrb
S span	85.1388	56.96	96.16	0.00048	0.00061	0.0000	0.000
L Span	45.0000	68.16	67.14	0.00034	0.00061	0.0000	0.000
Check for Deflection							
Stress Developed in Steel, F_s					=	94.85	N/Sq.m m
Maximum Provided % of reinforcements, P_t					=	0.42	%
Modification Factor, $K_t (M/F)$					=	3.40	-
Maximum Allowed Deflection Ratio, $\delta R_{\text{Allowed}}$					=	88.37	-
Actual Deflection Ratio, δR_{Actual}					=	42.07	-
DESIGN IS SAFE IN DEFLECTION							




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ROOF SLAB SCHEDULE:

Slab No.	Tk	Type of slab	Main Reinforcement	Distributor	Shorter Span	Longer Span
S1	150	Two Way	---	---	Y10 @ 150 C/C	Y10 @ 150 C/C
S2	150	Two Way	---	---	Y8 @ 150 C/C	Y8 @ 150 C/C
S3	150	One Way	Y10 @ 150 C/C	Y8 @ 150 C/C	---	---
S4	150	One Way	Y8 @ 150 C/C	Y8 @ 150 C/C	---	---


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Note:

Adequate Plinth beams, Continuous Lintels are provided for the earthquake resistance. Columns are designed for horizontal shear forces. The detailing of Reinforcement is done according to provisions of IS:13920 – Indian Standard Code of Practice for ductile detailing of R.C. Structures subjected to Seismic forces.

