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GRANDE SMITH PRIVATE LIMITED, CHENNAI.

LORDS ENGINEERING (STRUCTURAL DIVISION)

STRUCTURAL AND CIVIL ENGINEERING CONSULTANTS

H.O: No. 68, Madhavaram Redhills Highroad, Moolakadai, Chennai – 600 060.

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**STRUCTURAL STABILITY DESIGN BASIS
CALCULATION REPORT**

**FOR PROPOSED CONSTRUCTION OF RESIDENTIAL
BUILDING (STILT+05).**

OF

OWNER: M/s NEXA INFRA PROJECTS

AT

**AT NO.3, A.N.A. ANNAMALAI AVENUE, ATHIPATTU,
AMBATTUR, CHENNAI-53.**

12-04-2023

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STRUCTURAL STABILITY CERTIFICATE

I Hereby Certify That the Proposed Residential Building at **No:3, A.N.A. Annamalai Avenue, Athipattu, Ambattur, Chennai-53.**

Designed and Checked for Dead Load and Live Load and Also to Resist Earthquake Forces and Found Safe.

The following assumptions and basic values are considered for the design and analysis of the structure under the title of the project.

1. The least dimension of the column for Stilt floor is **300 mm**, and for other columns is **230mm**.
2. The grade of concrete for structural elements is **M30**.
3. Safe Bearing Capacity of soil is **80 MT** as per soil report.
4. The proposed type of foundation is **FRICTION CUM ENDBEARING PILE**.
5. The depth of foundation as per soil report is **22.00 m** below existing ground level.
6. The design, analysis, detailing and drafting is carried out using the various code of practice namely and as per **IS 875** for Loads and loading combinations, **IS 1786** for High Tensile Reinforcement bar, **IS 456** for Practice of plain and reinforced concrete and **IS 1893** for Seismic design. Etc.
7. The **Registered Construction Project Engineer** at site shall take necessary precautions in adhering to the relevant Code provisions while executing the above work.



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NOTE:

The structural design is prepared based on the copy of Approval Plan of the building. Any change/alteration/ addition/ modification of the building plan in further Approval or during construction hereafter, it should be brought to the knowledge of structural engineer and work shall be carried out with new structural design report by the engineer.



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CHENNAI METROPOLITAN DEVELOPMENT AUTHORITY
'Thalamuthu Natarajan Building', No. 1, Gandhi Irwin Road,
Egmore, Chennai-600 008

PROFESSIONAL REGISTRATION CERTIFICATE

This is to certify that **Thiru. VIJAYAN. S** has enrolled as **Registered Structural Engineer Grade-I (SE)** with Chennai Metropolitan Development Authority (CMDA) who has the qualification of **M.E., (Structural Engg.)** and experience in the relevant field for **5 years**.

He has been given the Registration No.**SE/GR-I/2022/11/346** on **08.11.2022**. He 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 **07.11.2027**.



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[Signature]
(A. MEGANATHAN)
Superintendent

STRUCTURAL DESIGN ANALYSIS FOR PROPOSED BUILDING

Name of work : No:3, A.N.A. Annamalai Avenue, Athipattu, Ambattur,
Chennai-53.

Preamble :

8. I hereby certify that the structural design is done by Limit State Method, as per relevant sections of IS 456.
9. I hereby certify that the structural design sheet is done for critical column, beam, slab and foundation for the proposed building.
10. The usage of the building is Car Parking and residence in Stilt floor and residential in other 3 Floors.
11. The building consists of For Proposed Construction of Third Floor & The Existing Stilt, First & Second Floor of the Residential Building.
12. The superstructure is built on 230 mm thick outer walls and 115 mm thick inner walls.
13. The structural design includes the earthquake Criteria as per the IS 1893.

IS Code :

- IS 456 for concrete
IS 875 for design load
IS 1786 for steel reinforcement
IS 1893 for earthquake design
IS 2911 for pile design



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General Notes on Structural Design

General:

Design Basis

Design Of The Building Is Carried Out By RCC Frame Analysis Using Staad Pro Software And Critical Members Are Checked And Designed By Manual Calculation As Enclosed Below.

Codes and Standards

All Structural design is carried out confirming to the provisions in National Building Code of India and other relevant Indian Codes and Standards.

I.S. Code used for design purposes were:

IS 456:2000 for Concrete

IS 875:2007 for Design Load

IS 1786: 2008 for Steel Reinforcement

IS 1893:2002 for Earthquake design

Concrete:

The concrete shall be in grades designated as per Table 5 of IS 456-2000. Recommended grades for the different members is as follows:

Foundation	M30
Columns	M30
Roof beams and slabs	M30
Pile	M30

Material, Workmanship, Inspection and testing:

The Proposed RCC Structure will consist of cement concrete and reinforcement as the main material used for construction of the structure. The specifications for these materials are discussed below.



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Design methods:

General Design and Detailing of concrete members / elements are done confirming to the provisions of I.S. 456 and other relevant IS codes. Limit State Method is followed for the design.

Seismic Zone III load is taken into account for the analysis. The predominant direction of vibration is horizontal. Relevant combination of forces has been taken into account.

Minimum Cover:

Slab	20mm
Beam	25mm
Columns	40mm
Pile	75mm



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DESIGN OF RECTANGULAR COLUMN UNDER BIAXIAL BENDING GRID D3

Depth of the column (D)	600	mm
Width of the column (b)	300	mm
Unsupported length (Longer direction) L_x	3	m
Unsupported length (Shorter direction) L_y	3	m
Effective length factor (Longer direction)	1	
Effective length (Longer direction) L_{ex}	3	m
Effective length factor (Shorter direction)	1	
Effective length (Shorter direction) L_{ey}	3	m
Factored axial load P_u	2542	KN
Factored moment (Longer direction) M_{ux1} (Top)	175	KNm
Factored moment (Shorter direction) M_{ly1} (Top)	166	KNm
Factored moment (Longer direction) M_{ux2} (Bottom)	175	KNm
Factored moment (Shorter direction) M_{ly2} (Bottom)	166	KNm
Clear cover	40	mm
Compressive strength of concrete f_{ck}	30	N/mm ²
Grade of steel f_y	500	N/mm ²
d'	60.5	mm

Braced/Unbraced and Double curvature

Braced =	NO	If yes = "YES", If no = "NO"
Bent in double curvature =	NO	If yes = "YES", If no = "NO"

Check for slenderness

Longer direction

Longer direction slenderness ratio L_{ex}/D	5.00	
	Short	
Additional moment due to slenderness M_{ux}	0.00	KNm

Shorter direction

Shorter direction slenderness ratio L_{ey}/b	10.00	
	Short	
Additional moment due to slenderness M_{ly}	0.00	KNm

Slenderness limit check



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60B = 18.00 m
OK

$100 \cdot b^2 / D = 15.00$ m
OK

Determination of P_{uz}

Assumed percentage of reinforcement P_t	3	%
Gross area of the column A_g	180000	mm ²
Area of steel reinforcement A_{sc}	5400	mm ²
Area of concrete A_c	174600	mm ²
Axial load capacity of the column P_{uz}	4382.1	KN

Determination of M'_{ax} and M'_{ay} (Only for slender columns)

Longer direction

d'/D	0.101	
Coefficient k_{1x}	0.207	
Coefficient k_{2x}	0.172	
Axial load corresponding to balanced failure P_{bx}	1209.61	KN
Reduction factor for additional moments k_x	0.580	
Moment due to slenderness M'_{ax}	0.000	KNm

Shorter direction

d'/b	0.202	
Coefficient k_{1y}	0.184	
Coefficient k_{2y}	-0.001	
Axial load corresponding to balanced failure P_{by}	993.06	KN
Reduction factor for additional moments k_y	0.543	
Moment due to slenderness M'_{ay}	0.000	KNm

Check for moments due to minimum eccentricity

Longer direction

Minimum eccentricity e_x	26.00	mm
Moment due to minimum eccentricity M_x	66.09	KNm



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Moment in longer direction M_{ux}	175.00	KNm
Shorter direction		
Minimum eccentricity e_x	16.00	mm
Moment due to minimum eccentricity M_y	0.00	KNm
Moment in shorter direction M_{uy}	166.00	KNm

Design moments

If braced

Moment in longer direction M_{ux}	175	KNm
OK		
Moment in longer direction M_{uy}	166	KNm
OK		
Design moment in longer direction M_{dx}	175.000	KNm
Design moment in shorter direction M_{dy}	166.000	KNm

If unbraced

Design moment in longer direction M_{dx}	175.00	KNm
Design moment in shorter direction M_{dy}	166.00	KNm

Check with interaction equation

Longer direction

For $d'/D =$	0.101	
$p/f_{ck} =$	0.100	
$P_u/f_{ck}bD =$	0.471	
$M_{ux1}/f_{ck}bD^2 =$	0.140	
Design moment capacity (longer direction) M_{ux1}	453.60	KNm

Shorter direction

For $d'/b =$	0.202	
$p'f_{ck} =$	0.100	
$P_{u'}f_{ck}bD =$	0.471	
$M_{ux1}f_{ck}Db^2 =$	0.120	
Design moment capacity (shorter direction) M_{ux1}	194.40	KNm



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$P_u/P_{uz} =$	0.580
$\alpha_n =$	1.633
$[M_{ux}/M_{ux1}]^{un} + [M_{uy}/M_{uy1}]^{un} \leq 1.0$	0.983794876
OK	

Longitudinal Reinforcement Calculation

DIA OF LONGITUDINAL BAR-1	25	mm
DIA OF LONGITUDINAL BAR-2	20	mm
DIA OF LONGITUDINAL BAR-3	16	mm
DIA OF LONGITUDINAL BAR-4	12	mm
DIA OF LATERAL TIES	8	mm

CALCULATION OF NO OF REBARS REQUIRED

	REQ NOS	PROV NOS
LONGITUDINAL BAR-1	11.00	10
LONGITUDINAL BAR-2	17.19	5
LONGITUDINAL BAR-3	26.86	0
LONGITUDINAL BAR-4	47.75	0

DIAMETER OF LONGITUDINAL REBARS USED

LONGITUDINAL BAR-1	25	mm
LONGITUDINAL BAR-2	20	mm
LONGITUDINAL BAR-3	NOT USED	mm
LONGITUDINAL BAR-4	NOT USED	mm

CALCULATION OF REBAR AREA (As)

Required (mm²)	Provided (mm²)
5400.00	6479.53

Transverse Reinforcement Calculation

Diameter of lateral ties

i) Not less than 1/4 th of the diameter of the largest longitudinal bar	6.25	mm
---	------	----



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ii) Not less than 6 mm

6

mm

Spacing of lateral ties

i) Least lateral dimension

300

mm

ii) 16*Smallest diameter of the longitudinal bar

320

mm

iii)

300

mm

Provide minimum value of the three as spacing

Provided rebar

8

mm

OK

Provided spacing

190

mm

OK



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DESIGN OF SINGLY REINFORCED BEAM - GRID E1-E5

Thickness of the slab	150	mm	Self-weight of slab	3.75	KN/m
Beam clear span (L)	5.61	m	Live Load	3	KN/m
Beam effective span (L_{eff})	5.84	m	Floor finish load	1.5	KN/m
Slab width - Beam length/2	2.805	m			
Slab width from the plan	4.5	m			
a) Load from the slab	23.14125	KN/m			
Assumed/Calculated					
b) load from the slab	37.125	KN/m			
Load from the slab considered	0	KN/m			
Floor height	3	m			
Beam depth (D)	450	mm			
Breadth of beam (b)	300	mm			
Beam self-weight/m	2.25	KN/m			
Wall thickness	230	mm			
Wall load/m	12.42	KN/m			
UDL on the beam	14.67	KN/m			
Factored UDL on beam	22.0	KN/m			
1) Bending moment	93.81	KNm			
2) Bending moment	62.54	KNm			
Bending moment considered	93.81	KNm			
Grade of concrete	25	N/mm ²			
Grade of steel	500	N/mm ²			
Clear cover	25	mm			
Effective depth (d)	417	mm			
d'	33	mm			
d'/d-	0.079	mm			
	OK				

Main Reinforcement calculation:



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DIA OF TENSION	16	mm
DIA OF COMPRESSION	16	mm
$X_u \text{ max} =$ $[(d*0.0035)/(0.0055+0.87*f_y/E_s)]$ mm		
$X_u \text{ max} =$	190.16	mm
$M_u \text{ lim} = [0.36*f_{ck}*b*X_u \text{ max}(d-0.416*X_u \text{ max})]$ mm		
$M_u \text{ lim} =$	173.49	KN.m
$X_u = (1.2-\text{SQRT}(1.44-((6.68*M_u)/(f_{ck}*b*d^2))))*d$ mm		
$X_u =$	91.93	mm

SINGLY REINFORCED	Note: Design as doubly or singly reinforced beam	
$A_{st} = M_u/(0.87f_y(d-0.416*X_u))$	mm	
$A_{st} =$	569.39	mm ²
$A_{st} \text{ lim} =$	1180.32	mm ²
$f_{sc} =$	417.01	N/mm ²
$f_{cc} =$	11.15	N/mm ²

CHECK FOR MIN & MAX TENSION REBAR AREA

1)			
a)	Min	212.67	mm ²
b)	Max	5400	mm ²

CALCULATION OF REBAR AREA (Ast)

Location	Required	Provided	
BOTTOM	569.39	602.88	mm ²
TOP	-	-	mm ²

CALCULATION OF NO OF REBARS REQUIRED

	REQ NOS	PROV NOS	Dia of combination bar	NOS
TENSION BAR NOS	2.83	3	0	0
HANGAR BAR NOS	-	3	0	0

Note: If required provide two layers for tension reinforcement. For eg 3-25 T + 2-12 T

$P_t \text{ lim} =$	$(100 A_{st} \text{ lim})/b*d$	%		
$P_t \text{ lim} =$	0.944	%		
$P_t \text{ prov} =$	0.482	%		

$P_c \text{ prov} =$

0.4819 %



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OK

Pt prov < Pt lim = OK

CHECK FOR DUCTILITY

Xu actual =97.13mm

OKXu actual < Xu max = OK

est =0.01153

OKest > 0.005 = OK

CALCULATION OF MOR FOR SRB

Mu =98.82KNm

OKMOR > Mu = OK

CALCULATION OF MOR FOR DRB

Note: As we provide two hangar bars, it will act as a doubly reinforced beam

Xu =6.51mm

OKXu < Xu max = OK

Mu =101.24KNm

OKMOR > Mu = OK

SHEAR REINFORCEMENT CALCULATION:

Vu=(Wu L/2)kNFor simply supported and fixed beam

Vu=64.255kN

Pt=(100 Ast)/b*d%

Pt=0.482%

Tv=(Vu)/(b*d)

Tv=0.514N/mm²

Tc=0.496N/mm²

REFER TABLE 19

λ (Pt prov)=0.482If Pt>3 interpolation not required

λ10.75

λ21

γ10.56

γ20.62



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τ_c max=	2.8	N/mm ²	REFER TABLE 20
	OK		$T_v < T_c \text{ max} = \text{OK}$
No of legs	2		
Dia of stirrups	8	mm	
f_y =	415	N/mm ²	NOTE: f_y should not be
A_{sv} =	100.48	mm ²	greater than 415 N/mm ²
V_{uc} =	62.007	kN	
V_{us} =	2.247	kN	

- | | | |
|--------------|-----------------|-----------------------|
| 1) CONDITION | IF: $T_v > T_c$ | SHEAR DESIGN REQ |
| 2) CONDITION | IF: $T_v < T_c$ | PROVIDE MINIMUM SHEAR |

MINIMUM REINFORCEMENT FORMULA

$$(A_{sv}/b * S_x) \geq (0.4/0.87 f_y)$$

S_v =	$(0.87 * f_y * A_{sv} * d) / (V_{us})$	mm
S_x =	6731.12	mm

CHECK FOR MINIMUM SHEAR AREA

A_{sv} =	2237.18	mm ²
------------	---------	-----------------

Spacing Condition

1	6731.12	mm
2	312.75	mm
3	300	mm

Provide minimum value of the three as spacing

Provided spacing	140	mm	
REQUIRED A_{sv}	2.09	mm ²	OK
PROVIDED A_{sv}	100.48	mm ²	
BAR DIA	8	mm	
LEG	2	NOS	
SPACING	140	mm C/C	

CHECK FOR DEFLECTION



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1) $\Delta = (5WL^4)/(384EI)$	3.901	mm
1)	span 250	23.36 mm
2)	span 350	16.68571429 mm
	or	20 mm
Minimum	16.68571429	mm
	OK	
	$E_c = 5000 \cdot \text{SQRT}(f_{ck})$	25000.00 N/mm ²
	$I = (bD^3)/12$	2278125000 mm ⁴
2) $f_s = 0.58 \cdot f_y \cdot (\text{Area of steel required} / \text{Area of steel provided})$		N/mm ²
$f_s =$	273.89	N/mm ²
$kt = 1 + (0.225 + 0.00322 \cdot f_s - 0.625 \cdot \log_{10}(l/d))$		
$kt =$	1.10	
$kc = 1 + (pe/pe + 3)$		
$kc =$	1.14	≤ 1.5
$(l/d)_{\max} =$	25.05	
Actual $(l/d) =$	14.00	
OK		

TABLE F STRESS IN COMPRESSION REINFORCEMENT f_{sc} , N/mm² IN DOUBLY REINFORCED BEAMS WITH COLD-WORKED BARS
(Clause 2.3.2)

f_y , N/mm ²	d'/d			
	0.05	0.10	0.15	0.20
415	355	353	342	329
500	424	412	395	370

Table 20 Maximum Shear Stress, $\tau_{c, \max}$, N/mm²
(Clauses 40.2.3, 40.2.3.1, 40.5.1 and 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35
$\tau_{c, \max}$, N/mm ²	2.5	2.8	3.1	3.5	3.7



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Table 19 Design Shear Strength of Concrete, τ_v , N/mm²
(Clauses 40.2.1, 40.2.2, 40.3, 40.4, 40.5.3, 41.3.2, 41.3.3 and 41.4.3)

$100 \frac{A_s}{bd}$	Concrete Grade					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤ 0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.50	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.66	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.00 and above	0.71	0.82	0.92	0.96	0.99	1.01

NOTE — The term A_s is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at support where the full area of tension reinforcement may be used provided the detailing conforms to 26.2.2 and 26.2.3



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ANALYSIS & DESIGN OF 3-PILE CAP
INPUT DATA FOR FOUNDATION

NGL TO FGL =0.75 m
Projection of pedestal above FGL =0.30 m
Founding lvl (top-pdstl-pile cap) D =22.00 m
Density of soil γ_s =14 KN/m³
Bearing capacity factor = 9.00
Clear cover d = 75mm

COLUMN SIZE = 230 mm X 600 mm
DEPTH OF PILE CAP = 1.10 m

TYPE OF PILE			-	Friction cum End bearing
DIAMETER OF PILE			-	500mm

CAPACITY OF PILE				
Compression (pile capacity)			-	80.00 t
Horiz. Thrust			-	10.50 t
Up lift			-	54.00t

Seismic zone			-	0.16	III
Wind pressure			-	50	m/s
Soil-foundation coeff			-	1	
Importance factor			-	1.5	

PILE CAP DIMENSIONS



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All load are in KN-m								
Sl NO	PILE	Load comb	F _y	F _x	F _z	MX	MZ	
1	C1	DL+LL	100	1672.269	-8.474	23.134		
			100	1187.493	-38.872	0.966		
			100	2051.957	0.67	21.219		
			100	1113.671	-2.322	-18.61		
			100	2075.089	24.113	17.138		
			100	-167.953	16.172	-41.752		
			100	143.373	30.948	-23.752		
			100	3647.764	16.305	25.478		
			100	1309.058	19.73	-3.278		
			100	1460.867	28.871	4.71		
			100	1760.818	37.865	1.727		
			100	1490.97	-43.98	-8.584		
			100	2287.403	-87.365	-27.307		
			100	1602.893	6.339	28.91		
				0				
				0				
				0				
Maximum forces of wl/sl comb			465.469	23.053	46.964	0	0	
Minimum forces of wl/sl comb			-167.953	-87.365	-41.752	0	0	
DL+LL			1672.269	-8.474	23.134	0	0	

No of piles = 1.760 DL+LL -COMB

No of piles = -0.736 DL+LL+WL/SL-COMB

FOR PEDESTAL DESIGN ONLY				
FOOTING	CASE	FY	MXX	MZZ
C1	100	1672.269	20.8206	-7.6266
	101	1187.493	0.8694	-34.9848
	102	2501.957	19.0971	0.603
	103	1113.671	-16.749	-2.0898
	104	2075.089	15.4242	21.7017
	105	-167.953	-37.5768	14.5548
	106	143.973	-21.3768	27.8532
	107	3647.764	22.9302	14.6745
	108	1309.058	-2.9502	17.757
	109	1460.867	4.239	25.9839
	110	1760.818	1.5543	34.0785
	111	1490.97	-7.7256	-39.582
	112	2287.403	-24.5763	-78.6285
	113	1602.893	26.019	5.70551




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ANALYSIS

Comp	lateral	pull
out		
safe	safe	safe
0	0	0
0	0	0
0	0	0
0	0	0
0	safe	0
0	0	0
0	0	0
safe	0	0
0	0	0
0	0	0
0	0	0
0	0	0
safe	0	0

SI.NO	PILE	Load comb		fy	fz	MX	fx	MZ
1	C1	DL+LL	100	1672.269	23.134	0.00	-8.474	0.00
			100	1187.493	0.966	0.00	-38.872	
			100	2051.957	21.219	0.00	0.67	
			100	1113.671	-18.61	0.00	-2.322	
			100	2075.089	17.138	0.00	24.113	
			100	-167.953	-41.752	0.00	16.172	
			100	143.373	-23.752	0.00	30.948	
			100	3647.764	25.478	0.00	16.305	
			100	1309.058	-3.278	0.00	19.73	
			100	1460.867	4.71	0.00	28.871	
			100	1760.818	1.727	0.00	37.865	
			100	1490.97	-8.584	0.00	-43.98	
			100	2287.403	-27.307	0.00	-87.365	

	DIA	NOS	Vu/d
--	-----	-----	------



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Capacity of pile

Vertical	P_{pv}	800 KN
Lateral	P_{ph}	95 KN
Pullout	P_{pp}	540 KN
Increment factor for wind loads	f_{inew}	1.00
Increment factor for seismic forces	f_{ines}	1.00
Increment for current load case	$f_{inch} =$	1.00

Calculations:

$$M_{xh} = 37.01 \quad 37.0144$$

$$M_{zh} = -13.6 \quad -13.5584$$

Vertical loads

Area of pile cap	$A_{pc} = 1/2 * 1 * b$	3.91468 m ²
	$A_{pc_{net}} = (A_{pc} - A_p)$	1.35 m ²
Soil weight W_{soil}	$DF * YS * A_{pc_{net}}$	54.00 KN
Overburden Wover	$w_{over} * A_{pc_{net}}$	6.75KN
Self-weight of pile cap	$W_{swpc} = A_{pc} * D_{pc} * 25$	25.3125KN
Self-weight of pedestal	$W_p = A_p * (d_{forcb} - D_{pc}) * 25$	8.44KN
Total	$\sum W$	94.50 KN
Vertical load	$P_v = \sum f_y + \sum W$	1766.77 KN

CG of load system

M	$\sum (F_y z)$	0.00
M	$\sum F_y * d_{forct}$	23.13
M	$\sum M_x$	0.00
M	$M_{xv} + M_{xh} + M_{xm}$	23.13 KNm
M	$\sum (F_y x)$	0.00
M	$\sum F_x * d_{forct}$	-8.47
M	$\sum M_z$	0.00




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M	$M_{xy} + M_{xh} + M_{xm}$	-8.47 KNm
e_x	M_{ze}/p_x	0.00 m
e_z	M_{xe}/p_x	0.01 m
Final Moment about X axis M_{xf}		23.134 KNm
Final Moment about z axis M_{zf}		-8.474 KNm

Pile forces

Sl NO	Pile design	x (m)	z (m)	P_v/n (KN)	F_p (KN)
1	P1	-0.75	0	220	220.000
2	P2	0.75	0	220.5	220.500

$$\sum F_p = 440.500$$

$\sum X^2$	1.125	Max	220.5 KN
$\sum Z^2$	0	Min	220.500 KN

Check for compressive force

Max force in pile	220.50 KN	safe
Permissible compressive force	$F_{inc} * P_{pv}$	220.50 KN

Check for lateral force

$$\sum H_x = -8.474 \text{ KN}$$

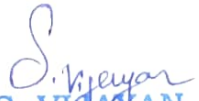
$$\sum H_z = 23.134 \text{ KN}$$

Resultant	$\text{sqrt}(\sum H_x^2 + \sum H_z^2)$	24.64 KN
Lateral force per pile	H/n	12.32 KN safe
Permissible lateral force	$F_{inc} * P_{ph}$	95 KN

Check for lateral force since all piles are in compression, there is no uplift force in the piles

Minimum compressive force	220.00 KN	safe
Permissible Tensile force	$F_{inc} * P_{pp}$	337.50 KN




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Analysis of 3 pile group

Pedestal size

Width $B = 300 \text{ mm}$

strength of concrete $f_{ck} = 30 \text{ N/mm}^2$

Depth $D = 750 \text{ mm}$

strength of steel $f_y = 550 \text{ N/mm}^2$

Height $h_{ped} = 0 \text{ m}$

Analyzing the Dia of main bar

20 mm 16 mm

Clear cover to the tie Reinf.

75 mm

Effective cover $75 + 20/2$

85 mm

Depth of pile cap deff

1020 mm

*****Pile cap

Overburden pressure

Due to grade slab

$= 0.00 \text{ KN/m}^2$

Due to soling

$= 0.00 \text{ KN/m}^2$

Due to live load

$= 0.00 \text{ KN/m}^2$

Soil weight of pile cap

$= 45 \text{ KN/m}^2$

Self-weight of pile cap

$= 15 \text{ KN/m}^2$

Total pressure

$= 60.00 \text{ KN/m}^2$

Depth of soil filling

$= 2.50 \text{ m}$

Unit wt. Of soil

$= 14.00 \text{ KN/m}^2$

Thickness of pc

$= 0.60 \text{ m}$

Pile diameter

$= 500 \text{ mm}$

Mex pile occur in pile

$= 250 + 250$

Pile from the face of pedestal

$= 600 \text{ mm}$

Distance from the face of col to edge of pc

$= 975 \text{ mm}$

Face of the pedestal

$= (250.00) \times 0.6 - 60 \times 2.25 \times 0.975^2/2$

$= 85.8328 + 0.00$

Mu

$= 85.8328 \text{ KN-m}$

*****Moment due to torsion:

Horizontal load in critical direction

$= 5 \text{ KN}$



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Torsion	T_u	$= 4.5 \text{ KN}$
Equivalent BM due to torsion	M_e	$= \frac{\tau_{tu} (1 + \frac{D}{b})}{1.7}$
	M_e	$= 4.76 \text{ KN-m}$
Total BM		$= M_u + M_e$
		$= 85.8 + 4.76$
		$= 90.60 \text{ KN-m}$

Shear force at d/2 from face of pedestal

Critical shear section $= 257.5 \text{ mm}$ from face of column

Shear force at critical section $= (250.00) \times 1.000 - 60 \times 2.25 \times 0.72$

$= 153.138 \text{ KN}$

$= (\text{As per cl 34.2.4.1 is 456:2000})$

Factor $= 1.00$

**** Shear due to torsion:

Horizontal load in critical direction	$= 5 \text{ KN}$
Torsion	$T_u = 4.5 \text{ KN}$
Equivalent shear due to torsion	$V_e = 1.6 T_u / b$

**** SHEAR FORCE

$V_e = 9.60 \text{ KN-m}$

$= V_u + V_e$

$= 153.1375 + 9.6$

$= 162.74 \text{ KN}$

factor	1.5	1.5	
Moment	181.20	181.20	181.20 KNm/m-width
shear	216.98	216.98	216.98 KN/m-width
M_u	$= 181.20$		
	$= 181.20$	KNm	



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$$M_u / bd^2 = 181.20 / (1000 \times 515^2)$$

$$= 0.68 \text{ N/mm}^2$$

$$P_c = 0.20 \%$$

$$P_c = 0.109 d' / d = 0.17$$

***** Reinforcement

$$A_{st} = 1030.00 \text{ mm}^2$$

$$\text{Required Spacing} = 905.01 \text{ mm} \quad 314.159$$

$$\text{Provide } 16\text{DIA } 8\text{Nos} = 3.2786$$

Hence provide	16DIA	8 Nos
---------------	-------	-------

***** reinforced beam provide minimum reinforcement otherwise provide P_c of doubly reinforced beam

Top Reinforcement

$$\text{Providing minimum \% of steel} = 0.12 \% \text{ or } 0.109$$

$$A_{st} = \frac{0.12 \times 1000 \times 600}{100}$$

$$= 0.12$$

$$= 720 \text{ mm}^2$$

$$\text{Required spacing} = 279.25 \text{ mm} \quad 201.062 \quad = 3.581$$

$$\text{Provide } 16 \text{ DIA } 4 \text{ Nos}$$

Hence provide	16DIA	4 Nos
---------------	-------	-------

Check for shear:

For column load:

$$\text{Maximum shear force} = 216.98 \text{ KN} \quad \text{load factor } 1.5$$

$$V_u = 1.5 \times 216.98$$

$$= 244.1 \text{ KN}$$

$$P_t \% = 0.20$$

$$\text{Value of } = V_u / d \quad t_c = 0.62$$

$$b_o = 750 \text{ mm}$$

Hence provide shear reinforcement

DIA=	10		LEG=	2
------	----	--	------	---



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Providing =2	LEG =10	DIA @ 150C/C
--------------	---------	--------------

DESIGN OF PILE

Length of pile $L = 22.00\text{m}$

Characteristic strength of concrete $f_{ck} = 30 \text{ N/mm}^2$

Diameter of pile $D = 60 \text{ cm}$

Length of pile cap $= 1.14 \text{ m}$

Depth of pile cap $= 1.1 \text{ m}$

Modulus of elasticity of concrete (E) $= 5000 \times f_{ck} = 5000 \times 30 \text{ N/mm}^2$
 [As per IS 456:2000 Pg .no 16]
 $= 150000 \text{ kg/cm}^2$

Moment of inertia of pile (I) $\pi \times D^4 \frac{3.14 \times 60^4}{64} \text{ cm}^4$
 $= 635850.0 \text{ cm}^4$

K_1 (constant) $= 0.146 \text{ kg/cm}^2$

[As per IS: 2911 (part 1/sec 4)-appendix -B]

$T(\text{constant}) = 5(E \times I) \frac{5(E \times I)}{K_1}$

[AS per IS: 2911 (part 1/sec 4)- appendix -

B. B-1.2]

$= 255.50 \text{ cm}$

Hence $L/R = \frac{L}{R}$

[AS per IS: 2911 (part 1/sec 4)- appendix -B]

$= 6.262$

For free head pile $\frac{L}{R} = 1.85$

[AS per IS: 2911 (part 1/sec 4)- appendix -B. B-1.1 fig 2]

Length of fixity $L_f = 472.67 \text{ cm}$ [AS per IS: 2911 (part 1/sec 4)-

Appendix

-B]



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$$=4.73 \text{ m}$$

Embedded length

$$l_e > 4 \times R \quad \text{AS per IS: 2911 (part 1/sec 4)-}$$

Appendix -B. B-1.2]

$$>10.2 \text{ m}$$

Length of pile L_1

$$= L - L_e$$

$$= 5.8$$

Calculation of lateral capacity of pile

[AS per IS: 2911 (part 1/sec 4, B- 1.3 -appendix B)]

$$\text{Lateral capacity ,Q} = \frac{12 E I Y}{(L_1 + L_f)^3}$$

where,

$$Y = \text{lateral deflection pf pile} = 5 \text{ mm (As per IS 2911, part 4)}$$

$$\text{Modulas of elasticity of concrete (E)} = 250000 \text{ Kg / cm}^2$$

$$\text{Moment of inertia of pile (I)} = 635850.0 \text{ cm}^4$$

$$L_1 + L_f = 0 + 481.37$$

$$= 2072.7 \text{ cm}$$

$$= 20.73 \text{ m}$$

Lateral capacity

$$Q = \frac{12 \times 223606.8 \times 306640.6 \times 0.5}{481.37 \times 481.37 \times 481.37}$$

$$= 0.1 \text{ Mt}$$

$$= 1 \text{ KN}$$

DESIGN OF MAXIMUM MOMENT IN THE PILE

$$\text{Lateral load on each pile from staad } Q = 3.90 \text{ Mt}$$

$$\text{Lateral load taken by the pile cap (for 2 piles)} = 3 \times 18 \times 1.14 \times 0.6$$

$$= 3.96 \text{ Mt}$$

Considering the effect of passive pressure on piles

$$\text{Angle of repose of soil} \quad \phi = 30$$



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$$\text{Coeff of passive pressure } K_p = \frac{1 + \sin \phi}{1 - \sin \phi}$$

$$K_p = 3.00$$

$$\text{Unit weight of submerged soil } \gamma = 14 \text{ kN/m}^3 \quad \text{dwg}$$

$$\text{Shape factor } s = 0.70$$

$$\text{Force due to passive pressure} = K_p \cdot \gamma \cdot s \cdot h$$

$$\text{Equating the passive pressure to the shear force acting on } 3 \times 18 \times 0.7 \times 0.6$$

$$\text{The pile} \quad 2.68 \text{ h}$$

$$22.68 h x h / 2 = 2.67 t$$

$$= \sqrt{(2.6688 \times 10 \times 2)} / 22.68$$

$$h = 1.53 \text{ m}$$

$$\text{depth of pile} = 1.53 \text{ m}$$

$$22.68 \text{ h} = 34.79$$

$$= 22.68 h x h / 2$$

$$F = 26.69 \text{ KN}$$

$$\text{Maximum moment} = F \cdot h - F \cdot (1/3) \cdot h$$

$$= 27.29 \text{ KNm}$$

$$M_u = 4.09 \text{ t-m}$$

$$\text{Vertical load on each pile} \quad p = 46.00 \text{ t}$$

$$\text{Maximum vertical load on pile} \quad p_u = 1.5 x p$$

$$P_u = 69.00 \text{ t}$$

$$\frac{P_u}{f_{ck} D^2} = 0.08$$

$$\frac{M_u}{f_{ck} D^3} = 0.01$$

$$\text{Diameter of bar} = 16 \text{ mm}$$

$$\text{Clear cover} = 75 \text{ mm}$$

$$d' = 50 + (16/2)$$

$$= 68 \text{ mm}$$

$$\frac{d'}{D} = \frac{58}{600}$$



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$$= 0.11$$

DESIGN OF PILL

$$\frac{P_u}{f_{ck} D^2} = 0.08$$

$$\frac{M_u}{f_{ck} D^3} = 0.01$$

$$\frac{P}{f_{ck}} = 0.018 \text{ (As per SP -16, chart 60)}$$

Percentage of steel required (P) = 0.45 %

Min. percentage of steel required = 0.40 %

[AS per IS: 2911 (part 1/sec 2)-1979 pg -14]

Percentage of steel = 0.45 %

Area of steel $A_{st} = 1271.7 \text{ mm}^2$

Using 20 mm dia bar 4.04795 nos

Hence provide 20 mm dia bar 8 no's

provided $A_{st} = 2513.274 \text{ mm}^2$

check for shear

lateral load on pile (v) = 26.688 KN

Area of pile (A) = 282743.34 mm^2

Shear stress $\tau_v = V_u/A$

$$= 26.688 \times 1000 / 282743.34$$

$$= 0.094 \text{ N/mm}^2$$

100 $A_{st}/bd = 0.889$

$$\tau_{vc} = 0.850 \text{ N/mm}^2$$

SAFE (table 19 IS 456:2000 Page:73)

Provide min shear reinforcement

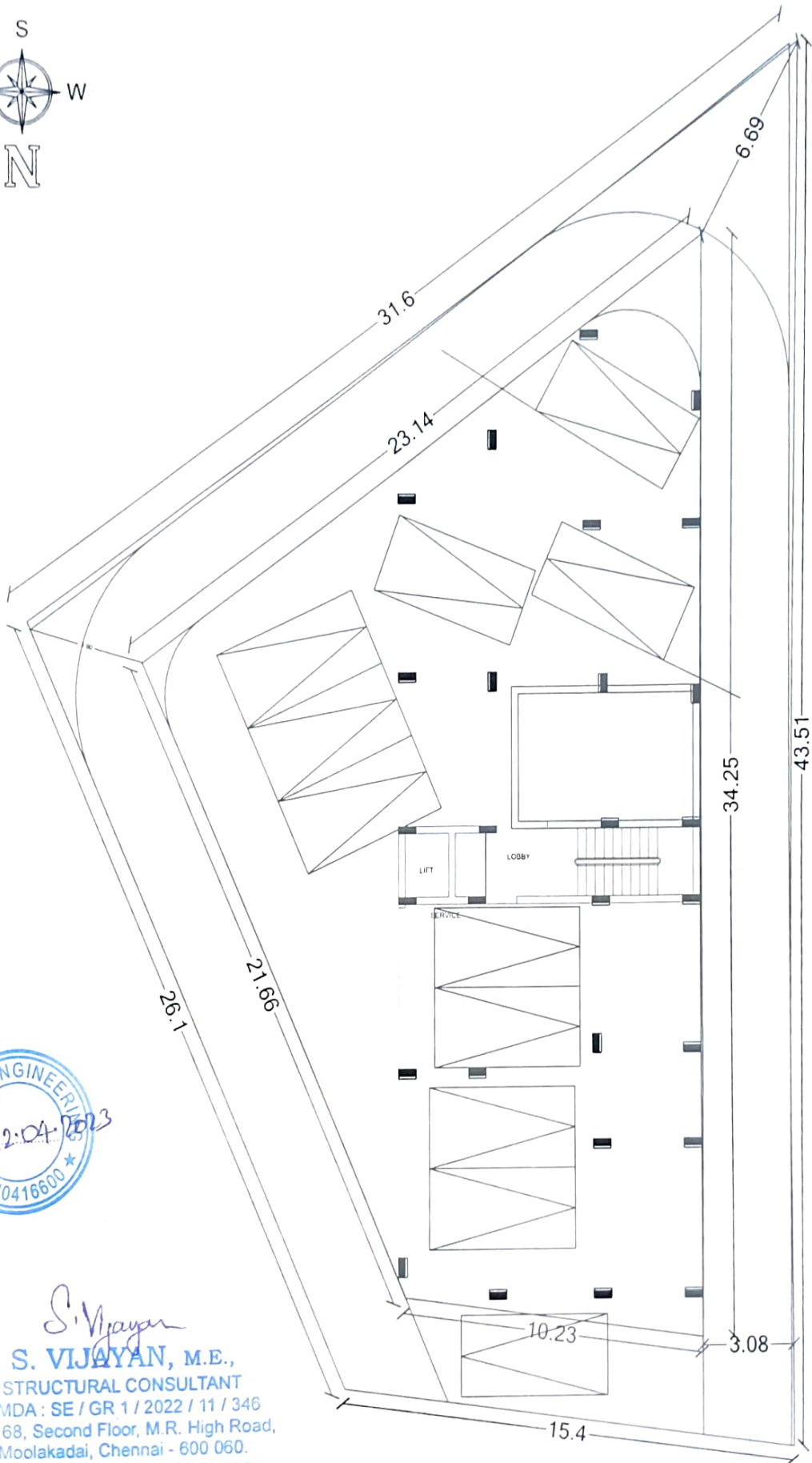
Hence provide 10 mm dia. rod ties at 200 mm c/c

[As per IS :2911(part / sec 2)-1979 pg-14]



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NEXA INFRA PROJECTS – RESIDENTIAL S+5 – AMBATTUR



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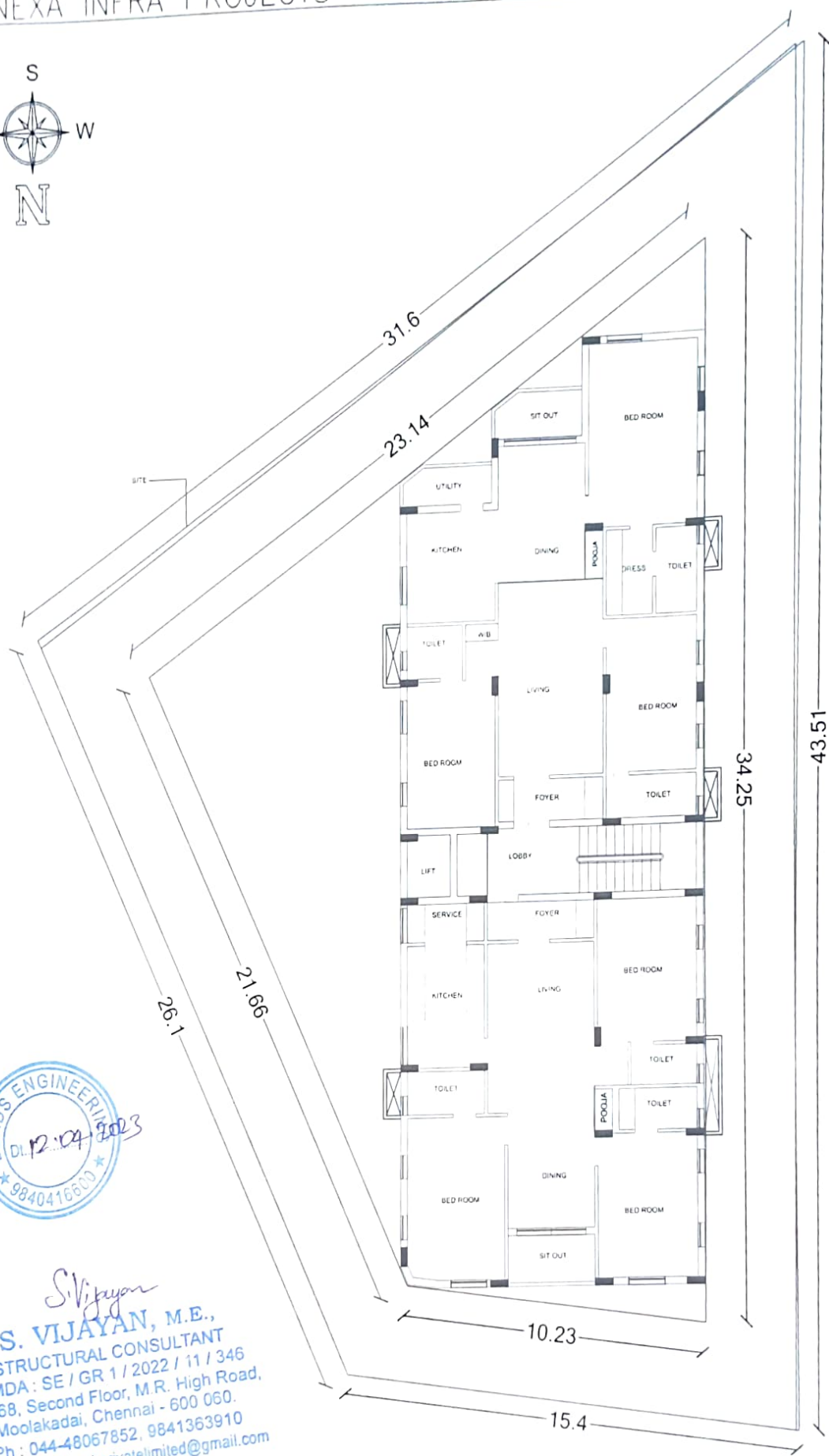
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STILT FLOOR PLAN

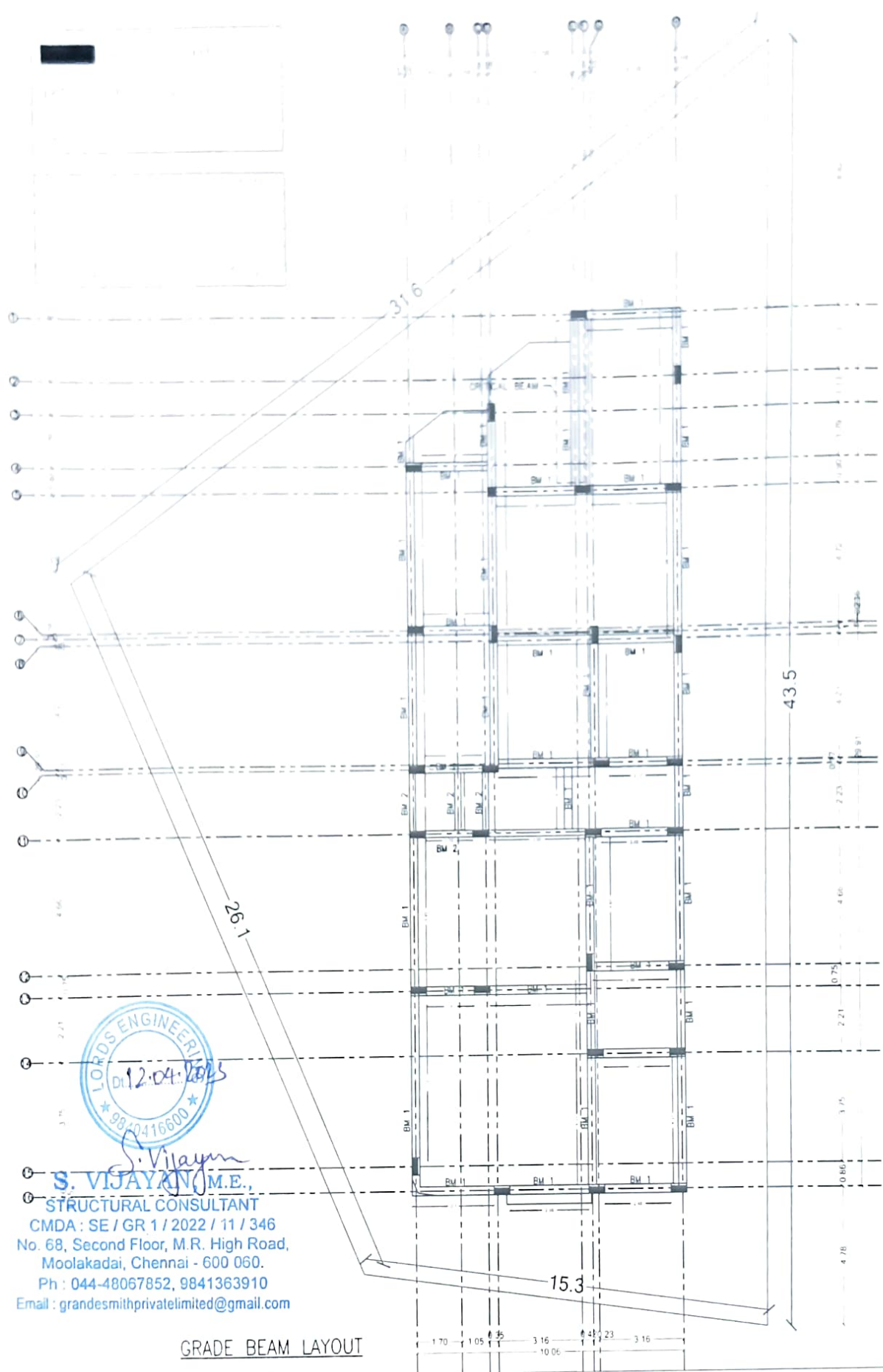
NEXA INFRA PROJECTS - RESIDENTIAL S+5 - AMBATTUR



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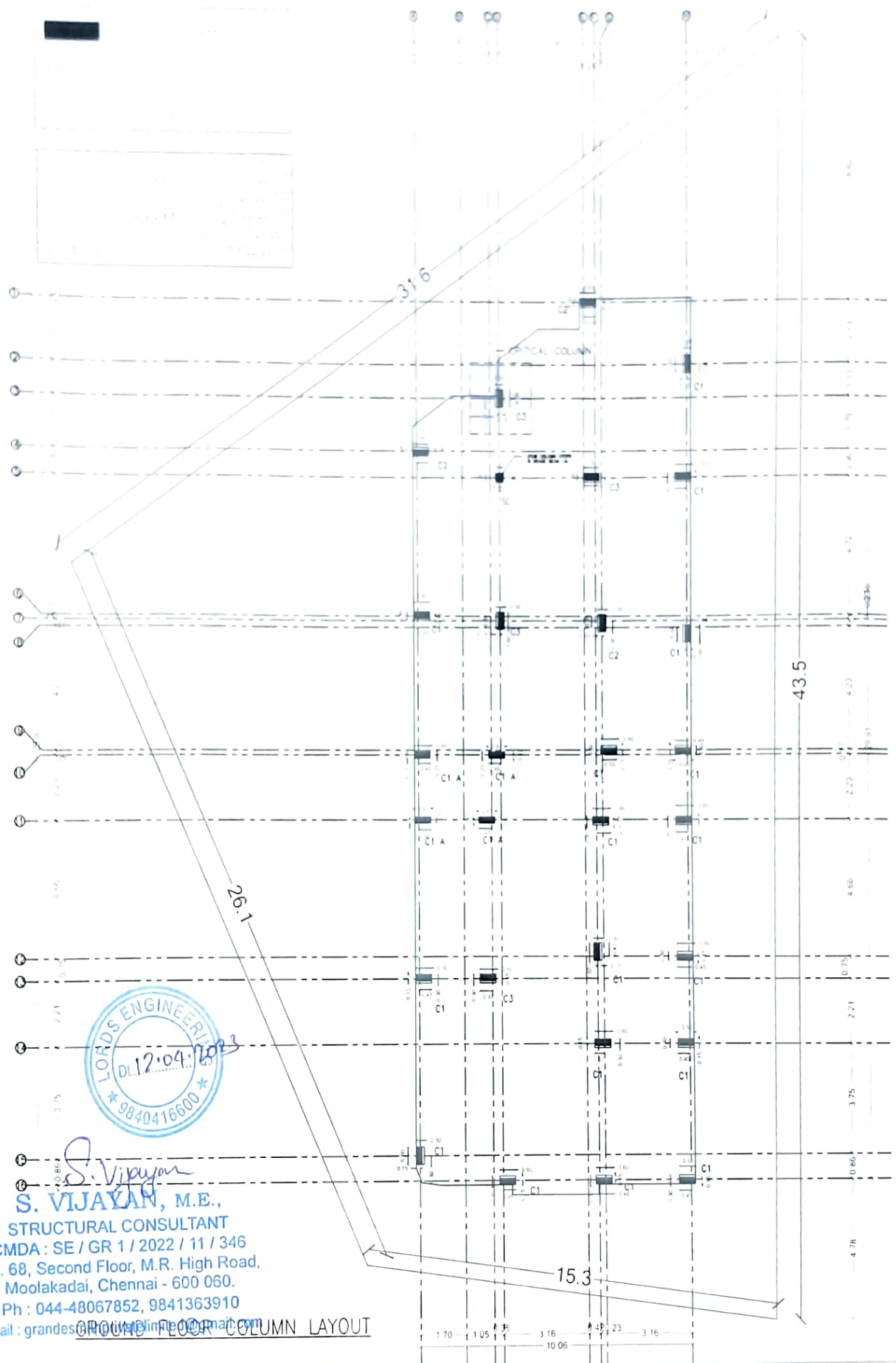
TYPICAL FLOOR PLAN

ANALYSIS



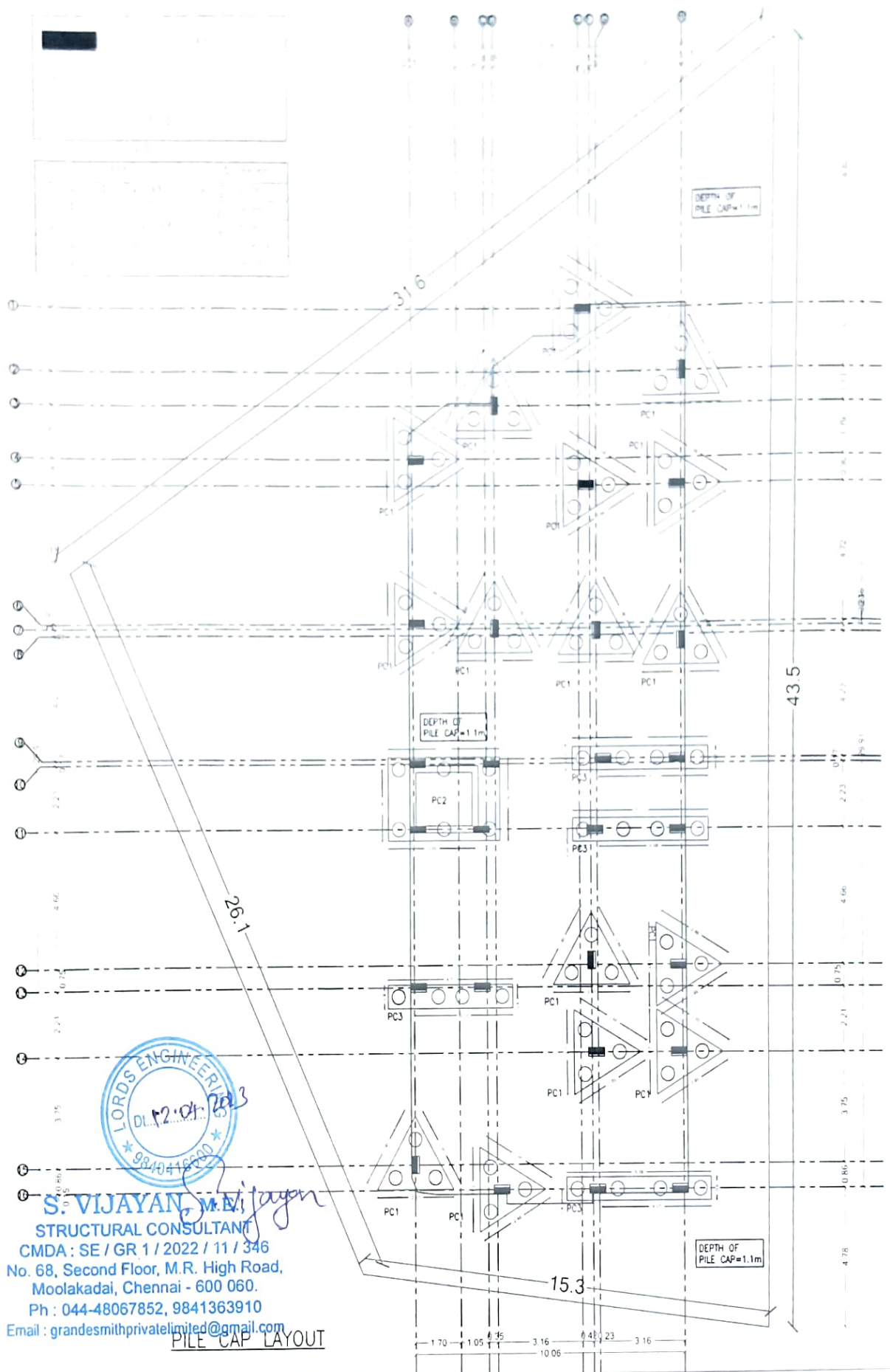
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GRADE BEAM LAYOUT



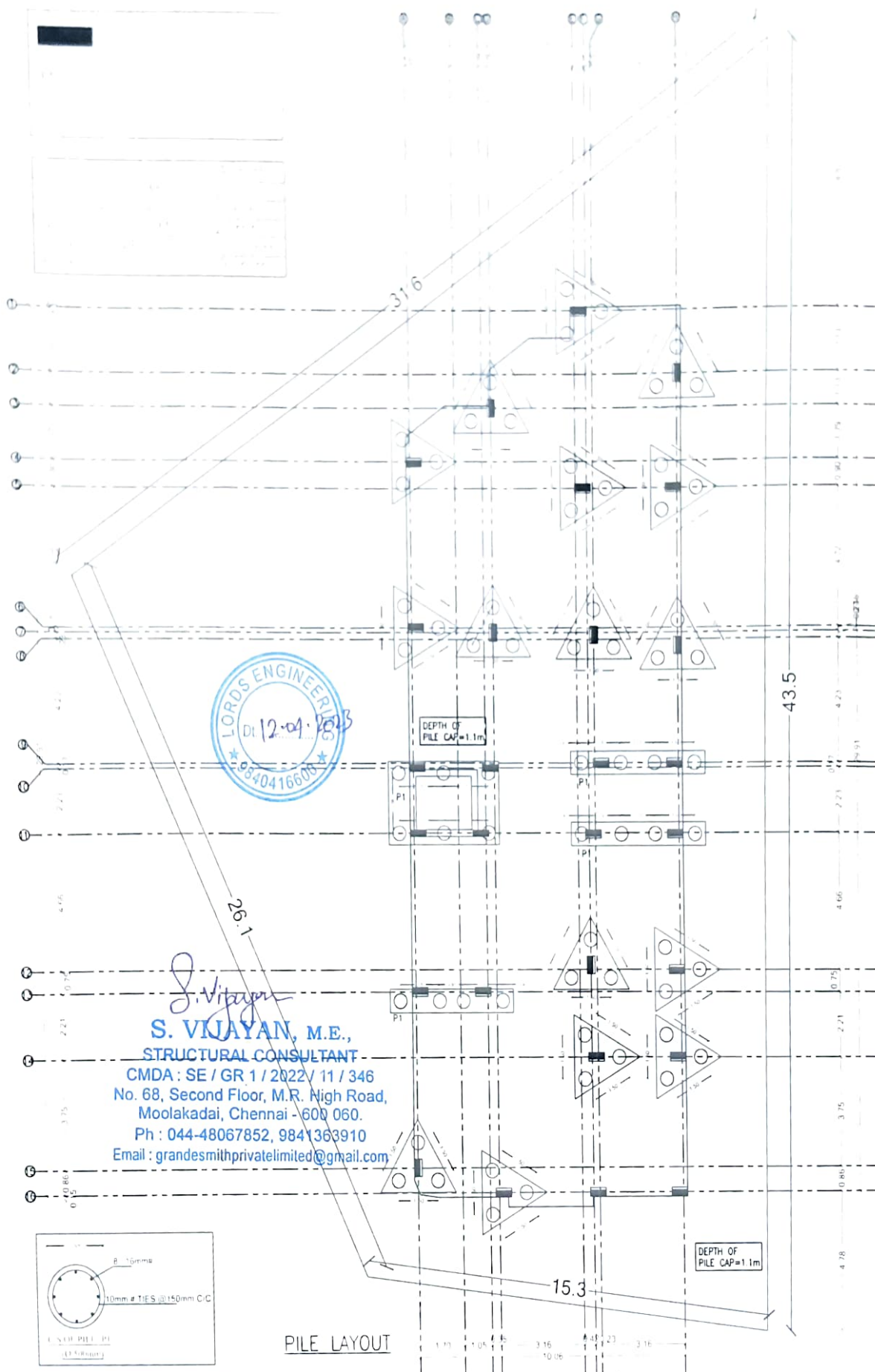
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PILE CAP LAYOUT

[illegible]

PILE LAYOUT

NEXA INFRA PROJECTS – RESIDENTIAL S+5 – AMBATTUR

GROUND FLOOR COLUMN SCHEDULE						REMARKS	
COLUMN NUMBER	COLUMN SIZE	COLUMN MAIN REINFORCEMENT DETAIL			COLUMN TIE REINFORCEMENT DETAIL		
		MAIN REIN	COLUMN SHAPE	COLUMN TIE SHAPE	AT BEAM JUNCTIONS AND NEAR LATERAL SUPPORTS FOR A LENGTH OF 0.2H	AT CENTRE OF COLUMN HEIGHT	
C1	300X600	● 8NOSx20mm dia ● 4NOSx20mm dia			AT COLUMN ENDS H _a = 0.2H(BOTTOM) H _a = 0.2H(TOP) 8 mm DIA @ 125mm C/C	AT CENTRE OF COLUMN HEIGHT H _b = 0.6H 8 mm DIA @ 150mm C/C	
C1A	230X600	● 8NOSx20mm dia			AT COLUMN ENDS H _a = 0.2H(BOTTOM) H _a = 0.2H(TOP) 8 mm DIA @ 125mm C/C	AT CENTRE OF COLUMN HEIGHT H _b = 0.6H 8 mm DIA @ 150mm C/C	
C2	300X600	● 4NOSx25mm dia ● 4NOSx20mm dia ○ 4NOSx20mm dia			AT COLUMN ENDS H _a = 0.2H(TOP) 8 mm DIA @ 125mm C/C	AT CENTRE OF COLUMN HEIGHT H _b = 0.6H 8 mm DIA @ 150mm C/C	
C3	300X600	● 10NOSx25mm dia ○ 5NOSx20mm dia			AT COLUMN ENDS H _a = 0.2H(BOTTOM) H _a = 0.2H(TOP) 8 mm DIA @ 125mm C/C	AT CENTRE OF COLUMN HEIGHT H _b = 0.6H 8 mm DIA @ 150mm C/C	
SC	300X300	○ 8NOSx12mm dia			AT COLUMN ENDS H _a = 0.2H(BOTTOM) H _a = 0.2H(TOP) 8 mm DIA @ 125mm C/C	AT CENTRE OF COLUMN HEIGHT H _b = 0.6H 8 mm DIA @ 150mm C/C	

ROOF BEAM

FLOOR LVL

H_a = 0.2H

H_b = 0.6H

H - UNSUPPORTED LENGTH OF COLUMN

ADDITIONAL CONFINEMENT TIES AROUND LAP

EX GR LEVEL

GRADE BEAM

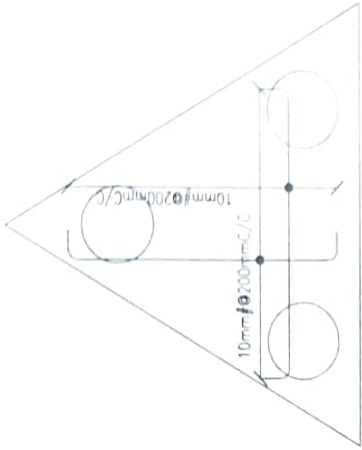
FOOTING

TYPICAL SECTION SHOWING COLUMN MAIN REINFORCEMENT LAPPING POSITION

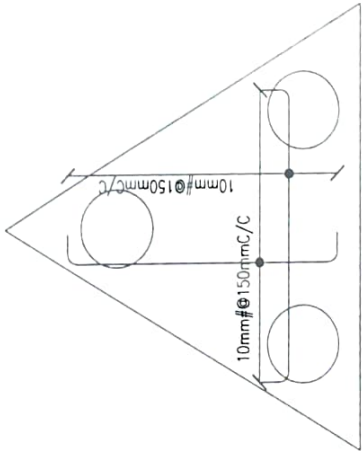
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NEXA INFRA PROJECTS – RESIDENTIAL S+5 – AMBATTUR



PC1 TOP REIN



PC1 BOTTOM REIN

LEGEND



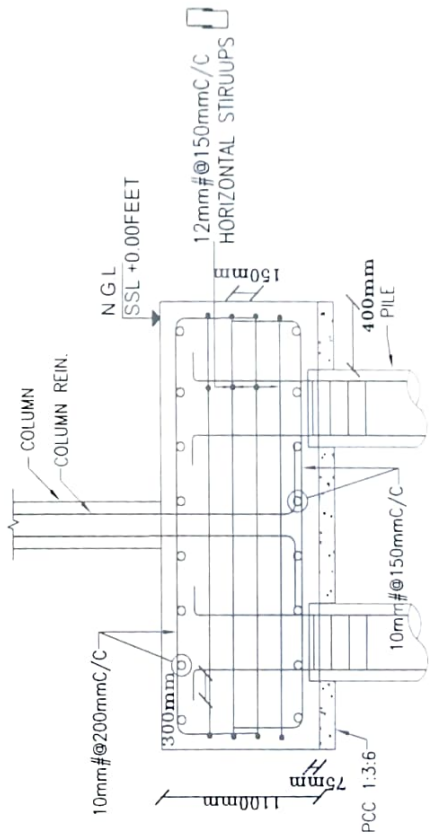
M 30

F 2.5

C 15

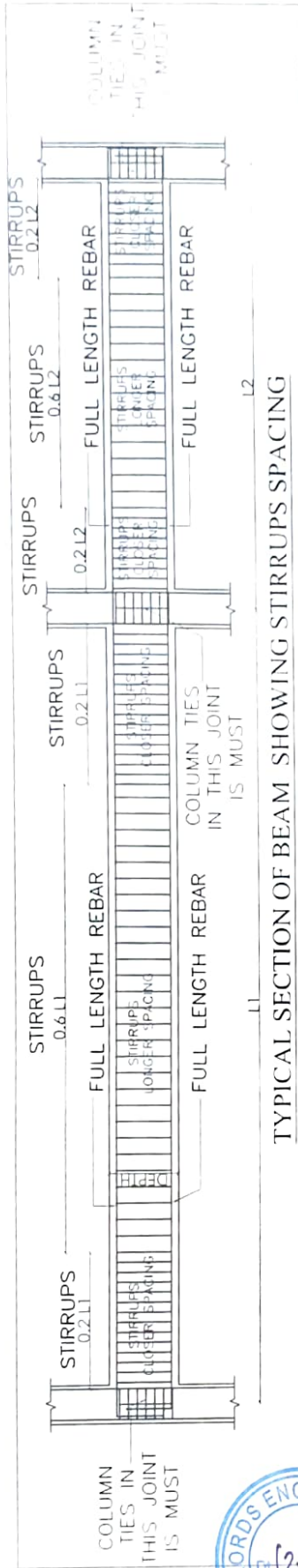
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1	2	3	4	5

S. Vijayan
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SECTION A

NEXA INFRA PROJECTS – RESIDENTIAL S+5 – AMBATTUR



TYPICAL SECTION OF BEAM SHOWING STIRRUPS SPACING

NO CRANKING OF RODS IS REQUIRED – ALL RODS ARE STRAIGHT – AT TOP AND BOTTOM
FOR CURTAILMENT OF MAIN RODS IS MENTIONED. ALL RODS ARE FULL LENGTH.
TOP REINFORCEMENT OF CANTILEVER BEAMS AS WELL AS RECTANGULAR BEAMS
SHALL NOT BE LAPPED.

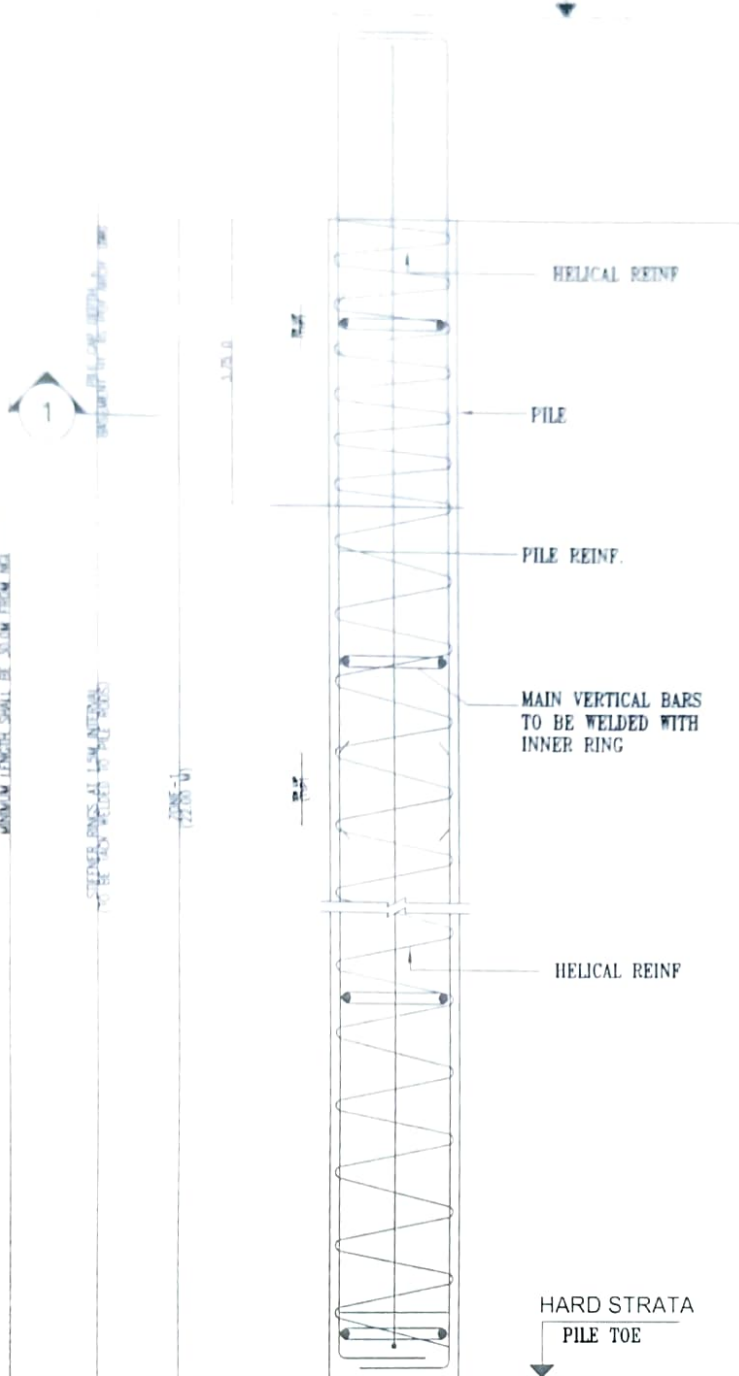
GRADE BEAM WITHOUT CURTAILMENT SCHEDULE

BEAM SIZE		FULL LENGTH BEAM MAIN REINFORCEMENT		VERTICAL STIRRUP DETAILS		
TYPE	B X D	MAIN REIN.	BEAM SHAPE	STIRRUP SHAPE	NEAR SUPPORT	AT MID SPAN
	300 X 450	TOP: FULL LENGTH ● 3NOSx16mm dia			TWO LEGGED STIRRUP	TWO LEGGED STIRRUP
		BOT: FULL LENGTH ● 3NOSx16mm dia			FOR A LENGTH=0.2L 8 mm @ 125mm C/C TWO LEGGED STIRRUPS	FOR A LENGTH=0.6L 8 mm @ 150mm C/C TWO LEGGED STIRRUPS
	SIDE FACE REIN.= 2NOS X 12mm #					
TYPE	230 X 450	TOP: FULL LENGTH ● 3NOSx16mm dia			TWO LEGGED STIRRUP	TWO LEGGED STIRRUP
		BOT: FULL LENGTH ● 3NOSx16mm dia			FOR A LENGTH=0.2L 8 mm @ 125mm C/C TWO LEGGED STIRRUPS	FOR A LENGTH=0.6L 8 mm @ 150mm C/C TWO LEGGED STIRRUPS
	SIDE FACE REIN.= 2NOS X 12mm #					

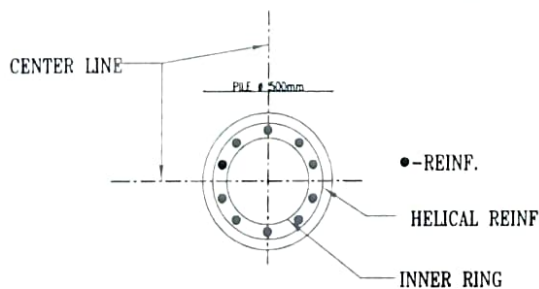


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NOTE: ALL PILES SHALL BE AS PER IIT MADRAS CONSTRUCTION
AND WILL NOT DEPEND ON THE SITE SOIL CONDITION.
MINIMUM LENGTH SHALL BE 30.0M FROM BGL



TYPICAL PILE DETAILS

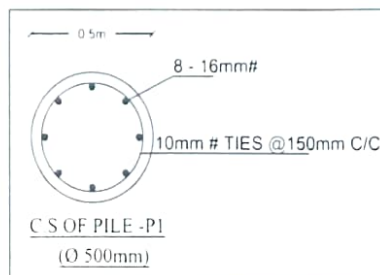


(BORED CAST INSITU PILE)

SECTION-1

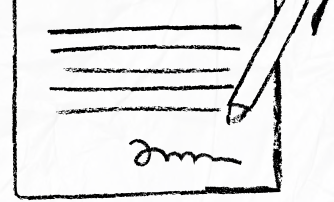


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CLEAR COVER

1. COLUMN	= 40mm.
2. ROOF BEAM	= 25mm.
3. ROOF SLAB	= 20mm.
4. GRADE BEAM	= 25mm.
5. PILE CAP	= 150mm.
6. PILE	= 75mm.



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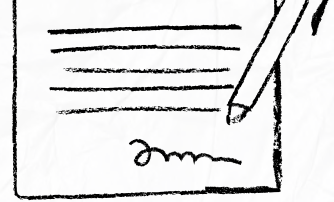
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