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DESIGN CALCULATION

PROPOSED CONSTRUCTION OF (G+3) RESIDENTIAL BUILDING AT OLD S.NO. 17/1, 17/2, 17/3, WARD NO.G, BLOCK NO.56, TOWN S.NO. 54/3, PLOT NO. B & C, MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM, CHENNAI - 600 073.

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1.0 DESIGN BRIEF:

This document furnishes the structural design calculation for PROPOSED CONSTRUCTION OF (G+3) RESIDENTIAL BUILDING AT OLD S.NO. 17/1, 17/2, 17/3, WARD NO.G, BLOCK NO.56, TOWN S.NO. 54/3, PLOT NO. B & C, MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM, CHENNAI - 600 073.

GENERAL DESIGN CRITERIA

2.1. Description of the structure

The Building consists of (G+3) Upper floors and the structure is designed as a RCC framed structure.

1. Size of the building	: (19.964m x 14.131m)
2. Height of the building	: 12.3m
3. Number of storeys above GL	: (G+3) Upper floors
4. Floor to floor height of this building	: Ground to Third floors - 3.05m:
5. Type of foundation	: Isolated footing as per geotechnical report

2.2. Materials Used

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

For Concrete Structure

- M25 grade concrete for all reinforced concrete parts of the structure
- Fe500 grade Structural reinforcement steel
- Masonry for all walls : Brick / Block

2.3. Design Codes & Standards

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

- IS 456 : 2000 Code of practice for Plain and Reinforced concrete
- SP: 16 Design Aids for Reinforced concrete
- IS 875 : 1987 Design Loads for building and structures
- IS 1893 : 2002 Criteria for Earthquake Resistant design of structures
- IS 13920: 1993 Ductile Detailing of R.C structures subjected to Seismic forces
- SP: 34 Handbook on Concrete Reinforcement & Detailing
- IS 1786: 1985 Specification for high strength deformed bars and wires for concrete
- Reinforcement

3.0 LOADING

Primary Load:

- Dead Load (DL)
- Live Load (LL)

3.1. Dead load (DL)

Material weight

- Density of Concrete =25 kN/m3
- D.L. due to partitions
- (Light Partition as per IS 875) =1.50 kN/m2
- D.L due to floor finish 50mm thick = 0.05x25 = 1.25 kN/m2

3.2. Live load (LL)

Minimum Uniformly Distributed Live Loads:

Occupancy or use Live Load (kn/m2)

- All rooms and kitchens 2.00
- Toilet and bath rooms 3.00
- Corridors, passages, staircases 3.00
- Balconies 3.00
- Terrace Floor 1.50
- Basic wind speed =50 m/sec
- Seismic Zone =Zone III

3.5 Loading Combinations

The following combinations of Loads as per IS 1893 is considered in the STAAD Analysis

- 1.5(DL +LL)
- 1.2(DL + LL)
- The below load combinations are used to check the structure in serviceability condition.
- 1.0(DL +LL)

4.0 STRUCTURAL MATERIAL:

4.1 Concrete Material Specifications:

Concrete compressive cube strength - 25 N/mm2

Minimum cement content - Refer standard drawings for general &

Legends

Cement type - OPC

Free water cement ratio - 0.5 (Maximum)

Nominal maximum size of aggregate - 20mm UNO

4.2 Reinforcement:

Steel reinforcement shall be deformed bars, the characteristic of reinforcement bars shall conform to the following specification:

Grade of Steel Actual Yield Strength (MPa)

Grade Fe500 Minimum 500 MPa,

4.3 Concrete cover:

Cover for Reinforcement is considered as per Table 16A of NBC, for a Fire resistance of 2 hrs.

- Raft Slab = 50mm
- Footing = 50mm
- RC Wall = 30mm
- Columns = 40mm
- Beams = 30mm
- Slabs = 25mm

5.0 Structural Description

5.1. Method of analysis

Structure analysis & design:

The structure is analyzed as a Space frame structure in STADD Pro Connect Edition. Columns, Footing, Basement walls, slabs, Beams and Staircases are designed as reinforced concrete members.

5.2. Framing System, Analysis and Design

Limit State concept of design will be used as per IS: 456-2000 & SP - 16. The detailing of reinforcement shall be done satisfying the requirements of IS: 456-2000, SP-16, IS: 13920, IS: 4326-1993 and SP-34.

The Slabs and beams are checked for deflection as per IS 456:2000. Cover for Reinforcement is considered as per Table 16A of NBC, for a Fire resistance of 2 hrs.

1.0 GENERAL NOTES

1.1 ALL DIMENSIONS ARE IN mm (UNLESS NOTED OTHERWISE)

1.2 BUILDING DESIGNED FOR (G+3) FLOOR .

2.0 SAFE BEARING CAPACITY

2.1 SBC AS PER SOIL TEST REPORT :
32T/Sqm AT 2.25m BELOW NGL.

2.2 EXCAVATION SHALL BE DONE UPTO THE LVL. SHOWN IN THE DRG. IF THE ABOVE MENTIONED SBC IS NOT AVAILABLE AT THIS LVL., EXCAVATION HAS TO GO UPTO THE REQUIRED DEPTH TO GET THIS SBC & THE DIFFERENCE IN LVL. FILLED WITH RIVER SAND IN LAYERS OF 150mm & COMPACTED THOROUGHLY TO OBTAIN 85% OF STANDARD PROCTOR DENSITY.

2.3 IF GOOD STRATA IS MET WITH AT EARLIER THAN THE SPECIFIED DEPTH, THE SAME HAS TO BE BROUGHT TO THE NOTICE OF DESIGN OFFICE FOR NECESSARY REVIEW.

3.0 CONCRETE WORKS

3.1 UOS MINIMUM CLEAR COVER TO MAIN REINFT. SHALL BE AS FOLLOWS :

ITEM		COVER
FOR PILE CAPS		75 mm
FOR FOOTINGS & RAFTS		50 mm
FOR COLUMNS		40 mm
FOR BEAMS/WALL		25 mm OR BAR DIA WHICHEVER IS GREATER
FOR SLABS		15 mm OR BAR DIA WHICHEVER IS GREATER
FOR WATER TANK INNER SURFACES IN CONTACT WITH WATER		25 mm OR BAR DIA WHICHEVER IS GREATER
FOR WATER TANK OUTER SURFACES IN CONTACT WITH EARTH FACE	DRY CONDITION	25 mm OR BAR DIA WHICHEVER IS GREATER
	SUBMERGED CONDITION	40 mm OR BAR DIA WHICHEVER IS GREATER

3.2 FOR SPECIAL CASES WHERE GROUND WATER CONTAINS SULPHATES/CHLORIDES MORE THAN THE ALLOWABLE LIMITS THEN COVER SHOULD BE SUITABLY INCREASED AS GIVEN IN THE STRUCTURAL DRGS.

3.3 WATER STOP SHALL BE 150mm WIDE WITH MIN. 6 THK. RIBBED/SERRATED PVC WITH A CENTRAL BULB UOS.

3.4 CONCRETING FOR WATER RETAINING AND BASEMENT STRUCTURES SHALL BE MADE WATER-TIGHT BY ADDITION OF WATERPROOFING ADMIXTURES IN THE PROPORTION AS RECOMMENDED BY MANUFACTURER.

3.5 EXPANSION/SETTLEMENT JOINTS SHALL BE PROVIDED AT LOCATIONS AS SHOWN IN THE DRG. AND AS PER STANDARD DETAILS. ALL REINFT. SHALL BE TERMINATED BEFORE THE JOINT.

3.6 GRADE OF CONC. FOR VARIOUS ELEMENTS - M25.

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4.0 REINFORCEMENT

4.1 Y DENOTES HYSD BARS (Fe 500) CONFORMING TO IS : 1786.
R DENOTES ROUND BARS (MS) CONFORMING TO IS : 432 (PART -I).

4.2 LAPS FOR HYSD BARS SHALL BE AS FOLLOWS
WHERE 'd' IS THE DIA OF THE SMALLER BAR.

GRADE OF CONC.	LAP LENGTH
M25	50 x d

FOR PLAIN BARS THE ABOVE VALUES SHOULD BE INCREASED BY 60%.

4.3 ALL HOOKS, BENDS AND LAPS SHALL BE AS PER IS : 2502 UNLESS OTHERWISE INDICATED.

4.4 LAPS SHALL BE STAGGERED SUCH THAT NOT MORE THAN 50% OF STEEL IS LAPPED AT ANY LOCATION.

5.0 MASONRY WORKS

5.1 UOS ALL BRICK WORK/PLASTERING SHALL BE AS FOLLOWS :

a) EXTERNAL BRICK WORK - 230mm THICK IN CM 1:5 FOR LOAD BEARING WALLS.
230mm THICK IN CM 1:6 FOR FRAMED STRUCTURES.

b) INTERNAL BRICK WORK - 230mm THICK IN CM 1:5 FOR LOAD BEARING WALLS.
230mm THICK IN CM 1:6 FOR FRAMED STRUCTURES.
115mm THICK IN CM 1:4.

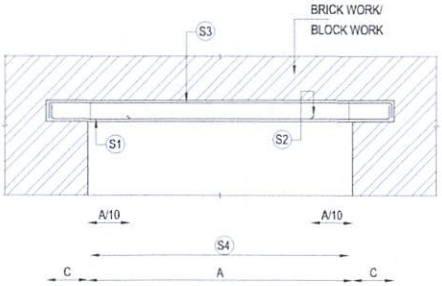
c) EXTERNAL PLASTER - 18mm THICK IN CM 1:4.

d) INTERNAL PLASTER - 12mm THICK IN CM 1:5.

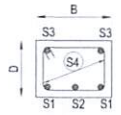
5.2 PARAPET WALLS SHALL BE 230THK. AND 450 HIGH IN THE CASE OF INACCESSIBLE TERRACES UOS. IN THE CASE OF ACCESSIBLE TERRACES, PARAPET WALL SHALL BE 900 HIGH

5.3 BED BLOCKS 600x230x175 IN PCC 1:2:4 SHALL BE PROVIDED UNDER BEAMS RESTING OVER BRICK WALLS.

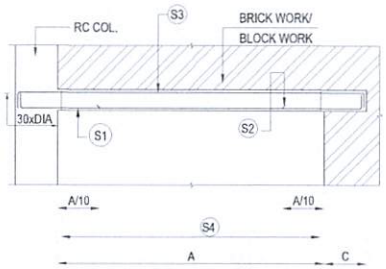
5.4 INSIDE SURFACE OF EXTERNAL WALLS SHALL BE CONSTRUCTED TO PLUMB.



L.S. OF LINTEL - TYPE-I



SEC. OF CUT LINTEL



L.S. OF LINTEL - TYPE-II

NOTES

1. ALL DIMENSIONS ARE IN mm.
2. CONC. GRADE SHALL BE M20
3. FOR LOCATION OF OPENINGS REFER RESPECTIVE ARCH.DWG.
4. IF FASCIA IS TO BE PROVIDED IN SUNSHADE, REINFT. SHOULD BE BENT ACCORDINGLY.
5. THIS DWG SHOULD READ IN CONJECTION WITH ALL STR. DWGS

SCHEDULE OF LINTEL BEAMS

B) WITH SUNSHADE WIDTH FROM 751mm TO 1050mm

BEAM No	OPENING WIDTH	MIN. BEARING WIDTH	BEAM SIZE		REINFT. DETAILS			
	A	C	B	D	S1	S2	S3	S4
LB8	UPTO 900	150	200/230	150	2-Y12	-	2-Y12	Y8 AT 100C/C
LB9	901 TO 1200	200	200/230	200	2-Y12	-	2-Y12	Y8 AT 125C/C
LB10	1201 TO 1500	200	200/230	200	2-Y12	2-Y12	2-Y12	Y8 AT 125C/C
LB11	1501 TO 1800	250	200/230	200	2-Y12	1-Y12	2-Y12	Y8 AT 150C/C
LB12	1801 TO 2100	250	200/230	250	2-Y12	1-Y12	2-Y12	Y8 AT 150C/C
LB13	2101 TO 2400	300	200/230	250	2-Y16	-	2-Y12	Y8 AT 150C/C
LB14	2401 TO 3000	300	200/230	300	2-Y16	1-Y16	2-Y12	Y8 AT 150C/C

(FOR DETAILS OF LINTEL BEAMS WITH SUNSHADE WIDTH MORE THAN 1050mm REFER RESPECTIVE DRGS.)

SCHEDULE OF SUNSHADES

TYPE	SPAN	THICKNESS		REINFT. DETAILS				REMARKS
	L	D1	D2	S5	S6	S7	S8	
I	UPTO 600	75	50	Y8 AT 200C/C	Y8 AT 250C/C	-	-	
I	601 TO 750	100	50	Y8 AT 200C/C	Y8 AT 250C/C	-	-	
I	751 TO 900	100	75	Y8 AT 200C/C	Y8 AT 250C/C	-	-	
I	901 TO 1050	125	75	Y8 AT 175C/C	Y8 AT 250C/C	-	-	
II	1051 TO 1200	150	100	Y8 AT 150C/C	Y8 AT 250C/C	Y8 AT 250C/C	Y8 AT 250C/C	BEAM SHOULD BE FROM COL. TO COL.
II	1201 TO 1500	175	125	Y8 AT 125C/C	Y8 AT 250C/C	Y8 AT 250C/C	Y8 AT 250C/C	

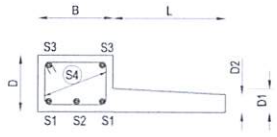
SCHEDULE OF LINTEL BEAMS

A) WITHOUT SUNSHADE AND WITH SUNSHADE WIDTH UPTO 750mm

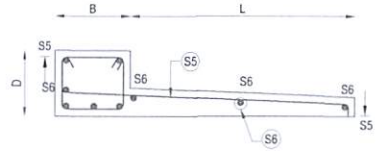
BEAM No	OPENING WIDTH	MIN. BEARING WIDTH	BEAM SIZE		REINFT. DETAILS			
	A	C	B	D	S1	S2	S3	S4
LB1	UPTO 900	150	200/230	150	2-Y12	-	2-Y12	Y8 AT 100C/C
LB2	901 TO 1200	150	200/230	200	2-Y12	-	2-Y12	Y8 AT 125C/C
LB3	1201 TO 1500	150	200/230	200	2-Y12	-	2-Y12	Y8 AT 125C/C
LB4	1501 TO 1800	200	200/230	200	2-Y12	-	2-Y12	Y8 AT 150C/C
LB5	1801 TO 2100	200	200/230	225	2-Y12	1-Y12	2-Y12	Y8 AT 150C/C
LB6	2101 TO 2400	250	200/230	225	2-Y12	1-Y12	2-Y12	Y8 AT 150C/C
LB7	2401 TO 3000	250	200/230	250	2-Y16	1-Y16	2-Y12	Y8 AT 150C/C

SCHEDULE OF LINTEL BEAMS FOR PARTITION WALLS

BEAM No	OPENING WIDTH	MIN. BEARING WIDTH	BEAM SIZE		REINFT. DETAILS			
	A	C	B	D	S1	S2	S3	S4
LB15	UPTO 900	150	100/115	115	2-Y12	-	2-Y12	Y8 AT 75C/C
LB16	901 TO 1200	150	100/115	150	2-Y12	-	2-Y12	Y8 AT 100C/C

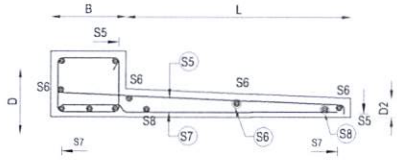


SEC. OF LINTEL WITH SUNSHADE



C.S. OF SUNSHADE - TYPE-I

(FOR 'L' = 600 TO 1050)



C.S. OF SUNSHADE - TYPE-II

(FOR 'L' = 1051 TO 1500)

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Footing Design

JM CONSTRUCTION		PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM			
		Client:	Mr.G.EKAMBARAM		
		Job No:	142	Date:	
Page Type:	Calculation Sheet	Prepared By:		GRP	F1
Section:	Design of Rectangular Footing	Checked By:			

1 **Super imposed with self load** = **450** kN 450000 **N/mm²**

2 **Column size** = **0.23** x **0.38** mtr

3 **Concrete** Grade of concrete = **M-25** FCK **25** N/mm²

4 **Steel** Grade of STEEL = **fe-500** FY **500** N/mm²

5 **Soil** **Soil Bearing capacity** = **320** kN/m² **Soil wt** = **320000** N/m³

6 **Nomial cover** = **75** **Effective cover** = **81** mm

Wt of concrete = **24000** N/m³

Using M- 25 Grade concrete fe = **fe-500** N/mm² **Cover** = **75** **mm**

1 *Footing Size*

W = 450 kN Let w' be equal to 10 % w = 45 kN

$\therefore$ Base Area = $\frac{495}{320}$ = 1.55 m² Let ratio of L to B = 1

1.0 B x B = 1.55 or **B** = 1.2 m say = **1.20** m

L = 1 x 1.2 = 1.2 m say = **1.20** m

However, provide a footing size = **1.20** x **1.20** = 1.44 sqm

Net upward pressure p_o = $\frac{450}{1.20 \text{ X } 1.20}$ = 313 kN/m²

3 *Design of section :-*

Bending moment M_1, about longer dir is given by

$M_1 = \frac{po}{8}(L\text{-}a)^2 \times 10^{\prime 6} = \frac{313}{8.00} \times (1.20 - 0.23)^2 \times 10^{\prime 6}$ N-mm

= 55219069 N-mm

Bending moment M_1, about shorter dir is given by

$M_2 = \frac{po}{8}(B\text{-}a)^2 \times 10^{\prime 6} = \frac{313}{8.00} \times (1.20 - 0.38)^2 \times 10^{\prime 6}$ N-mm

= 39461475 N-mm Thus, $M_1 > M_2$

M1 is greater than M2 therefore use M1 to calculate depth of footing

Moment limit: $\frac{M_{u\ lim}}{f_{ck} \cdot bd^2}$ = 0.133 mm

$d = \sqrt{\frac{55219069}{25 \times 1000 \times 0.133}}$ 128.87

eff depth		mm
Manual d	375	mm

Over all Depth = **eff depth + cover**

D = 450 mm

Keep d = **375** mm and the total depth **450** mm

Provide uniform thickness for entire footing.

The efffective depth found above has to be checked for shear

4 *Check for shear:-*

One way shear:

For the beam action total S.F. along section AB is

$$V = po \times B\left(\frac{L}{2} - \frac{a}{2} - d\right) = po \times B \times \left(\frac{L-a}{2} - d\right)$$

$$= 313 \times 1.20 \times \left(\frac{1.20 - 0.23}{2} - 0.38\right) = 41.316 \text{ kN} \quad 41316 \text{ N}$$

Nominal shear stress(T_v)

$$t_v = \frac{V \times 1.5}{B \times d} = \frac{61974}{1200 \times 375} = 0.138 \text{ N/mm}^2$$

Allowable shear stress(T_c)

For M 25 concrete and fe '= 500 steel, p = 0.17 %

tc = **0.304** N/mm²

Also for D > 300 mm , k = 1.00 from table 3.2

k. tc = 1.00 x 0.30 = 0.304

Allowable shear stress is greater than nomial shear stress	safe

Two way shear:

For the two way action or punching shear action along *ABCD*.

Column Perimeter = **2 (a+d)+(b+d)** = **2.00 x** (0.23 + 0.38)+(0.38 + 0.38))= 2.72 mtr

Column Area = **(a+ d) +(b+ d)** =(0.23 + 0.38)x(0.38 + 0.38) = 0.457 m²

Punching shear = 313 x ((1.20 x 1.20) - 0.46)= 307.749 kN or 307749.425 N

Nominal shear stress(T_v)

$$\therefore \quad t_v = \frac{V \times 1.5}{B \times d} = \frac{307749.425 \times 1.5}{2720 \times 375} = 0.453 \text{ N/mm}^2$$

Allowable shear stress(T_c)

tc = **1.25** N/mm²

Also for D > 300 mm , k = 1.00 from table 3.2

k. tc = 1.00 x 1.25 = 1.250

Allowable shear stress is greater than nomial shear stress	safe

5 *Design for reinforcement:-*

Area A_{st} of long bars calculated for moment M_1 is given by

$$Ast = (50/100) \times (fck/fy) \times \left(1 - \sqrt{1 - \left(\frac{4.6Mu}{fck * b * d^2}\right)}\right)$$

$$Ast = \left(\frac{50}{100}\right) \times \left(\frac{25}{500}\right) \times \left(1 - \sqrt{1 - \frac{4 \times 55219068.75}{25 \times 1000 \times 375\text{^}2}}\right)$$

Ast req = 345 mm²

using 12 mm Φ bars area = $\frac{3.14 \times dia^2}{4} = \frac{3.14 \times 12 \times 12}{4} = 113$ mm²

Spacinf of bars (S) = $\frac{ast}{AST} \times 1000 = 328$ mm

Hence Provided 12 mm ϕ bars @ 328 C/C

These are to be distributed uniformaly in a width L= 1.20 **m**

The area A_{st2} of short bars calculated for M_2 is given by

$$Ast = (50/100) \times (fck/fy) \times \left(1 - \sqrt{1 - \left(\frac{4.6Mu}{fck * b * d^2}\right)}\right)$$

$$Ast = \left(\frac{50}{100}\right) \times \left(\frac{25}{500}\right) \times \left(1 - \sqrt{1 - \frac{4 \times 39461475}{25 \times 1000 \times 375\text{^}2}}\right)$$

$$Ast = 245.238 \ mm^2$$

$$A_{st2(B)} = \frac{2A_{st2}}{\beta + 1} = \frac{2 \times 245.2}{1 + 1} = 245.238 \ mm^2$$

using 12 mm Φ bars area $= \frac{3.14 \times dia^2}{4} = \frac{3.14 \times 12 \times 12}{4} = 113 \ mm^2$

Spacinf of bars (S) $= \frac{ast}{AST} \times 1000 = 461$ mm

To be Provided in central band width = 12 mm ϕ bars @ 461 C/C

Remaning area in each end band strip $= \frac{1}{2} \times (245.2 - 245) = 0 \ mm^2$

Spacinf of bars (S) $= \frac{ast}{AST} \times 1000 =$ ##### mm

$= \frac{1}{2} \times (L - B) = \frac{1}{2} \times (1.20 - 1.20) = 0$ m

6 However provide 3 bars in each end band

Test for devlopment length:-

Ld = 47ϕ = 564 mm

564

Providing 150 mm side cover, lentgh available =

$= \frac{1}{2} \times (B-b)-60 = \frac{1}{2} \times (1200 - 380) - 150 = 260$ mm

Which is < than Ld

7

Check for transfer of load at base:-

$A_2 = 230 \times 230 = 52900 \ mm^2$

At the rate of spread 2:1, $A_1 = (230 + 2 \times (2 \times 380))^2 = 3062500$

$\therefore \sqrt{\frac{A1}{A2}} = \frac{3062500}{52900} = 7.61 > 2$

Adopt max. value of $\sqrt{A1 / A2} =$

$\therefore$ permissible bearing stress = $0.25 fck\sqrt{A/ / A2}$ "$= 0.25 \times 25 \times 2 = 12.5 \ N/mm^2$

$= \frac{450000}{230 \times 230} = 8.51 \ N/mm^2$ Hence satisfactory.

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Reinforcement :-

shown in drawing

JM CONSTRUCTION		PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM			
		Client:	Mr.G.EKAMBARAM		
		Job No:	142	Date:	
Page Type:	Calculation Sheet	Prepared By:		GRP	F2
Section:	Design of Rectangular Footing	Checked By:			

1 **Super imposed with self load** = **650** kN 650000 **N/mm²**

2 **Column size** = **0.23** x **0.45** mtr

3 **Concrete** Grade of concrete = **M-25** FCK **25** N/mm²

4 **Steel** Grade of STEEL = **fe-500** FY **500** N/mm²

5 **Soil** **Soil Bearing capacity** = **320** kN/m² **Soil wt** = **320000** N/m³

6 **Nomial cover** = **75** **Effective cover** = **81** mm

Wt of concrete = **24000** N/m³

Using M- 25 Grade concrete fe = **fe-500** N/mm² **Cover** = **75** **mm**

1 *Footing Size*

W = 650 kN Let w' be equal to 10 % w = 65 kN

∴ Base Area = $\frac{715}{320}$ = 2.23 m² Let ratio of L to B = 1

1.0 B x B = 2.23 or **B** = 1.5 m say = **1.60** m

L = 1 x 1.5 = 1.6 m say = **1.60** m

However, provide a footing size = **1.60** x **1.60** = 2.56 sqm

Net upward pressure p_o = $\frac{650}{1.60 \times 1.60}$ = 254 kN/m²

3 *Design of section :-*

Bending moment M_1, about longer dir is given by

$M_1 = \frac{po}{8}(L-a)^2 \times 10^{\prime 6} = \frac{254}{8.00} \times (1.60 - 0.23)^2 \times 10^{\prime 6}$ N-mm

= 89387363 N-mm

Bending moment M_1, about shorter dir is given by

$M_2 = \frac{po}{8}(B-a)^2 \times 10^{\prime 6} = \frac{254}{8.00} \times (1.60 - 0.45)^2 \times 10^{\prime 6}$ N-mm

= 62984063 N-mm Thus, $M_1 > M_2$

M1 is greater than M2 therefore use M1 to calculate depth of footing

Moment limit: $\frac{M_{u\ lim}}{f_{ck} \cdot bd^2}$ = 0.133 mm

$d = \sqrt{\frac{89387363}{25 \times 1000 \times 0.133}}$ 163.96

eff depth		mm
Manual d	450	mm

Over all Depth = **eff depth + cover**

D = 525 mm

Keep d = **450** mm and the total depth **525** mm

Provide uniform thickness for entire footing.

The efffective depth found above has to be checked for shear

4 ***Check for shear:-***

One way shear:

For the beam action total S.F. along section AB is

$$V = po \times B\left(\frac{L}{2} - \frac{a}{2} - d\right) = po \times B \times \left(\frac{L-a}{2} - d\right)$$

$$= 254 \times 1.60 \times \left(\frac{1.60 - 0.23}{2} - 0.45\right) = 95.504 \text{ kN} \quad 95504 \text{ N}$$

Nominal shear stress(Tv)

$$t_v = \frac{V \times 1.5}{B \times d} = \frac{143256}{1600 \times 450} = 0.199 \text{ N/mm}^2$$

Allowable shear stress(T_c)

For M 25 concrete and fe '= 500 steel, p = 0.10 %

tc = **0.29** N/mm^2

Also for D > 300 mm , k = 1.00 from table 3.2

k. tc = 1.00 x 0.29 = 0.290

Allowable shear stress is greater than nomial shear stress	safe

Two way shear:

For the two way action or punching shear action along *ABCD*.

Column Perimeter = **2 (a+d)+(b+d)** = **2.00 x** (0.23 + 0.45)+(0.45 + 0.45)]= 3.16 mtr

Column Area = **(a+ d) +(b+ d)** =(0.23 + 0.45)x(0.45 + 0.45) = 0.612 m^2

Punching shear = 254 x ((1.60 x 1.60) - 0.61)= 494.792 kN or 494792 N

Nominal shear stress(Tv)

$$\therefore \quad t_v = \frac{V \times 1.5}{B \times d} = \frac{494792 \times 1.5}{3160 \times 450} = 0.522 \text{ N/mm}^2$$

Allowable shear stress(T_c)

tc = **1.25** N/mm^2

Also for D > 300 mm , k = 1.00 from table 3.2

k. tc = 1.00 x 1.25 = 1.250

Allowable shear stress is greater than nomial shear stress	safe

5 ***Design for reinforcement:-***

Area A_{st} of long bars calculated for moment M_1 is given by

$$Ast = (50/100) \times (fck/fy) \times \left(1 - \sqrt{1 - \left(\frac{4.6Mu}{fck * b * d^2}\right)}\right)$$

$$Ast = \left(\frac{50}{100}\right) \times \left(\frac{25}{500}\right) \times \left(1 - \sqrt{1 - \frac{4 \times 89387362.5}{25 \times 1000 \times 450\text{^}2}}\right)$$

Ast req = 466.5 mm^2

using 12 mm Φ bars area = $\frac{3.14 \times dia^2}{4} = \frac{3.14 \times 12 \times 12}{4} = 113$ mm^2

Spacinf of bars (S) = $\frac{ast}{AST} \times 1000 = 242$ mm

Hence Provided 12 mm ϕ bars @ 242 C/C

These are to be distributed uniformaly in a width L= 1.60 **m**

The area A_{st2} of short bars calculated for M_2 is given by

$$Ast = (50/100) \times (fck/fy) \times \left(1 - \sqrt{1 - \left(\frac{4.6Mu}{fck * b * d^2}\right)}\right)$$

$$Ast = \left(\frac{50}{100}\right) \times \left(\frac{25}{500}\right) \times \left(1 - \sqrt{1 - \frac{4 \times 62984062.5}{25 \times 1000 \times 450\text{^}2}}\right)$$

$$Ast = 326.661 \ mm^2$$

$$A_{st2(B)} = \frac{2A_{st2}}{\beta + 1} = \frac{2 \times 326.7}{1 + 1} = 326.661 \ mm^2$$

using 12 mm Φ bars area $= \frac{3.14 \times dia^2}{4} = \frac{3.14 \times 12 \times 12}{4} = 113 \ mm^2$

Spacinf of bars (S) $= \frac{ast}{AST} \times 1000 = 346$ mm

To be Provided in central band width = 12 mm ϕ bars @ 346 C/C

Remaning area in each end band strip $= \frac{1}{2} \times (326.7 - 327) = 0 \ mm^2$

Spacinf of bars (S) $= \frac{ast}{AST} \times 1000 = \#\#\#\#\#$ mm

$= \frac{1}{2} \times (L - B) = \frac{1}{2} \times (1.60 - 1.60) = 0$ m

6 However provide 3 bars in each end band

Test for devlopment length:-

Ld = 47ϕ = 564 mm

564

Providing 150 mm side cover, lentgh available =

$= \frac{1}{2} \times (B-b) - 60 = \frac{1}{2} \times (1600 - 450) - 150 = 425$ mm

Which is < than Ld

7

Check for transfer of load at base:-

$A_2 = 450 \times 450 = 202500 \ mm^2$

At the rate of spread 2:1, $A_1 = (450 + 2 \times (2 \times 230))^2 = 1876900$

$\therefore \sqrt{\frac{A1}{A2}} = \frac{1876900}{202500} = 3.04 > 2$

Adopt max. value of $\sqrt{A1 / A2}$ =

$\therefore$ permissible bearing stress = $0.25 \ fck \sqrt{A/ / A2}$ = 0.25 X 25 X 2 = 12.5 N/mm²

$= \frac{650000}{450 \times 450} = 3.21$ N/mm² Hence satisfactory.

8

Reinforcement :-

shown in drawing

JM CONSTRUCTION		PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM			
		Client:	Mr.G.EKAMBARAM		
		Job No:	142	Date:	
Page Type:	Calculation Sheet	Prepared By:		GRP	F3
Section:	Design of Rectangular Footing	Checked By:			

1 **Super imposed with self load** = **991** kN 991000 **N/mm²**

2 **Column size** = **0.30** x **0.60** mtr

3 **Concrete** Grade of concrete = **M-25** FCK **25** N/mm²

4 **Steel** Grade of STEEL = **fe-500** FY **500** N/mm²

5 **Soil** **Soil Bearing capacity** = **320** kN/m² **Soil wt** = **320000** N/m³

6 **Nomial cover** = **75** **Effective cover** = **81** mm

Wt of concrete = **24000** **N/m³**

Using M- 25 Grade concrete fe = **fe-500** N/mm² **Cover = 75 mm**

1 *Footing Size*

W = 991 kN Let w' be equal to 10 % w = 99.1 kN

∴ Base Area = $\frac{1090}{320}$ = 3.41 m² Let ratio of L to B = 1

1.0 B x B = 3.41 or **B** = 1.8 m say = **2.00** m

L = 1 x 1.8 = 2.0 m say = **2.00** m

However, provide a footing size = **2.00** x **2.00** = 4.00 sqm

Net upward pressure p_o = $\frac{991}{2.00 \times 2.00}$ = 248 kN/m²

3 *Design of section :-*

Bending moment M_1, about longer dir is given by

$M_1 = \frac{po}{8}(L-a)^2 \times 10^{\prime 6} = \frac{248}{8.00} \times (2.00 - 0.30)^2 \times 10^{\prime 6}$ N-mm

= 134385000 N-mm

Bending moment M_1, about shorter dir is given by

$M_2 = \frac{po}{8}(B-a)^2 \times 10^{\prime 6} = \frac{248}{8.00} \times (2.00 - 0.60)^2 \times 10^{\prime 6}$ N-mm

= 91140000 N-mm Thus, $M_1 > M_2$

M1 is greater than M2 therefore use M1 to calculate depth of footing

Moment limit: $\frac{M_{u\ lim}}{f_{ck} \cdot bd^2}$ = 0.133 mm

$d = \sqrt{\frac{134385000}{25 \times 1000 \times 0.133}}$ 201.04

eff depth		mm
Manual d	525	mm

Over all Depth = **eff depth + cover**

D = 600 mm

Keep d = **525** mm and the total depth **600** mm

Provide uniform thickness for entire footing.

The efffective depth found above has to be checked for shear

4 ***Check for shear:-***

One way shear:

For the beam action total S.F. along section AB is

$$V = po \times B\left(\frac{L}{2} - \frac{a}{2} - d\right) = po \times B \times \left(\frac{L-a}{2} - d\right)$$

$$= 248 \times 2.00 \times \left(\frac{2.00 - 0.30}{2} - 0.53\right) = 161.2 \text{ kN} \quad 161200 \text{ N}$$

Nominal shear stress(Tv)

$$t_v = \frac{V \times 1.5}{B \times d} = \frac{241800}{2000 \times 525} = 0.230 \text{ N/mm}^2$$

Allowable shear stress(T_c)

For M 25 concrete and fe '= 500 steel, p = 0.18 %

tc = **0.311** N/mm²

Also for D > 300 mm , k = 1.00 from table 3.2

k. tc = 1.00 x 0.31 = 0.311

Allowable shear stress is greater than nomial shear stress	safe

Two way shear:

For the two way action or punching shear action along *ABCD*.

Column Perimeter = **2 (a+d)+(b+d)** = **2.00 x** (0.30 + 0.53)+(0.60 + 0.53))= 3.9 mtr

Column Area = **(a+ d) +(b+ d)** =(0.30 + 0.53)x(0.60 + 0.53) = 0.928 m²

Punching shear = 248 x ((2.00 x 2.00) - 0.93)= 761.825 kN or 761825 N

Nominal shear stress(Tv)

$$\therefore \quad t_v = \frac{V \times 1.5}{B \times d} = \frac{761825 \times 1.5}{3900 \times 525} = 0.558 \text{ N/mm}^2$$

Allowable shear stress(T_c)

tc = **1.25** N/mm²

Also for D > 300 mm , k = 1.00 from table 3.2

k. tc = 1.00 x 1.25 = 1.250

Allowable shear stress is greater than nomial shear stress	safe

5 ***Design for reinforcement:-***

Area A_{st} of long bars calculated for moment M_1 is given by

$$Ast = (50/100) \times (fck/fy) \times \left(1 - \sqrt{1 - \left(\frac{4.6Mu}{fck * b * d^2}\right)}\right)$$

$$Ast = \left(\frac{50}{100}\right) \times \left(\frac{25}{500}\right) \times \left(1 - \sqrt{1 - \frac{4 \times 134385000}{25 \times 1000 \times 525\text{^}2}}\right)$$

Ast req = 602.6 mm²

using 12 mm Φ bars area = $\frac{3.14 \times dia^2}{4} = \frac{3.14 \times 12 \times 12}{4} = 113$ mm²

Spacinf of bars (S) = $\frac{ast}{AST} \times 1000 = 188$ mm

Hence Provided 12 mm ϕ bars @ 188 C/C

These are to be distributed uniformaly in a width L= 2.00 **m**

The area A_{st2} of short bars calculated for M_2 is given by

$$Ast = (50/100) \times (fck/fy) \times \left(1 - \sqrt{1 - \left(\frac{4.6Mu}{fck * b * d^2}\right)}\right)$$

$$Ast = \left(\frac{50}{100}\right) \times \left(\frac{25}{500}\right) \times \left(1 - \sqrt{1 - \frac{4 \times 91140000}{25 \times 1000 \times 525\text{^}2}}\right)$$

Ast = 405.545 mm^2

$$A_{st2(B)} = \frac{2A_{st2}}{\beta + 1} = \frac{2 \times 405.5}{1 + 1} = 405.545 \quad mm^2$$

using 12 mm Φ bars area $= \frac{3.14 \times dia^2}{4} = \frac{3.14 \times 12 \times 12}{4} = 113 \quad mm^2$

Spacinf of bars (S) $= \frac{ast}{AST} \times 1000 = 279$ mm

To be Provided in central band width = 12 mm ϕ bars @ 279 C/C

Remaning area in each end band strip $= \frac{1}{2} \times (405.5 - 406) = 0 \quad mm^2$

Spacinf of bars (S) $= \frac{ast}{AST} \times 1000 = \#\#\#\#\#$ mm

$= \frac{1}{2} \times (L - B) = \frac{1}{2} \times (2.00 - 2.00) = 0$ m

6 However provide 3 bars in each end band

Test for devlopment length:-

Ld = 47ϕ = 564 mm

564

Providing 150 mm side cover, lentgh available =

$= \frac{1}{2} \times (B-b)-60 = \frac{1}{2} \times (2000 - 600) - 150 = 550$ mm

Which is < than Ld

7

Check for transfer of load at base:-

$A_2 = 600 \times 600 = 360000 \quad mm^2$

At the rate of spread 2:1, $A_1 = (600 + 2 \times (2 \times 300))^2 = 3240000$

$$\therefore \sqrt{\frac{A1}{A2}} = \frac{3240000}{360000} = 3.00 > 2$$

Adopt max. value of $\sqrt{A1 / A2}$ =

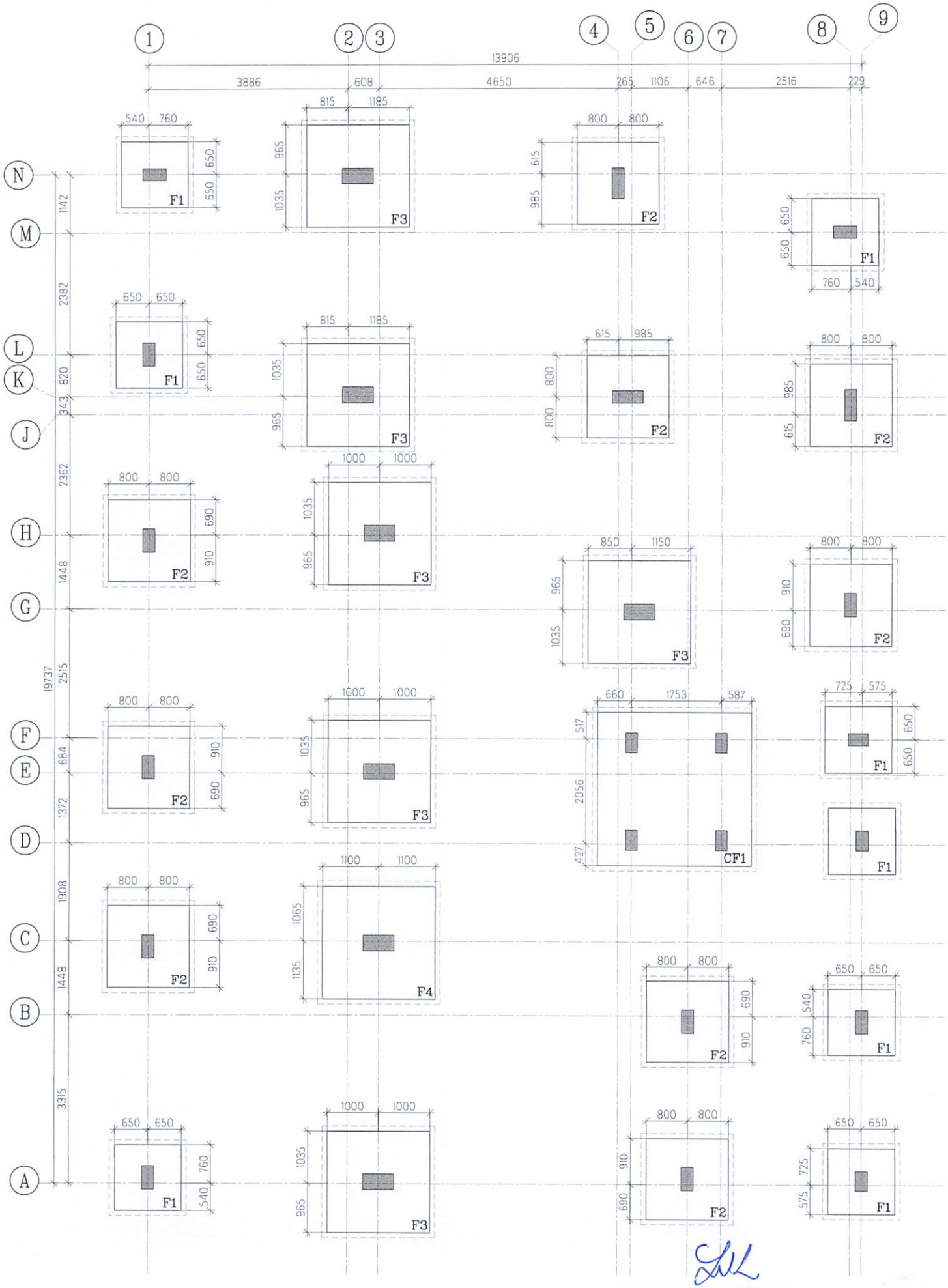
∴ permissible bearing stress = $0.25 fck\sqrt{A/ / A2}$ = 0.25 X 25 X 2 = 12.5 N/mm^2

$= \frac{991000}{600 \times 600} = 2.75 \quad N/mm^2$ Hence satisfactory.

8

Reinforcement :-

shown in drawing

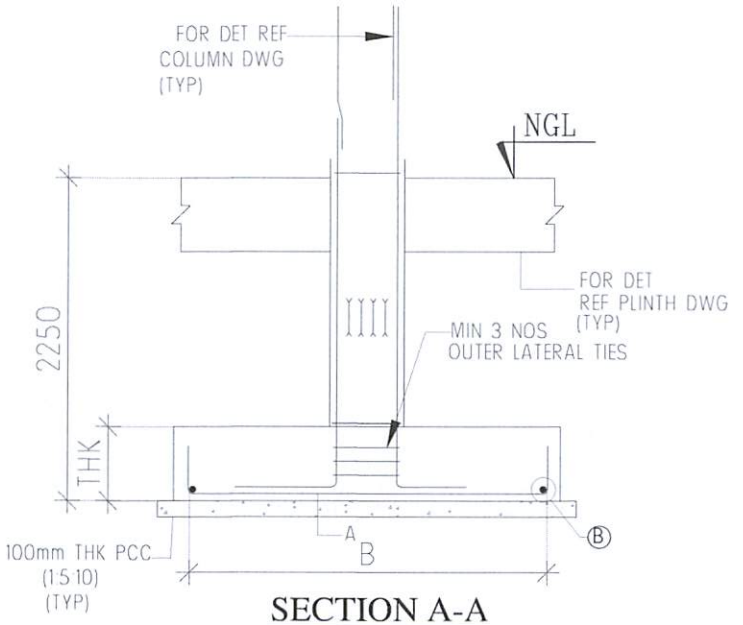


FOOTING LAYOUT

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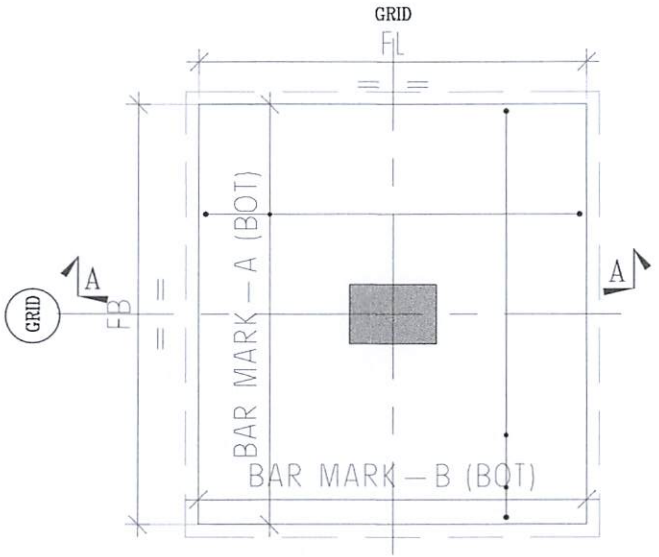
SCHEDULE OF FOOTING REINFORCEMENT

SL NO	FOOTING MKD	LENGTH OF FOOTING (FL)	WIDTH OF FOOTING (FB)	THICKNESS OF FOOTING (THK)	NO.	REINFORCEMENT DETAILS	
						BAR MARK – A (BOT)	BAR MARK – B (BOT)
1	F1	1300	1300	450	08	Y12 AT 150mmc/c	Y12 AT 150mmc/c
2	F2	1600	1600	525	09	Y12 AT 150mmc/c	Y12 AT 150mmc/c
3	F3	2000	2000	600	06	Y12 AT 125mmc/c	Y10 AT 125mmc/c
4	F4	2200	2200	600	01	Y12 AT 125mmc/c	Y10 AT 125mmc/c
5	CF1	3000	3000	525	01	REFER SECTION – 1	



SECTION A-A

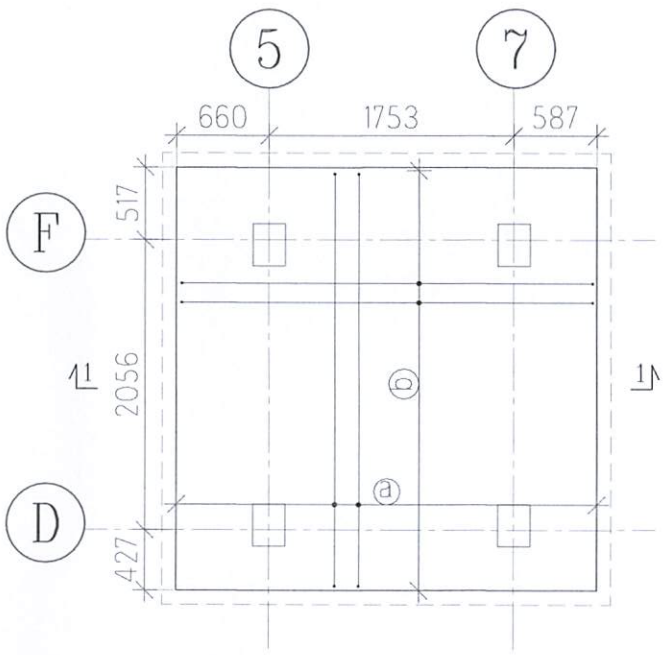
SCALE – NTS



PLAN SHOWING REINFORCEMENT FOR FOOTING

(COLUMN SHOWN IS INDICATIVE, FOR LOCATION REFER LAYOUT)

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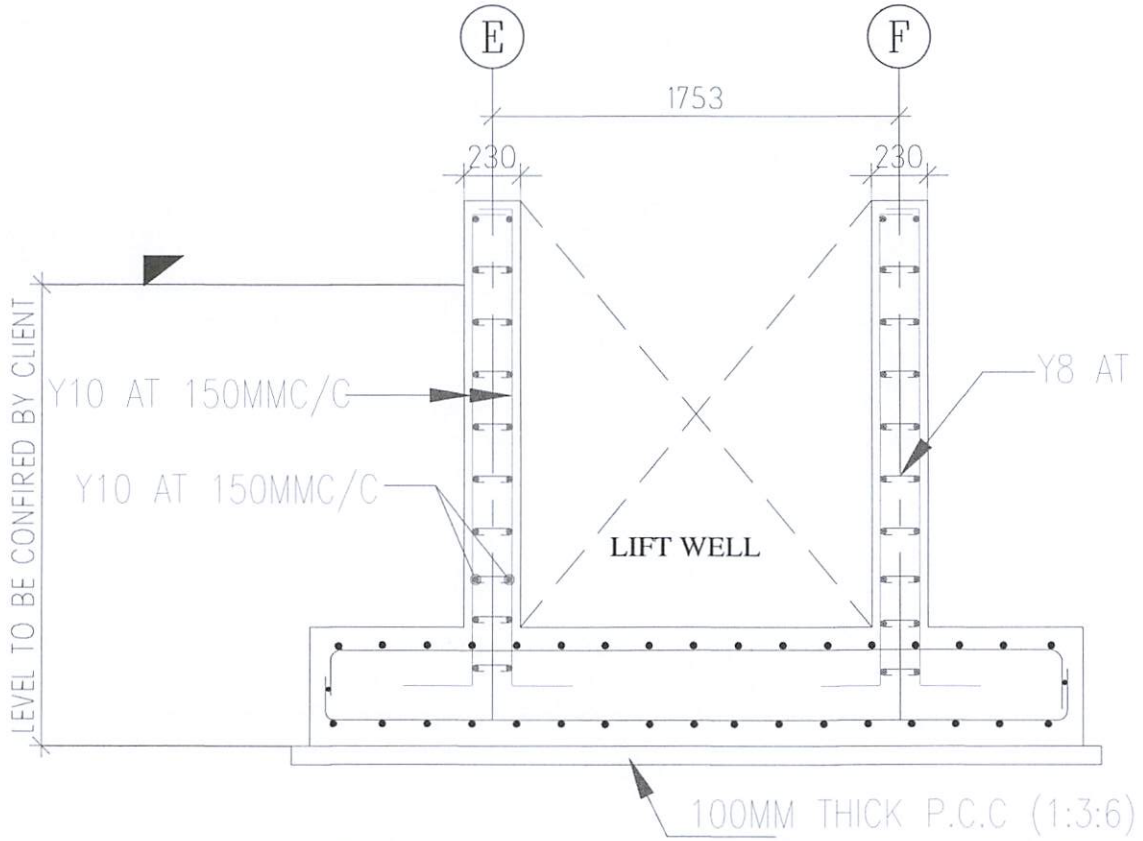


COMBINED FOOTING

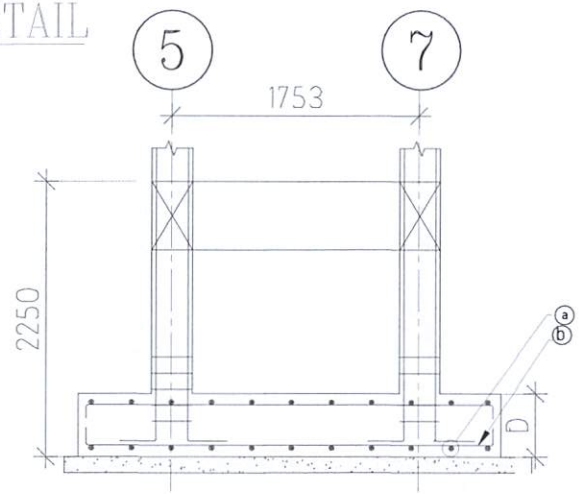
RFT: ⓐ –Y16 AT 150mm C/C – BAR MARK BOT.
ⓑ –Y16 AT 150mm C/C – BAR MARK BOT.

TOP REINFORCEMENT Y12 AT 150mm C/C

COMBINED FOOTING DETAIL



LIFT WALL REINFORCEMENT DETAIL



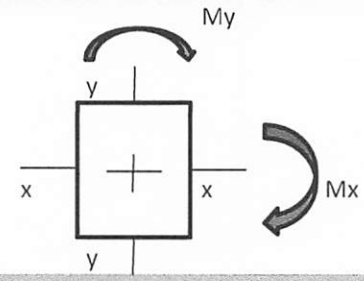
SECTION 1-1

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Column Design

	JM CONSTRUCTION	PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR, TAMBARAM			EAST
		Client:	**Mr.G.EKAMBARAM**		
		Job No:	**142**	Date:	
Page Type:	Calculation Sheet	Prepared By:		Column group	**C1**
Section:	Design of Column	Checked By:			

DESIGN OF COLUMN



1.1 MATERIAL DATA

Grade of Concrete	=	M	25	
Grade of steel	=	Fe	500	

1.2 Codal Data

Nominal Cover	=	40	mm	Cl 26.4.2.1,IS 456 : 2000
Dia of the Bar	=	16	mm	
d'	=	48	mm	

1.3 Column Basic Dimension =

Unsupported length	Lo	=	3000	mm	
	Lex	=	3000	mm	
	Ley	=	3000	mm	
	Size				
	B	=	230	mm	
	D	=	380	mm	

1.4 Analysis Result

Factored Axial Load	P_u	=	450	kN	From STAAD Output
Factored Moment	M_{ux}	=	15	kN.m	From STAAD Output
Factored Moment	M_{uy}	=	2	kN.m	From STAAD Output

1.5 Check for Slenderness =

About Major axis	Lex / D	=	7.895		
About minor axis	Ley / B	=	13.0435		
Leff / LLD > 12	**Design as Long Column**				Cl 25.1.2,IS 456 : 2000

Check for Eccentricity

Minimum Eccentricity	e_x	=	l/500 + D/30		Cl 25.4,IS 456 : 2000
		=	18.6667	< 20	
	e_y	=	l/500 + D/30		
		=	13.6667	< 20	
Moment Due to Minimum Eccentricity					
	Me_x	=	8.40	kN.m	
	Me_y	=	6.15	kN.m	

1.7 Design Moment

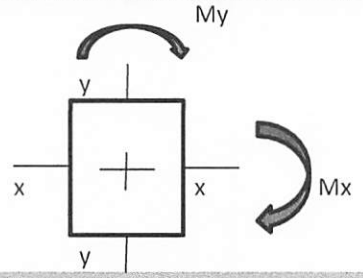
About X axis	M_{ux}	=	15.00	kN.m	
About Y axis	M_{uy}	=	6.15	kN.m	

	JM CONSTRUCTION	PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM			
		Client:	Mr.G.EKAMBARAM		
		Job No:	142	Date:	
Page Type:	Calculation Sheet	Prepared By:		Column group	C1
Section:	Design of Column	Checked By:			

1.8	**Uniaxial Moment Carrying Capacity of the Sec abt X & Y**					
	d' / D		=	0.12632		
	d' / B		=	0.2087		
	Assume the Rein %	P	=	1.2		
	P/ fck		=	0.048		
	Pu / fck BD		=	0.206		
	Reinforcement distributed equally on four sides					
	Side		=	4		
	d' /D		=	0.15		
	d' / B		=	0.2		
	$M_{ux1} / f_{ck} BD^2$		=	1.2		Chart 49 , SP 16
	$M_{uy1} / f_{ck} BD^2$		=	1		Chart 50 , SP 16
	Moment Carring Capacity			.		
	About X axis	M_{ux1}	=	996.36	kN.m	
	About Y axis	M_{uy1}	=	502.55	kN.m	
1.9	**Calculation of Puz**					
	Puz = 0.45 fck Ac + 0.75 fy Asc					Cl 39.6,IS 456 : 2000
	Puz		=	1361.21	kN	safe
	Pu/puz		=	0.33059		
	α^n		=	1		Cl 39.6,IS 456 : 2000
2	**Criteria for Biaxial Bending**					
	$(M_{ux}/M_{ux1})^{\alpha n} + (M_{uy}/M_{uy1})^{\alpha n}$		=	0.0190	< 1	Safe
2.1	**Longitudinal Reinforcement**					
	Area of Compression Steel	Asc	=	1048.8	mm^2	
	No of bars required		=	5.21895		
	Provide 6 Nos of 16 mm dia bars					

JM CONSTRUCTION		PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM			
		Client:	Mr.G.EKAMBARAM		
		Job No:	142	Date:	
Page Type:	Calculation Sheet	Prepared By:		Column group	C2
Section:	Design of Column	Checked By:			

DESIGN OF COLUMN



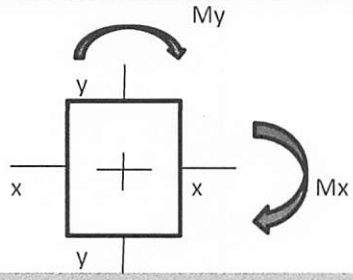
1.1	**MATERIAL DATA**					
	Grade of Concrete		=	M	25	
	Grade of steel		=	Fe	500	
1.2	**Codal Data**					
	Nominal Cover		=	40	mm	Cl 26.4.2.1,IS 456 : 2000
	Dia of the Bar		=	16	mm	
	d'		=	48	mm	
1.3	**Column Basic Dimension**		=			
	Unsupported length	Lo	=	3000	mm	
		Lex	=	3000	mm	
		Ley	=	3000	mm	
		Size				
		B	=	230	mm	
		D	=	450	mm	
1.4	**Analysis Result**					
	Factored Axial Load	P_u	=	750	kN	From STAAD Output
	Factored Moment	M_{ux}	=	150	kN.m	From STAAD Output
	Factored Moment	M_{uy}	=	20	kN.m	From STAAD Output
1.5	**Check for Slenderness**		=			
	About Major axis	Lex / D	=	6.667		
	About minor axis	Ley / B	=	13.0435		
	Leff / LLD > 12	**Design as Long Column**				Cl 25.1.2,IS 456 : 2000
	Check for Eccentricity					
	Minimum Eccentricity	e_x	=	l/500 + D/30		Cl 25.4,IS 456 : 2000
			=	21 > 20		
		e_y	=	l/500 + D/30		
			=	13.6667 < 20		
	Moment Due to Minimum Eccentricity					
		Me_x	=	15.75	kN.m	
		Me_y	=	10.25	kN.m	
1.7	**Design Moment**					
	About X axis	M_{ux}	=	150.00	kN.m	
	About Y axis	M_{uy}	=	20.00	kN.m	

	JM CONSTRUCTION	PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM			
		Client:	Mr.G.EKAMBARAM		
		Job No:	142	Date:	
Page Type:	Calculation Sheet	Prepared By:		Column group	C2
Section:	Design of Column	Checked By:			

1.8	**Uniaxial Moment Carrying Capacity of the Sec abt X & Y**						
	d' / D		=	0.10667			
	d' / B		=	0.2087			
	Assume the Rein %	P	=	1.4			
	P/ fck		=	0.056			
	Pu / fck BD		=	0.290			
	Reinforcement distributed equally on four sides						
	Side		=	4			
	d' /D		=	0.15			
	d' / B		=	0.2			
	$M_{ux1} / f_{ck} BD^2$		=	1.2			Chart 49 , SP 16
	$M_{uy1} / f_{ck} BD^2$		=	1			Chart 50 , SP 16
	Moment Carring Capacity			.			
	About X axis	M_{ux1}	=	1397.25	kN.m		
	About Y axis	M_{uy1}	=	595.125	kN.m		
1.9	**Calculation of Puz**						
	Puz = 0.45 fck Ac + 0.75 fy Asc						Cl 39.6,IS 456 : 2000
	Puz		=	1686.56	kN		safe
	Pu/puz		=	0.44469			
	α^n		=	1			Cl 39.6,IS 456 : 2000
2	**Criteria for Biaxial Bending**						
	$(M_{ux}/M_{ux1})^{\alpha n} + (M_{uy}/M_{uy1})^{\alpha n}$		=	0.1410	<	1	Safe
2.1	**Longitudinal Reinforcement**						
	Area of Compression Steel	Asc	=	1449	mm^2		
	No of bars required		=	7.21039			
	Provide 8 Nos of 16 mm dia bars						

JM CONSTRUCTION		PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM			
		Client:	Mr.G.EKAMBARAM		
		Job No:	142	Date:	
Page Type:	Calculation Sheet	Prepared By:		Column group	C3
Section:	Design of Column	Checked By:			

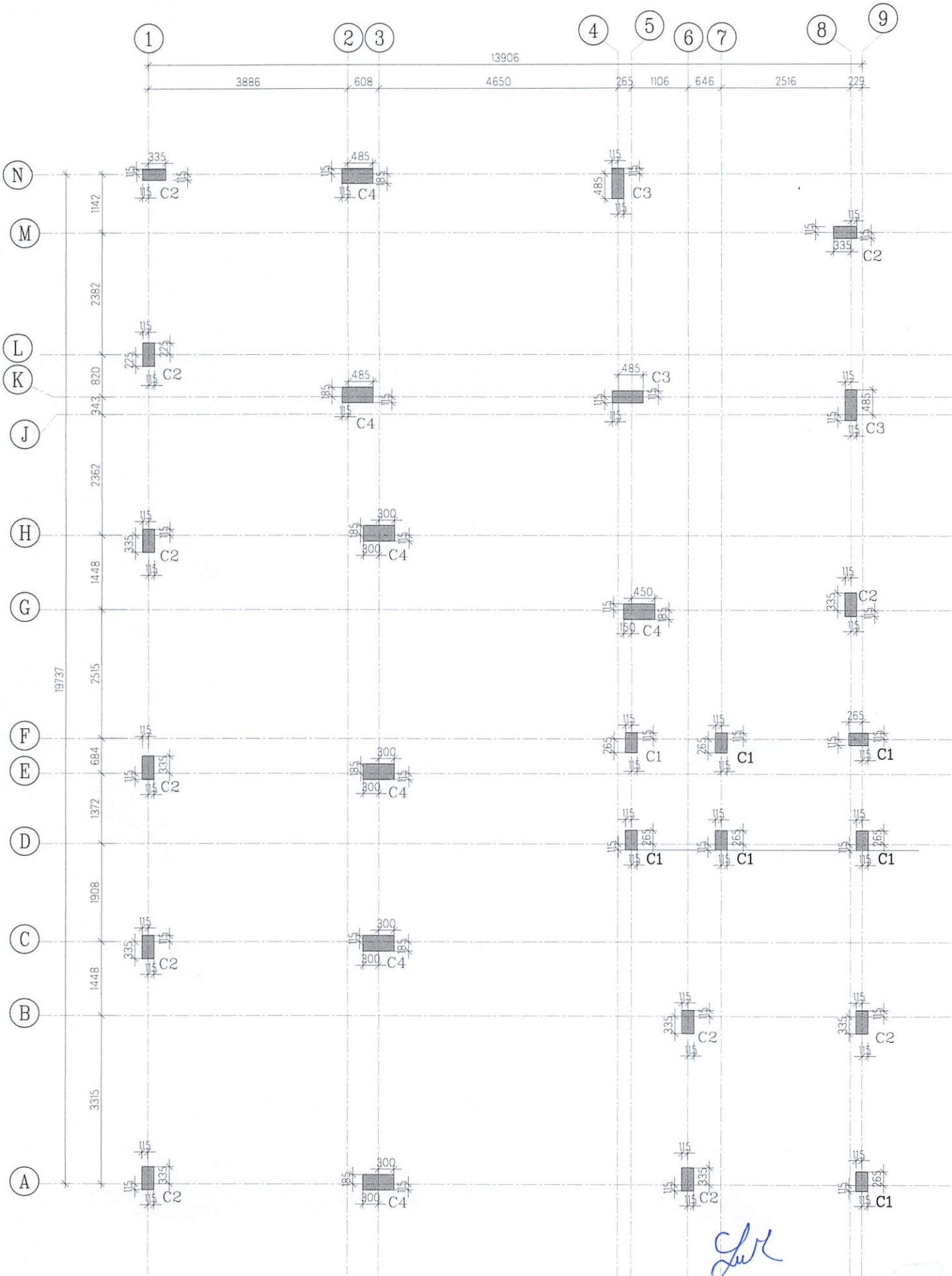
DESIGN OF COLUMN



1.1	**MATERIAL DATA**						
	Grade of Concrete		=	M	25		
	Grade of steel		=	Fe	500		
1.2	**Codal Data**						
	Nominal Cover		=	40	mm	Cl 26.4.2.1,IS 456 : 2000	
	Dia of the Bar		=	16	mm		
	d'		=	48	mm		
1.3	**Column Basic Dimension**		=				
	Unsupported length	Lo	=	3000	mm		
		Lex	=	3000	mm		
		Ley	=	3000	mm		
		Size					
		B	=	230	mm		
		D	=	600	mm		
1.4	**Analysis Result**						
	Factored Axial Load	P_u	=	1215	kN	From STAAD Output	
	Factored Moment	M_{ux}	=	157	kN.m	From STAAD Output	
	Factored Moment	M_{uy}	=	30	kN.m	From STAAD Output	
1.5	**Check for Slenderness**		=				
	About Major axis	Lex / D	=	5.000			
	About minor axis	Ley / B	=	13.0435			
	Leff / LLD > 12	**Design as Long Column**				Cl 25.1.2,IS 456 : 2000	
	Check for Eccentricity						
	Minimum Eccentricity	e_x	=	l/500 + D/30		Cl 25.4,IS 456 : 2000	
			=	26 > 20			
		e_y	=	l/500 + D/30			
			=	13.6667 < 20			
	Moment Due to Minimum Eccentricity						
		Me_x	=	31.59	kN.m		
		Me_y	=	16.61	kN.m		
1.7	**Design Moment**						
	About X axis	M_{ux}	=	157.00	kN.m		
	About Y axis	M_{uy}	=	30.00	kN.m		

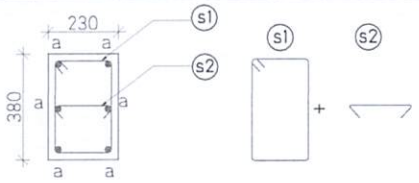
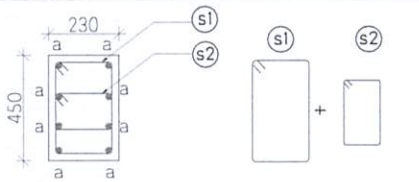
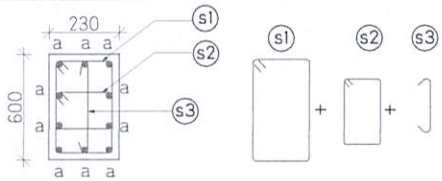
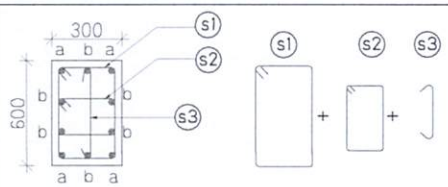
	JM CONSTRUCTION	PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR, EAST TAMBARAM			
		Client:	Mr.G.EKAMBARAM		
		Job No:	142	Date:	
Page Type:	Calculation Sheet	Prepared By:		Column group	C3
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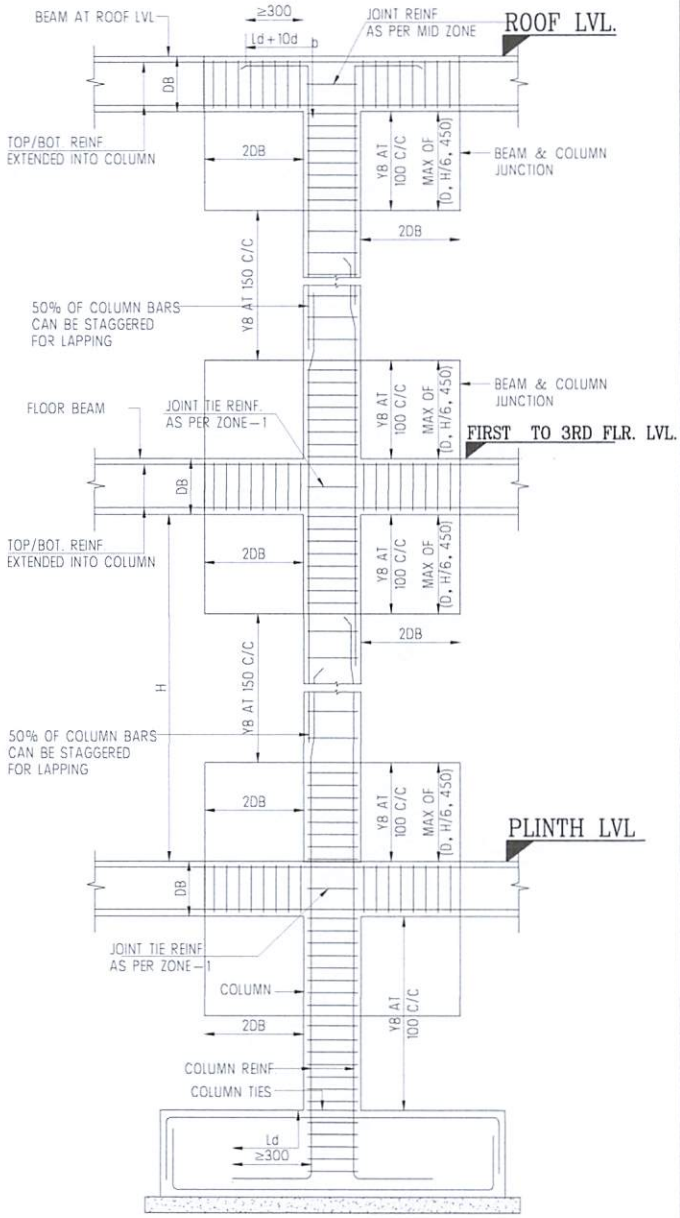
1.8	**Uniaxial Moment Carrying Capacity of the Sec abt X & Y**					
	d' / D		=	0.08		
	d' / B		=	0.2087		
	Assume the Rein %	P	=	1.2		
	P/ fck		=	0.048		
	Pu / fck BD		=	0.352		
	Reinforcement distributed equally on four sides					
	Side		=	4		
	d' /D		=	0.15		
	d' / B		=	0.2		
	$M_{ux1} / f_{ck} BD^2$		=	1.2		Chart 49 , SP 16
	$M_{uy1} / f_{ck} BD^2$		=	1		Chart 50 , SP 16
	Moment Carring Capacity			.		
	About X axis	M_{ux1}	=	2484	kN.m	
	About Y axis	M_{uy1}	=	793.5	kN.m	
1.9	**Calculation of Puz**					
	Puz = 0.45 fck Ac + 0.75 fy Asc					Cl 39.6,IS 456 : 2000
	Puz		=	2149.28	kN	safe
	Pu/puz		=	0.56531		
	α^n		=	1		Cl 39.6,IS 456 : 2000
2	**Criteria for Biaxial Bending**					
	$(M_{ux}/M_{ux1})^{\alpha n} + (M_{uy}/M_{uy1})^{\alpha n}$		=	0.1010	< 1	Safe
2.1	**Longitudinal Reinforcement**					
	Area of Compression Steel	Asc	=	1656	mm^2	
	No of bars required		=	8.24045		
	Provide 10 Nos of 16 mm dia bars					



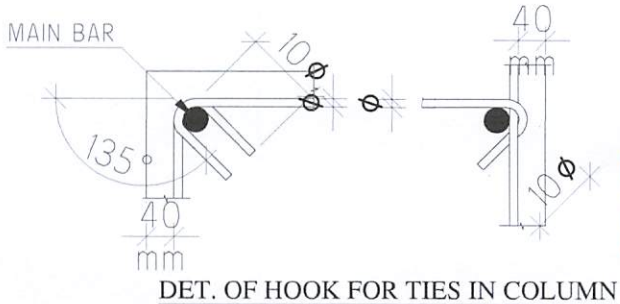
COLUMN LAYOUT

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C1 (230x380)	 VERT. BARS ⓐ - 6 - Y16 TIES = Y8 spacing as per elevation
NOs. (07)	COLUMN.C1
C2 (230x450)	 VERT. BARS ⓐ - 8 - Y16 TIES = Y8 spacing as per elevation
NOs. (11)	COLUMN.C2
C3 (230x600)	 VERT. BARS ⓐ - 10 - Y16 TIES = Y8 spacing as per elevation
NOs. (03)	COLUMN.C3
C4 (300x600)	 VERT. BARS ⓐ - 4 - Y20 +ⓑ- 6 - Y16 TIES = Y8 spacing as per elevation
NOs. (07)	COLUMN.C4



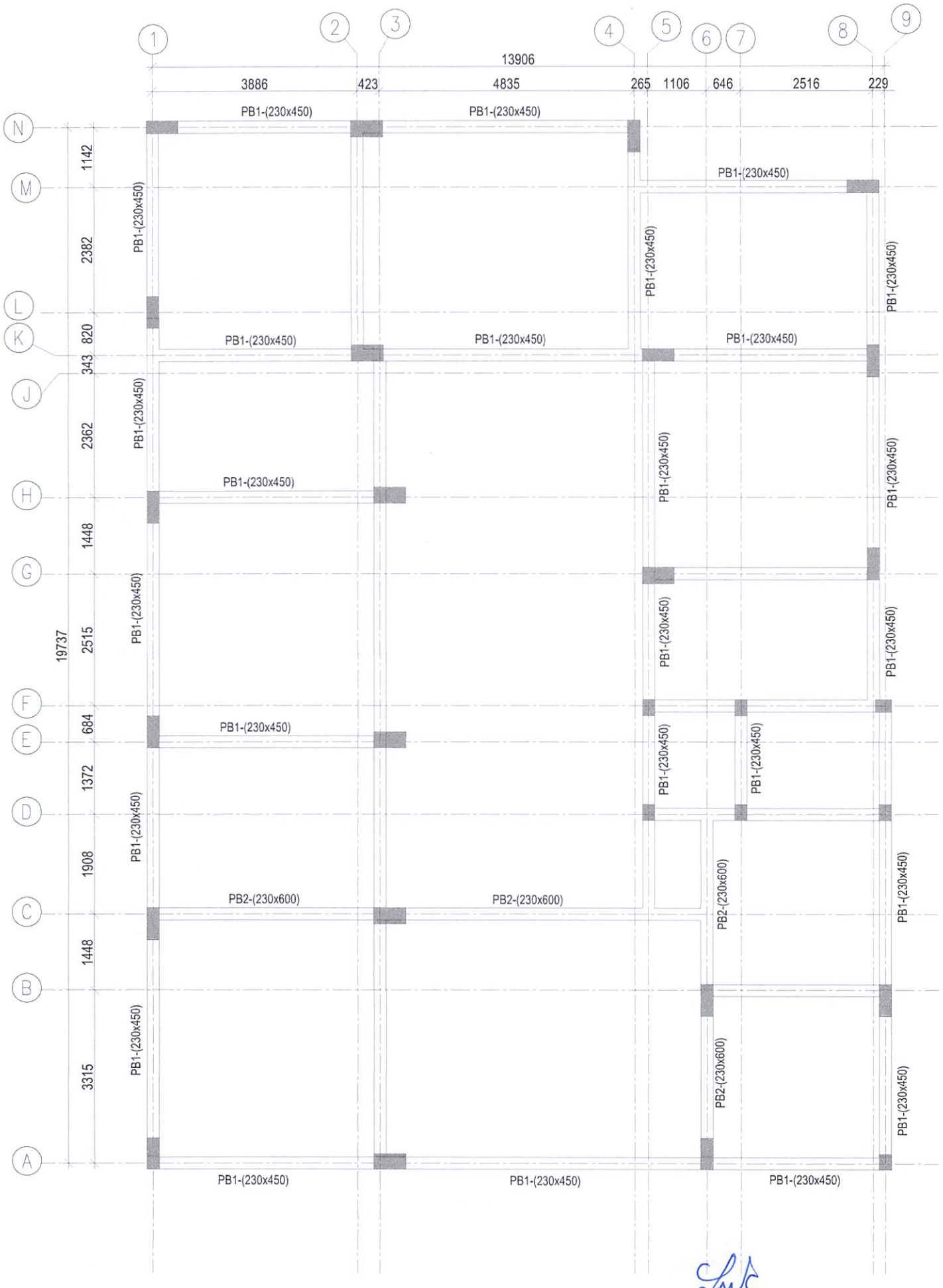
TYP — LONGITUDINAL SECTION OF COLUMN
SCALE — NTS



DET. OF HOOK FOR TIES IN COLUMN

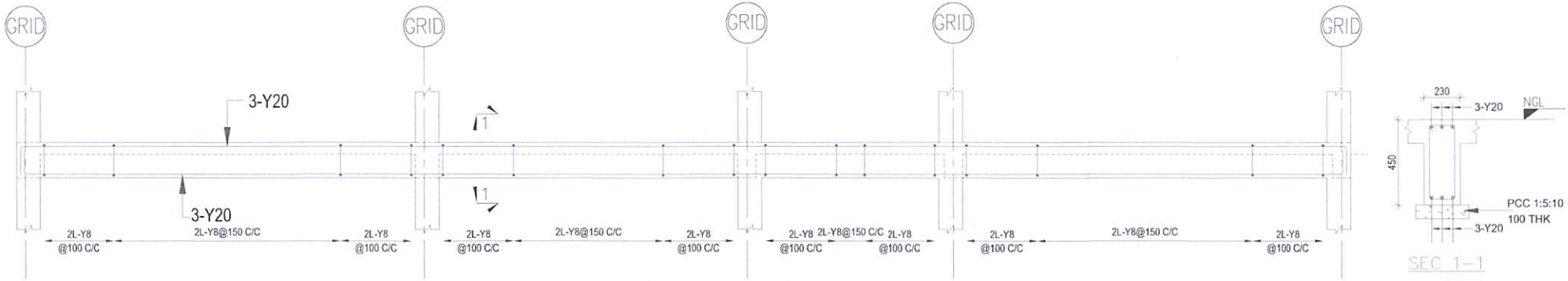
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Kosapet, Chennai - 600 012

PLINTH BEAM LAYOUT & DETAIL

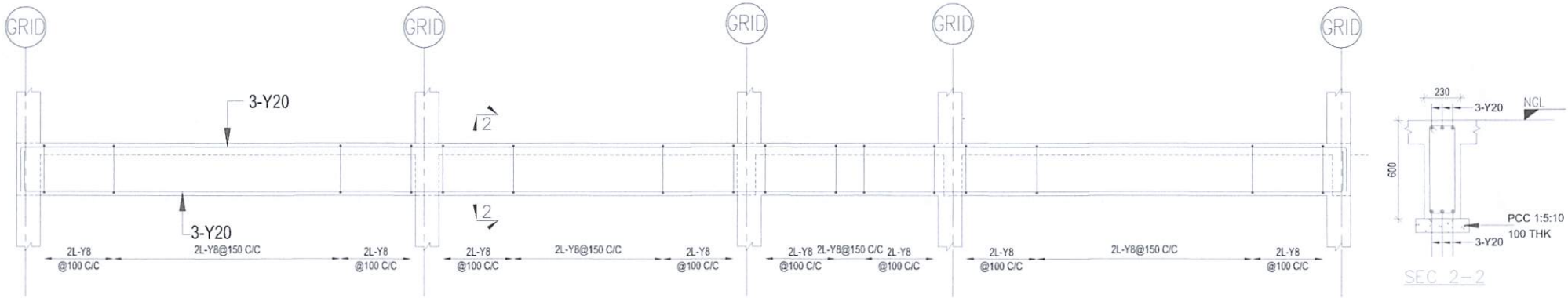


PLINTH BEAM LAYOUT

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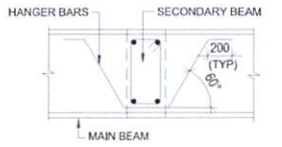


BEAM REINFORCEMENT DETAIL(TYP.)-PB1-(230x450)

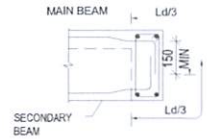


BEAM REINFORCEMENT DETAIL(TYP.)-PB2-(230x600)

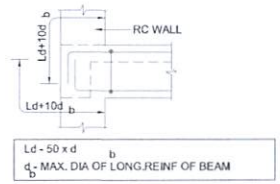
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TYP - DETAIL OF HANGER BAR
SCALE:NTS



TYP - BEAM ANCHORAGE DETAIL
SCALE:NTS



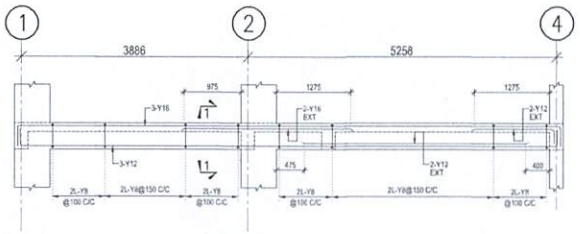
TYP - EXTERNAL JOINT DETAIL
SCALE:NTS

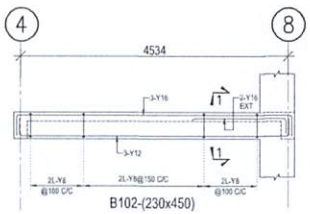
FLOOR BEAM AND ROOF BEAM
LAYOUT & DETAIL

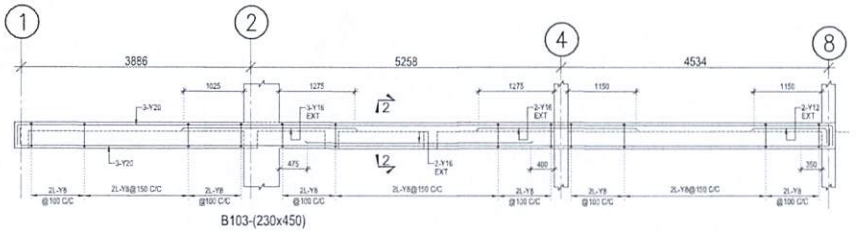


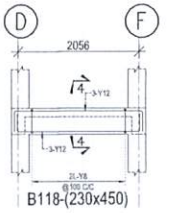
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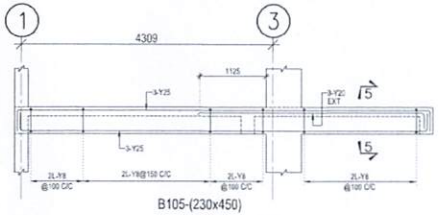
FLOOR BEAM AND ROOF BEAM LAYOUT

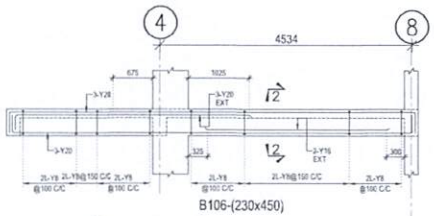


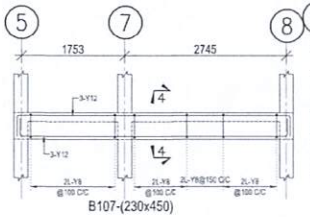


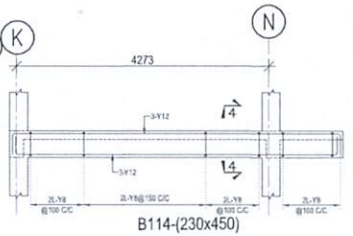


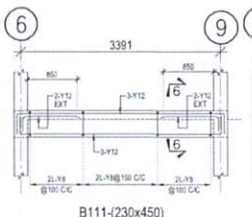


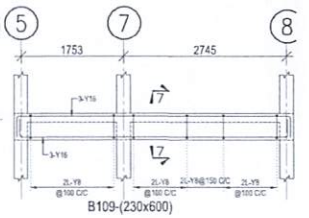


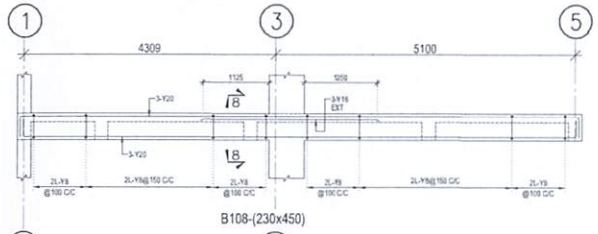


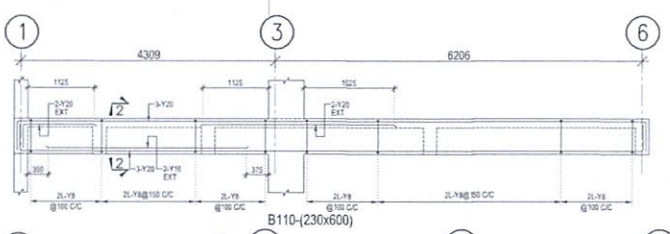


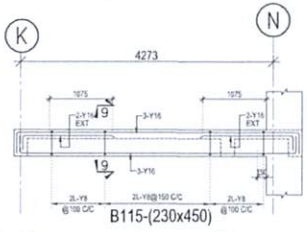


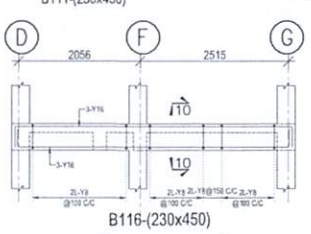


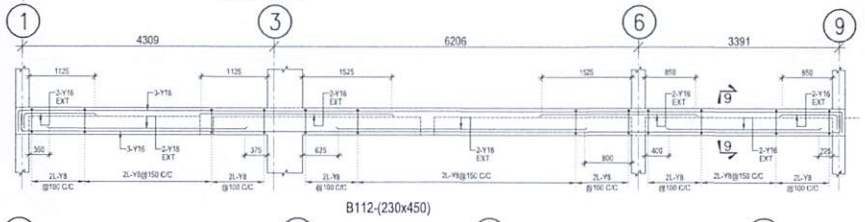


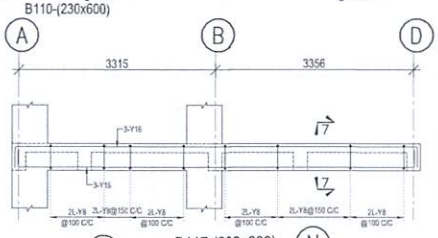


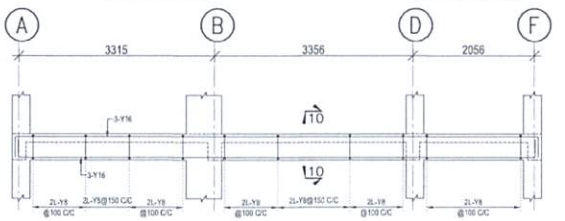


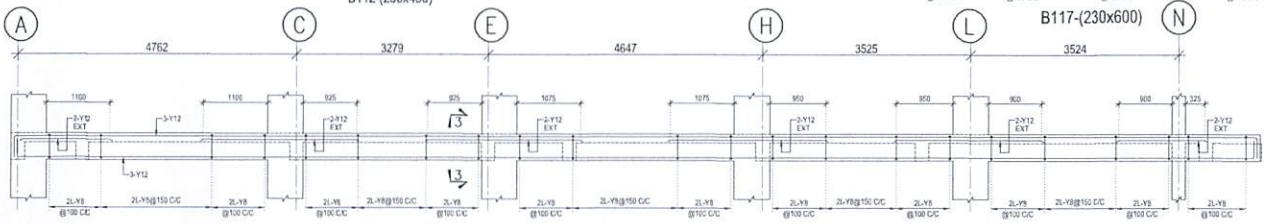


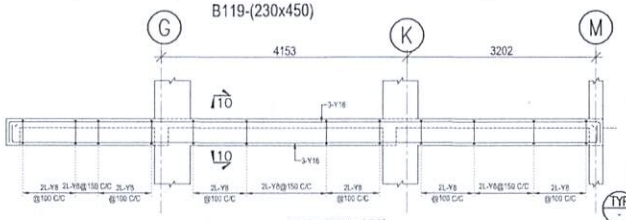


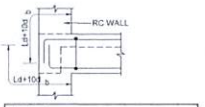




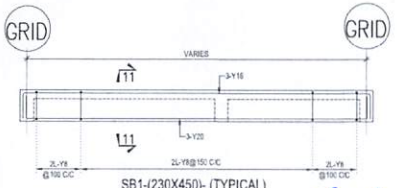


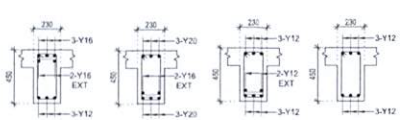


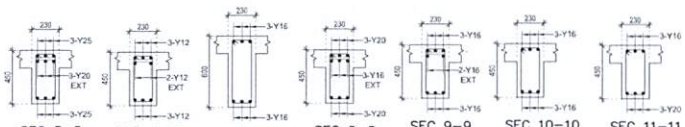


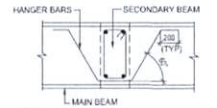


TYP EXTERNAL JOINT DETAIL
SCALE:NTS

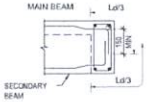








TYP DETAIL OF HANGER BAR
SCALE:NTS



TYP BEAM ANCHORAGE DETAIL
SCALE:NTS

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FLOOR BEAM AND ROOF BEAM REINFORCEMENT DETAILS

FLOOR SLAB AND ROOF SLAB LAYOUT & DETAIL

JM CONSTRUCTION	PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR,EAST TAMBARAM			
	Client:	Mr.G.EKAMBARAM		
	Job No:	142	Date:	
Page Type: Calculation Sheet	Prepared By:		GRP	S1
Section: Design of Slab	Checked By:			

SPANS :

Shorter span c/c w.r.to beam	B	=	3.00	m
Longer span c/c w.r.to beam	L	=	6.70	m
Width of support - shorter span - Left	bxl	=	230	mm
Width of support - shorter span - Right	bxr	=	230	mm
Width of support - longer span - Left	byl	=	230	mm
Width of support - longer span - Right	byr	=	230	mm
Clear Cover of slab	cv	=	15	mm
Ratio Ly / Lx	ly/lx	=	2.24	> 2

LOAD CALCULATION :

Assume the thickness of the slab	=	127	mm
Effective Depth of the of the slab	=	107.00	mm
Self Weight of slab	=	3.175	KN / m^2
Weathering course / Floor finishing	=	1.250	KN / m^2
Wall load	=		KN / m^2
Live Load	=	2.800	KN / m^2
D.L+L.L	=	7.225	KN / m^2
1.5D.L+1.5L.L	=	10.838	KN / m^2

DESIGN AS ONE WAY SLAB

Support Condition — **ONE SHORT END CONTINUOUS**

EFFECTIVE SPANS :

Shorter span: (Lx)

Clear span + effective depth	=	2.88	m
Center to centre of supports	=	3.00	m
Effective span	=	3.00	m

Longer span: (Ly)

Clear span + effective depth	=	6.58	m
Center to centre of supports	=	6.70	m
Effective span	=	6.70	m
Aspect Ratio Ly / Lx	=	2.23	> 2

As per IS456 Table 26 & 27

+ ve Alpha - x	=	0.100
- ve Alpha - x	=	0.100
+ ve Alpha - y	=	0.000
- ve Alpha - y	=	0.000

Compressive Stength of Concrete	=	20	N/mm^2
Grade of steel	=	500	N/mm^2

BENDING MOMENTS:

EFFECTIVE DEPTH OF SLAB

Positive moment along Shorter dir.	Mx +ve	=	9.75	KNm / m	dx+	=	107.00	mm	Kx+	=	0.043	Zx+	=	101.650 mm
Negative moment along Shorter dir.	Mx -ve	=	9.75	KNm / m	dx-	=	107.00	mm	Kx-	=	0.043	Zx-	=	101.650 mm
Positive moment along Longer dir.	My +ve	=	0.00	KNm / m	dy+	=	97.00	mm	Ky+	=	0.000	Zy+	=	92.150 mm
Negative moment along Longer dir.	My -ve	=	0.00	KNm / m	dy-	=	107.00	mm	Ky-	=	0.000	Zy-	=	101.650 mm

REQUIRED AREA OF STEEL :

Ast req. at midspan along x- dir.	ast +ve x	=	202.01	mm^2
Ast req. at midspan along y- dir.	ast +ve y	=	0.00	mm^2
Ast req. at support along x- dir.	ast -ve x	=	202.01	mm^2
Ast req. at support along y- dir.	ast -ve y	=	0.00	mm^2
Minimum Ast required	ast min	=	165.10	mm^2

REQUIRED SPACING OF STEEL:

10	mm dia at midspan along x- dir.	=	389	mm c/c
10	mm dia at midspan along y- dir.	=	476	mm c/c
10	mm dia at support along x- dir.	=	389	mm c/c
10	mm dia at support along y- dir.	=	476	mm c/c
10	mm dia for distribution steel	=	476	mm c/c
0	mm dia for top of mid span	=	200	mm c/c

PROVIDED SPACING OF STEEL

Bottom reinforcement along x- dir.	Y	10	150	mm c/c
Bottom reinforcement along y- dir.	Y	10	150	mm c/c
Top reinforcement along x- dir.	Y	10	150	mm c/c
Top reinforcement along y- dir.	Y	10	150	mm c/c
Distribution Steel	Y	10	0	mm c/c

CHECK FOR DEFLECTION :

Provided area of reinforcement	=	523.60	mm^2
Mu/bd^2	=	0.85	
Actual Span / Effective depth Ratio	=	28.04	
Design service stress	=	128.60	
Modification factor - Tension reinf.	=	2.00	
Modification factor - Comp. reinf.	=	1.00	
Allowable Span /Effective depth Ratio	=	52.00	

==<<< Hence SAFE In Deflection >>>==

JM CONSTRUCTION		PROPOSED RESIDENTIAL BUILDING AT MALAVIKA AVENUE, SELAIYUR,EAST TAMBARAM			
		Client:	Mr.G.EKAMBARAM		
		Job No:	142	Date:	
Page Type:	Calculation Sheet	Prepared By:		GRP	S2
Section:	Design of Slab	Checked By:			

SPANS :

Shorter span c/c w.r.to beam	B	=	3.60	m
Longer span c/c w.r.to beam	L	=	6.50	m
Width of support - shorter span - Left	bxl	=	230	mm
Width of support - shorter span - Right	bxr	=	230	mm
Width of support - longer span - Left	byl	=	230	mm
Width of support - longer span - Right	byr	=	230	mm
Clear Cover of slab	cv	=	15	mm
Ratio Ly / Lx	ly/lx	=	1.81	< 2

LOAD CALCULATION :

Assume the thickness of the slab	=	127	mm
Effective Depth of the of the slab	=	107.00	mm
Self Weight of slab	=	3.175	KN / m^2
Weathering course / Floor finishing	=	1.250	KN / m^2
Wall load	=		KN / m^2
Live Load	=	2.800	KN / m^2
D.L+L.L	=	7.225	KN / m^2
1.5D.L+1.5L.L	=	10.838	KN / m^2

DESIGN AS TWO WAY SLAB

Support Condition — **ONE SHORT EDGE DISCONTINUOUS**

EFFECTIVE SPANS :

As per IS456 Table 26 & 27

Shorter span: (Lx)			
Clear span + effective depth	=	3.48	m
Center to centre of supports	=	3.60	m
Effective span	=	3.60	m
Longer span: (Ly)			
Clear span + effective depth	=	6.38	m
Center to centre of supports	=	6.50	m
Effective span	=	6.50	m
Aspect Ratio Ly / Lx	=	1.81	< 2

+ ve Alpha - x	=	0.048	
- ve Alpha - x	=	0.064	
+ ve Alpha - y	=	0.028	
- ve Alpha - y	=	0.038	

Compressive Stength of Concrete	=	20	N/mm^2
Grade of steel	=	500	N/mm^2

BENDING MOMENTS:

EFFECTIVE DEPTH OF SLAB

Positive moment along Shorter dir.	Mx +ve	=	6.74	KNm / m	dx+	=	107.00	mm	Kx+	=	0.029	Zx+	=	101.650 mm
Negative moment along Shorter dir.	Mx -ve	=	8.99	KNm / m	dx-	=	107.00	mm	Kx-	=	0.039	Zx-	=	101.650 mm
Positive moment along Longer dir.	My +ve	=	3.93	KNm / m	dy+	=	97.00	mm	Ky+	=	0.021	Zy+	=	92.150 mm
Negative moment along Longer dir.	My -ve	=	5.34	KNm / m	dy-	=	107.00	mm	Ky-	=	0.023	Zy-	=	101.650 mm

REQUIRED AREA OF STEEL :

Ast req. at midspan along x- dir.	ast +vc x	=	139.63	mm^2
Ast req. at midspan along y- dir.	ast +ve y	=	89.85	mm^2
Ast req. at support along x- dir.	ast -ve x	=	186.17	mm^2
Ast req. at support along y- dir.	ast -ve y	=	110.54	mm^2
Minimum Ast required	ast min	=	165.10	mm^2

REQUIRED SPACING OF STEEL:

10	mm dia at midspan along x- dir.	=	476	mm c/c
10	mm dia at midspan along y- dir.	=	476	mm c/c
10	mm dia at support along x- dir.	=	422	mm c/c
10	mm dia at support along y- dir.	=	476	mm c/c
10	mm dia for distribution steel	=	476	mm c/c
0	mm dia for top of mid span	=	200	mm c/c

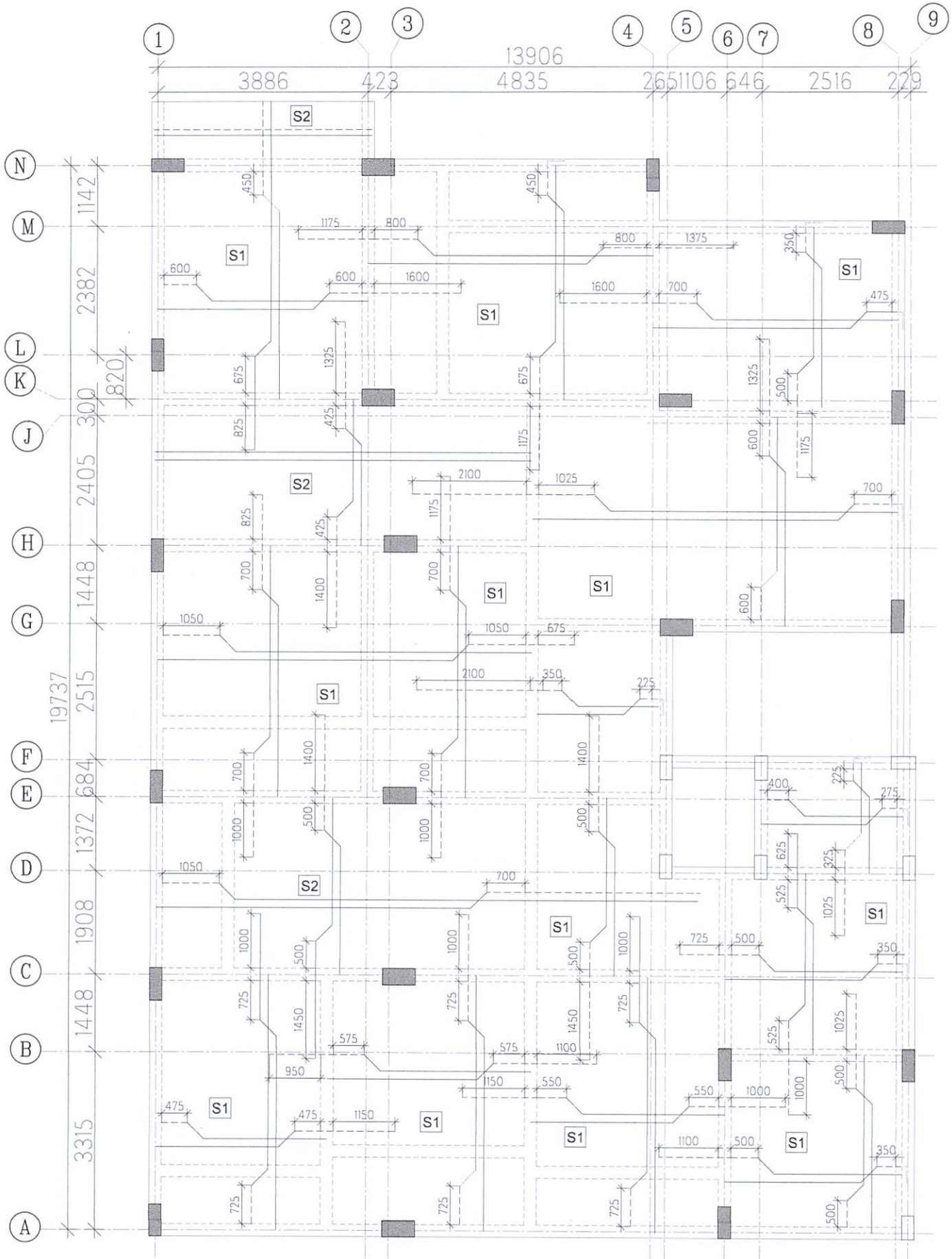
PROVIDED SPACING OF STEEL

Bottom reinforcement along x- dir.	Y	10	150	mm c/c
Bottom reinforcement along y- dir.	Y	10	150	mm c/c
Top reinforcement along x- dir.	Y	10	150	mm c/c
Top reinforcement along y- dir.	Y	10	150	mm c/c
Distribution Steel	Y	10	0	mm c/c

CHECK FOR DEFLECTION :

Provided area of reinforcement	=	523.60	mm^2
Mu/bd^2	=	0.59	
Actual Span / Effective depth Ratio	=	33.64	
Design service stress	=	88.89	
Modification factor - Tension reinf.	=	2.00	
Modification factor - Comp. reinf.	=	1.00	
Allowable Span /Effective depth Ratio	=	52.00	

<<< Hence SAFE in Deflection >>>



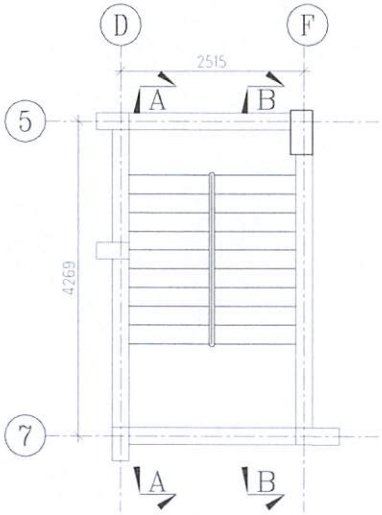
SLAB S1	TWO WAY SLAB		
Rein. Bars	Along	SLAB tk	
#10 @ 150 c/c	SLAB S.S	130mm tk	TWO WAY
#10 @ 150 c/c	SLAB L.S		

SLAB S2	ONE WAY SLAB		
Rein. Bars	Along	SLAB tk	
#10 @ 150 c/c	SLAB S.S	130mm tk	ONE WAY
#10 @ 150 c/c	SLAB L.S		

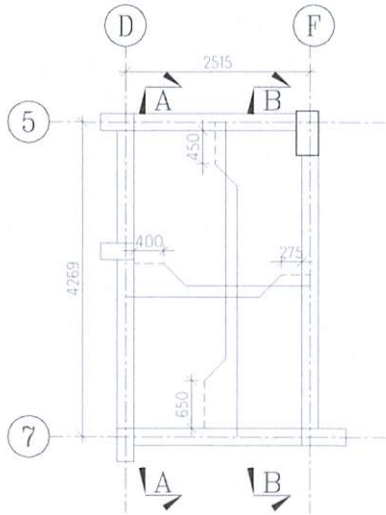
FLOOR SLAB AND ROOF SLAB LAYOUT

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STAIRCASE LAYOUT & OHT DETAIL

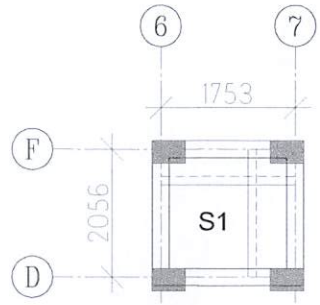


STAIRCASE PLAN



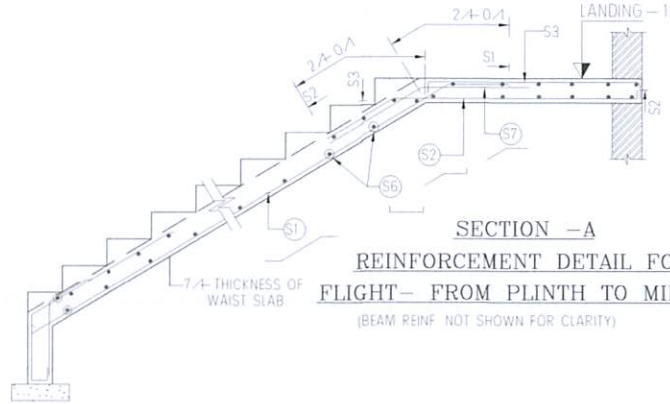
HEAD ROOM SLAB LAYOUT

SLAB S1	TWO WAY SLAB		
Rein. Bars	Along	SLAB tk	
#12 @ 150 c/c	SLAB S.S	130mm tk	TWO WAY
#12 @ 150 c/c	SLAB L.S		

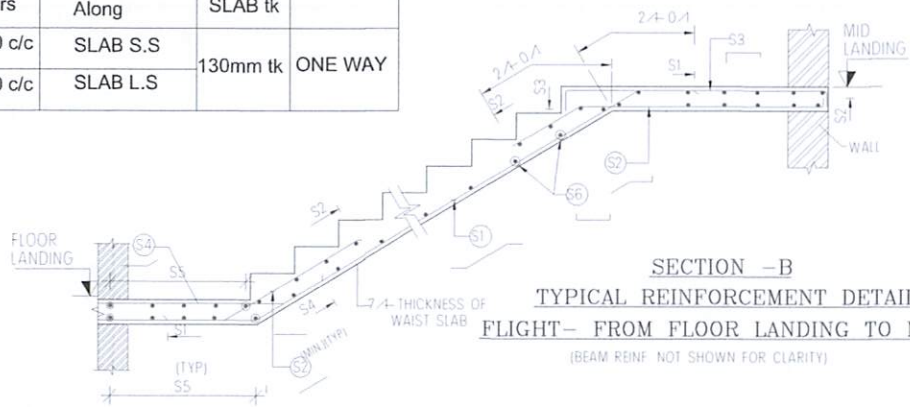


LIFT ROOM SLAB LAYOUT

SLAB S2	ONE WAY SLAB		
Rein. Bars	Along	SLAB tk	
#10 @ 150 c/c	SLAB S.S	130mm tk	ONE WAY
#10 @ 150 c/c	SLAB L.S		



SECTION – A
REINFORCEMENT DETAIL FOR
FLIGHT– FROM PLINTH TO MID LANDING
(BEAM REINF. NOT SHOWN FOR CLARITY)



SECTION – B
TYPICAL REINFORCEMENT DETAIL FOR
FLIGHT– FROM FLOOR LANDING TO MID LANDING
(BEAM REINF. NOT SHOWN FOR CLARITY)

STAIRCASE REINFORCEMENT:

WAIST SLAB THICKNESS	– 150mm
ALL MAIN BARS (S1 to S5)	Y12 AT 125 C/C
DISTRIBUTION REINFT. (S6)	Y8 AT 200 C/C



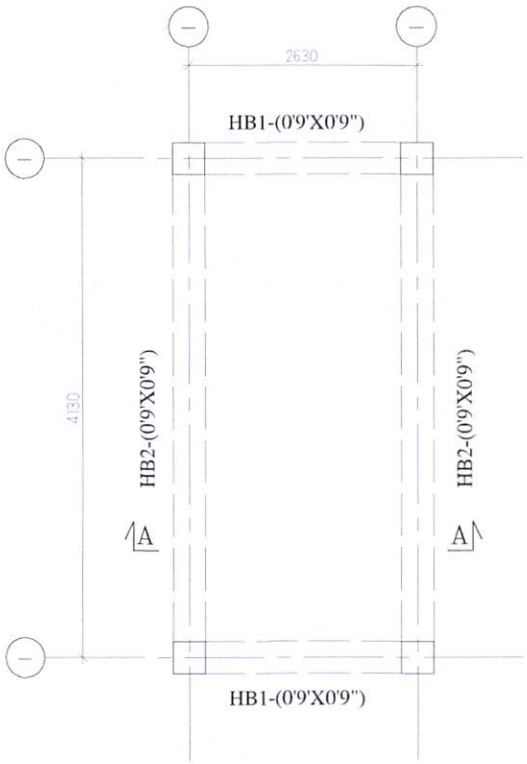
SECTION – B
TYPICAL REINFORCEMENT DETAIL FOR
FLIGHT– FROM MID LANDING TO FLOOR LANDING
(BEAM REINF. NOT SHOWN FOR CLARITY)

DISTRIBUTION REINFORCEMENT NOT SHOWN IN PLAN FOR CLARITY HOWEVER PROVIDE DISTRIBUTION REINFORCEMENT WHEREVER REQUIRED.

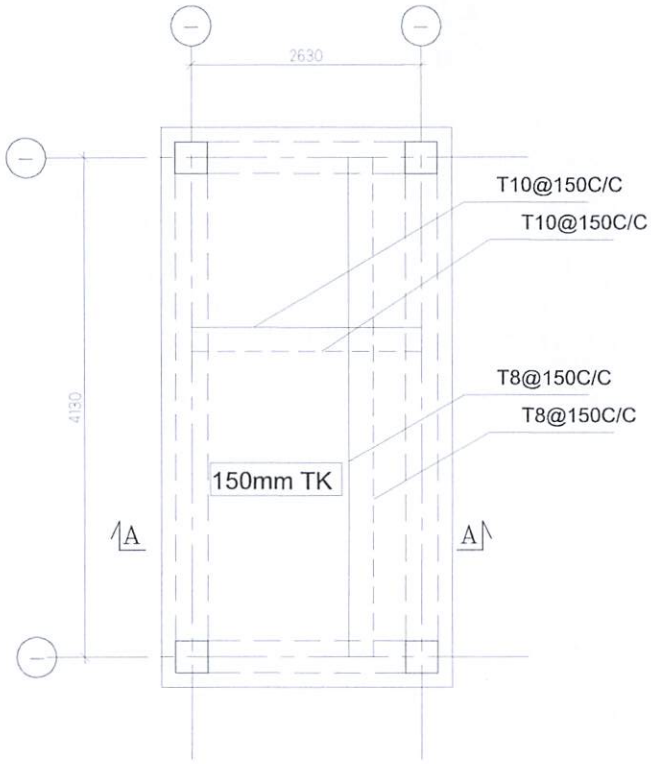
BARS SHOWN AS DOT /\/\IN SECTIONS ARE INDICATIVE EXTENT /\ SPACING OF THESE BARS SHALL BE PROVIDED AS PER PLAN DETAIL /\ SCHEDULE OF REINFORCEMENT.

STAIRCASE DETAIL

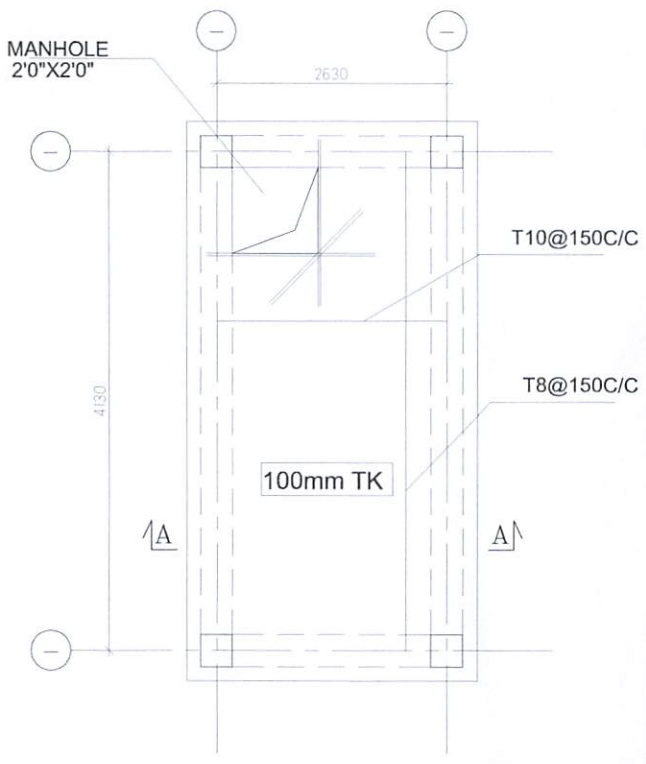
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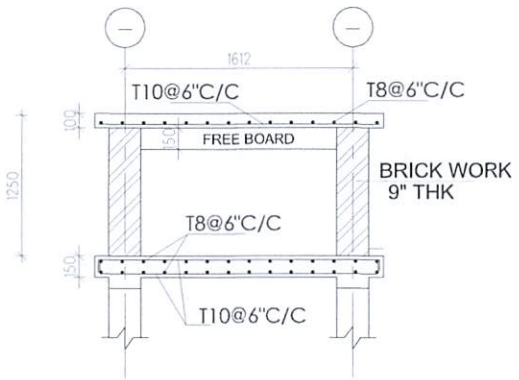
OHT BEAM LAYOUT



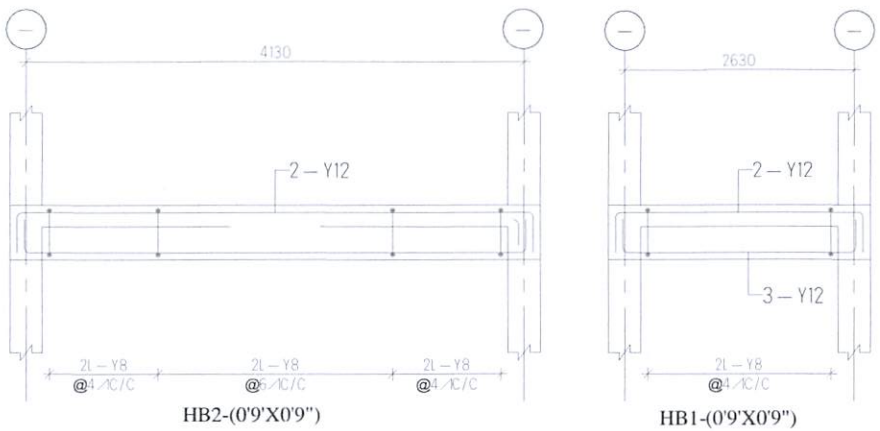
OHT BOTTOM SLAB LAYOUT



OHT TOP SLAB LAYOUT



SECTION A-A



OHT BEAM LS

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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 the **Thiru. J.MADHAN KUMAR** has enrolled as **Registered Structural Engineer Grade-II (SE)** with Chennai Metropolitan Development Authority (CMDA) who has the qualification of **M.E., (Structural Engg.)** and experience in the relevant field for **1 year**.

He has been given the Registration No.**SE/GR-II/19/08/144** on **05.08.2019.** 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 **05.08.2024**.



CHENNAI METROPOLITAN DEVELOPMENT AUTHORITY ★ CHENNAI-600 008 ★

5.8.19

N. MUNIPRASAD
ASSISTANT PLANNER
CHENNAI METROPOLITAN DEVELOPMENT AUTHORITY
CHENNAI - 600 008.

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