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CHENNAI METROPOLITAN DEVELOPMENT AUTHORITY
Thalamuthu Natarajan Building, No.1, Gandhi Irwin Road, Egmore,
Chennai - 600 008
Phone : 28414855 Fax: 91-044-28548416
E-mail: mscmda@tn.gov.in
Web site: www.cmdachennai.gov.in.

PROFESSIONAL REGISTRATION CERTIFICATE

This is to certify that the Thiru / Tmt. / M/s. **M.SHANKAR** has enrolled as **Registered Structural Engineer Grade-I(SE)** with Chennai Metropolitan Development Authority (CMDA) who has the qualification of **M.E Structures** and experience in the relevant field for 7 years.

He/She has been given the Registration No. **SE/I/2024/4/0037** on **12-Apr-2024** He/She shall be required to comply with the duties and responsibilities prescribed by the Government of Tamil Nadu in Part V, "Registration of Professionals", Tamil Nadu Combined Development and Building Rules, 2019 approved vide G.O.(ms) No.18 MAWS (MA1) Dept. Dated. 04.02.2019.

This Registration is valid upto **11-Apr-2029**.

Signature valid
Name :A Meganathan
Designation :Superintendent
Date :04/12/2024 3:58:02 PM



A Meganathan

Superintendent

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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Thalamuthu Natarajan Building, No.1, Gandhi Irwin Road, Egmore,
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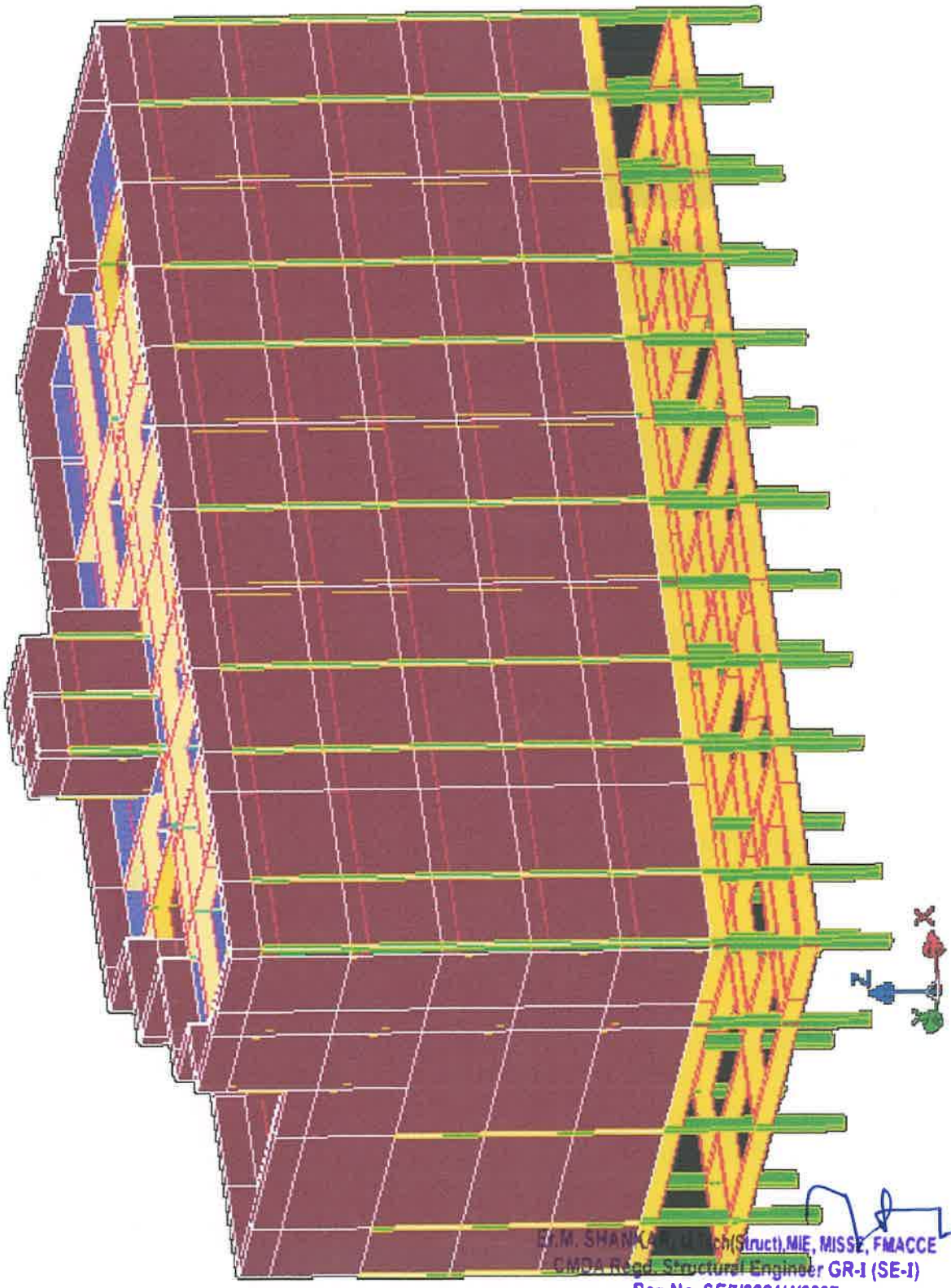
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CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar87@yahoo.com



E.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

Design Data Report :

Footing Data:

Soil Parameters :

SBC of soil	= Bored pile foundation 22M depth
SBC Increment Factor for Seismic Load	= 1.500
SBC Increment Factor for Wind Load	= 1.500
Angle of friction,(phi)	= 30.00 degrees
Cohesion	= 60.00
Permissible factor of safty	= 1.400
Depth From Ground	= 2000 mm

Material Properties :

fck	= 25.00 N/mm²
fy	= 500.00 N/mm²

Footing Parameters :

Minimum steel Percentage	= 0.12
Minimum spacing between bars	= 100 mm
Maximum spacing between bars	= 200 mm
Diameters of selected bars	= 10 mm , 12 mm .
Bottom Cover	= 50 mm
End Cover	= 50 mm
Minimum Footing Depth	= 150 mm
Top Offset	= 50 mm
Bottom PCCOffset	= 150 mm
Bottom PCCDepth	= 150 mm
Pedestal Offset	= 150 mm
Pedestal Depth	= 150 mm
Size RoundingFactor	= 25 mm
Depth RoundingFactor	= 25 mm
Number of Steps	= 1
Minimum Step Depth	= 150 mm

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Column Data:

Material Properties :

fck	= 30.00 N/mm²
fy	= 500.00 N/mm²
fy - Stirrups	= 500.00 N/mm²

Column Parameters :

Effective Length Factor (X - Axis)	= 1.00
Effective Length Factor (Y - Axis)	= 1.00
Minimum Eccentricity @ X-Axis	= 20 mm
Minimum Eccentricity @ Y-Axis	= 20 mm
Clear Cover	= 40 mm

Design Considerations :

Design is done as per IS 456.

Detailing is done as per IS 13920.

Confinement reinforcement is provided.

Live Load reduction is considered.

Minimum steel percentage is based on provided section.

Reinforcement Details :

Steel Ratio(AstX/AstY)	= 1.00
ReinForcement Placing (on two sides or on four sides)	= 2
Minimum steel Percentage	= 0.80
Maximum steel Percentage	= 4.00
Minimum spacing between main bars	= 25 mm
Maximum spacing between main bars	= 300 mm
Minimum spacing between Stirrups	= 100 mm
Diameters of selected bars	= 12 mm , 16 mm , 20 mm , 25 mm , 32 mm .

Slab Data: Use Bent-up bars for steel detailing.

Material Properties :

Er.M. SHANKAR, M.Tech(Struct.)ME, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
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No.11, Theradi Street, Pallikaranai, Chennai-600 100.
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RCC Grade	= M25
Main Steel Grade	= Fe500
Distribution Steel Grade	= Fe500

Slab Parameters :

Effective Cover	= 25 mm
Minimum spacing between Main bars	= 75 mm
Maximum spacing between Main bars	= 200 mm
Minimum spacing between Dist.bars	= 75 mm
Maximum spacing between Dist.bars	= 300 mm
Maximum Main/Dist. bar Diameter	= 32 mm
Minimum Main bar Diameter	= 8 mm
Minimum Dist.bar Diameter	= 8 mm
Maximum Dimension to be used for Trapizoidal Slab	= Average of two parellel sides
Maximum Dimension to be used for General Slab	= Maximum of two parellel sides

---

Beam Data:

Shear Design Data :

Minimun Stirrup Dia	= 8 mm
Stirrup Legs	= 2
Minimum Stirrups Spacing	= 100 mm
Stirrups Steel Grade Name	= Fe500

Effect of Concrete in shear considered

Beam Covers :

Tension Cover	= 33 mm
Compression Cover	= 33 mm
Side Cover	= 25 mm

Beam sections are designed as Doubly Reinforced Section

Main Bars Detailing Straight + Curtailed

Detailing provision As per IS 13920 is followed

End Moments are calcultaed at c/c distance of the beam span for design

Torsion Moments are considered in beam design

Anchor bar Dia = 8 mm

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
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No.11, Theradi Street, Pallikaranai, Chennai-600 100.
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Email: structshankar97@yahoo.com

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Main Steel Grade	= Fe500
No. of division of the beam	= 10
Max. Aggregate size	= 20

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CMDA Regd. Structural Engineer GR-I (SE-I)
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No.11, Theradi Street, Pallikaranai, Chennai-600 100.
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Annexure – XIV

[See rule 51 (3)]

Form 1

Structural Design Basis Report

(1) This report to accompany the application for Building Developments

Part1	**General Data**		
S.No.	**Description**	**Information**	**Notes**
1.	Site Address		
2	Name of Owner		
3	Name of Registered Developer along with the Registration Number		
4	Name of Registered Architect/Engineer along with the Registration Number		
5	Name of Registered Structural engineer along with the Registration Number	**SE/GR-I/2024/4/037**	
6	Use of the building	**RESIDENTIAL**	
7	Number of stories above ground level (including storeys to be added later, if any)	**STILT+4FLOORS+5 FLOOR (PART)**	
8	Number of basements below ground level	**NIL**	
9	Type of structure Load bearing walls R.C.C. frame R.C.C. frame and Shear Walls Steel frame	**RC FRAME**	
10	Soil data Type of soil Design safe bearing capacity	**BORED PILE FOUNDATION 22 M DEPTH AS PER SOIL**	IS:1893 C1.6.3.5.2 IS: 1904 IS: 1892 IS: 2131 IS: 2720

Er.M. SHANKAR, M.Tech(Struct),ME, MISSE, FMACCE
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		REPORT	
11	Dead loads (unit weight adopted) • Earth: **18KN/CUM** • Water: **10KN/CUM** • Brick masonry:**19.2KN/CUM** • Plain cement concrete **24KN/CUM** • Reinforced cement concrete:**25 KN/CUM** • Floor finish **50KG/SQM** • Other fill materials		IS: 875 Part 1
12	Imposed (live) loads Floor loads Roof loads	**4KN/SQM** **1.5KN/SQM**	IS: 875 Part 2

13	Cyclone/Wind • Speed • Design pressure intensity	**N/A**	IS: 875 Part 3
14	Seismic zone	**III**	IS: 1893 (2002)
15	Importance factor	**1**	IS:1893(2002) Table 6
16	Seismic zone factor (Z)	**0.16**	IS:1893 Table 2
17	Response reduction factor		IS:1893 Table 7
18	Fundamental natural period – approx.	**0.29SEC**	IS:1893 C1.7.6
19	Design horizontal acceleration spectrum value (A_h)		IS: 1893 C1.6.4.2
20	♥ Expansion/Separation Joints	**N/A**	
21	Building is regular/irregular	**REGULAR**	IS 1893

♥ Enclose detailed drawings drawn to scale for each floor

♦ In case terrace garden is provided, indicate additional fill load and live load along with the detailed drawings drawn to scale

Part 2	**Load bearing masonry buildings**		
Sl.No.	**Description**	**Information**	**Notes**
1	Building category	**N/A**	IS: 4326 C1.7 Read with IS:1893

Er.M. SHANKAR, M.Tech(Struct),MIE,MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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			Zone II III IV V Bldg. Ordinary B C D E Important C D E E
2	Basement Provided	**N/A**	
3	Number of floors including Ground Floor (all floors including stepped floors in hill slopes)	**N/A**	
4	Type of wall masonry	**N/A**	
5	Type and mix of Mortar	**N/A**	IS: 4326 C1.8.1.2
6	Re: size and position of openings (See note No.[i]) • Minimum distance (b5) • Ratio $(b_1+b_2+b_3)/l_1$ or $(b_6+b_7)/l_2$ • Minimum pier width between consequent opening (b_4) • Vertical distance (h_3) • Ratio of wall height to thickness 4 Ratio of wall length between cross wall to thickness	**N/A**	IS: 4326 Table 4, Fig.7
7	Horizontal seismic band • at plinth level **N/A** • at window sill level **N/A** • at lintel level **N/A** • at ceiling level **N/A**	P IP NA } } } } } } } } } } } }	(see note No.2) IS:4326 C1 8.4.6 IS: 4326 C1 8.3 IS: 4326 C1 8.4.2 IS: 4326 C1 8.4.3 IS: 4326 C1 8.4.3
	• at eave level of sloping roof **N/A** • at top of gable walls **N/A** • at top of ridge walls **N/A**	} } } } } } } } }	IS: 4326 C1 8.4.3 IS: 4326 C1 8.4.4
8	Vertical reinforcing bar **N/A** at corners and T junction of walls at jambs of doors and window openings **N/A**	} } } } } }	IS: 4326 C1 8.4.8. IS: 4326 C1 8.4.9

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9	Integration of prefab roofing/flooring elements through reinforced concrete screed. **N/A**	} } }	IS:4326 C1 9.1.4
10	Horizontal bracings in pitched truss in horizontal plane at the level of ties in the slopes of pitched roofs **N/A**	} } } } } }	

Notes: (i) Information in Item 6 should be given on separate A4 sheets for all walls with large number of openings

(ii) IP indicated "Information Provided"

IP indicates "Information to be provided" NA indicates "Not Applicable"

Tick mark one box

Part 3	**Reinforced concrete framed buildings**		
Sl. No.	**Description**	**Information**	**Notes**
1	Type of building • Regular frames • Regular frames with Shear Walls • Irregular frames • Irregular frames with Shear Walls • Soft storey	**REGULAR FRAMES**	IS: 1893 C1 7.1
2	Number of basements	**NIL**	
3	Number of floors including ground floor	**S+4+5 TH FLOOR (PART)**	
4	Horizontal floor system • Beams and slabs • Waffles • Ribbed floor • Flat slab with drops • Flat plate without drops	**BEAM AND SLAB**	

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Reg No. SE/I/2024/4/0037
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No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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5	Soil data • Type of soil • Recommended type of foundation -Independent footings -Raft -Piles • Recommended bearing capacity of soil	**CLAYELY SOIL WITH BORED PILE FOUNDATIO N**	IS: 1498

	• Recommended type, length, diameter and load capacity of piles • Depth of water table • Chemical analysis of ground water • Chemical analysis of soil		
6	Foundations Depth below ground level Type Independent Interconnected Raft Piles	**BORED PILE FOUNDATIO N**	
7	System of interconnecting foundations Plinth beams Foundation beams	**PLINTH BEAM**	IS: 1893 C1 7.12.1
8	Grades if concrete used in different parts of building	**M30 FOR COLUMNS& M25 FOR REST**	
9	Method of analysis used	**FEM**	
10	Computer software used	**STRUDS V12**	
11	Torsion included	**INCLUDED**	IS: 1893 C1 7.9

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CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
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12	Base shear a. Based on approximate fundamental period b. Based on dynamic analysis c. Ratio of a/b	**BASED ON APPROXIMATE FUNDAMENTAL PERIOD**	IS: 1893 Cl. 7.5.3
13	Distribution of seismic forces along the height of the building.	**REFER SESIMIC JUSTIFICATION**	IS:1893 Cl.7.7 (Provide sketch)
14	The Column of soft ground storey specially designed.	**YES**	IS: 1893 Cl.7.10
15	Clear minimum cover provided in • **Footing 50MM** • **Column 40MM** • **Beams 25MM** • **Slabs 20MM** • **Walls**		IS: 456 Cl. 26.4
16	Ductile detailing of RC frame • Type of reinforcement used **Fe 500D** • Minimum dimension of beams **230MM** • Minimum dimension of columns **230MM** • Minimum percentage of reinforcement of beams at any cross section **0.20%** • Maximum percentage of reinforcement at any section of beam **4%** • Spacing of transverse reinforcement in 2-d length of beams near the ends **100MM** • Ratio of capacity of beams in shear to capacity of beams in flexure **<1** • Maximum percentage of reinforcement in column **4%**		IS: 456 Cl. 5.6 IS: 13920 Cl.6.1 IS: 13920 Cl.7.1.2 IS: 456 Cl.26.5.1(a) IS: 13920 Cl.6.2.1 IS: 456 Cl.26.5.1.1(b) IS: 13920 Cl.6.2.2 IS: 13920 Cl.6.3.5 IS: 456 Cl.26.5.3.1 IS: 13920 Cl. 7.4

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
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No.11, Theradi Street, Pallikaranai, Chennai-600 100.
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	• Confining stirrups near ends of columns and in beam-column joints Diameter **8mm** Spacing **100MM** • Ratio of shear capacity of columns to maximum seismic shear in the storey. **<1**		
17	Does the features require clearance by SDRP Example: High rise building Prefab building Building in hazard prone areas	**N/A**	

Foundation

i. In case raft foundation has been adopted, indicate K value used for analysis ofthe raft.

ii. Incase pile foundations have been used, give full particulars of the piles, type, dia, length, capacity

iii. In case of high water table, indicate system of countering water pressure, and indicate the existing water table, and that assumed to design foundations.

Idealization for Earthquake analysis

i. In case of composite system of shear walls and rigid frames, give distribution of base shear in the two systems on the basis of analysis, and that used for design of each system

ii. Indicate the idealization of frames and shear walls adopted in the analysis with the help of sketches.

iii. Submit framing plans of each floor

iv. In case of basements, indicate the system used to contain earth pressures.

Part 4	**Buildings in structural steel**		
1	Adopted method of design	o Simple o Semi-rigid o Rigid	IS:800 Cl 3.4.4 IS:800 Cl.3.4.5 IS:800 Cl.3.5.6

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2	Design based on	o Elastic analysis o Plastic analysis	IS: 800 Section-9 SP;6 (6)
3	Floor construction	o Composite o Non composite o Boarded	
4	Roof construction	o Composite o Non composite o Metal o Any other	
5	Horizontal force resisting system adopted	Frames Braced frames Frames & shear walls	Note: Seismic force As per IS:1893 Would depend on system
6	Slenderness ratios maintained	Members defined in Table 3.1, IS:800	IS 800; Cl.3.7
7	Member deflection	Beams, Rafters	IS:800 Cl.3.13
	limited to	Crane Girders, Purlins Top of columns	
8	Structural members	o Encased in concrete Not encased	IS: 800 Section-10
9	Proposed material	o General weld-able o High strength o Cold formed o Tubular	IS: 2062 IS: 8500 IS: 801, 811 IS: 806
10.	Minimum metal thickness Specified for corrosion protection	o Hot rolled sections o Cold formed sections o Tubes	IS: 800, Cl.3.8 Cl.3.8.1 to Cl.3.8.4 Cl.3.8.5
11	Structural connections	o Rivets o CT Bolts o S H F G Bolts o Black Bolts o Welding field Shop (Specify welding type proposed) Composite	IS: 800, Section-8 IS: 1929, 2155, 1149 IS 6639, 1367 IS 3757, 4000 IS: 1363, 1367 IS: 816, 814,1395, 7280, 3613, 6419, 6560, 813, 9595

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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12	Minimum Fire rating proposed, with method	o Rating..... hours o Method proposed. -In tumescent painting -Spraying -Quilting -Fire retardant boarding	IS: 1641, 1642, 1643

Er.M. SHANKAR, M.Tech.(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STRUCTURAL ENGINEER

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Foundation Design Detail Report :

(Design by Limit State Method as per IS 456 : 2000)

Standard Conventions:

Design of FPG1

Size of col./ped, (l x b) = 0.230 x 0.450 m

f_{ck} = 25000.00 kN/m²
f_y = 500000.00 kN/m²
σ_{cc} = 6000.00 kN/m²
σ_{cbc} = 8500.00 kN/m²
σ_{sc} = 230000.00 kN/m²

Soil Parameters :

Sr.No.	Soil Type	Layer Name	Height (m)	N	C (kN/m²)	ϕ°	γ (kN/m³)	σ (kN/m²)
1	Cohesive	Layer1	3.00	6	20.00	10.00	14.00	10.00
2	Cohesive	Layer2	11.00	6	150.00	30.00	16.00	75.00
3	Cohesive	Layer3	8.00	6	20.00	10.00	14.00	10.00

Pile Parameters :

Minimum diameter = 0.50 m
Maximum diameter = 0.90 m
Minimum Length = 22.00 m
Maximum Length = 22.00 m
Spacing = 1.25 m
Max. No of Pile(s) per group = 25
Load combination = 1.50DL + 1.50EQL Y-

Loads On Footing :

Factored Load
Axial load ,(Pu) = 566.52 kN (From analysis results)
BMu @ x-x,(M_x) = 74.38 kN-m
BMu @ y-y,(M_y) = 0.50 kN-m

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
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Horizontal Force,HFu_x	= 0.20 kN
Horizontal Force,HFu_y	= 41.05 kN
Working Load	
Axial load ,(P)	= 377.68 kN (From analysis results)
BM @ x-x,(M_x)	= 49.59 kN-m
BM @ y-y,(M_y)	= 0.33 kN-m
Horizontal Force,HF_x	= 0.13 kN
Horizontal Force,HF_y	= 27.37 kN
Self Weight of pile cap assumed	= 10.00 % of P = 37.77 kN
Total load,(P)	= 415.45 kN
Uplift	= 0.00 kN
Moment due to Shear Force in X-direction	= SF_x x Pile Cap Depth (from Reynolds minimum depth equation, Ref. Reynolds Handbook)
	= 0.13 x 1.100
	= 0.14 kN-m
Moment due to Shear Force in Y-direction	= SF_y x Pile Cap Depth
	= 27.37 x 1.100
	= 30.10 kN-m
Total BM @ x-x,(M_x)	= 49.73 kN-m
Total BM @ y-y,(M_y)	= 30.44 kN-m

Provide 2 - 0.500 m dia. piles of length 22.000 m.
Provide 16 dia. 6 nos. in each pile.

Calculation Of Axial Load On A Pile :

Axial load on a pile = Direct axial load + additional loads due to moments
+ self weight of pile

Direct axial load,(F) = P/(no.of piles)
= 415.45 / 2 = 207.73 kN

For X-axis :

Additional load due to moments, (FaX) = km x M_x

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
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SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
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Calculation of factor km :
Reference point = center of first pile row,
a = distance of a pile row from reference point,
x = distance of a pile row from CG of pile group,
n = no. of piles in a pile row from reference point.

Pile row	a (m)	n	a.n (m)	x (m)	$n.x^2$
1	0.000	2	0.000	0.000	0.000
		2	0.000		0.000

Distance of CG from reference point,(xBar) = Σ(a.n)/(no.of piles)=0.000/2=0.000 m

I = $\Sigma(n.x^2)$ = 0.000

Maximum distance from CG of any pile row,(xMax) = 0.000 m

km = 0.000

Additional load due to moments, (FaX) = km x M_x = 0.000 x 49.73 = 0.00 kN

For Y-axis :

Additional load due to moments, (FaY) = km x M_y

Calculation of factor km :
Reference point = center of first pile row,
a = distance of a pile row from reference point,
x = distance of a pile row from CG of pile group,
n = no. of piles in a pile row from reference point.

Pile row	a (m)	n	a.n (m)	x (m)	$n.x^2$
1	0.000	1	0.000	-0.750	0.563
2	1.500	1	1.500	0.750	0.563
		2	1.500		1.125

Distance of CG from reference point,(xBar) = Σ(a.n)/(no.of piles)=1.500/2=0.750 m

I = $\Sigma(n.x^2)$ = 1.125

Maximum distance from CG of any pile row,(xMax) = 0.750 m

km = xMax /I = 0.750/1.125 = 0.667

Additional load due to moments, (FaY) = km x M_y = 0.667 x 30.44 = 20.29

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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kN

Self weight of pile,(Fsw) = $\pi/4$ x (PileDia)^2 x pile length x density
(Fsw) = $\pi/4$ x 0.500^2 x 22.000 x 25.000
Self weight of pile,(Fsw) = 107.99 kN

Axial load on a pile,(Fa) = F + FaX + FaY + Fsw = 207.73 + 0.00 + 20.29+ 107.99 = 336.01 kN

Calculation of a Pile Capacity

User Defined Pile Capacity = 800.00 kN

Factor of Safety = 1.50
Safe Pile Capacity(Q_u) = 800.00 / 1.50 = 533.33 kN > 336.01 kN. Hence SAFE

Calculation Of Pile Spacing

Min Pile Spacing as per IS:2911 = 1.500 m
Pile Spacing Provided = 1.500 m

Design Of Pile Cap :

Calculation Of Pile Cap Size :
Pile cap overhang = 0.150 m
Length = 0.150 + 0.500 + 0.150 = 0.800 m
Width = 0.150 + 2.000 + 0.150 = 2.300 m
Provide pile cap of size,(L x B) 0.800 m x 2.300 m

Bending Moment Calculations :

As per IS 456: 2000 /Clause 33.2.3.2,critical section for checking bending moment
in the design of an isolated concrete footing which supports a column shall be a section located at the face of the column or pedestal.

As per IS 2911: (Part III)1980 /Clause 5.4.2,the reaction of any pile whose centre is
located D/2 or more outside the section shall be assumed as producing shear on the section;
the reaction of any pile whose centre is located D/2 or more inside the section shall

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

be
assumed as producing no shear on the section.

For X-axis :
x-dist and y-dist are the distances of pile centres from column centres.

Pile no.	x-dist (m)	y-dist (m)	Distance from column face,a (m)	Location w.r.t. critical section	Pile reaction,R (kN)	BM (kN-m)
1	0.000	-0.750	-0.115	Inside (a <= 0.250)	90.13	0.045 x 90.13 = 4.06
2	0.000	0.750	-0.115	Inside (a <= 0.250)	90.13	0.045 x 90.13 = 4.06
					Total	8.11

Design BM_x,(M_{ux}) = 8.11 kN-m= 8.11 kN-m

For Y-axis :
x-dist and y-dist are the distances of pile centres from column centres.

Pile no.	x-dist (m)	y-dist (m)	Distance from column face,a (m)	Location w.r.t. critical section	Pile reaction,R (kN)	BM (kN-m)
2	0.000	0.750	0.525	Outside (a >= 0.250)	333.82	0.525 x 333.82 = 175.25
					Total	175.25

Design BM_y,(M_{uy}) = 175.25 kN-m= 175.25 kN-m

Depth Calculations :
Depth required for punching shear:
As per IS : 456 - 2000 - Clause 31.6.3.1,the calculated shear stress at the critical section shall not exceed ks x TauC

where,
ks = 0.5 + β_c, but not greater than 1,β_c being the ratio of short side to the long side of column, and
TauC = 0.25 x √(fck)in Limit state method of design.

Depth required for punching shear is calculated by equating the actual stress to permissible stress and by solving the resulting equation.

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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Minimum depth calculation :

(Ref.Reynolds Handbook.)

Minimum depth	= (2.00 x pile size.) + 0.100 = (2.00 x 0.500) + 0.100
	= 1.100 m
User defined minimum pile cap Depth,	= 0.800 m
Depth assumed,(D)	= 0.800 m
Bottom cover	= 0.075 m
Effective depth,(d_{effX})	= 0.719 m
Effective depth,(d_{effY})	= 0.707 m
Effective depth,(d_{eff})	= 0.719 m
Resisting area,(A_r)	= d_{eff} x 2 x (b + d_{eff}/2 + 1 + d_{eff})/2 = 0.719 x 2 x (0.230 + 0.450)
	= 2.012 m²
Shear force,(V)	= 566.52 kN
Actual shear stress	= V / A_r = 566.52 / 2.012 = 281.61 kN/m²
Maximum permissible shear stress	= 0.25 x √(fck) x ks
ks	= 0.5 + β_c
β_c	= short column side/ long column side
	= 0.230 / 0.450 = 0.511
ks	= 0.5 + 0.511 = 1.011 > 1 , ks = 1
Maximum permissible shear stress	= 0.25 x 5.0000 x 1.000
	= 1.250 N/mm² = 1250.00 kN/m²
Actual shear stress	< Maximum permissible shear stress.
Hence,Safe in punching shear.	

Calculation Of Moment Of Resistence :

Total depth,(D) = 0.800 m For (Fe500),k = 0.456

R	= 0.36 x fck x k x (1 - 0.42 x k)
	= 0.36 x 25000.000 x 0.456 x (1 - 0.42 x 0.456)
	= 3318.505
Moment of resistanceX	= R x width x d_{effX} x d_{effX}
	= 3318.505 x 2.300 x 0.719 x 0.719
	= 3945.74 kN-m> 8.11 kN-m

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050543
Email: structshankar97@yahoo.com

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Moment of resistanceY = R x width x d_{effY} x d_{effY}
= 3318.505 x 0.800 x 0.707 x 0.707
= 1327.00 kN-m> 175.25 kN-m

Steel Calculations :

For M_{ux} :

Cross sectional area,(csArea) = width at NA x d_{effX} = 2.300 x 0.719 = 1.654 m²
val = 1 - [(4.6 x MuX)/(fck x B x deffX²)]
Required steel area,(AstX) = [0.5(fck/fy) x (1 - sqrt(val))] x csArea
= 26 mm²
Required steel pt. = 0.00157 % < minimum steel pt.(min-pt = 0.120 %)
Required steel area,(AstX) = min-pt x csArea/100 = 0.120 x 1.654/100
= 1984 mm²

Provide 12 Tor 19 nos.(2149 mm²)
(considering flexural and shear check requirements)

Spacing = (B - dia. - (2 x end cover))/(no - 1) = (2.300 - 0.012 - (2 x 0.050))/(19 - 1)
= 0.122 m < maximum spacing(0.200 m)

For M_{uy} :

Cross sectional area,(csArea) = width at NA x d_{effY} = 0.800 x 0.707 = 0.566 m²
val = 1 - [(4.6 x MuY)/(fck x B x deffY²)]
Required steel area,(AstY) = [0.5 (fck/fy) x (1 - sqrt(val))] x csArea
= 582 mm²
Required steel pt. = 0.10292 % < minimum steel pt.(min-pt = 0.120 %)
Required steel area,(AstY) = min-pt x csArea/100 = 0.120 x 0.566/100
= 679 mm²

Provide 12 Tor 21 nos.(2375 mm²)
(considering flexural and shear check requirements)

Spacing = (L - dia. - (2 x end cover))/(no - 1) = (0.800 - 0.012 - (2 x 0.050))/(21 - 1)
= 0.034 m < maximum spacing(0.200 m)

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
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SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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Check For One Way Shear :

As per IS:2911(Part III)-1980 /Clause 5.4.2,the critical section for this condition shall be assumed as a vertical section located at a distance equal to the half the
depth of pile cap from the face of the column or pedestal.

For X-axis :

Projection-X,(proj) = (L - l)/2 - (d_{effX}/2) = (0.800 - 0.230)/2 - (0.719/2) = -0.074 m

As the critical section for one way shear falls outside
the footing,oneway shear check is not required to be taken in this direction.

For Y-axis :

Projection-Y,(proj) = (B - b)/2 - (d_{effY}/2) = (2.300 - 0.450)/2 - (0.707/2) = 0.571 m

x-dist and y-dist are the distances of pile centres from column centres.

Distance of critical section from the face of column = 0.400 m

Pile no.	x-dist (m)	y-dist (m)	Distance from column face (m)	Distance from critical section,a (m)	Location w.r.t. critical section	SF (kN)
2	0.000	0.750	0.525	0.125	Outside (a < 0.250)	250.36
					Total	250.36

Shear force,(Vy) = 250.36 kN

Resisting area,(A_r) = d_{effY} x L = 0.707 x 0.800 = 0.566 m²

Actual shear stress = Vy / A_r = 250.36 / 0.566

= 440.78 kN/m² < permissible shear stress(454.11 kN/m²)

Summary :

Provide 2 - 0.500 m dia. piles
of length 22.000 m.
Provide 16 dia. 6 nos. in each pile.
Provide pile cap of size,(L x B x D) :
0.800 x 2.300 x 0.800 m
Provide 12 dia. 19 nos along L
Provide 12 dia. 21 nos along B.

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
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COLUMN DESIGN CALCULATIONS

As per IS 13920:1993 Clause 7.1.2 ,
The minimum dimension of column shall not be less than 200 mm.
For the columns with unsupported length exceeding 4 m ,
the shortest dimension of the column shall not be less than 300 mm.
As per IS 13920:1993 Clause 7.1.3 of IS 13920:1993,
The ratio of the sohrtest cross sectional dimensions to the perpendicular dimension shall preferably not be less than 0.4.

Terms Used In Calculation :

In biaxial column design,

$P_u = P_{uc} + P_{us(Total)}$

where,

P_u = external axial compressive load,

P_{uc} = axial compressive resistance offered by concrete,

$P_{us(Total)}$ = total axial compressive resistance offered by steel at different levels in the section.

$M_u = M_{uc} + M_{us(Total)}$

where,

M_u = external moment about centroidal axis,

M_{uc} = moment of resistance offered by concrete in compression,

$M_{us(Total)}$ = total moment of resistance offered by steel at different levels in the section.

i = serial no. of row of reinforcement,

A_{si} = cross-sectional area of reinforcement in the i th row,

f_{si} = stress in the reinforcement in the i th row (compression + ve, tension - ve),

f_{ci} = compressive stress in concrete at the level of i th row of reinforcement,

e_i = strain at the i th row of reinforcement from the stress-strain curve of steel and concrete,

x_i = distance of the bars in the i th row from the centroid of the section

Values of stress in steel :

Design strength in bending compression (f_{yd}) = 0.87 f_y

(a) Fe 250,

For $e_i >= f_{yd} / E_s$, $f_{si} = f_{yd}$

For $e_i < f_{yd} / E_s$, $f_{si} = e_i \times f_{yd}$

(b) Fe 415,

For $e_i >= 0.8 \times f_{yd} / E_s$, f_{si} = value obtained from stress-strain curve

For $e_i < 0.8 \times f_{yd} / E_s$, $f_{si} = e_i \times f_{yd}$

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Stress	Stress level (N/mmB)	Strain
0.800 f_{yd}	288.7	0.00144
0.850 f_{yd}	306.7	0.00163
0.900 f_{yd}	324.8	0.00192
0.950 f_{yd}	342.8	0.00241
0.975 f_{yd}	351.8	0.00276
1.000 f_{yd}	360.9	0.00380

Values of stress in concrete :

For $e_i >= 0.002$, $f_{ci} = 0.446\ f_{ck}$

For $e_i < 0.002$, $f_{ci} = (446.e_i \times (1 - 250.e_i)) \times f_{ck}$

Values of strain :

Values of strain at different levels are obtained by taking maximum strain as 0.0035 as the reference value at the highly compressed edge.

Transverse Reinforcement:

As per IS 13920 : 1993,the design shear force for columns shall be the maximum of

i)calculated factored shear force as per analysis, and

ii)a factored shear force given by

Vu = 1.4 x (MubL,lim + MubR,lim)/storey height

where MubL,lim,MubR,lim are moments of resistance,of oppsite sign framing into the column from oppsite faces (to be calculated as per IS 456 : 1978)

Design of CG1 :

General Design Parameters :

Below FIV.FLOOR - Default Level at 21.000 m

Column size, (B x D) = 230 x 450 mm Column height,(L) = 2400 mm

From analysis results,loads on column

Axial load,(P) = 39.21 kN

MomentX,(M_x) = 15.42 kN-m

MomentY,(M_y) = 13.36 kN-m

f_{ck} = 30.00 N/mmB

f_y = 500.00 N/mmB

Load combination = 1.50 DL + 1.50 LL

Column Name	Orientation Angle
C1	0.00

Er.M. SHANKAR, M.Tech(Struct),ME, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth
= (2400 x 1.000) / 450 = 5.33 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width
= (2400 x 1.000) / 230 = 10.43 <= 12.00,

column is not slender in this direction.

$M_{x_MinEccen}$ = 0.784 kN.m
$M_{y_MinEccen}$ = 0.784 kN.m
$M_x = \max(M_x, M_{x_MinEccen}) + M_{uaddX}$ = 15.415 kN.m
$M_y = \max(M_y, M_{y_MinEccen}) + M_{uaddY}$ = 13.365 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity
equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 15.42/39.21 = 393 mm
Actual $eccen_Y$ = M_y / P = 13.36/39.21 = 341 mm
$eccen_{XMin}$ = (L/500) + (D/30) = (2400/500) + (450/30) = 20 mm
$eccen_{XMin}$ = max(20,20)= 20 mm
$eccen_{YMin}$ = (L/500) + (B/30) = (2400/500) + (230/30) = 12 mm
$eccen_{YMin}$ = max(12,20) = 20 mm
$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(393,20) = 393 mm
$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(341,20) = 341 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension, the column will be designed as an axially loaded column.
eccenX(393 mm) > 0.05 x 230(12 mm)
and eccenY(341 mm) > 0.05 x 450(23 mm)
Hence the column will be designed as a column with biaxial moments.
Provided steel :
Provide #16 - 6 nos. (1206 mmB)

Biaxial Check Calculations :

For X axis :

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00183	357.58	13.29	344.29	138.45	177	24.51
2	402	-0.00431	-434.80	0.00	-434.80	-174.84	0	0.00
3	402	-0.01045	-434.80	0.00	-434.80	-174.84	-177	30.95
					Total	-211.24		55.45

x_{ux} = 101 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 101/450 = 0.081

P_{uc} = 0.081 x 30.00 x 230 x 450

= 250.52 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$

= 250.52 + (-211.24)

= 39.28 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.D - C_2.D)$

C_2 = 0.416 k_u = 0.416 x 101/450

= 0.093

M_{uc} = 250.52 x (0.5 x 450 - 0.093 x 450) = 45.86 kN-m

$M_{ux1} = M_{uc} + M_{us(Total)}$

= 45.86 + (55.45)

= 101.31 kN-m

For Y axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	603	0.00036	72.78	4.43	68.35	41.23	67	2.76
2	603	-0.00839	-434.80	0.00	-434.80	-262.27	-67	17.57
					Total	-221.04		20.33

x_{uy} = 54 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 54/230 = 0.084

P_{uc} = 0.084 x 30.00 x 450 x 230

= 260.35 kN

$P_{uy1} = P_{uc} + P_{us(Total)}$

= 260.35 + (-221.04)

= 39.31 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.B - C_2.B)$

C_2 = 0.416 k_u = 0.416 x 54/230

= 0.097

M_{uc} = 260.35 x (0.5 x 230 - 0.097 x 230) = 24.14 kN-m

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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$M_{uy1} = M_{uc} + M_{us(Total)}$

$= 24.14 + (20.33)$

$= 44.47$ kN-m

$P_u/P_{uz} = 0.021$

For $P_u/P_{uz} <= 0.2$, $\alpha_n = 1.0$

For $P_u/P_{uz} >= 0.8$, $\alpha_n = 2.0$

For $P_u/P_{uz} = 0.021$, $\alpha_n = 1.000$

$((M_{ux} / M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n})$

$= ((15.42/101.31)^{1.000}) + ((13.36/44.47)^{1.000})$

$= 0.453 < 1.0$,hence O.K.

Ties Details :

Load combination for ties design = 1.20 DL + 1.20 LL + 1.20 EQL X+

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c),the diameter of the polygonal links or lateral ties shall be not less than the following

i)one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

ii)in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 16 mm diameter for ties.

(considering the requirement for shear)

Calculation of spacing of ties :

Calculation of τ_{auc} (as per IS 456: 2000 Clause 40.2.2)

τ_{auc} = 0.695 MPa for 1.17% steel

$\Delta = 1 + (3.P_u/A_g.f_{ck>})$ but not exceeding 1.5 = 1.03

$\tau_{auc}=$

$\tau_{auc} \times \Delta$ = 0.718 MPa

For X - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

Design shear force,(V_{ud})	= 6.50 kN(from analysis)
V_{ud}	= 6.50 kN
Capacity of concrete,(V_{uc})	= τauc x Area = 74.27 kN
V_{us}	= $V_{ud} - V_{uc}$ = -67.77 kN
Stirrup legs assumed	= 4
V_{umin}	= $0.87 \times f_{ys} \times A_{sv} \times d / s_{vmin}$
	= 0.87 x 500 x 804 x 182 /115 = 553672.12 N

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

= 553.67 kN

$V_{us} < V_{usmin}$

Spacing from min.shear R/F requirement = $(0.87 \times f_{ys} \times a_{sv})/(0.4 \times \text{width})$

= ((0.87 x 500.000 x 804.248)/(0.4 x 450.000))x (415.0/500.0f)

= 1613 mm

Required spacing = min(1613,115) = 115 mm

For Y - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

Design shear force,(V_{ud}) = 8.78 kN(from analysis)

V_{ud} = 8.78 kN

Capacity of concrete,(V_{uc}) = τauc x Area = 74.27 kN

V_{us} = V_{ud} - V_{uc} = -65.49 kN

Stirrup legs assumed = 2

V_{umin} = $0.87 \times f_{ys} \times A_{sv} \times d / s_{vmin}$

= 0.87 x 500 x 402 x 402 /115 = 611473.08 N

= 611.47 kN

$V_{us} < V_{usmin}$

Spacing from min.shear R/F requirement = $(0.87 \times f_{ys} \times a_{sv})/(0.4 \times \text{width})$

= ((0.87 x 500.000 x 402.124)/(0.4 x 230.000))x (415.0/500.0f)

= 1578 mm

Required spacing = min(1578,115) = 115 mm

Axial Stess Check :

Actual Axial Stress = 0.325 N/mm2

Allowable Axial Stress = 0.1 x fck = 3.000 N/mm2

Actual Axial Stress is less than Allowable Axial Stress,

Hence, No need to provide Ductile detailing as per 7.1.1 clause,IS 13920

Provide ties #16 @ 110 mm c/c

SUMMARY :

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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CG1 : FIV.FLOOR - Default Level at 21.000 m
Provide rectangular section : 230 x 450 mm
Provide #16 - 6 nos. (1206 mmB)
Provide ties #16 @ 110 mm c/c
Provide 4 legged ties along width and
2 legged ties along depth.

General Design Parameters :

Below FO. FLOOR - Default Level at 18.000 m

Column size, (B x D) = 230 x 450 mm Column height,(L) = 2400 mm

From analysis results,loads on column

Axial load,(P) = 102.85 kN

MomentX,(M_x) = 19.35 kN-m

MomentY,(M_y) = 15.47 kN-m

f_{ck} = 30.00 N/mmB

f_y = 500.00 N/mmB

Load combination = 1.20 DL + 1.20 LL + 1.20 EQL X-

Column Name	Orientation Angle
C1	0.00

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth
= (2400 x 1.000) / 450 = 5.33 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width
= (2400 x 1.000) / 230 = 10.43 <= 12.00,

column is not slender in this direction.

$M_{x_MinEccen}$ = 2.057 kN.m

$M_{y_MinEccen}$ = 2.057 kN.m

$M_x = \max(M_x, M_{x_MinEccen}) + M_{uaddX}$ = 19.346 kN.m

$M_y = \max(M_y, M_{y_MinEccen}) + M_{uaddY}$ = 15.466 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity

Er.M. SHANKAR, M.Tech(Struct),MIE,MISSE,FMACCE
CMDA Regd. Structural Engineer GR-1 (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 19.35/102.85 = 188 mm

Actual $eccen_Y$ = M_y / P = 15.47/102.85 = 150 mm

$eccen_{XMin}$ = (L/500) + (D/30) = (2400/500) + (450/30) = 20 mm

$eccen_{XMin}$ = max(20,20)= 20 mm

$eccen_{YMin}$ = (L/500) + (B/30) = (2400/500) + (230/30) = 12 mm

$eccen_{YMin}$ = max(12,20) = 20 mm

$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(188,20) = 188 mm

$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(150,20) = 150 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension, the column will be designed as an axially loaded column.
eccenX(188 mm) > 0.05 x 230(12 mm)
and eccenY(150 mm) > 0.05 x 450(23 mm)
Hence the column will be designed as a column with biaxial moments.
Provided steel :
Provide #16 - 6 nos. (1206 mmB)

Biaxial Check Calculations :

For X axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00211	380.61	13.38	367.23	147.67	177	26.14
2	402	-0.00303	-421.04	0.00	-421.04	-169.31	0	0.00
3	402	-0.00816	-434.80	0.00	-434.80	-174.84	-177	30.95
					Total	-196.48		57.09

x_{ux} = 121 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 121/450 = 0.097

P_{uc} = 0.097 x 30.00 x 230 x 450

= 299.64 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$

= 299.64 + (-196.48)

= 103.16 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.D - C_2.D)$

C_2 = 0.416 k_u = 0.416 x 121/450

= 0.112

M_{uc} = 299.64 x (0.5 x 450 - 0.112 x 450) = 52.38 kN-m

$M_{ux1} = M_{uc} + M_{us(Total)}$

= 52.38 + (57.09)

Er.M. SHANKAR, MTech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Channai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

= 109.47 kN-m

For Y axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	603	0.00067	134.43	7.48	126.95	76.57	67	5.13
2	603	-0.00722	-434.80	0.00	-434.80	-262.27	-67	17.57
					Total	-185.69		22.70

x_{uy} = 59 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 59/230 = 0.093

P_{uc} = 0.093 x 30.00 x 450 x 230

= 288.73 kN

$P_{uy1} = P_{uc} + P_{us(Total)}$

= 288.73 + (-185.69)

= 103.04 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.B - C_2.B)$

C_2 = 0.416 k_u = 0.416 x 59/230

= 0.107

M_{uc} = 288.73 x (0.5 x 230 - 0.107 x 230) = 26.07 kN-m

$M_{uy1} = M_{uc} + M_{us(Total)}$

= 26.07 + (22.70)

= 48.77 kN-m

P_u/P_{uz} = 0.056

For P_u/P_{uz} <= 0.2, α_n = 1.0

For P_u/P_{uz} >= 0.8, α_n = 2.0

For P_u/P_{uz} = 0.056, α_n = 1.000

$((M_{ux} / M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n})$

$= ((19.35/109.47)^{1.000}) + ((15.47/48.77)^{1.000})$

= 0.494 < 1.0,hence O.K.

Ties Details :

Load combination for ties design = 1.20 DL + 1.20 LL + 1.20 EQL Y-

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c),the diameter of the polygonal links or lateral ties shall be not less than the following

i)one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

ii)in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 16 mm diameter for ties.

(considering the requirement for shear)

Calculation of spacing of ties :

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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Calculation of τ_{auc} (as per IS 456: 2000 Clause 40.2.2)

τ_{auc} = 0.695 MPa for 1.17% steel

$\Delta = 1 + (3.P_u/A_g.f_{ck>})$ but not exceeding 1.5 = 1.10

τ_{auc}=

$\tau_{auc} \times \Delta$ = 0.765 MPa

For X - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

Design shear force,(V_{ud})	= 5.77 kN(from analysis)
V_{ud}	= 5.77 kN
Capacity of concrete,(V_{uc})	= τauc x Area = 79.15 kN
V_{us}	= V_{ud} - V_{uc} = -73.39 kN
Stirrup legs assumed	= 4
V_{umin}	= 0.87 x f_{ys} x A_{sv} x d /s_{vmin}
	= 0.87 x 500 x 804 x 182 /115 = 553672.12 N
	= 553.67 kN
$V_{us} < V_{usmin}$	
Spacing from min.shear R/F requirement	= (0.87 x f_{ys} x a_{sv})/(0.4 x width)
	= ((0.87 x 500.000 x 804.248)/(0.4 x 450.000))x (415.0/500.0f)
	= 1613 mm
Required spacing	= min(1613,115) = 115 mm

For Y - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

Design shear force,(V_{ud})	= 15.96 kN(from analysis)
V_{ud}	= 15.96 kN

Er.M. SHANKAR, M.Tech(Structl),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102585 / 9952050643
Email: structshankar97@yahoo.com

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Capacity of concrete,(V_{uc})	= τauc x Area = 79.15 kN
V_{us}	= V_{ud} - V_{uc} = -63.20 kN
Stirrup legs assumed	= 2
V_{umin}	= 0.87 x f_{ys} x A_{sv} x d /s_{vmin}
	= 0.87 x 500 x 402 x 402 /115 = 611473.08 N
	= 611.47 kN
V_{us} < V_{usmin}	
Spacing from min.shear R/F requirement	= (0.87 x f_{ys} x a_{sv})/(0.4 x width)
	= ((0.87 x 500.000 x 402.124)/(0.4 x 230.000))x (415.0/500.0f)
	= 1578 mm
Required spacing	= min(1578,115) = 115 mm

Axial Stess Check :

Actual Axial Stress = 1.004 N/mm2
Allowable Axial Stress = 0.1 x fck = 3.000 N/mm2
Actual Axial Stress is less than Allowable Axial Stress,
Hence, No need to provide Ductile detailing as per 7.1.1 clause,IS 13920

Provide ties #16 @ 110 mm c/c

SUMMARY :

CG1 : FO. FLOOR - Default Level at 18.000 m
Provide rectangular section : 230 x 450 mm
Provide #16 - 6 nos. (1206 mmB)
Provide ties #16 @ 110 mm c/c
Provide 4 legged ties along width and
2 legged ties along depth.

General Design Parameters :

Below TH FLOOR - Default Level at 15.000 m

Column size, (B x D) = 230 x 450 mm Column height,(L) = 2400 mm

From analysis results,loads on column

Axial load,(P) = 181.47 kN

MomentX,(M_x) = 34.15 kN-m

MomentY,(M_y) = 20.66 kN-m

f_{ck} = 30.00 N/mmB

f_y = 500.00 N/mmB

Load combination = 1.50 DL + 1.50 EQL X-

Column Name	Orientation Angle
C1	0.00

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth
= (2400 x 1.000) / 450 = 5.33 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width
= (2400 x 1.000) / 230 = 10.43 <= 12.00,

column is not slender in this direction.

$M_{x_MinEccen}$ = 3.629 kN.m
$M_{y_MinEccen}$ = 3.629 kN.m
$M_x = \max(M_x, M_{x_MinEccen}) + M_{uaddX}$ = 34.150 kN.m
$M_y = \max(M_y, M_{y_MinEccen}) + M_{uaddY}$ = 20.658 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity
equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 34.15/181.47 = 188 mm
Actual $eccen_Y$ = M_y / P = 20.66/181.47 = 114 mm
$eccen_{XMin}$ = (L/500) + (D/30) = (2400/500) + (450/30) = 20 mm
$eccen_{XMin}$ = max(20,20)= 20 mm
$eccen_{YMin}$ = (L/500) + (B/30) = (2400/500) + (230/30) = 12 mm
$eccen_{YMin}$ = max(12,20) = 20 mm
$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(188,20) = 188 mm
$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(114,20) = 114 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension, the column will be designed as an axially loaded column.
eccenX(188 mm) > 0.05 x 230(12 mm)
and eccenY(114 mm) > 0.05 x 450(23 mm)
Hence the column will be designed as a column with biaxial moments.
Provided steel :
Provide #16 - 6 nos. (1206 mmB)

Biaxial Check Calculations :

For X axis :

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00232	394.01	13.38	380.63	153.06	177	27.09
2	402	-0.00201	-374.07	0.00	-374.07	-150.42	0	0.00
3	402	-0.00635	-434.80	0.00	-434.80	-174.84	-177	30.95
					Total	-172.20		58.04

x_{ux} = 143 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 143/450 = 0.114

P_{uc} = 0.114 x 30.00 x 230 x 450

= 354.77 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$

= 354.77 + (-172.20)

= 182.57 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.D - C_2.D)$

C_2 = 0.416 k_u = 0.416 x 143/450

= 0.132

M_{uc} = 354.77 x (0.5 x 450 - 0.132 x 450) = 58.75 kN-m

$M_{ux1} = M_{uc} + M_{us(Total)}$

= 58.75 + (58.04)

= 116.78 kN-m

For Y axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	603	0.00102	203.01	10.14	192.88	116.34	67	7.79
2	603	-0.00592	-434.80	0.00	-434.80	-262.27	-67	17.57
					Total	-145.92		25.37

x_{uy} = 68 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 68/230 = 0.106

P_{uc} = 0.106 x 30.00 x 450 x 230

= 328.57 kN

$P_{uy1} = P_{uc} + P_{us(Total)}$

= 328.57 + (-145.92)

= 182.65 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.B - C_2.B)$

C_2 = 0.416 k_u = 0.416 x 68/230

= 0.122

M_{uc} = 328.57 x (0.5 x 230 - 0.122 x 230) = 28.54 kN-m

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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$M_{uy1} = M_{uc} + M_{us(Total)}$

$= 28.54 + (25.37)$

$= 53.91$ kN-m

$P_u/P_{uz} = 0.099$

For $P_u/P_{uz} <= 0.2$, $\alpha_n = 1.0$

For $P_u/P_{uz} >= 0.8$, $\alpha_n = 2.0$

For $P_u/P_{uz} = 0.099$, $\alpha_n = 1.000$

$((M_{ux} / M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n})$

$= ((34.15/116.78)^{1.000}) + ((20.66/53.91)^{1.000})$

$= 0.676 < 1.0$,hence O.K.

Ties Details :

Load combination for ties design = 1.50 DL + 1.50 EQL Y-

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c),the diameter of the polygonal links or lateral ties shall be not less than the following

i)one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

ii)in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 16 mm diameter for ties.

(considering the requirement for shear)

Calculation of spacing of ties :

Calculation of τ_{auc} (as per IS 456: 2000 Clause 40.2.2)

$\tau_{auc} = 0.695$ MPa for 1.17% steel

$\Delta = 1 + (3.P_u/A_g.f_{ck>})$ but not exceeding 1.5 = 1.18

$\tau_{auc}=$

$\tau_{auc} \times \Delta = 0.819$ MPa

For X - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

Design shear force,(V_{ud})	= 4.15 kN(from analysis)
V_{ud}	= 4.15 kN
Capacity of concrete,(V_{uc})	= τauc x Area = 84.80 kN
V_{us}	= V_{ud} - V_{uc} = -80.65 kN
Stirrup legs assumed	= 4
V_{umin}	= $0.87 \times f_{ys} \times A_{sv} \times d / s_{vmin}$
	= 0.87 x 500 x 804 x 182 /115 = 553672.12 N

Er.M.SHANKAR, M.Tech.,(Struct.Engg.), ME, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

= 553.67 kN

$V_{us} < V_{usmin}$

Spacing from min.shear R/F requirement = $(0.87 \times f_{ys} \times a_{sv})/(0.4 \times \text{width})$

= ((0.87 x 500.000 x 804.248)/(0.4 x 450.000))x (415.0/500.0f)

= 1613 mm

Required spacing = min(1613,115) = 115 mm

For Y - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

Design shear force,(V_{ud}) = 38.99 kN(from analysis)

V_{ud} = 38.99 kN

Capacity of concrete,(V_{uc}) = τauc x Area = 84.80 kN

V_{us} = V_{ud} - V_{uc} = -45.81 kN

Stirrup legs assumed = 2

V_{umin} = $0.87 \times f_{ys} \times A_{sv} \times d / s_{vmin}$

= 0.87 x 500 x 402 x 402 /115 = 611473.08 N

= 611.47 kN

$V_{us} < V_{usmin}$

Spacing from min.shear R/F requirement = $(0.87 \times f_{ys} \times a_{sv})/(0.4 \times \text{width})$

= ((0.87 x 500.000 x 402.124)/(0.4 x 230.000))x (415.0/500.0f)

= 1578 mm

Required spacing = min(1578,115) = 115 mm

Axial Stess Check :

Actual Axial Stress = 1.788 N/mm2

Allowable Axial Stress = 0.1 x fck = 3.000 N/mm2

Actual Axial Stress is less than Allowable Axial Stress,

Hence, No need to provide Ductile detailing as per 7.1.1 clause,IS 13920

Provide ties #16 @ 110 mm c/c

SUMMARY :

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

CG1 : TH FLOOR - Default Level at 15.000 m
Provide rectangular section : 230 x 450 mm
Provide #16 - 6 nos. (1206 mmB)
Provide ties #16 @ 110 mm c/c
Provide 4 legged ties along width and
2 legged ties along depth.

General Design Parameters :

Below S FLOOR - Default Level at 12.000 m

Column size, (B x D) = 230 x 450 mm Column height,(L) = 2400 mm

From analysis results,loads on column

Axial load,(P) = 284.07 kN

MomentX,(M_x) = 30.82 kN-m

MomentY,(M_y) = 24.40 kN-m

f_{ck} = 30.00 N/mmB

f_y = 500.00 N/mmB

Load combination = 1.50 DL + 1.50 EQL X-

Column Name	Orientation Angle
C1	0.00

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth

= (2400 x 1.000) / 450 = 5.33 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width

= (2400 x 1.000) / 230 = 10.43 <= 12.00,

column is not slender in this direction.

$M_{x_MinEccen}$ = 5.681 kN.m

$M_{y_MinEccen}$ = 5.681 kN.m

$M_x = \max(M_x, M_{x_MinEccen}) + M_{uaddX}$ = 30.821 kN.m

$M_y = \max(M_y, M_{y_MinEccen}) + M_{uaddY}$ = 24.398 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity

E.M.SHANKAR, B.E., M.I.E., M.I.S.E., M.I.A.C.C.E
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 30.82/284.07 = 108 mm

Actual $eccen_Y$ = M_y / P = 24.40/284.07 = 86 mm

$eccen_{XMin}$ = (L/500) + (D/30) = (2400/500) + (450/30) = 20 mm

$eccen_{XMin}$ = max(20,20)= 20 mm

$eccen_{YMin}$ = (L/500) + (B/30) = (2400/500) + (230/30) = 12 mm

$eccen_{YMin}$ = max(12,20) = 20 mm

$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(108,20) = 108 mm

$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(86,20) = 86 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension, the column will be designed as an axially loaded column.
eccenX(108 mm) > 0.05 x 230(12 mm)
and eccenY(86 mm) > 0.05 x 450(23 mm)
Hence the column will be designed as a column with biaxial moments.
Provided steel :
Provide #16 - 6 nos. (1206 mmB)

Biaxial Check Calculations :

For X axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	(f_{si} - f_{ci}) (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00248	400.57	13.38	387.19	155.70	177	27.56
2	402	-0.00129	-258.29	0.00	-258.29	-103.86	0	0.00
3	402	-0.00506	-434.80	0.00	-434.80	-174.84	-177	30.95
					Total	-123.01		58.51

x_{ux} = 164 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 164/450 = 0.131

P_{uc} = 0.131 x 30.00 x 230 x 450

= 408.26 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$

= 408.26 + (-123.01)

= 285.25 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.D - C_2.D)$

C_2 = 0.416 k_u = 0.416 x 164/450

= 0.152

M_{uc} = 408.26 x (0.5 x 450 - 0.152 x 450) = 63.94 kN-m

$M_{ux1} = M_{uc} + M_{us(Total)}$

= 63.94 + (58.51)

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

= 122.45 kN-m

For Y axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	603	0.00139	277.42	12.12	265.30	160.02	67	10.72
2	603	-0.00451	-434.80	0.00	-434.80	-262.27	-67	17.57
					Total	-102.24		28.29

x_{uy} = 80 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 80/230 = 0.124

P_{uc} = 0.124 x 30.00 x 450 x 230

= 386.43 kN

$P_{uy1} = P_{uc} + P_{us(Total)}$

= 386.43 + (-102.24)

= 284.19 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.B - C_2.B)$

C_2 = 0.416 k_u = 0.416 x 80/230

= 0.144

M_{uc} = 386.43 x (0.5 x 230 - 0.144 x 230) = 31.66 kN-m

$M_{uy1} = M_{uc} + M_{us(Total)}$

= 31.66 + (28.29)

= 59.95 kN-m

P_u/P_{uz} = 0.155

For P_u/P_{uz} <= 0.2, α_n = 1.0

For P_u/P_{uz} >= 0.8, α_n = 2.0

For P_u/P_{uz} = 0.155, α_n = 1.000

$((M_{ux} / M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n})$

$= ((30.82/122.45)^{1.000}) + ((24.40/59.95)^{1.000})$

= 0.659 < 1.0,hence O.K.

Ties Details :

Load combination for ties design = 1.50 DL + 1.50 EQL Y-

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c),the diameter of the polygonal links or lateral ties shall be not less than the following

i)one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

ii)in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 16 mm diameter for ties.

(considering the requirement for shear)

Calculation of spacing of ties :

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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Calculation of τ_{auc} (as per IS 456: 2000 Clause 40.2.2)

τ_{auc} = 0.695 MPa for 1.17% steel

$\Delta = 1 + (3.P_u/A_g.f_{ck>})$ but not exceeding 1.5 = 1.28

τ_{auc}=

$\tau_{auc} \times \Delta$ = 0.893 MPa

For X - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

Design shear force,(V_{ud})	= 3.15 kN(from analysis)
V_{ud}	= 3.15 kN
Capacity of concrete,(V_{uc})	= τauc x Area = 92.39 kN
V_{us}	= V_{ud} - V_{uc} = -89.24 kN
Stirrup legs assumed	= 4
V_{umin}	= 0.87 x f_{ys} x A_{sv} x d /s_{vmin}
	= 0.87 x 500 x 804 x 182 /115 = 553672.12 N
	= 553.67 kN
$V_{us} < V_{usmin}$	
Spacing from min.shear R/F requirement	= (0.87 x f_{ys} x a_{sv})/(0.4 x width)
	= ((0.87 x 500.000 x 804.248)/(0.4 x 450.000))x (415.0/500.0f)
	= 1613 mm
Required spacing	= min(1613,115) = 115 mm

For Y - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

Design shear force,(V_{ud})	= 45.81 kN(from analysis)
V_{ud}	= 45.81 kN

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

Capacity of concrete,(V_{uc})	= τauc x Area = 92.39 kN
V_{us}	= V_{ud} - V_{uc} = -46.58 kN
Stirrup legs assumed	= 2
V_{umin}	= 0.87 x f_{ys} x A_{sv} x d /s_{vmin} = 0.87 x 500 x 402 x 402 /115 = 611473.08 N = 611.47 kN
V_{us} < V_{usmin}	
Spacing from min.shear R/F requirement	= (0.87 x f_{ys} x a_{sv})/(0.4 x width) = ((0.87 x 500.000 x 402.124)/(0.4 x 230.000))x (415.0/500.0f) = 1578 mm
Required spacing	= min(1578,115) = 115 mm

Axial Stess Check :

Actual Axial Stress = 2.843 N/mm2
Allowable Axial Stress = 0.1 x fck = 3.000 N/mm2
Actual Axial Stress is less than Allowable Axial Stress,
Hence, No need to provide Ductile detailing as per 7.1.1 clause,IS 13920

Provide ties #16 @ 110 mm c/c

SUMMARY :

CG1 : S FLOOR - Default Level at 12.000 m
Provide rectangular section : 230 x 450 mm
Provide #16 - 6 nos. (1206 mmB)
Provide ties #16 @ 110 mm c/c
Provide 4 legged ties along width and
2 legged ties along depth.

General Design Parameters :

Below F FLOOR - Default Level at 9.000 m

Column size, (B x D) = 230 x 450 mm
Column height,(L) = 2400 mm
From analysis results,loads on column
Axial load,(P) = 173.51 kN
MomentX,(M_x) = 23.64 kN-m
MomentY,(M_y) = 18.88 kN-m
f_{ck} = 30.00 N/mmB
f_y = 500.00 N/mmB
Load combination = 0.90 DL + 1.50 EQL X+

Column Name	Orientation Angle
C1	0.00

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth
= (2400 x 1.000) / 450 = 5.33 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width
= (2400 x 1.000) / 230 = 10.43 <= 12.00,

column is not slender in this direction.

$M_{x_MinEccen}$ = 3.470 kN.m
$M_{y_MinEccen}$ = 3.470 kN.m
$M_x = \max(M_x, M_{x_MinEccen}) + M_{uaddX}$ = 23.641 kN.m
$M_y = \max(M_y, M_{y_MinEccen}) + M_{uaddY}$ = 18.875 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity
equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 23.64/173.51 = 136 mm
Actual $eccen_Y$ = M_y / P = 18.88/173.51 = 109 mm
$eccen_{XMin}$ = (L/500) + (D/30) = (2400/500) + (450/30) = 20 mm
$eccen_{XMin}$ = max(20,20)= 20 mm
$eccen_{YMin}$ = (L/500) + (B/30) = (2400/500) + (230/30) = 12 mm
$eccen_{YMin}$ = max(12,20) = 20 mm
$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(136,20) = 136 mm
$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(109,20) = 109 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension, the column will be designed as an axially loaded column.
eccenX(136 mm) > 0.05 x 230(12 mm)
and eccenY(109 mm) > 0.05 x 450(23 mm)
Hence the column will be designed as a column with biaxial moments.
Provided steel :
Provide #16 - 6 nos. (1206 mmB)

Biaxial Check Calculations :

For X axis :

Er.M. SHANKAR, M.Tech.(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00231	393.23	13.38	379.85	152.75	177	27.04
2	402	-0.00210	-380.10	0.00	-380.10	-152.85	0	0.00
3	402	-0.00651	-434.80	0.00	-434.80	-174.84	-177	30.95
					Total	-174.94		57.98

x_{ux} = 141 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 141/450 = 0.113

P_{uc} = 0.113 x 30.00 x 230 x 450

= 349.31 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$

= 349.31 + (-174.94)

= 174.37 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.D - C_2.D)$

C_2 = 0.416 k_u = 0.416 x 141/450

= 0.130

M_{uc} = 349.31 x (0.5 x 450 - 0.130 x 450) = 58.16 kN-m

$M_{ux1} = M_{uc} + M_{us(Total)}$

= 58.16 + (57.98)

= 116.14 kN-m

For Y axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	603	0.00098	196.32	9.91	186.41	112.44	67	7.53
2	603	-0.00605	-434.80	0.00	-434.80	-262.27	-67	17.57
					Total	-149.83		25.11

x_{uy} = 67 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 67/230 = 0.104

P_{uc} = 0.104 x 30.00 x 450 x 230

= 324.21 kN

$P_{uy1} = P_{uc} + P_{us(Total)}$

= 324.21 + (-149.83)

= 174.38 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.B - C_2.B)$

C_2 = 0.416 k_u = 0.416 x 67/230

= 0.121

M_{uc} = 324.21 x (0.5 x 230 - 0.121 x 230) = 28.29 kN-m

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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$M_{uy1} = M_{uc} + M_{us(Total)}$

$= 28.29 + (25.11)$

$= 53.39$ kN-m

$P_u/P_{uz} = 0.095$

For $P_u/P_{uz} <= 0.2$, $\alpha_n = 1.0$

For $P_u/P_{uz} >= 0.8$, $\alpha_n = 2.0$

For $P_u/P_{uz} = 0.095$, $\alpha_n = 1.000$

$((M_{ux} / M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n})$

$= ((23.64/116.14)^{1.000}) + ((18.88/53.39)^{1.000})$

$= 0.557 < 1.0$,hence O.K.

Ties Details :

Load combination for ties design = 1.50 DL + 1.50 EQL Y-

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c),the diameter of the polygonal links or lateral ties shall be not less than the following

i)one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

ii)in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 16 mm diameter for ties.

(considering the requirement for shear)

Calculation of spacing of ties :

$V_u = 1.4 \times (M_{ubL,lim} + M_{ubR,lim})$ / storey height

Calculation of τ_{auc} (as per IS 456: 2000 Clause 40.2.2)

$\tau_{auc} = 0.695$ MPa for 1.17% steel

$\Delta = 1 + (3.P_u/A_g.f_{ck>}) $ but not exceeding 1.5 = 1.39

$\tau_{auc}=$

$\tau_{auc} \times \Delta = 0.968$ MPa

For X - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

V_u for columns in the group :

Column.No	$M_{ubL,lim}$(kN-m)	$M_{ubR,lim}$(kN-m)	V_u(kN)
C1	294.29	0.00	137.33

V_u = 137.33 kN

Design shear force,(V_{ud}) = 2.13 kN(from analysis)

V_{ud} = max(V_{ud},V_u) = 137.33 kN

Capacity of concrete,(V_{uc}) = τauc x Area = 100.22 kN

Er.M.SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

V_{us} = V_{ud} - V_{uc} = 37.12 kN

Stirrup legs assumed = 4

V_{umin} = 0.87 x f_{ys} x A_{sv} x d /s_{vmin}

= 0.87 x 500 x 804 x 182 /115 = 553672.12 N

= 553.67 kN

V_{us} < V_{usmin}

Spacing from min.shear R/F requirement = (0.87 x f_{ys} x a_{sv})/(0.4 x width)

= ((0.87 x 500.000 x 804.248)/(0.4 x 450.000))x (415.0/500.0f)

= 1613 mm

Required spacing = min(1613,115) = 115 mm

For Y - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

V_u for columns in the group :

Column.No	$M_{ubL,lim}$(kN-m)	$M_{ubR,lim}$(kN-m)	V_u(kN)
C1	294.29	0.00	137.33

V_u = 137.33 kN

Design shear force,(V_{ud}) = 49.14 kN(from analysis)

V_{ud} = max(V_{ud},V_u) = 137.33 kN

Capacity of concrete,(V_{uc}) = τauc x Area = 100.22 kN

V_{us} = V_{ud} - V_{uc} = 37.12 kN

Stirrup legs assumed = 2

V_{umin} = 0.87 x f_{ys} x A_{sv} x d /s_{vmin}

= 0.87 x 500 x 402 x 402 /115 = 611473.08 N

= 611.47 kN

V_{us} < V_{usmin}

Spacing from min.shear R/F requirement = (0.87 x f_{ys} x a_{sv})/(0.4 x width)

= ((0.87 x 500.000 x 402.124)/(0.4 x 230.000))x (415.0/500.0f)

= 1578 mm

Required spacing = min(1578,115) = 115 mm

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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Axial Stess Check :

Actual Axial Stress = 3.932 N/mm2
Allowable Axial Stress = 0.1 x fck = 3.000 N/mm2
Actual Axial Stress is greater than Allowable Axial Stress,
Hence, Provide Ductile detailing as per 7.1.1 clause,IS 13920

Provide ties #16 @ 110 mm c/c

Confinement reinforcement :

Area of concrete core,(A_k)

Size of concrete core,(B-core x D-core) = (B - 2 x cover + 2 x dia.of Ties) x (D - 2 x cover + 2 x dia.of Ties)
= (230 - 2 x 40 + 2 x 16) x (450 - 2 x 40 + 2 x 16)
= 182 x 402
= 73164 mmB

Maximum hoop length,(h) = 201 mm

As per IS 13920 : 1993 , the spacing shall not exceed the smaller of
i) 1/4 th of the minimum member dimension = 230/4 = 58 mm
ii) 100 mm
Spacing need not be less than 75 mm

Required Spacing,(S) = 75 mm ~ 75 mm

Gross Area,(A_g) = 103500 mmB

C/S area of hoop bar,(A_{sh}) = $0.18 \times S \times h \times f_{ck}/f_{ys} \times ((A_g/A_k) - 1)$
= 68 mmB

As per IS 13920 : 1993 , the length for confining reinforcement shall not be less than
i) larger lateral dimension of the member at the section where yielding occurs = 450 mm
ii) 1/6 th of the clear span of the member = 400 mm
iii) 450 mm
Length = 450 ~ 450 mm
Provide confinement reinforcement for 450 mm at both the junctions.

SUMMARY :

CG1 : F FLOOR - Default Level at 9.000 m
Provide rectangular section : 230 x 450 mm
Provide #16 - 6 nos. (1206 mmB)
Provide confinement reinforcement #16 @ 75mm c/c for a length 450mm on both ends of column.
Provide ties #16 @ 110 mm c/c for remaining length of the column
Provide 4 legged ties along width and 2 legged ties along depth.

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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General Design Parameters :

Below G FLOOR - Default Level at 6.000 m

Column size, (B x D) = 230 x 450 mm Column height,(L) = 2400 mm

From analysis results,loads on column

Axial load,(P) = 196.09 kN

MomentX,(M_x) = 18.93 kN-m

MomentY,(M_y) = 23.97 kN-m

f_{ck} = 30.00 N/mmB

f_y = 500.00 N/mmB

Load combination = 0.90 DL + 1.50 EQL X+

Column Name	Orientation Angle
C1	0.00

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth
= (2400 x 1.000) / 450 = 5.33 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width
= (2400 x 1.000) / 230 = 10.43 <= 12.00,

column is not slender in this direction.

$M_{x_MinEccen}$ = 3.922 kN.m

$M_{y_MinEccen}$ = 3.922 kN.m

$M_x = \max(M_x, M_{x_MinEccen}) + M_{uaddX}$ = 18.933 kN.m

$M_y = \max(M_y, M_{y_MinEccen}) + M_{uaddY}$ = 23.972 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity
equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 18.93/196.09 = 97 mm

Actual $eccen_Y$ = M_y / P = 23.97/196.09 = 122 mm

$eccen_{XMin}$ = (L/500) + (D/30) = (2400/500) + (450/30) = 20 mm

$eccen_{XMin}$ = max(20,20)= 20 mm

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.co

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$eccen_{YMin}$ = (L/500) + (B/30) = (2400/500) + (230/30) = 12 mm

$eccen_{YMin}$ = max(12,20) = 20 mm

$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(97,20) = 97 mm

$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(122,20) = 122 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension, the column will be designed as an axially loaded column.
eccenX(97 mm) > 0.05 x 230(12 mm)
and eccenY(122 mm) > 0.05 x 450(23 mm)
Hence the column will be designed as a column with biaxial moments.
Provided steel :
Provide #16 - 6 nos. (1206 mmB)

Biaxial Check Calculations :

For X axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	(f_{si} - f_{ci}) (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00235	395.21	13.38	381.83	153.54	177	27.18
2	402	-0.00188	-362.48	0.00	-362.48	-145.76	0	0.00
3	402	-0.00611	-434.80	0.00	-434.80	-174.84	-177	30.95
					Total	-167.06		58.12

x_{ux} = 146 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 146/450 = 0.117

P_{uc} = 0.117 x 30.00 x 230 x 450
= 363.50 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$
= 363.50 + (-167.06)
= 196.44 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.D - C_2.D)$

C_2 = 0.416 k_u = 0.416 x 146/450
= 0.135

M_{uc} = 363.50 x (0.5 x 450 - 0.135 x 450) = 59.66 kN-m

$M_{ux1} = M_{uc} + M_{us(Total)}$
= 59.66 + (58.12)
= 117.78 kN-m

For Y axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	(f_{si} - f_{ci}) (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	603	0.00107	214.31	10.50	203.81	122.94	67	8.24

S.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

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2	603	-0.00571	-434.80	0.00	-434.80	-262.27	-67	17.57
					Total	-139.33		25.81

x_{uy} = 69 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 69/230 = 0.108

P_{uc} = 0.108 x 30.00 x 450 x 230

= 336.21 kN

$P_{uy1} = P_{uc} + P_{us(Total)}$

= 336.21 + (-139.33)

= 196.88 kN > P ,hence O.K.

$M_{uc} = P_{uc}.(0.5.B - C_2.B)$

C_2 = 0.416 k_u = 0.416 x 69/230

= 0.125

M_{uc} = 336.21 x (0.5 x 230 - 0.125 x 230) = 28.99 kN-m

$M_{uy1} = M_{uc} + M_{us(Total)}$

= 28.99 + (25.81)

= 54.80 kN-m

P_u/P_{uz} = 0.107

For P_u/P_{uz} <= 0.2, α_n = 1.0

For P_u/P_{uz} >= 0.8, α_n = 2.0

For P_u/P_{uz} = 0.107, α_n = 1.000

$((M_{ux} / M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n})$

$= ((18.93/117.78)^{1.000}) + ((23.97/54.80)^{1.000})$

= 0.598 < 1.0,hence O.K.

Ties Details :

Load combination for ties design = 0.90 DL + 1.50 EQL Y-

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c),the diameter of the polygonal links or lateral ties shall be not less than the following

i)one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

ii)in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 16 mm diameter for ties.

(considering the requirement for shear)

Calculation of spacing of ties :

$V_u = 1.4 \times (M_{ubL,lim} + M_{ubR,lim})$ / storey height

Calculation of τ_{auc} (as per IS 456: 2000 Clause 40.2.2)

τ_{auc} = 0.695 MPa for 1.17% steel

$\Delta = 1 + (3.P_u/A_g.f_{ck>})$ but not exceeding 1.5 = 1.34

τ_{auc}=

$\tau_{auc} \times \Delta$ = 0.929 MPa

For X - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE,FMACCE
Consulting Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

V_u for columns in the group :

Column.No	$M_{ubL,lim}$(kN-m)	$M_{ubR,lim}$(kN-m)	V_u(kN)
C1	294.29	0.00	137.33

V_u	= 137.33 kN
Design shear force,(V_{ud})	= 0.12 kN(from analysis)
V_{ud}	= max(V_{ud},V_u) = 137.33 kN
Capacity of concrete,(V_{uc})	= τauc x Area = 96.17 kN
V_{us}	= V_{ud} - V_{uc} = 41.17 kN
Stirrup legs assumed	= 4
V_{umin}	= 0.87 x f_{ys} x A_{sv} x d /s_{vmin}
	= 0.87 x 500 x 804 x 182 /115 = 553672.12 N
	= 553.67 kN
V_{us} < V_{usmin}	
Spacing from min.shear R/F requirement	= (0.87 x f_{ys} x a_{sv})/(0.4 x width)
	= ((0.87 x 500.000 x 804.248)/(0.4 x 450.000))x (415.0/500.0f)
	= 1613 mm
Required spacing	= min(1613,115) = 115 mm

For Y - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

V_u for columns in the group :

Column.No	$M_{ubL,lim}$(kN-m)	$M_{ubR,lim}$(kN-m)	V_u(kN)
C1	294.29	0.00	137.33

V_u	= 137.33 kN
Design shear force,(V_{ud})	= 39.77 kN(from analysis)

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050543
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

V_{ud}	= max(V_{ud},V_u) = 137.33 kN
Capacity of concrete,(V_{uc})	= τauc x Area = 96.17 kN
V_{us}	= V_{ud} - V_{uc} = 41.17 kN
Stirrup legs assumed	= 2
V_{umin}	= 0.87 x f_{ys} x A_{sv} x d /s_{vmin}
	= 0.87 x 500 x 402 x 402 /115 = 611473.08 N
	= 611.47 kN
V_{us} < V_{usmin}	
Spacing from min.shear R/F requirement	= (0.87 x f_{ys} x a_{sv})/(0.4 x width)
	= ((0.87 x 500.000 x 402.124)/(0.4 x 230.000))x (415.0/500.0f)
	= 1578 mm
Required spacing	= min(1578,115) = 115 mm

Axial Stess Check :

Actual Axial Stress = 3.368 N/mm2
Allowable Axial Stress = 0.1 x fck = 3.000 N/mm2
Actual Axial Stress is greater than Allowable Axial Stress,
Hence, Provide Ductile detailing as per 7.1.1 clause,IS 13920

Provide ties #16 @ 110 mm c/c

Confinement reinforcement :

Area of concrete core,(A_k)	
Size of concrete core,(B-core x D-core)	= (B - 2 x cover + 2 x dia.of Ties) x (D - 2 x cover + 2 x dia.of Ties)
	= (230 - 2 x 40 + 2 x 16) x (450 - 2 x 40 + 2 x 16)
	= 182 x 402
	= 73164 mmB
Maximum hoop length,(h)	= 201 mm

As per IS 13920 : 1993 , the spacing shall not exceed the smaller of
i) 1/4 th of the minimum member dimension = 230/4 = 58 mm
ii) 100 mm
Spacing need not be less than 75 mm

Required Spacing,(S)	= 75 mm ~ 75 mm
Gross Area,(A_g)	= 103500 mmB
C/S area of hoop bar,(A_{sh})	= 0.18 x S x h x f_{ck}/f_{ys} x ((A_g/A_k) - 1)
	= 68 mmB

As per IS 13920 : 1993 , the length for confining reinforcement shall not be less than
i) larger lateral dimension of the member at the section where yielding occurs = 450 mm
ii) 1/6 th of the clear span of the member = 400 mm

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FIMACCE
Chief Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

iii) 450 mm

Length = 450 ~ 450 mm

Provide confinement reinforcement for 450 mm at both the junctions.

SUMMARY :

CG1 : G FLOOR - Default Level at 6.000 m
Provide rectangular section : 230 x 450 mm Provide #16 - 6 nos. (1206 mmB) Provide confinement reinforcement #16 @ 75mm c/c for a length 450mm on both ends of column. Provide ties #16 @ 110 mm c/c for remaining length of the column Provide 4 legged ties along width and 2 legged ties along depth.

General Design Parameters :

Below BASEMENT - Default Level at 3.000 m

Column size, (B x D) = 230 x 450 mm

Column height,(L) = 2700 mm

From analysis results,loads on column

Axial load,(P) = 129.24 kN

MomentX,(M_x) = 69.02 kN-m

MomentY,(M_y) = 1.64 kN-m

f_{ck} = 30.00 N/mmB

f_y = 500.00 N/mmB

Load combination = 0.90 DL + 1.50 EQL Y+

Column Name	Orientation Angle
C1	0.00

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth

= (2700 x 1.000) / 450 = 6.00 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width

= (2700 x 1.000) / 230 = 11.74 <= 12.00,

column is not slender in this direction.

$M_{x_MinEccen}$ = 2.636 kN.m

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Tiruvadi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

$M_{y_MinEccen}$ = 2.585 kN.m

$M_x = \max(M_x, M_{x_MinEccen}) + M_{uaddX}$ = 69.024 kN.m

$M_y = \max(M_y, M_{y_MinEccen}) + M_{uaddY}$ = 2.585 kN.m

Calculation Of Eccentricities :

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricity
equal to the unsupported length of the column/500 plus lateral dimension/30, subject to a minimum of 20 mm.

Actual $eccen_X$ = Mx / P = 69.02/129.24 = 534 mm

Actual $eccen_Y$ = M_y / P = 1.64/129.24 = 13 mm

$eccen_{XMin}$ = (L/500) + (D/30) = (2700/500) + (450/30) = 20 mm

$eccen_{XMin}$ = max(20,20)= 20 mm

$eccen_{YMin}$ = (L/500) + (B/30) = (2700/500) + (230/30) = 13 mm

$eccen_{YMin}$ = max(13,20) = 20 mm

$eccen_X$ = max(Actual $eccen_X$,$eccen_{XMin}$) = max(534,20) = 534 mm

$eccen_Y$ = max(Actual $eccen_Y$,$eccen_{YMin}$) = max(13,20) = 20 mm

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension, the column will be designed as an axially loaded column.
eccenX(534 mm) > 0.05 x 230(12 mm)
and eccenY(20 mm) > 0.05 x 450(23 mm)

Hence the column will be designed as a column with biaxial moments.

Provided steel :
Provide #16 - 6 nos. (1206 mmB)

Biaxial Check Calculations :

For X axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	$(f_{si} - f_{ci})$ (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	402	0.00219	386.45	13.38	373.07	150.02	177	26.55
2	402	-0.00264	-407.34	0.00	-407.34	-163.80	0	0.00
3	402	-0.00746	-434.80	0.00	-434.80	-174.84	-177	30.95
					Total	-188.62		57.50

x_{ux} = 128 mm $P_{uc} = C_1.f_{ck}.B.D$

C_1 = 0.36 k_u = 0.36 x 128/450 = 0.103

P_{uc} = 0.103 x 30.00 x 230 x 450
= 318.75 kN

$P_{ux1} = P_{uc} + P_{us(Total)}$
= 318.75 + (-188.62)

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

= 130.12 kN > P ,hence O.K.

M_{uc} = P_{uc}.(0.5.D - C_2.D)

C_2 = 0.416 k_u = 0.416 x 128/450

= 0.119

M_{uc} = 318.75 x (0.5 x 450 - 0.119 x 450) = 54.70 kN-m

M_{ux1} = M_{uc} + $M_{us(Total)}$

= 54.70 + (57.50)

= 112.20 kN-m

For Y axis :

Row No. (i)	A_{si} (mmB)	e_i	f_{si} (N/mmB)	f_{ci} (N/mmB)	(f_{si} - f_{ci}) (N/mmB)	P_{usi} (kN)	x_i (mm)	M_{usi} (kN-m)
1	603	0.00079	158.98	8.52	150.45	90.75	67	6.08
2	603	-0.00676	-434.80	0.00	-434.80	-262.27	-67	17.57
					Total	-171.51		23.65

x_{uy} = 62 mm P_{uc} = C_1.f_{ck}.B.D

C_1 = 0.36 k_u = 0.36 x 62/230 = 0.097

P_{uc} = 0.097 x 30.00 x 450 x 230

= 301.83 kN

P_{uy1} = P_{uc} + $P_{us(Total)}$

= 301.83 + (-171.51)

= 130.31 kN > P ,hence O.K.

M_{uc} = P_{uc}.(0.5.B - C_2.B)

C_2 = 0.416 k_u = 0.416 x 62/230

= 0.112

M_{uc} = 301.83 x (0.5 x 230 - 0.112 x 230) = 26.91 kN-m

M_{uy1} = M_{uc} + $M_{us(Total)}$

= 26.91 + (23.65)

= 50.56 kN-m

P_u/P_{uz} = 0.070

For P_u/P_{uz} <= 0.2, α_n = 1.0

For P_u/P_{uz} >= 0.8, α_n = 2.0

For P_u/P_{uz} = 0.070, α_n = 1.000

$((M_{ux} / M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n})$

= $((69.02/112.20)^{1.000}) + ((2.58/50.56)^{1.000})$

= 0.666 < 1.0,hence O.K.

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952950643
Email: structshankar37@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

Ties Details :

Load combination for ties design = 0.90 DL + 1.50 EQL Y-

Calculation of diameter of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c),the diameter of the polygonal links or lateral ties shall be not less than the following

i)one-fourth the diameter of the largest longitudinal bar = 16/4 = 4 mm

ii)in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm

Provide 16 mm diameter for ties.

(considering the requirement for shear)

Calculation of spacing of ties :

$V_u = 1.4 \times (M_{ubL,lim} + M_{ubR,lim})$ / storey height

Calculation of τ_{auc} (as per IS 456: 2000 Clause 40.2.2)

τ_{auc} = 0.695 MPa for 1.17% steel

$\Delta = 1 + (3.P_u/A_g.f_{ck>})$ but not exceeding 1.5 = 1.38

τ_{auc}=

$\tau_{auc} \times \Delta$ = 0.959 MPa

For X - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

V_u for columns in the group :

Column.No	$M_{ubL,lim}$(kN-m)	$M_{ubR,lim}$(kN-m)	V_u(kN)
C1	294.29	0.00	137.33

V_u	= 137.33 kN
Design shear force,(V_{ud})	= 0.37 kN(from analysis)
V_{ud}	= max(V_{ud},V_u) = 137.33 kN
Capacity of concrete,(V_{uc})	= τauc x Area = 99.22 kN
V_{us}	= V_{ud} - V_{uc} = 38.11 kN
Stirrup legs assumed	= 4
V_{umin}	= 0.87 x f_{ys} x A_{sv} x d /s_{vmin}
	= 0.87 x 500 x 804 x 182 /115 = 553672.12 N
	= 553.67 kN
$V_{us} < V_{usmin}$	
Spacing from min.shear R/F requirement	= (0.87 x f_{ys} x a_{sv})/(0.4 x width)
	= ((0.87 x 500.000 x 804.248)/(0.4 x 450.000))x (415.0/500.0f)
	= 1613 mm
Required spacing	= min(1613,115) = 115 mm

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FNACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

For Y - axis :

As per IS 456: 2000 , the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 230 mm

ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied

= 16 x 16 = 256 mm

iii) 300 mm

Required spacing = minimum of (230, 256, 300) = 230 mm

Maximum ties spacing as per IS 13920 = the least lateral dimension/2 = 115 mm

Required spacing = minimum of (230, 115)= 115 mm

V_u for columns in the group :

Column.No	$M_{ubL,lim}$(kN-m)	$M_{ubR,lim}$(kN-m)	V_u(kN)
C1	294.29	0.00	137.33

V_u	= 137.33 kN
Design shear force,(V_{ud})	= 40.17 kN(from analysis)
V_{ud}	= max(V_{ud},V_u) = 137.33 kN
Capacity of concrete,(V_{uc})	= τauc x Area = 99.22 kN
V_{us}	= V_{ud} - V_{uc} = 38.11 kN
Stirrup legs assumed	= 2
V_{umin}	= 0.87 x f_{ys} x A_{sv} x d /s_{vmin}
	= 0.87 x 500 x 402 x 402 /115 = 611473.08 N
	= 611.47 kN
V_{us} < V_{usmin}	
Spacing from min.shear R/F requirement	= (0.87 x f_{ys} x a_{sv})/(0.4 x width)
	= ((0.87 x 500.000 x 402.124)/(0.4 x 230.000))x (415.0/500.0f)
	= 1578 mm
Required spacing	= min(1578,115) = 115 mm

Axial Stess Check :

Actual Axial Stress = 3.793 N/mm2

Allowable Axial Stress = 0.1 x fck = 3.000 N/mm2

Actual Axial Stress is greater than Allowable Axial Stress,

Hence, Provide Ductile detailing as per 7.1.1 clause,IS 13920

Provide ties #16 @ 110 mm c/c

Confinement reinforcement :

Area of concrete core,(A_k)	
Size of concrete core,(B-core x D-core)	= (B - 2 x cover + 2 x dia.of Ties) x (D - 2 x cover + 2 x dia.of Ties)
	= (230 - 2 x 40 + 2 x 16) x (450 - 2 x 40 + 2 x 16)

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0057
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY

= 182 x 402

= 73164 mmB

Maximum hoop length,(h) = 201 mm

As per IS 13920 : 1993 , the spacing shall not exceed the smaller of

i) 1/4 th of the minimum member dimension = 230/4 = 58 mm

ii) 100 mm

Spacing need not be less than 75 mm

Required Spacing,(S) = 75 mm ~ 75 mm

Gross Area,(A_g) = 103500 mmB

C/S area of hoop bar,(A_{sh}) = $0.18 \times S \times h \times f_{ck}/f_{ys} \times ((A_g/A_k) - 1)$

= 68 mmB

As per IS 13920 : 1993 , the length for confining reinforcement shall not be less than

i) larger lateral dimension of the member at the section where yielding occurs = 450 mm

ii) 1/6 th of the clear span of the member = 450 mm

iii) 450 mm

Length = 450 ~ 450 mm

Provide confinement reinforcement for 450 mm at both the junctions.

SUMMARY :

CG1 : BASEMENT - Default Level at 3.000 m
Provide rectangular section : 230 x 450 mm
Provide #16 - 6 nos. (1206 mmB)
Provide confinement reinforcement
#16 @ 75mm c/c for a length
450mm on both ends of column.
Provide ties #16 @ 110 mm c/c
for remaining length of the column
Provide 4 legged ties along width and
2 legged ties along depth.

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050[illegible]
Email: structshankar97@[illegible]

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY.

Beam Detail Report

F FLOOR - Default Level

Load Combinations used in beam design :

Load Combi No	Load Combination	Load Combi No	Load Combination
1	1.50 DL + 1.50 LL	2	1.20 DL + 1.20 LL + 1.20 EQL X+
3	1.20 DL + 1.20 LL + 1.20 EQL X-	4	1.20 DL + 1.20 LL + 1.20 EQL Y+
5	1.20 DL + 1.20 LL + 1.20 EQL Y-	6	1.50 DL + 1.50 EQL X+
7	1.50 DL + 1.50 EQL X-	8	1.50 DL + 1.50 EQL Y+
9	1.50 DL + 1.50 EQL Y-	10	1.20 DL + 1.20 LL + 1.20 WL X+
11	1.20 DL + 1.20 LL + 1.20 WL X-	12	1.20 DL + 1.20 LL + 1.20 WL Y+
13	1.20 DL + 1.20 LL + 1.20 WL Y-	14	1.50 DL + 1.50 WL X+
15	1.50 DL + 1.50 WL X-	16	1.50 DL + 1.50 WL Y+
17	1.50 DL + 1.50 WL Y-	18	0.90 DL + 1.50 EQL X+
19	0.90 DL + 1.50 EQL X-	20	0.90 DL + 1.50 EQL Y+
21	0.90 DL + 1.50 EQL Y-	22	0.90 DL + 1.50 WL X+
23	0.90 DL + 1.50 WL X-	24	0.90 DL + 1.50 WL Y+
25	0.90 DL + 1.50 WL Y-		

Design of Beam Group : BG1

Beam : B1

Material Properties :

fck = 20.000 N/mm²

fy = 500.000 N/mm²

1.Design Data :

1.1)Dimensions:

Beam Type = 'Rectangular Beam'

Width (b) = 230 mm

Total depth (D) = 600 mm

Effective Tension Cover = 33mm

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY.

Effective Compression Cover = 33mm
Clear side Cover = 25mm
Effective depth (d) = 567mm
Span = 2.730m
Clear Span (L) = 2.730m

1.2)Loading on the Beam :

1.2.1)Self wt. of beam : -3.450 kN/m

1.2.2)Slab Load

Dead Load :

Trapezoidal Load in kN/m
St.Pt 1.580 m L = 2.730 m Max. Value = -6.517 kN/m

LiveLoad

Trapezoidal Load in kN/m
St.Pt 1.580 m L = 2.730 m Max. Value = -4.740 kN/m

1.2.3)Wall Load

UDL in kN/m
St.Pt 0.000 m L = 2.730 m Max. Value = -10.488 kN/m

2.Computation of Area of steel :

2.1) Flexural Steel at Top and Bottom

2.1.1)Computation of theoretical steel

At length	0.000 L	0.100 L	0.200 L	0.300 L	0.400 L	0.500 L	0.600 L	0.700 L	0.800 L	0.900 L	1.000 L
B.M. Hog in kN-m	43.395	33.532	25.173	18.428	12.614	8.321	8.542	16.523	30.502	46.383	64.165
Load Combi-No	7	7	19	19	19	19	6	6	6	6	6
B.M. Sag in kN-m	34.415	30.493	25.998	20.838	14.129	6.259	0.021	0.000	0.000	0.280	0.000
Load Combi-No	18	18	6	6	6	6	20	1	1	19	19
Torsion in kN-m	5.347	5.347	5.347	5.347	5.347	5.347	5.347	5.347	5.347	5.347	5.347
Load Combi-No	9	9	9	9	9	9	9	9	9	9	9
Eq.B.M.Hog in kN-m	54.746	44.883	25.173	18.428	12.614	19.671	19.893	27.874	41.853	57.724	75.515
Eq.B.M.Sag	34.415	30.493	37.349	32.189	25.480	3.030	2.808	0.000	0.000	0.280	0.000

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg.No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.co

STABILITY CALCULATIONS FOR THE PROPOSED CONSTRUCTION STILT FLOOR +4 FLOORS +5 TH FLOOR (PART) RESIDRNTIAL BUILDING (HEIGHT 18.3 M)WITH 39 DWELLING UNITS AVAILING PREMIUM FSI AT PLOT NO.19,20&21,11 TH WEST STREET, JAI NAGAR EXTENSION,METHA NAGAR ,KUNDRATHUR ,CHENNAI-600069 COMPRISED IN S.NO.90/1B,90/1B1C,90/1B1A2,OF KUNDRATHUR -B VILLAGE,WITH IN THE LIMITS OF KUNDRATHUR MUNICIPALITY.

in kN-m											
Top $A_{st_req.}$ in mm²	232.309	188.806	152.543	123.669	99.043	81.014	81.942	115.571	175.601	245.643	326.621
Bot $A_{st_req.}$ in mm²	192.670	175.560	156.098	133.948	105.435	72.400	46.519	0.000	0.000	47.592	0.000

Max.Area of steel req. at Top = 326.621 mm² at 1.000L
Max.Area of steel req. at Bottom = 192.670 mm² at 0.000L
2.1.2)Minimum and Maximum Steel Checks :
a)Minimum Steel Check as per IS 456:2000 Clause 26.5.1.1(a)

$A_{st} = (0.85 \times b \times d) / f_y$
$= 221.697$ mm²

b)Maximum Steel Check as per IS 456:2000 Clause 26.5.1.1(b)

$A_{st} = 0.04 \times b \times D$
$= 5520.000$ mm²

2.1.3)Provided Steel
2.1.3.1)Provided Bottom Steel :

Straight bars = 2 - #12 +1 - #10
Curtail Bars = ---
Total Area of Steel at bottom at 0.000L = 304.734 mm² > $A_{st_req.}$ (O.K.)

2.1.3.2)Provided Top Steel :

Straight bars = 2 - #12 +1 - #10
Extra bars at Left = ---
Extra bars at Right = 2 - #12 1.365,1.400
Total Area of Steel at Top
At Left Support = 304.734 mm²
At Right Support = 530.929 mm²

2.2)Computation of Shear Reinforcement(Stirrups)

τ_{auc} = 0.440 N/mm²

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

S_{vmin} = 425 mm

Note :
Design Shear = (Equivalent S.F.)-(V_{uc})
V_{usT} = (T_u/b_1 + $V_u/2.5$)(d/d_1)
Where,
T_u = Torsional moment,
V_u = Ultimate shear force
b_1 = Center to center distance between corner bars in the direction of width
d_1 = Center to center distance between corner bars in the direction of depth
(As per Clause 41.4.3 of IS 456:2000)
If Design Shear < V_{usT} then Design Shear = V_{usT}
V_{usT} = Minimum shear for torsional transverse Reinforcement
2.2.1) Computation of Steel

At length	0.000L	0.100 L	0.200 L	0.300 L	0.400 L	0.500 L	0.600 L	0.700 L	0.800 L	0.900L	1.000L
S.F.in kN	38.354	33.829	28.842	23.395	-27.571	-33.941	-40.761	-47.725	-54.689	-61.653	-68.616
T_u in kN-m	5.347	5.347	5.347	5.347	5.347	5.347	5.347	5.347	5.347	5.347	5.347
Eq. S.F. in kN	75.552	71.026	66.040	60.592	64.768	71.138	77.959	84.923	91.886	98.850	105.814
V_{usT} in kN	55.730	53.733	51.532	49.129	50.971	53.782	56.792	59.864	62.937	66.010	69.083
Vuc in kN	46.843	46.843	46.843	46.843	46.843	57.436	57.436	57.436	57.436	57.436	57.436
Design Shear in kN	108.580	83.315	58.050	49.129	50.971	53.782	56.792	68.276	93.541	118.806	144.071
Spacing req. in mm for 8 Dia	100	100	100	100	100	100	100	100	100	100	100

2.2.2)Provided Shear Steel (Stirrups) :

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

Nominal Zone :#8 @ 100 mm c/c from 0.000 to 2.700 .

Note : Nominal zone is where No Shear Reinforcement is needed.
Hence provide minimum Shear Reinforcement of 0.75 d
(As per Clause 26.5.1.5 of IS 456:2000)

Detailing Provisions as per IS 13920:1993

6.1 General :

Clause 6.1.1 :

Factored Axial stress = 0.000 N/mm² < 0.1 f_{ck}

Clause 6.1.2 :

Width to Depth Ratio = 0.383

'Width to Depth Ratio is more than 0.3' Hence O.K.

Clause 6.1.3 :

Width of the beam = 230

Provided Width of the beam is more than 200 mm' Hence O.K.

Clause 6.1.4 :

Depth of the beam = 0.600 m

Provided Depth of the beam is less than 1/4 of clear span(2.730)' Hence O.K.

6.2 Longitudinal Reinforcement :

Clause 6.2.1 :

Minimum tension steel ratio on any face at any section = 0.24 x $\sqrt{(f_{ck})}/f_y$

Minimum tension steel ratio on any face at any section = 0.0021

Provided Minimum tension steel ratio on any face at any section = 0.0022 >= 0.0021

Clause 6.2.2 :

Provided Maximum tension steel ratio on any face at any section = 0.0038 < 0.025 Hence O.K.

Clause 6.2.3 :

Maximum negative steel provided at top face of left joint = 305 mm²

Maximum negative moment steel provided at top face of right joint = 531 mm²

Provided steel on bottom face of left joint = 305mm² > (531)/2 Hence O.K.

Provided steel on bottom face of right joint = 305 mm²> (531)/2 Hence O.K.

Clause 6.2.4 :

Provided Minimum steel on bottom face at any section = 305 mm²> (531)/4 Hence this clause satisfied

Provided Minimum steel on top face at any section = 305 mm²> (531)/4 Hence this clause satisfied

6.3 Web Reinforcement :

Clause 6.3.2 :

Provided Minimum Dia of stirrup = 8mm >= 6mm(for span less than 5m.) Hence

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

O.K.

Clause 6.3.3 :

The Shear force to be resisted by the vertical stirups shall be the maximum of

a)Calculated shear force as per the analysis

b)Shear force due to formation of plastic hinges at both ends plus the

factored gravity load on the span this is given by

i)FOR SWAY TO RIGHT

$V_{ua} = V_{a(D+L)} - 1.4[(M_{uAs,lim} + M_{uBh,lim})/L_{AB}]$

and $V_{ub} = V_{b(D+L)} + 1.4[(M_{uAs,lim} + M_{uBh,lim})/L_{AB}]$

$V_{ua} = 27.006 - 1.4[(79.533 + 135.963)/2.730]'$

$V_{ua} = -83.505$ kN'

$V_{ub} = 33.560 + 1.4[(79.533 + 135.963)/2.730]'$

$V_{ub} = 144.071$ kN'

i)FOR SWAY TO LEFT

$V_{ua} = V_{a(D+L)} + 1.4[(M_{uAh,lim} + M_{uBs,lim})/L_{AB}]$

and $V_{ub} = V_{b(D+L)} - 1.4[(M_{uAh,lim} + M_{uBs,lim})/L_{AB}]$

$V_{ua} = 27.006 + 1.4[(79.533 + 79.536)/2.730]'$

$V_{ua} = 108.580$ kN'

$V_{ub} = 33.560 - 1.4[(79.536 + 79.536)/2.730]'$

$V_{ub} = -48.014$ kN'

The final V_{ua} & V_{ub} are the maximum of Right and Left sway.

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com

V_{ua} = 108.580

V_{ub} = 144.071

If the Design Shear is less than V_{ua} & V_{ub} at left & right end

then Design shear = V_{ua} or Design shear = V_{ub}

NOTE:

i)All notations are as per IS13920

Clause 6.3.5 :
6.3.5.a:
Stirrup spacing over a length 2d at either end of a beam shall not exceed

a) d/4 , b) 8 x smallest longitudinal dia.

however it shall not be less than 100mm.
Provided Stirrup spacing in the 2d zone = 100 mm
6.3.5.b.:
Stirrup spacing in the rest portion <= d/2

Provided Stirrup spacing in the rest portion = 0 mm

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050[illegible]
Email: structshankar97@ya[illegible]

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Design Of Slabs : Default Level

Design Method : Limit State Method
Design Code : IS 456:2000

Design Of Slab

Slab Name : S1
Slab Type : TwoWay Slab
Grade of Concrete :M20
Grade of Steel : Fe500
Dimensions : Lx = 6.230 m ,Ly = 3.160 m ,Thickness= 125 mm
Span Ratio : Longer/Shorter = 1.972
Boundary Condition : One short and one long edge continuous

Loading on Slab :(Table 1)

Total DL = (1) + (2) + (3)

Self Weight (1) kN/m²	Floor Finish (2) kN/m²	Sunk Load (3) kN/m²	Total DL (4) kN/m²	Live Load (5) kN/m²	Total LL (6) kN/m²	Total Design Load 1.5 x [(4)+(6)] kN/m²
3.125	1.000	0.000	4.125	3.000	3.000	10.688

Deflection Check :(Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
25.280	1.000	32.000	32.000	OK

Moment And Steel Calculations :

Coefficients (α_x and α_y) are computed as per clause D-1.1 of IS 456:2000:

$M_x = \alpha_x \times W \times L_x \times L_x$:
$M_y = \alpha_y \times W \times L_x \times L_x$:
where
M_x,M_y = moments on strips of unit width spanning L_x and L_y respectively,
W = Total Design Load = 10.688 kN/m^2
L_x = Shorter span = 3.160 m

Position	Moment Coeff.α_x or α_y	Moment kN-m	A_{st_rqd} per metre mm²	Steel Detail dia @ spc mm c/c	A_{st_prv} per metre mm²	Remark

EXNO SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 1
Cell: 9282102566 / 9952050643
Email: structshankar97@yahoo.com

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MidShort	0.068	7.291	175.286	#8 @ 200	251.327	Main
MidLong	0.035	3.735	87.795	#8 @ 200	251.327	Other
SuppDown	0.000	0.000	0.000	--	0.000	Extra at Top
SuppTop	0.090	9.627	235.119	--	0.000	Extra at Top
SuppLeft	0.000	0.000	0.000	--	0.000	Extra at Top
SuppRight	0.047	5.016	118.838	--	0.000	Extra at Top

Minimum Steel Check :

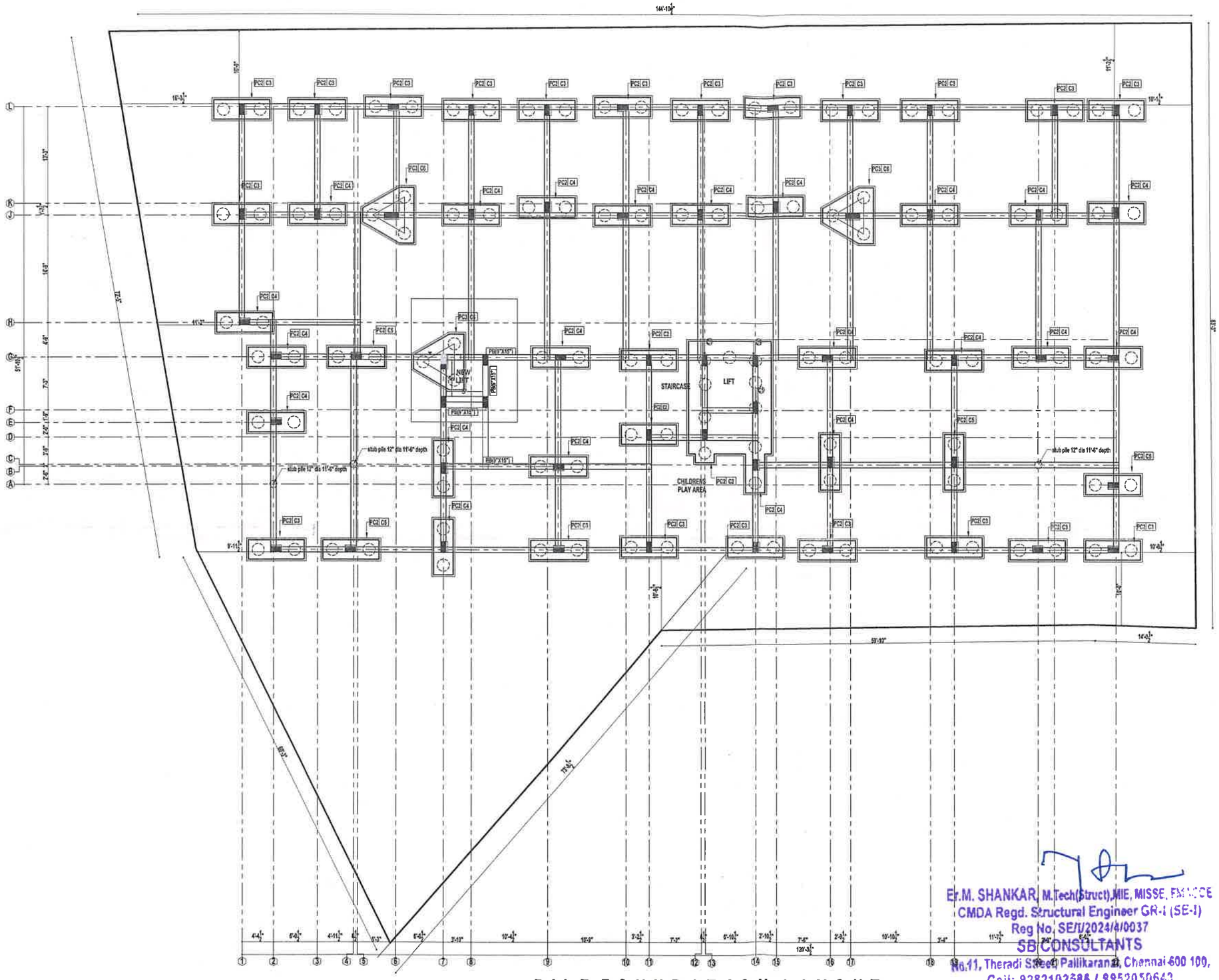
As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = 0.12 x C/S Area
= 150.000 mm^2

Notes :

Extra steel at Top support is computed considering the bent-ups,if any,coming from the adjoining span. It is the maximum of the extra steel required for each slab at that common support.

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102585 / 99[illegible]
Email: structshankar87@yahoo.co



PILE FOUNDATION LAYOUT

PROJECT:
PROPOSED RESIDENTIAL APARTMENTS AT PLOT NO.19,20&21,SURVEY NO 90/1B1 A2,90/1B1A3&90/1B1B,90/1B1C, 11 TH WEST STREET, METHA NAGAR JAI NAGAR EXTENSION,KUNDRATHUR,CHENNAI-600069

PROJECT NAME:ADI YOGI

NOTES:

1. ALL DIMENSIONS ARE IN FEET&INCHES UNLESS OTHER WISE SPECIFIED
2. M20 GRADE OF CONCRETE AND FE 500 GRADE SHALL BE USED
3. CLEAR COVER TO MAIN STEEL SHALL BE
   1" FOR BEAMS
   1.5" FOR COLUMNS.
   2"FOR FOOTINGS,
   5/8"FOR SLABS&SUNSHADES
4. FOUNDATION IS DESIGNED FOR STILT+5 FLOOR ONLY.
5. SBC CONSIDERED AT 15 M BELOW EXG. GL IS BORED PILE FOUNDATION AS PER SOIL REPORT.
6. BUILDING IS DESIGNED FOR GRAVITY +LATERAL LOADS AS AS PER IS 1893-2002.
7. 'Y' DENOTES TOR 40 STEEL CONFIRMING TO IS 1786-1985.
8. 'R' DENOTES MILD STEEL CONFIRMING TO I.S.432-1982.
9. FORM WORK TO BE REMOVED FOR SPANS
   UPTO 0M—3M 7DAYS
   3M—4.5M 14DAYS
   4.5M—6M 21DAYS
10. AVOID CLAY SOIL FILLING IN BASEMENT.
11. USE 1:1:2 MIX FOR STARTERS
12. USE OPC FOR ALL CONCRTE WORKS.
13. LAP LENGTH SHALL BE 47 TIMES THE DIA OF BAR.
14. ALL COLUMN RODS TO BE STAGGERED
15. NO DIMENSION SHOULD BE SCALED OFF FROM THE DRG. ONLY WRITTEN DIMENSION SHOULD BE FOLLOWED.
16. USE CHAIR RODS FOR TOP REINFORCEMENT
17. CANTILEVER PROPS SHOULD NOT BE RELEASED UNTIL ROOF IS CONCRETED.
18. ALL FOOTING RODS BENT 4"UP.
19. ALL BEAM TOP RODS SHALL BE PROPERLY ANCHORED IN COLUMNS

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R.NO.	REVISION	DATE	SIGN
1.			
2.			
3.			

PROJECT NO:

DRG TITLE: **FOUNDATION LAYOUT**

STRUCTURAL DRAWING

DRG.NO : SD-01 ONLY.

SCALE : 1"=4'-0" & 1"=2'-0"

DATE : 21.1.2024

SHEET SIZE : A1 | V1

CAD FILE: E:\SHANKAR\2023\AVITTAM HOMES\ KUNDRATHUR\FDN

GOOD FOR CONSTRUCTION

GENERAL NOTES ON PILES.

1. MINIMUM CEMENT CONTENT FOR PILES SHALL BE 400KG/M3 WITH M30 GRADE
2. WATERCEMENT RATIO SHALL NOT EXCEED 0.55
3. COVER BLOCK OF HOLLOW CIRCULAR SHAPE IN 1:1C:M SHALL BE USED
4. TOP SLURRY CONCRETE IN PILES IS TO BE DISMANTLED FOR A LENGTH OF 3'-3" BEFORE CASTING PILECAP
5. PILES INSERT IN A PILE CAP SHOULD BE A MINIMUM OF 2"OF GOOD QUALITY CONCRETE
6. THE LONGITUDINAL BARS OF COLUMN SHALL BE ANCHORED INTO THE PILE CAP PREFERABLY TAKEN UPTO THE BOTTOM OF PILE

N S N S

DEVELOPERS:
`M/S AVITTAM HOMES'
PLOT NO.84,6 TH CROSS STREET,
SHANKAR NAGAR,PAMMAL,
CHENNAI-600075.

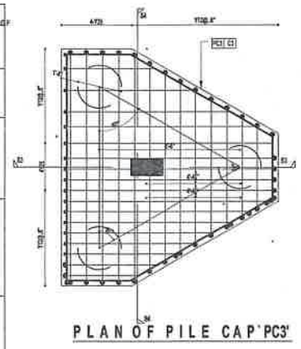
STRUCTURAL CONSULTANTS:
Er.M.SHANKAR, M.Tech(STRUCT),MIE,C.ENG(IND),MISSE,FMACCE(I)
CHARTERED ENGINEER/STRUCTURAL ENGINEER
SENIOR PROFESSIONAL ENGINEER
SB CONSULTANTS,
11,THERADI STREET,
PALLIKKARANAI.
CHENNAI-600100.
PH:65419436
MOBILE-9282102586.
e-mail m_shankarengineer@yahoo.co.in

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai 600 100.
Cell: 9282102586 / 8952050643
Email: structshankar97@yahoo.com

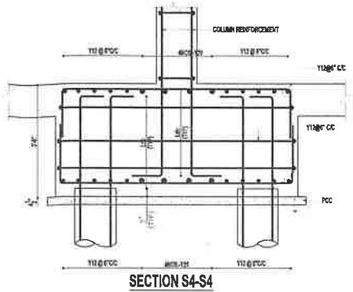
COLUMN SCHEDULE

S.NO	COLUMN REFERENCE	SIZE	COLUMN SHAPE	MAINSTEEL	BINDERS	BINDERS	TIE DETAILS
01.	C1	18"x9"		8Y16	-REFER- -SECTION-	M_{25}	
02.	C2	18"x9"		8Y20	-REFER- -SECTION-	M_{25}	
03.	C3	18"x12"	4Y12	8Y16	-REFER- -SECTION-	M_{25}	
04.	C4	18"x12"	4Y12	4Y25 + 4Y20	-REFER- -SECTION-	M_{25}	
05.	C5	18"x12"	4Y12	8Y25	-REFER- -SECTION-	M_{25}	
06.	C6	24"x12"	2'-6"	10Y25	-REFER- -SECTION-	M_{25}	

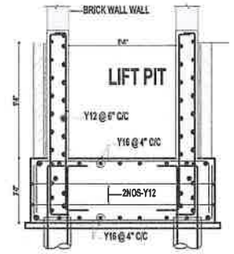
--portion can be removed above stilt floor roof &reinforcement



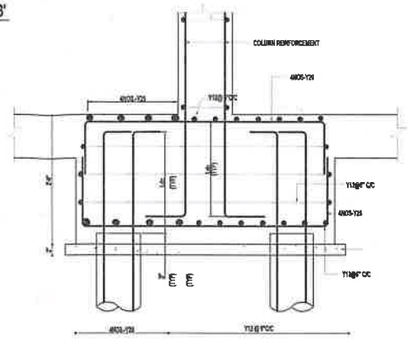
PLAN OF PILE CAP`PC3'



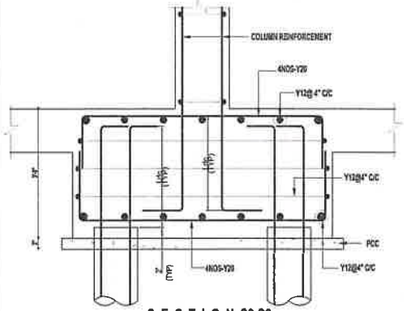
SECTION S4-S4



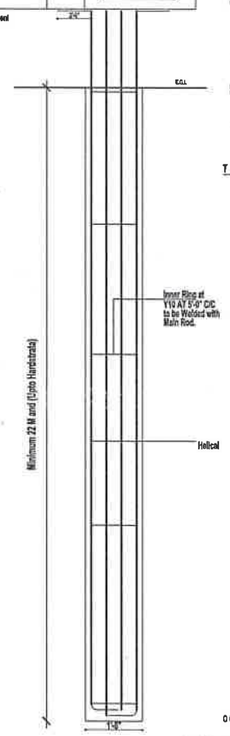
TYPICAL DETAILS OF LIFT WELL

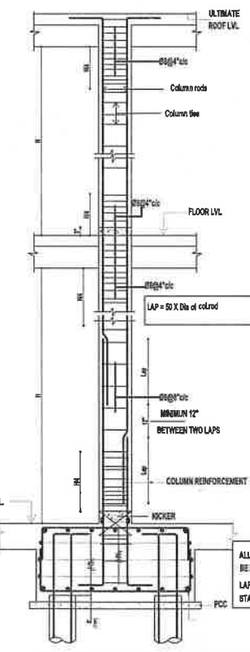


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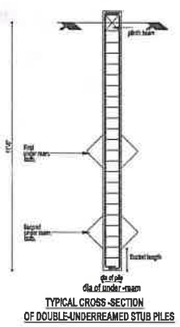


SECTION S2-S2

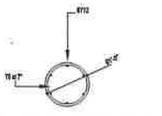




TYPICAL SECTION THROUGH COLUMN FOOTING



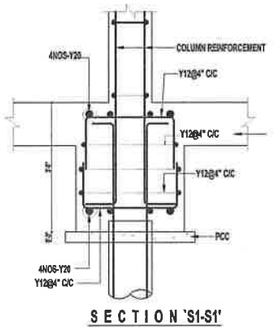
TYPICAL CROSS-SECTION OF DOUBLE-UNDERREAMED STUB PILES



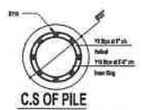
CROSS-SECTION OF STUB PILE



C.S OF PB

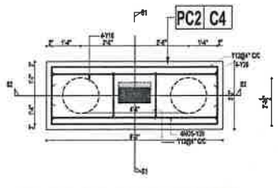


SECTION `S1-S1'

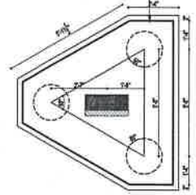


C.S OF PILE

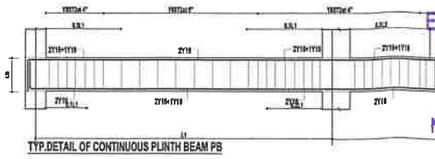
PC3 C6



PLAN OF PILE CAP`PC2'



PLAN OF PILE CAP`PC3'



TYP.DETAIL OF CONTINUOUS PLINTH BEAM PB

PROJECT:

PROPOSED RESIDENTIAL APARTMENTS AT PLOT NO.19,20&21,SURVEY NO. 90/1B1 A2,90/1B1A3&90/1B1B,90/1B1C, 11 TH WEST STREET, METHA NAGAR, JAI NAGAR EXTENSION,KUNDRATHUR,CHENNAI-600069

NOTES:

1. ALL DIMENSIONS ARE IN FEET&INCHES UNLESS OTHER WISE SPECIFIED
2. M20 GRADE OF CONCRETE AND FE 500 GRADE SHALL BE USED
3. CLEAR COVER TO MAIN STEEL SHALL BE 1" FOR BEAMS 1.5" FOR COLUMNS. 2"FOR FOOTINGS, 5/8"FOR SLABS&SUNSHADES
4. FOUNDATION IS DESIGNED FOR STILT+5 FLOOR ONLY.
5. SBC CONSIDERED AT 15 M BELOW EXG. GL IS BORED PILE FOUNDATION AS PER SOIL REPORT.
7. Y DENOTES TOR 40 STEEL CONFIRMING TO IS 1786-1985.
8. 'R' DENOTES MILD STEEL CONFIRMING TO I.S.432-1982.
9. FORM WORK TO BE REMOVED FOR SPANS
   UPTO 0M---3M 7DAYS
   3M---4.5M 14DAYS
   4.5M---6M 21DAYS
10. AVOID CLAY SOIL FILLING IN BASEMENT.
11. USE 1:1:2 MIX FOR STARTERS
12. USE OPC FOR ALL CONCRTE WORKS.
13. LAP LENGTH SHALL BE 47 TIMES THE DIA OF BAR.
14. ALL COLUMN RODS TO BE STAGGERED
15. NO DIMENSION SHOULD BE SCALED OFF FROM THE DRG. ONLY WRITTEN DIMENSION SHOULD BE FOLLOWED.
16. USE CHAIR RODS FOR TOP REINFORCEMENT
17. CANTILEVER PROPS SHOULD NOT BE RELEASED UNTIL ROOF IS CONCRETED.
18. ALL FOOTING RODS BENT 4"UP.
19. ALL BEAM TOP RODS SHALL BE PROPERLY ANCHORED IN COLUMNS

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ELEVATION LEVELS ARE FINISHED FLOOR LEVEL.

STRUCTURAL ENGINEER RESPONSIBILITY IS LIMITED TO DESIGN & ISSUAL OF STRUCTURAL DRAWING ONLY

ONUS OF CONSTRUCTION LIES WITH CONTRACTOR/OWNER

R.NO.	REVISION	DATE	SIGN
1.	OLD COLUMN POSITIONS C2&F2 MOVED FOR CAR PARKING	7.7.2023	
2.			
3.			

PROJECT NO:

DRG TITLE: **COLUMN&PILE CAP DETAILS**

STRUCTURAL DRAWING

DRG.NO : SD-01 ONLY.

SCALE : 1"=4'-0" & 1"=2'-0"

DATE : 21.1.2024

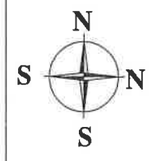
SHEET SIZE : A1 1/2

CAD FILE: E:\SHANKAR\2023\AVITTAM HOMES\ KUNDRATHUR\FDN

GOOD FOR CONSTRUCTION

GENERAL NOTES ON PILES.

1. MINIMUM CEMENT CONTENT FOR PILES SHALL BE 400KG/M3 WITH M30 GRADE
2. WATERCEMENT RATIO SHALL NOT EXCEED 0.55
3. COVER BLOCK OF HOLLOW CIRCULAR SHAPE IN 1:1C:M SHALL BE USED
4. TOP SLURRY CONCRETE IN PILES IS TO BE DISMANTLED FOR A LENGTH OF 3'-3" BEFORE CASTING PILECAP
5. PILES INSERT IN A PILE CAP SHOULD BE A MINIMUM OF 2"OF GOOD QUALITY CONCRETE
6. THE LONGITUDINAL BARS OF COLUMN SHALL BE ANCHORED INTO THE PILE CAP PREFERABLY TAKEN UPTO THE BOTTOM OF PILE



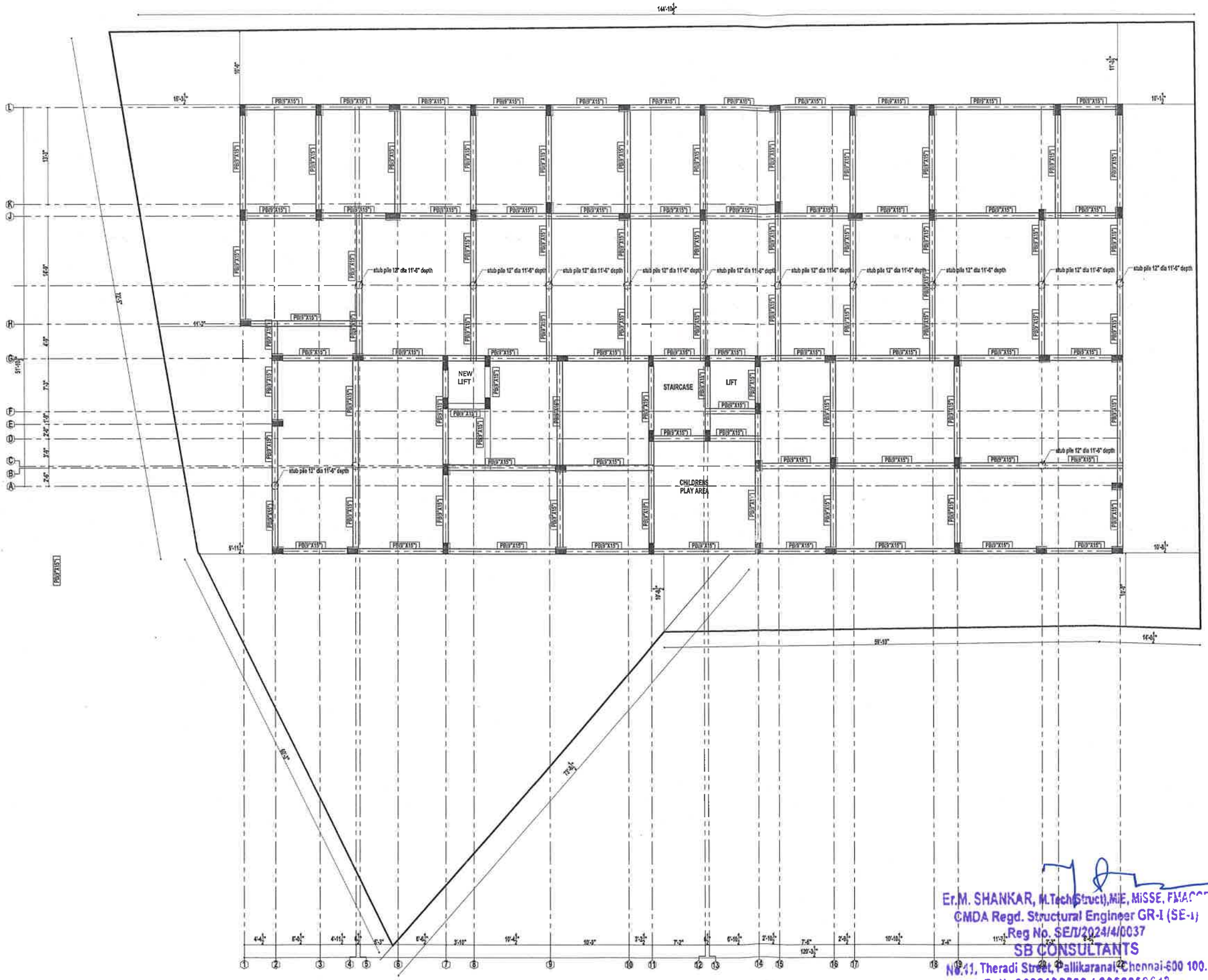
DEVELOPERS:

`M/S AVITTAM HOMES'
PLOT NO.3,6 TH CROSS STREET,
SHANKAR NAGAR,PAMMAL,
CHENNAI-600075.

STRUCTURAL CONSULTANTS:

Er.M.SHANKAR, M.Tech(STRUCT),MIE,C.ENG(IND),MISSE,FMACCE(I)
CHARTERED ENGINEER/STRUCTURAL ENGINEER
SENIOR PROFESSIONAL ENGINEER
SB CONSULTANTS,
11,THERADI STREET,
PALLIKKARANAI
CHENNAI-600100.
PH:65419436
MOBILE:9282102586.
e-mail:m_shankarengineer@yahoo.co.in

Er.M. SHANKAR, M.Tech(STRUCT),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE)
Reg No. SE/I/2024/A/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikkaranai, Chennai-600100
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com



PLINTH BEAM LAYOUT

PROJECT:

PROPOSED RESIDENTIAL APARTMENTS AT PLOT NO.19,20&21,SURVEY NO. 90/1B1 A2,90/1B1A3&90/1B1B,90/1B1C, 11 TH WEST STREET, METHA NAGAR, JAI NAGAR EXTENSION,KUNDRATHUR,CHENNAI-600069

NOTES:

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   2"FOR FOOTINGS,
   5/8"FOR SLABS&SUNSHADES
4. FOUNDATION IS DESIGNED FOR STILT+5 FLOOR ONLY.
5. SBC CONSIDERED AT 15 M BELOW EXG. G.L IS BORED PILE FOUNDATION AS PER SOIL REPORT.
6. BUILDING IS DESIGNED FOR GRAVITY +LATERAL LOADS AS AS PER IS 1893-2002.
7. 'X' DENOTES TOR 49 STEEL CONFIRMING TO IS 1786-1985.
8. 'R' DENOTES MILD STEEL CONFIRMING TO I.S.432-1982.
9. FORM WORK TO BE REMOVED FOR SPANS
   UPTO 0M—3M 7DAYS
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10. AVOID CLAY SOIL FILLING IN BASEMENT.
11. USE 1:1:2 MIX FOR STARTERS
12. USE OPC FOR ALL CONCRETE WORKS.
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R.NO.	REVISION	DATE	SIGN
1.			
2.			
3.			

PROJECT NO:

DRG TITLE: **PLINTH BEAM LAYOUT**

STRUCTURAL DRAWING

DRG.NO : SD-01 ONLY.

SCALE : 1"=4'-0" & 1"=2'-0"

DATE : 21.1.2024

SHEET SIZE : A1 (1/1)

CAD FILE: E:\SHANKAR\2023\AVITTAM HOMES\ KUNDRATHUR\FDN

GOOD FOR CONSTRUCTION

GENERAL NOTES ON PILES.

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N S N S

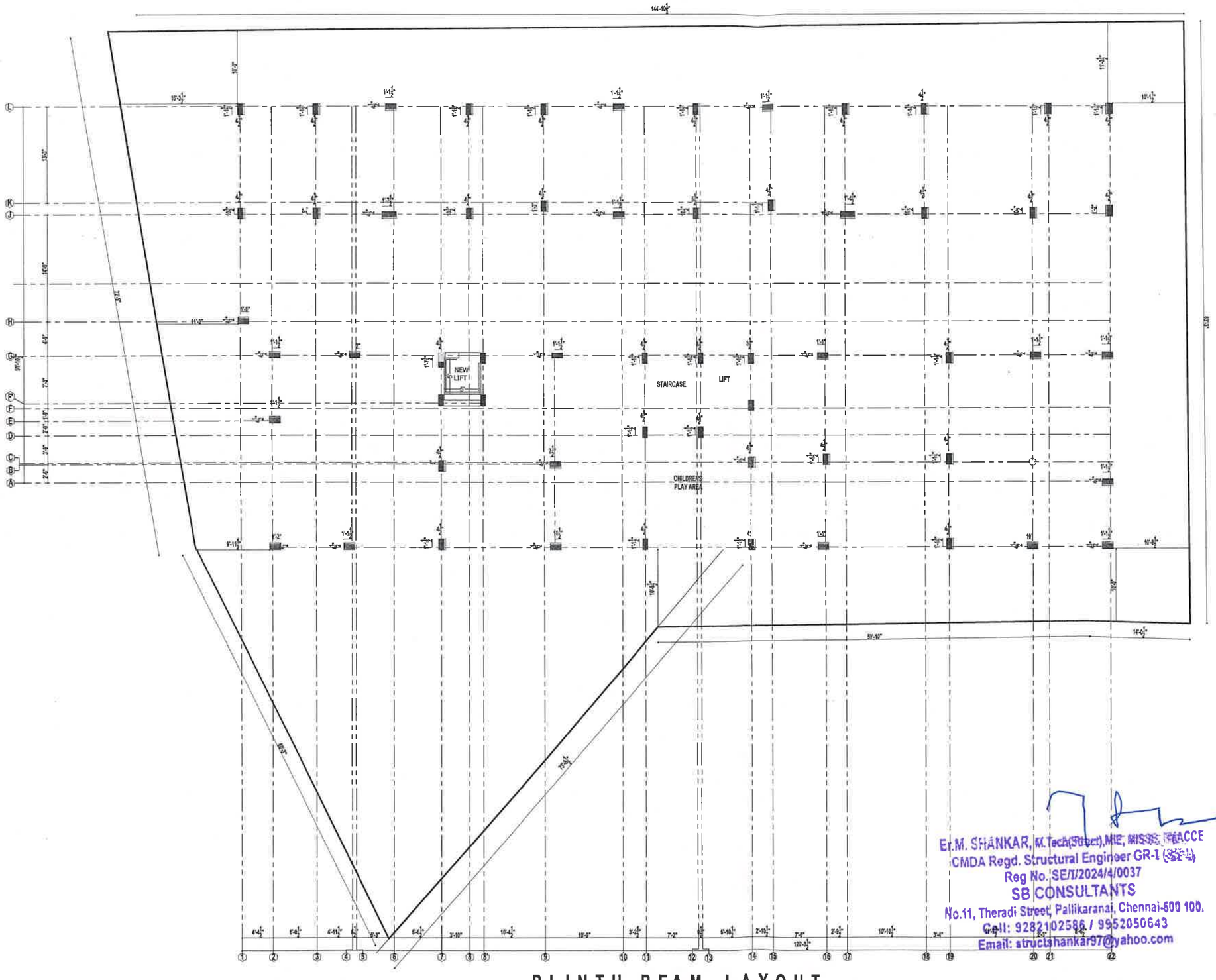
DEVELOPERS:

'M/S AVITTAM HOMES'
PLOT NO.84,5 TH CROSS STREET,
SHANKAR NAGAR,PAMMAL,
CHENNAI-600075.

STRUCTURAL CONSULTANTS:

Er.M.SHANKAR, M.Tech(STRUCT),MIE,C.ENG(IND),MISSE,FMACCE(I)
CHARTERED ENGINEER/STRUCTURAL ENGINEER
SENIOR PROFESSIONAL ENGINEER
SB CONSULTANTS,
11,THERADI STREET,
PALLIKKARANAI.
CHENNAI-600100.
PH:65419436
MOBILE:9282102586.
e-mail:m_shankarengineer@yahoo.co.in

Er.M. SHANKAR, M.Tech(Struct),MIE, MISSE, FMACCE
CMDA Regd. Structural Engineer GR-I (SE-I)
Reg No. SE/I/2024/4/0037
SB CONSULTANTS
No.11, Theradi Street, Pallikaranai, Chennai-600 100.
Cell: 9282102586 / 9952050643
Email: structshankar97@yahoo.com



PLINTH BEAM LAYOUT

PROJECT:

PROPOSED RESIDENTIAL APARTMENTS AT PLOT NO.19,20&21,SURVEY NO. 90/1B1 A2,90/1B1A3&90/1B1B,90/1B1C, 11 TH WEST STREET, METHA NAGAR JAI NAGAR EXTENSION,KUNDRATHUR,CHENNAI-600069

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R.NO.	REVISION	DATE	SIGN
1.			
2.			
3.			

PROJECT NO:

DRG TITLE: **COLUMN MARKING LAYOUT**

STRUCTURAL DRAWING

DRG.NO : SO-01 ONLY.

SCALE : 1"=4'-0" & 1"=2'-0"

DATE : 21.1.2024

SHEET SIZE : A1 | 1/1

CAD FILE: E\SHANKAR\2023\AVITTAM HOMES\ KUNDRATHUR\FDN

GOOD FOR CONSTRUCTION

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empowering your real estate decisions with smart tools

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Zoning & Land Use Analysis

Land Use Zone Finder, CRZ Zone Check, Aquifer Zone Check, CBA Check, EWS Check, Road Width Check, Proposed Road Widening Check, FSI Calculator

Property Documentation & Workflow Tools

Forms & Templates, Document Generator, Document Translation, Dictionary

Vastu & Property Management Tools

Home Vastu Shastra, Plot Vastu Shastra, Property Management

Verified RealEstate™

We're honored to be part of your real estate journey

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

hello@verified.realestate
+91 9996 12 9996
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