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J.PRABAKARAN ,M.E
STRUCTURAL ENGINEER
No. 1/4, PIDARIAMMAN KOIL ST
UTHANDI
CHENNAI- 600119.

prabakaran_j@yahoo.co.in
Mobile: 9841757884

Date : 14.10.2024

STRUCTURAL STABILITY CERTIFICATE

It is certified that the

PLAN SHOWING THE PROPOSED NEW CONSTRUCTION OF A RESIDENTIAL BUILDING AT PLOT NO - 1 TO PLOT NO- 26 AND ONE CLUB HOUSE BUILDING, KALAIGAR KARUNANITHI SALAI, SHOLINGANALLUR, CHENNAI - 600 119. COMPRESSED IN S.NO: 180/ ,181/3A2,181/3B,181/3C AND 181/3D OF SHOLINGANALLUR VILLAGE, GREATER CHENNAI CORPORATION DIVISION - 197, ZONE - XV.

is designed to resist for earthquake. Check has been made and found safe and I hereby certify that

1. The minimum grade of concrete is M25
2. The design analysis using the code of practice for loading . standards as per BIS-875 and Seismic forces as per the latest code BIS-1893 & BIS-456
3. Least dimension of column is 200mm in stilt floor.
4. The proposed structure will be structurally safe and sound.

J. Prabakaran.

J. PRABAKARAN, M.E.,
Structural Engineer
CMDA REG. No: SE/GR-I/2020/03/263
GCC REG No : RSE 101182020
No:1/4, Pidariamman Kovil Street,
Uthandi, Chennai - 600 119.
Mail : prabakaran_j@yahoo.co.in
Mobile : 9841757834

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TYPICAL STRUCTURAL DESIGN CALCULATION

1 **General:**

Proposed building is G + 1 upper floors for residential purpose. The floor height is 3.50 m. Typical one plot design calculation given.

2 **Design Basis:**

Latest Design codes of BIS-456, 875 and 1893 are followed with SP-16 Design Aid book. A typical Frame was taken for analysis to arrive at the bending moment for column design. Since the building is 2 stories, wind force is neglible effect. No Live load reduction factor is applied in design. Isolated foundation is proposed as per soil report. Fe- 500 CRS steel is used as main steel and M30 grade concrete.

3 **Earthquake consideration:**

For better stability during earhquake, the columns are tied with plinth beams. Continuous lintels are specified for outer walls and column ties are closely spaced for better ductility.

4 **WIND LOAD**

Vz	=	Vb x K1 x K2 x k3	
Vb	=	50 m/sec	in Chennai
K1	=	Probability Factor	= 1
K2	=	Terrain, Height Factor =	1
K3	=	Topography Factor	= 1
p	=	0.6 x Vz x Vz	
	=	0.6 x 50 x 50	
	=	1500	N/Sq.m

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GCC REG No : RSE 101182020
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Mail : prabakaran_j@yahoo.co.in
Mobile : 9841757884

5 **Design Loads:**

Slab thk	130	=	3.25 KN/sq.m
Live load		=	2.00 KN/sq.m
Finishes		=	1.60 KN/sq.m
			6.85 KN/sq.m

Say 6.85 KN/sq.m/floor for slab and beam design
Applying live load reduction of = 0 ,
For column and foundation design, 6.85 Kn/sqm is taken.

This value is also taken for frame analysis

AAC block 200 thick for exterior walls

	0.2	x	3	x	8		=	4.80	KN/m/floor
Beams	0.2	x	0.45	x	25		.=	2.25	KN/m/floor
Columns	0.2	x	0.45	x	3.60	x 25	.=	8.10	KN/floor

These are taken for analysis with WL+LL+DL combination.

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6 **SLAB DESIGN:** S1

Thickness	=	150	mm	Concrete mix	Fck	= M 25	N/sq.mm
Cover	=	25	mm	Steel Grade	Fy	=Fe 500	N/sq.mm
Eff.Depth	=	120	mm				

Panel (4.3 x 5.7) m

=

Twoway slab Ly / Lx 5.7 / 4.3 .= 1.3

Support condition - 2 TWO SHORT EDGE DISCONTINUOUS

short span=	α x	=	0.056	(- BM at continuous edge)	Table:26
		=	0.043	(+ BM at midspan)	in IS-456-2000
long span=	α y	=	0.000	(- BM at continuous edge)	
		=	0.035	(+ BM at midspan)	

Designed for 0.056 co.efficient

Mu .= 0.056 x 6.85 x 1.5 x 4.30 x 4.30 = 10.64

$$\text{Mu / b.d.d} = \frac{10.64 \times 1000 \times 1000}{1000 \times 120 \times 120} = 0.74 \text{ N/sq.mm}$$

From Table-3 of SP-16, Pt .= 0.229 %

Minimum Pt = 0.12 % as per code, for Fe-500

At = 0.229 x 1000 x 120 ./ 100 = **275** sq.mm/m

Provide 10 dia @ 150 c/c = **523.6** sq.mm/m Short span

Provide 8 dia @ 150 c/c = **335.1** sq.mm/m Long span

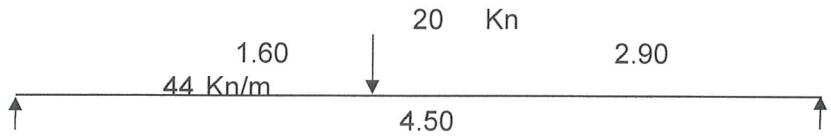
J. Prabakaran.

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7 **Beam design:** B1

Effective span = 4.50 m Beam size 200 x 450

Fck = M 25 d = 410 mm



Max. Bending Moment at support =

$$= \frac{44 \times 4.5 \times 4.5}{10} + 13.3 = 102 \text{ Knm}$$

Mu = 102.4 x 1.5 x 10-e6 = 153.6 x 10-e6

$$Mu/b.d.d = \frac{154 \times 10\text{-e6}}{200 \times 410 \times 410} = 4.57 \text{ N/sq.mm}$$

From table-4 SP-16 for d'/d = 70 / 410 = 0.17

Pt = **0.250** % Pc = **0.125** %

At = 0.250 x 600 x 410 / 100 = **615** sq.mm (bot)

Ac = 0.125 x 600 x 410 / 100 = **307.5** sq.mm (top)

provided 4 No 16 = 804 sq.mm

804 sqmm Top support

Shear Design:

Shear force = $\frac{44 \times 4.5}{2} + 260 = 359$ KN

Factored shear force = 1.5 x 359 = 538.5 KN

for 0.250 % steel Shear resisted by concrete section

from table-61 0.68 x 600 x 410 / 1000 = 167 KN

Balance shear (Vus) = 538.5 - 167 = 371 KN

Vus/d = 371 / 53 = 7.0 KN/cm

from table 62 of SP-16

Required 2 legged	8 stirrups	@	150	mm c/c
Provided 2 legged	**8** stirrups	@	**110**	mm c/c

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Structural Engineer
CMDA REG. No: SE/GR-I/2020/03/263
GCC REG No : RSE 101182020
No:1/4, Pidariamman Kovil Street,
Uthandi, Chennai - 600 119.
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Mobile : 9841757884

8 **COLUMN DESIGN:** C1 COL SIZE 200 X

Maximum load on column in basement P = 30 t

Pu = 1.5 x 30 x10 = 450

Max.Bending Moment = 3 t.m Mu = 1.5 x 3.0 x10= 45

Cover = 40 mm

d'/d = 0.09

Fck = 25 N/sq.mm

Provide bars equaly all sides

Refer chart 48 of SP-16

$$\frac{Pu}{Fck\ b\ d} = \frac{450 \times 1000}{25 \times 200 \times 450} =$$

$$\frac{Mu}{Fck\ b\ d^2} = \frac{45 \times 10^6}{25 \times 200 \times 450 \times 450} =$$

$$\frac{p}{Fck} = 0.01\ p\% = 0.01 \times 25 = 0.25$$

Minimum steel = 0.8 %

$$As = \frac{0.8 \times 200 \times 450}{100} = 720 \text{ sq.mm}$$

Provided 6 Nos 16 dia = **1206 sq.mm**

Ties 8 @ 110 C/C AND 170 C/C

J. Prabakaran

J. PRABAKARAN, M.E.,
Structural Engineer
CMDA REG. No: SE/GR-I/2020/03/253
GCC REG No : RSE 101182020
No:1/4, Pidariamman Kovil Street,
Uthandi, Chennai - 600 119.
Mail : prabakaran_j@yahoo.co.in
Mobile : 9841757884

9 **FOOTING DESIGN** for **TYPE-A** refer sheet - 1 of drawing

Maximum load on column type " A P = 30 t Fck =
Add 10% footing load = 33 t
Max.Bending Moment = 0.5 t.m

S.B.C of Soil at 8 t/sqm at depth = 2 m
Assume footing base of 2.15 X 2.15 m and depth = 450 mm
So pressure on soil is Eff. Depth = 394 mm

$$\frac{33.00}{2.15 \times 2.15} = 7.1 \text{ t/sqm} < 8 \text{ t/sqm}$$

Cantilever projection of footing = $2.15 - \frac{0.23}{2} = 0.96$ m

B.M on footing at face of column pedestal

BM = $\frac{7.14 \times 0.96 \times 0.96}{2} = 3.29$ tm

Mu = 3.29 x 1.5= 4.93 tm

Mu/b.d.d = $\frac{4.93 \times 1000 \times 10000}{1000 \times 394 \times 394} = 0.32$ N/sqm

Fck = 25 N/sqmm
From Table-3 of SP-16, Pt 0.153 % min pt% .= 0.150 %

At = $\frac{0.153 \times 1000 \times 394}{100}$ = **602.8** sq.mm

FOR 2.55m = 602.8
PER METER RUN = 280.4 sq.mm

Provide 10 dia @ 125 c/c = **753** sq.mm

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Structural Engineer
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GCC REG No : RSE 101182020
No:1/4, Pidariamman Kovil Street,
Uthandi, Chennai - 600 119.
Mail : prabakaran_j@yahoo.co.in
Mobile : 9841757884



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3/350/2 Mettukuppam,
OMR Thoraipakkam,
Chennai, India

hello@verified.realestate
+91 9996 12 9996
www.verified.realestate