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18-2-2022

STABILITY CERTIFICATE

The Proposed Residential Building at Plot no.30 & 31, Old Door no. 28 & 29, New Door no.51 & 53, 28 th Cross Street, Indra nagar, Thiruvanmiyur, Chennai- 600 020, Comprised in S.NO.: 50/3, T.S. NO : 27 & 28, S.NO. : 50/ 2B1A4 & 50/2B1R, in block no 13 of Thiruvanmiyur Village, is designed to resist for earthquake, a check has been made and found safe.

1. The least dimension of the column is 300mm. in Stilt Floor .
2. The min grade of concrete is M25.
3. Foundation is designed for Stilt + Five upper floors only.
4. The design analysis using the code of practice,

For loading Standards as per IS – 875 part-2 and seismic force as per the code IS 1893. The building is designed and Detailing as per IS 456 and IS13920.

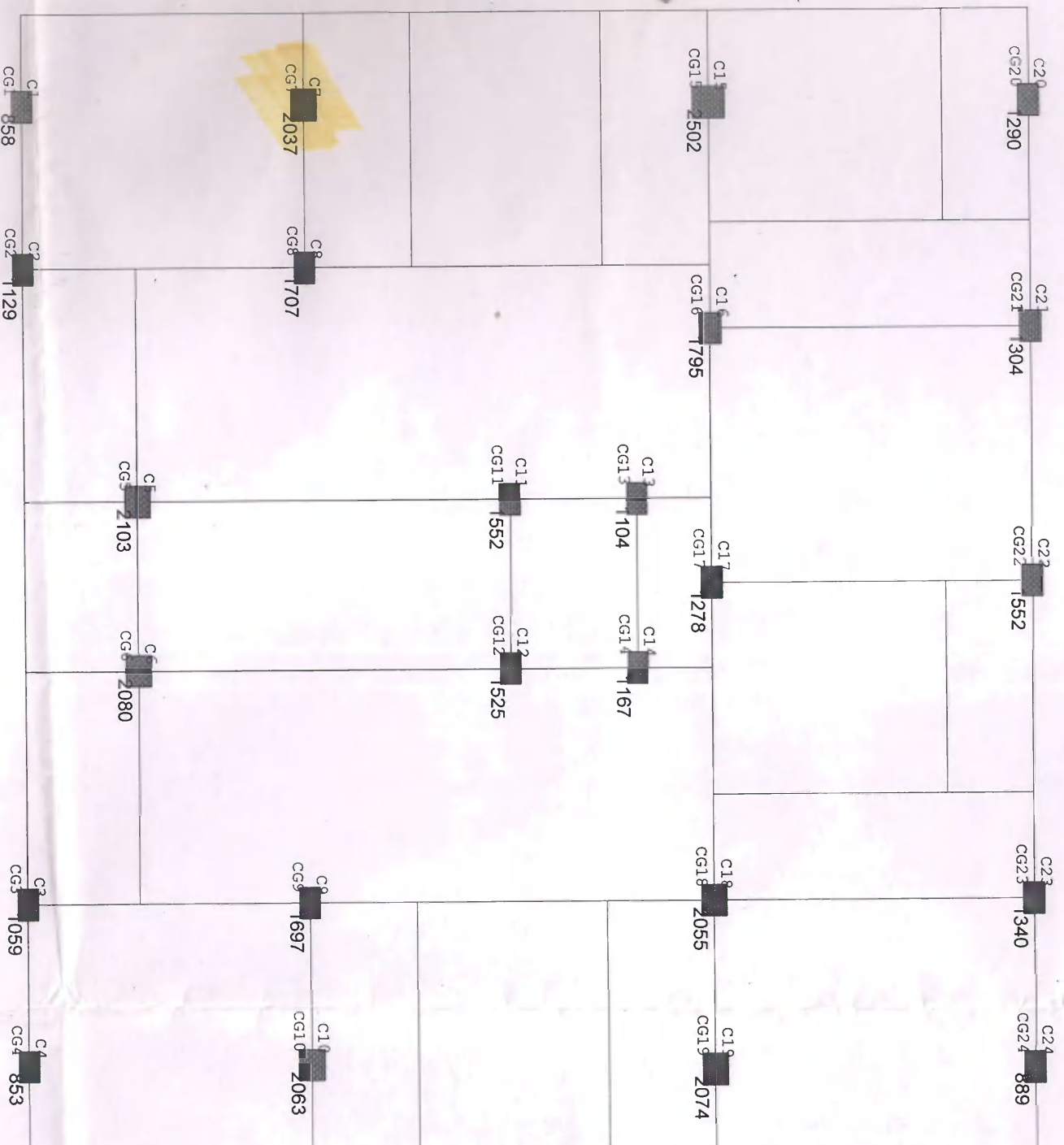
The building is earthquake resistant as per above IS Codes.

5. Pile Foundation is provided as per soil report from Prasad Armstrong Yen GEC (P) Ltd. Report no 064/GI/20-21 dt : March 5 ,2021.

We certify that the structural plan and details of the building submitted satisfy the structural safety requirements and the information given therein is factually correct to the best of our knowledge and understanding. The above building is structurally safe and sound as said above.

P. SANDHYA

Structural Engineer
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COLUMN LAYOUT

Sandhya

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PROJECT : INDRNGR

PLAN : INDRNGR

JOB NO. : 0

REF. NO. :

DATE : 02/20/22

TIME : 12:35:41

Building Version : 1.760

STRUDS Version : 7.0.0

Terms Used In Calculation :

In biaxial column design,

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

where,

P_u = external axial compressive load,

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

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

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

where,

M_u = external moment about centroidal axis,

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

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

i = serial no. of row of reinforcement,

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

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

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

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

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

Values of stress in steel :

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

(a) Fe 250,

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

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

(b) Fe 415,

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

curve

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

Stress	Stress level (N/mm ²)	Strain
0.800 f_{yd}	288.7	0.00144
0.850 f_{yd}	306.7	0.00163
0.900 f_{yd}	324.8	0.00192
0.950 f_{yd}	342.8	0.00241
0.975 f_{yd}	351.8	0.00276
1.000 f_{yd}	360.9	0.00380

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Values of stress in concrete :

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

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

Values of strain :

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

General Design Parameters :

Below FIRST FLOOR - Default Level at 6.000 m

Column size, (B x D) = 450 x 375 mm

Column height,(L) = 2550 mm

From analysis results, loads on column

Axial load,(P) = 1702.21 kN

MomentX,(M_x) = 10.53 kN-m

MomentY,(M_y) = 93.42 kN-m

$f_{ck} = 25.00 \text{ N/mm}^2$

$f_y = 415.00 \text{ N/mm}^2$

Load combination = 1.50 DL + 1.50 LL

Column Name	Orientation Angle
C7	0.00

Check For Slenderness :

$$\begin{aligned}\text{Slenderness RatioX} &= (L_o \times \text{Effective Length FactorX}) / \text{Depth} \\ &= (2550 \times 1.000) / 375 = 6.80 \leq 12.00,\end{aligned}$$

column is not slender in this direction.

$$\begin{aligned}\text{Slenderness RatioY} &= (L_o \times \text{Effective Length FactorY}) / \text{Width} \\ &= (2550 \times 1.000) / 450 = 5.67 \leq 12.00,\end{aligned}$$

column is not slender in this direction.

$$M_{x_MinEccen} = 34.044 \text{ kN.m}$$

$$M_{y_MinEccen} = 34.214 \text{ kN.m}$$

$$M_x = \max(M_x, M_{x_MinEccen}) + M_{uaddX} = 34.044 \text{ kN.m}$$

$$M_y = \max(M_y, M_{y_MinEccen}) + M_{uaddY} = 93.424 \text{ kN.m}$$

Calculation Of Eccentricities :

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

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

$$\text{Actual eccen}_x = M_x / P = 10.53/1702.21 = 6 \text{ mm}$$

$$\text{Actual eccen}_y = M_y / P = 93.42/1702.21 = 55 \text{ mm}$$

$$\text{eccen}_{x\text{Min}} = (L/500) + (D/30) = (2550/500) + (375/30) = 18 \text{ mm}$$

$$\text{eccen}_{x\text{Min}} = \max(18, 20) = 20 \text{ mm}$$

$$\text{eccen}_{y\text{Min}} = (L/500) + (B/30) = (2550/500) + (450/30) = 20 \text{ mm}$$

$$\text{eccen}_{y\text{Min}} = \max(20, 20) = 20 \text{ mm}$$

$$\text{eccen}_x = \max(\text{Actual eccen}_x, \text{eccen}_{x\text{Min}}) = \max(6, 20) = 20 \text{ mm}$$

$$\text{eccen}_y = \max(\text{Actual eccen}_y, \text{eccen}_{y\text{Min}}) = \max(55, 20) = 55 \text{ mm}$$

As per IS 456: 2000 Clause 39.3,
when the minimum eccentricity does not exceed 0.05 times the lateral dimension,
the column will be designed as an axially loaded column.

$$\text{eccen}_x(20 \text{ mm}) > 0.05 \times 450(23 \text{ mm})$$

$$\text{and eccen}_y(55 \text{ mm}) > 0.05 \times 375(19 \text{ mm})$$

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

Provided steel :

Provide #16 - 8 nos. (1608 mm²)

Biaxial Check Calculations :

For X axis :

Row No. (I)	A _{sl} (mm ²)	e _i	f _{sl} (N/mm ²)	f _d (N/mm ²)	(f _{sl} - f _d) (N/mm ²)	P _{usi} (kN)	x _i (mm)	M _{usi} (kN-m)
1	804	0.00301	354.02	11.15	342.87	275.75	140	38.47
2	804	0.00019	37.87	2.01	35.86	28.84	-140	-4.02
Total						304.59		34.44

$$x_{ux} = 346 \text{ mm} \quad P_{uc} = C_1 \cdot f_{ck} \cdot B \cdot D$$

$$C_1 = 0.36 k_u = 0.36 \times 346/375 = 0.332$$

$$P_{uc} = 0.332 \times 25.00 \times 450 \times 375$$

$$= 1400.10 \text{ kN}$$

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

$$= 1400.10 + (304.59)$$

$$= 1704.69 \text{ kN} > P, \text{ hence O.K.}$$

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

$$C_2 = 0.416 k_u = 0.416 \times 346/375$$

$$= 0.384$$

$$M_{uc} = 1400.10 \times (0.5 \times 375 - 0.384 \times 375) = 61.17 \text{ kN-m}$$

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

$$= 61.17 + (34.44)$$

$$= 95.61 \text{ kN-m}$$

For Y axis :

Row No. (I)	A _{sl} (mm ²)	e _i	f _{sl} (N/mm ²)	f _{cd} (N/mm ²)	(f _{sl} - f _{cd}) (N/mm ²)	P _{usi} (kN)	x _i (mm)	M _{usi} (kN-m)
1	402	0.00308	354.64	11.15	343.49	138.13	177	24.45
2	402	0.00206	330.05	11.15	318.90	128.24	59	7.57
3	402	0.00104	208.28	8.59	199.69	80.30	-59	-4.74
4	402	0.00002	3.98	0.22	3.76	1.51	-177	-0.27
Total						348.17		27.01

$$x_{uy} = 404 \text{ mm} \quad P_{uc} = C_1 \cdot f_{ck} \cdot B \cdot D$$

$$C_1 = 0.36 k_u = 0.36 \times 404/450 = 0.323$$

$$P_{uc} = 0.323 \times 25.00 \times 375 \times 450 \\ = 1364.50 \text{ kN}$$

$$P_{uy1} = P_{uc} + P_{us(\text{Total})} \\ = 1364.50 + (348.17) \\ = 1712.68 \text{ kN} > P, \text{ hence O.K.}$$

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

$$C_2 = 0.416 k_u = 0.416 \times 404/450 \\ = 0.374$$

$$M_{uc} = 1364.50 \times (0.5 \times 450 - 0.374 \times 450) = 77.52 \text{ kN-m}$$

$$M_{uy1} = M_{uc} + M_{us(\text{Total})} \\ = 77.52 + (27.01) \\ = 104.53 \text{ kN-m}$$

$$P_u/P_{uz} = 0.715$$

$$\text{For } P_u/P_{uz} \leq 0.2, \quad \alpha_n = 1.0$$

$$\text{For } P_u/P_{uz} \geq 0.8, \quad \alpha_n = 2.0$$

$$\text{For } P_u/P_{uz} = 0.715, \quad \alpha_n = 1.858$$

$$((M_{ux} / M_{ux1})^{\alpha_n}) + ((M_{uy} / M_{uy1})^{\alpha_n}) \\ = ((34.04/95.61)^{1.858}) + ((93.42/104.53)^{1.858}) \\ = 0.958 < 1.0, \text{ hence O.K.}$$

Ties Details :

Load combination for ties design = 1.50 DL + 1.50 LL

Calculation of diameter of ties :

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

- i) one-fourth the diameter of the largest longitudinal bar = $16/4 = 4 \text{ mm}$
- ii) in no case less than = 6 mm

Required diameter = maximum of (4, 6) = 6 mm
Provide 8 mm diameter for ties.

Calculation of spacing of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c), the pitch of transverse reinforcement shall be not more than the least of the following distances:

- i) the least lateral dimension of the compression member = 375 mm
- ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to

be tied

$$= 16 \times 16 = 256 \text{ mm}$$

- iii) 300 mm

Required spacing = minimum of (375, 256, 300) = 256 mm
Provide ties 8 @ 250 mm c/c

SUMMARY :

CG7 : FIRST FLOOR - Default Level at 6.000 m
Provide rectangular section : 450 x 375 mm
Provide #16 - 8 nos. (1608 mm²)
Provide ties #8 @ 250 mm c/c
Provide 2 legged ties along width and
4 legged ties along depth.

General Design Parameters :

Below Floor1 - Default Level at 3.000 m

Column size, (B x D) = 450 x 375 mm

Column height,(L) = 2775 mm

From analysis results, loads on column

Axial load,(P) = 2037.27 kN

MomentX,(M_x) = 9.71 kN-m

MomentY,(M_y) = 86.71 kN-m

f_{ck} = 25.00 N/mm²

f_y = 415.00 N/mm²

Load combination = 1.50 DL + 1.50 LL

Column Name	Orientation Angle
C7	0.00

Check For Slenderness :

Slenderness RatioX = (L_o x Effective Length FactorX) / Depth
= (2775 x 1.000) / 375 = 7.40 <= 12.00,

column is not slender in this direction.

Slenderness RatioY = (L_o x Effective Length FactorY) / Width
= (2775 x 1.000) / 450 = 6.17 <= 12.00,

column is not slender in this direction.

M_{x_MinEccen} = 40.745 kN.m

M_{y_MinEccen} = 41.866 kN.m

M_x = max(M_x, M_{x_MinEccen}) + M_{uaddX} = 40.745 kN.m

M_y = max(M_y, M_{y_MinEccen}) + M_{uaddY} = 86.710 kN.m

Calculation Of Eccentricities :

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

eccentricity

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

$$\text{Actual eccen}_x = M_x / P = 9.71/2037.27 = 5 \text{ mm}$$

$$\text{Actual eccen}_y = M_y / P = 86.71/2037.27 = 43 \text{ mm}$$

$$\text{eccen}_{x\text{Min}} = (L/500) + (D/30) = (2775/500) + (375/30) = 18 \text{ mm}$$

$$\text{eccen}_{x\text{Min}} = \max(18, 20) = 20 \text{ mm}$$

$$\text{eccen}_{y\text{Min}} = (L/500) + (B/30) = (2775/500) + (450/30) = 21 \text{ mm}$$

$$\text{eccen}_{y\text{Min}} = \max(21, 20) = 20 \text{ mm}$$

$$\text{eccen}_x = \max(\text{Actual eccen}_x, \text{eccen}_{x\text{Min}}) = \max(5, 20) = 20 \text{ mm}$$

$$\text{eccen}_y = \max(\text{Actual eccen}_y, \text{eccen}_{y\text{Min}}) = \max(43, 21) = 43 \text{ mm}$$

As per IS 456: 2000 Clause 39.3,

when the minimum eccentricity does not exceed 0.05 times the lateral dimension, the column will be designed as an axially loaded column.

$$\text{eccen}_x(20 \text{ mm}) > 0.05 \times 450(23 \text{ mm})$$

$$\text{and eccen}_y(43 \text{ mm}) > 0.05 \times 375(19 \text{ mm})$$

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

Provided steel :

Provide #12 - 16 nos. + #16 - 4 nos. (2614 mm²)

Biaxial Check Calculations :

For X axis :

Row No. (I)	A _{st} (mm ²)	e _i	f _{st} (N/mm ²)	f _{sd} (N/mm ²)	(f _{st} - f _{sd}) (N/mm ²)	P _{usl} (kN)	x _i (mm)	M _{usl} (kN-m)
1	1307	0.00304	354.29	11.15	343.14	448.45	140	62.56
2	1307	0.00040	79.91	4.01	75.90	99.20	-140	-13.84
Total						547.65		48.72

$$x_{uc} = 369 \text{ mm} \quad P_{uc} = C_1 \cdot f_{ck} \cdot B \cdot D$$

$$C_1 = 0.36 \quad k_u = 0.36 \times 369/375 = 0.354$$

$$P_{uc} = 0.354 \times 25.00 \times 450 \times 375 \\ = 1495.02 \text{ kN}$$

$$P_{ux1} = P_{uc} + P_{us(\text{Total})} \\ = 1495.02 + (547.65) \\ = 2042.67 \text{ kN} > P, \text{ hence O.K.}$$

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

$$C_2 = 0.416 \quad k_u = 0.416 \times 369/375 \\ = 0.410$$

$$M_{uc} = 1495.02 \times (0.5 \times 375 - 0.410 \times 375) = 50.74 \text{ kN-m}$$

$$M_{ux1} = M_{uc} + M_{us(\text{Total})} \\ = 50.74 + (48.72) \\ = 99.46 \text{ kN-m}$$

For Y axis :

Row No. (I)	A _{sl} (mm ²)	e _i	f _{sl} (N/mm ²)	f _{cl} (N/mm ²)	(f _{sl} - f _{cl}) (N/mm ²)	P _{usi} (kN)	x _i (mm)	M _{usi} (kN-m)
1	402	0.00310	354.79	11.15	343.64	138.19	177	24.46
2	226	0.00276	351.82	11.15	340.67	77.06	136	10.49
3	226	0.00244	343.57	11.15	332.42	75.19	97	7.31
4	226	0.00212	332.05	11.15	320.90	72.59	58	4.23
5	226	0.00179	316.98	11.03	305.94	69.20	19	1.35
6	226	0.00147	291.73	10.37	281.36	63.64	-19	-1.24
7	226	0.00115	229.88	9.13	220.74	49.93	-58	-2.91
8	226	0.00083	165.35	7.31	158.04	35.75	-97	-3.48
9	226	0.00050	100.82	4.91	95.91	21.69	-136	-2.95
10	402	0.00016	32.98	1.76	31.22	12.55	-177	-2.22
Total						615.79		35.04

$$x_{uy} = 422 \text{ mm} \quad P_{uc} = C_1 \cdot f_{ck} \cdot B \cdot D$$

$$C_1 = 0.36 k_u = 0.36 \times 422/450 = 0.338$$

$$P_{uc} = 0.338 \times 25.00 \times 375 \times 450 \\ = 1423.83 \text{ kN}$$

$$P_{uy1} = P_{uc} + P_{us(Total)} \\ = 1423.83 + (615.79) \\ = 2039.62 \text{ kN} > P, \text{ hence O.K.}$$

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

$$C_2 = 0.416 k_u = 0.416 \times 422/450 \\ = 0.390$$

$$M_{uc} = 1423.83 \times (0.5 \times 450 - 0.390 \times 450) = 70.48 \text{ kN-m}$$

$$M_{uy1} = M_{uc} + M_{us(Total)} \\ = 70.48 + (35.04) \\ = 105.52 \text{ kN-m}$$

$$P_u/P_{uz} = 0.759$$

$$\text{For } P_u/P_{uz} \leq 0.2, \quad \alpha_n = 1.0$$

$$\text{For } P_u/P_{uz} \geq 0.8, \quad \alpha_n = 2.0$$

$$\text{For } P_u/P_{uz} = 0.759, \quad \alpha_n = 1.932$$

$$((M_{ux}/M_{ux1})^{\alpha_n}) + ((M_{uy}/M_{uy1})^{\alpha_n}) \\ = ((40.75/99.46)^{1.932}) + ((86.71/105.52)^{1.932}) \\ = 0.863 < 1.0, \text{ hence O.K.}$$

Ties Details :

Load combination for ties design = 1.50 DL + 1.50 LL

Calculation of diameter of ties :

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

i) one-fourth the diameter of the largest longitudinal bar = $16/4 = 4 \text{ mm}$

ii) in no case less than = 6 mm

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

Provide 8 mm diameter for ties.

Calculation of spacing of ties :

As per IS 456: 2000 Clause 26.5.3.2 (c), the pitch of transverse reinforcement shall be not more than the least of the following distances:

i) the least lateral dimension of the compression member = 375 mm

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

= $16 \times 12 = 192$ mm

iii) 300 mm

Required spacing = minimum of (375, 192, 300) = 192 mm

Provide ties 8 @ 190 mm c/c

SUMMARY :

CG7 : Floor1 - Default Level at 3.000 m

Provide rectangular section : 450 x 375 mm

Provide #12 - 16 nos. + #16 - 4 nos. (2614 mm²)

Provide ties #8 @ 190 mm c/c

Provide 2 legged ties along width and

4 legged ties along depth.

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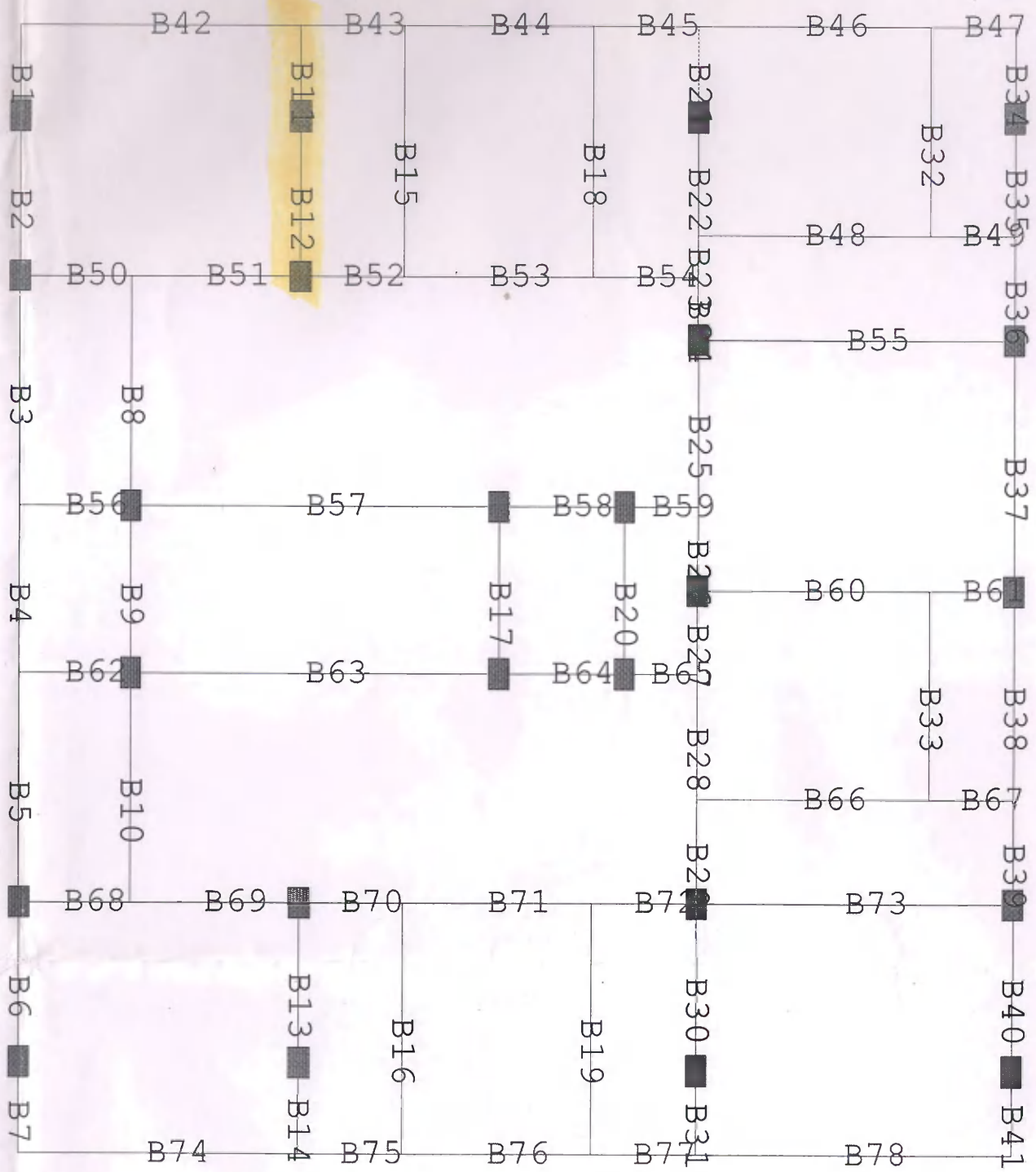
No. SE/GR-II/19/03/014

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BEAM LAYOUT

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PROJECT : INDRNGR

PLAN : INDRNGR

JOB NO. : 0

REF. NO. :

DATE : 02/20/22

TIME : 12:12:52

Building Version : 1.760

STRUDS Version : 7.0.0

Beam Detail Report

Floor1 - Default Level

Load Combinations used in beam design :

Load Combi No	Load Combination	Load Combi No	Load Combination
1	1.50 DL + 1.50 LL	2	1.20 DL + 1.20 LL + 1.20 EQL X+
3	1.20 DL + 1.20 LL + 1.20 EQL X-	4	1.20 DL + 1.20 LL + 1.20 EQL Y+
5	1.20 DL + 1.20 LL + 1.20 EQL Y-	6	1.50 DL + 1.50 EQL X+
7	1.50 DL + 1.50 EQL X-	8	1.50 DL + 1.50 EQL Y+
9	1.50 DL + 1.50 EQL Y-		

Design of Beam Group : BG11

Beam : B11

Material Properties :

$f_{ck} = 20.000 \text{ N/mm}^2$

$f_y = 415.000 \text{ N/mm}^2$

1.Design Data :

1.1)Dimensions:

Beam Type = 'Rectangular Beam'

Width (b) = 230 mm

Total depth (D) = 800 mm

**P. SANDHYA, B.E. (Civill) M.E (Struct.),
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Sandhya

[illegible]

Eq.B.M. Hog in kN-m	51.71 5	77.08 2	106.3 07	131.7 18	159.4 35	187.7 16	216.5 85	246.06 6	276.18 2	306.95 9	338.42 0
Eq.B.M.Sag in kN-m	123.7 89	98.42 3	69.19 8	43.78 6	16.07 0	0.000	0.000	0.000	0.000	0.000	0.000
Top A_{st_req.} in mmB	191.0 42	288.1 14	402.9 85	505.7 30	621.1 09	742.7 46	871.4 27	1008.1 00	1153.9 44	1310.4 47	1479.5 47
Bot A_{st_req.} in mmB	473.3 71	371.6 60	257.6 90	161.1 72	58.43 2	0.000	0.000	0.000	0.000	0.000	0.000

Max.Area of steel req. at Top = 1479.547 mmB at 1.000L

Max.Area of steel req. at Bottom = 473.371 mmB at 0.000L

2.1.2)Minimum and Maximum Steel Checks :

a)Minimum Steel Check as per IS 456:2000 Clause 26.5.1.1(a)

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

$$= 361.322 \text{ mmB}$$

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

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

$$= 7359.997 \text{ mmB}$$

2.1.3)Provided Steel

2.1.3.1)Provided Bottom Steel :

Straight bars = 2 - #16 +1 - #12

Curtail Bars = ---

Total Area of Steel at bottom at 0.000L = 515.221 mmB > A_{st_req.} (O.K.)

2.1.3.2)Provided Top Steel :

Straight bars = 4 - #20 +2 - #12

Extra bars at Left = ---

Extra bars at Right = 2 - #20 0.940,0.940 +1 - #12 0.940,0.940

Total Area of Steel at Top

At Left Support = 1482.832 mmB

At Right Support = 2224.248 mmB

2.2)Computation of Shear Reinforcement(Stirrups) :

$$\tau_{auc} = 0.675 \text{ N/mmB}$$

$$S_{vmin} = 450 \text{ mm}$$

Note :

Design Shear = (Equivalent S.F.)-(V_{uc})

$$V_{usT} = (T_u/b_1 + V_u/2.5)(d/d_1)$$

Where,

T_u = Torsional moment,

V_u = Ultimate shear force

b_1 = Center to center distance between corner bars in the direction of width

d_1 = Center to center distance between corner bars in the direction of depth

(As per Clause 41.4.3 of IS 456:2000)

If Design Shear < V_{usT} then Design Shear = V_{usT}

V_{usT} = Minimum shear for torsional transverse Reinforcement

2.2.1) Computation of Steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
S.F.in kN	- 201.27 3	- 205.79 8	- 210.32 2	- 214.84 6	- 219.37 0	- 223.89 5	- 228.41 9	- 232.94 3	- 237.46 7	- 241.99 2	- 246.51 6
T_u in kN-m	33.312	33.312	33.312	33.312	33.312	33.312	33.312	33.312	33.312	33.312	33.312
Eq. S.F. in kN	433.00 8	437.53 2	442.05 7	446.58 1	451.10 5	455.62 9	460.15 4	464.67 8	469.20 2	473.72 6	478.25 0
V_{usT} in kN	330.12 3	332.07 8	334.03 3	335.98 8	337.94 3	339.89 8	341.85 3	343.80 8	345.76 3	347.71 8	349.67 3
V_{uc} in kN	103.08 2	103.08 2	103.08 2	119.14 4	119.14 4	119.14 4	119.14 4	119.14 4	119.14 4	119.14 4	119.14 4
Design Shear in kN	330.12 3	334.45 0	338.97 4	335.98 8	337.94 3	339.89 8	341.85 3	345.53 3	350.05 8	354.58 2	359.10 6
Spacin g req. in mm for 8 Dia	167	166	164	164	163	162	161	160	159	157	78

2.2.2) Provided Shear Steel (Stirrups) :

Nominal Zone :#8 @ 125 mm c/c from 0.000 to 1.250 .

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

Design of Beam Group : BG12

Beam : B12

Material Properties :

$f_{ck} = 20.000 \text{ N/mm}^2$

$f_y = 415.000 \text{ N/mm}^2$

1.Design Data :

1.1)Dimensions:

Beam Type	= 'Rectangular Beam'
Width (b)	= 230 mm
Total depth (D)	= 450 mm
Effective Tension Cover	= 33mm
Effective Compression Cover	= 33mm
Clear side Cover	= 25mm
Effective depth (d)	= 417mm
Span	= 2.300m
Clear Span (L)	= 2.300m

1.2)Loading on the Beam :

1.2.1)Self wt. of beam : -2.588 kN/m

1.2.2)Slab Load

Dead Load :

Trapezoidal Load in kN/m

St.Pt 0.000 m L = 2.300 m Max. Value = -6.825 kN/m

UDL in kN/m

St.Pt 0.000 m L = 2.300 m Max. Value = -7.688 kN/m

LiveLoad

Trapezoidal Load in kN/m

St.Pt 0.000 m L = 2.300 m Max. Value = -2.600 kN/m

UDL in kN/m

St.Pt 0.000 m L = 2.300 m Max. Value = -1.500 kN/m

1.2.3)Wall Load

UDL in kN/m

St.Pt 0.000 m L = 2.300 m Max. Value = -4.845 kN/m

2.Computation of Area of steel :

2.1) Flexural Steel at Top and Bottom

2.1.1) Computation of theoretical steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
B.M. Hog in kN-m	134.281	105.732	79.020	54.223	31.297	10.145	18.485	29.725	42.452	56.570	71.984
Load Combi-No	7	7	7	7	7	7	6	6	6	6	6
B.M. Sag in kN-m	12.684	12.103	9.685	5.352	0.000	0.000	9.328	27.219	43.621	58.633	72.349
Load Combi-No	6	6	6	6	1	1	7	7	7	7	7
Torsion in kN-m	7.917	7.917	7.917	7.917	7.917	7.917	7.917	7.917	7.917	7.917	7.917
Load Combi-No	9	9	9	9	9	9	9	9	9	9	9
Eq.B.M.Hog in kN-m	148.050	119.501	92.789	67.992	45.066	23.914	32.254	43.494	42.452	56.570	71.984
Eq.B.M.Sag in kN-m	12.684	12.103	9.685	5.352	0.000	3.624	9.328	27.219	57.390	72.402	86.118
Top A_{st req.} in mmB	1189.941	984.023	732.333	507.275	321.716	164.702	225.205	309.627	409.740	527.360	665.344
Bot A_{st req.} in mmB	278.097	178.754	161.418	130.698	0.000	24.198	158.872	290.494	419.204	545.198	668.747

Max.Area of steel req. at Top = 1189.941 mmB at 0.000L

Max.Area of steel req. at Bottom = 668.747 mmB at 1.000L

2.1.2) Minimum and Maximum Steel Checks :

a) Minimum Steel Check as per IS 456:2000 Clause 26.5.1.1(a)

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

$$= 196.442 \text{ mmB}$$

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

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

$$= 4140.000 \text{ mmB}$$

2.1.3) Provided Steel

2.1.3.1) Provided Bottom Steel :

Straight bars = 3 - #16 + 1 - #12

Curtail Bars = ---

Total Area of Steel at bottom at 1.000L = 716.283 mmB > $A_{st_req.}$ (O.K.)

2.1.3.2) Provided Top Steel :

Straight bars = 2 - #20 + 1 - #12

Extra bars at Left = +2 - #20 0.940, 0.940 + 1 - #12 0.940, 0.940

Extra bars at Right = ---

Total Area of Steel at Top

At Left Support = 1482.832 mmB

At Right Support = 741.416 mmB

2.2) Computation of Shear Reinforcement (Stirrups) :

$\tau_{auc} = 0.566 \text{ N/mmB}$

$S_{vmin} = 313 \text{ mm}$

Note :

Design Shear = (Equivalent S.F.) - (V_{uc})

$V_{usT} = (T_u/b_1 + V_u/2.5)(d/d_1)$

Where,

T_u = Torsional moment,

V_u = Ultimate shear force

b_1 = Center to center distance between corner bars in the direction of width

d_1 = Center to center distance between corner bars in the direction of depth

(As per Clause 41.4.3 of IS 456:2000)

If Design Shear < V_{usT} then Design Shear = V_{usT}

V_{usT} = Minimum shear for torsional transverse Reinforcement

2.2.1) Computation of Steel

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
S.F. in kN	127.98 2	120.20 2	112.00 6	103.67 8	95.754	88.246	81.155	74.481	68.223	64.270	69.695
T_u in kN-m	7.917	7.917	7.917	7.917	7.917	7.917	7.917	7.917	7.917	7.917	7.917
Eq. S.F. in kN	183.05 7	175.27 8	167.08 2	158.75 4	150.82 9	143.32 2	136.23 1	129.55 6	123.29 8	119.34 6	124.77 0

V_{usT} in kN	121.26 2	117.65 8	113.86 0	110.00 1	106.33 0	102.85 1	99.566	96.473	93.574	91.742	94.256
V_{uc} in kN	69.372	69.372	69.372	69.372	69.372	54.322	54.322	54.322	54.322	54.322	54.322
Design Shear in kN	121.26 2	117.65 8	113.86 0	110.00 1	106.33 0	102.85 1	99.566	96.473	93.574	91.742	94.256
Spacin g req. in mm for 8 Dia	122	126	130	135	139	144	149	153	158	161	157

2.2.2) Provided Shear Steel (Stirrups) :

Nominal Zone : #8 @ 100 mm c/c from 0.000 to 2.300 .

Note : Nominal zone is where No Shear Reinforcement is needed

Hence provide minimum Shear Reinforcement of 0.75 d

(As per Clause 26.5.1.5 of IS 456:2000)

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Sandhya

PROJECT : INDRNGR

PLAN : INDRNGR

JOB NO. : 0

REF. NO. :

DATE : 02/20/22 TIME : 12:07:46

Building Version : 1.760

STRUDS Version : 7.0.0

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Design Of Slabs : Default Level

Design Method : Limit State Method

Design Code : IS 456:2000

Design Of Slab

Slab Name : S11

Slab Type : OneWay Slab

Grade of Concrete : M25

Grade of Steel : Fe415

Dimensions : Lx = 3.000 m , Ly = 1.200 m , Thickness = 150 mm

Span Ratio : Longer/Shorter = 2.500

Boundary Condition : Simply supported slab

Loading on Slab : (Table 1)

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

Self Weight (1) kN/m ²	Floor Finish (2) kN/m ²	Sunk Load (3) kN/m ²	Total DL (4) kN/m ²	Live Load (5) kN/m ²	Total LL (6) kN/m ²	Total Design Load 1.5 x [(4)+(6)] kN/m ²
3.750	1.500	3.000	8.250	2.000	2.000	15.375

Deflection Check : (Table 2)

As per clause 23.2.1 of IS 456:2000:

(Span/Depth) Ratio	Modification Factor (a)	Basic Factor (b)	Permissible Ratio (a x b)	Status
9.600	2.114	20.000	42.281	OK

Moment And Steel Calculations :

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

$$M_x = \alpha_x \times W \times L_x \times L_x:$$

$$M_y = \alpha_y \times W \times L_x \times L_x:$$

where

M_x, M_y = moments on strips of unit width spanning L_x and L_y respectively,

W = Total Design Load = 15.375 kN/m²

L_x = Shorter span = 1.200 m

Position	Moment kN-m	A _{st, req} per metre mm ²	Steel Detail dia @ spc mm c/c	A _{st, prv} per metre mm ²	Remark
Main Mid Span	2.768	180.000	#8 @ 200	251.327	

Provide #8 @ 275 (182.784 mm^2) as Distribution Steel

Minimum Steel Check :

As per clause 26.5.2.1 of IS 456:2000:

Minimum area of steel For Main Steel = $0.12 \times \text{C/S Area}$

= 180.000 mm^2

Minimum area of steel For Distribution Steel = $0.12 \times \text{C/S Area}$

= 180.000 mm^2

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PROJECT : INDRNGR
PLAN : INDRA NAGAR

JOB NO. : 0 REF. NO. :
DATE : 02/20/22 TIME : 11:52:29
Building Version : 1.140 STRUDS Version : Struds v12

EARTHQUAKE LOAD REPORT : As per IS:1893(Part 1) - 2002

Earthquake Load Generation Method : Response Spectrum Method
Wall Load Transfer Type : Half top and half bottom load is transferred on floor

Modal Combination Method: SRSS

Earthquake Load Parameters:

Parameters	Values
Seismic zone factor,Z	0.160
Importance factor,I	1.000
Response reduction factor,R	3.000
Percentage damping	5 %
No of Modes	3
Soil type	Medium Soil

Percentage Design Live Load:

Floor	%	Floor	%	Floor	%	Floor	%
Floor1	25.000	FIRST FLOOR	25.000	SECOND FLOOR	25.000	THIRD FLOOR	25.000
FOURTH FLOOR	25.000	TERRACE	25.000	HEADRM	25.000	OHT	0.000

EQ Loads Generated By Response Spectrum:

Floor Wise Distribution of Base Shear

Floor	Base Shear			
	X - Direction EQX		Y - Direction EQY	
	Value (kN)	% Contribution	Value (kN)	% Contribution
Floor1	20.391	3.663	16.490	3.909
FIRST FLOOR	50.990	9.160	40.735	9.657
SECOND FLOOR	81.290	14.604	63.772	15.118

THIRD FLOOR	108.165	19.432	83.085	19.696
FOURTH FLOOR	129.688	23.298	97.400	23.090
TERRACE	141.482	25.417	103.764	24.598
HEADRM	17.735	3.186	12.031	2.852
OHT	6.903	1.240	4.558	1.080
Σ	556.644	100.000	421.834	100.000

Distribution of Floor Base Shear to Column

Floor1 EQX = 20.391 kN EQY = 16.490 kN

Column

Node No.	Base Shear					
	X - Direction EQX			Y - Direction EQY		
	Value (kN)	% Contribution		Value (kN)	% Contribution of	
		Floor	Total		Floor	Total
41	0.505	2.477	0.091	0.418	2.537	0.099
42	0.411	2.014	0.074	0.346	2.100	0.082
47	0.722	3.543	0.130	0.643	3.902	0.153
48	0.754	3.696	0.135	0.683	4.142	0.162
29	1.053	5.163	0.189	0.932	5.655	0.221
30	1.072	5.255	0.193	0.966	5.855	0.229
76	1.022	5.014	0.184	0.764	4.633	0.181
77	0.809	3.970	0.145	0.617	3.740	0.146
79	0.826	4.051	0.148	0.607	3.681	0.144
25	0.894	4.385	0.161	0.794	4.816	0.188
26	1.081	5.299	0.194	0.924	5.604	0.219
71	1.538	7.542	0.276	1.193	7.238	0.283
68	1.052	5.160	0.189	0.844	5.117	0.200
63	1.222	5.994	0.220	0.921	5.584	0.218
62	1.259	6.176	0.226	0.875	5.307	0.207
73	0.539	2.645	0.097	0.353	2.140	0.084
74	0.874	4.286	0.157	0.625	3.791	0.148
66	0.816	4.003	0.147	0.644	3.902	0.153
38	0.426	2.091	0.077	0.386	2.339	0.091
37	0.571	2.802	0.103	0.540	3.276	0.128
34	0.515	2.524	0.092	0.458	2.778	0.109
33	0.424	2.081	0.076	0.346	2.099	0.082
54	1.110	5.441	0.199	0.859	5.209	0.204
55	0.895	4.390	0.161	0.751	4.556	0.178
Σ	20.391	100.000	3.663	16.490	100.000	3.909

FIRST FLOOR EQX = 50.990 kN EQY = 40.735 kN
Column

Node No.	Base Shear					
	X - Direction EQX			Y - Direction EQY		
	Value (kN)	% Contribution		Value (kN)	% Contribution of	
		Floor	Total		Floor	Total
102	1.439	2.822	0.259	1.088	2.671	0.258
103	1.323	2.594	0.238	1.020	2.504	0.242
108	1.781	3.493	0.320	1.417	3.477	0.336
109	1.838	3.605	0.330	1.491	3.660	0.353
89	2.529	4.960	0.454	2.099	5.152	0.498
90	2.570	5.040	0.462	2.175	5.340	0.516
134	2.476	4.856	0.445	1.750	4.295	0.415
114	2.044	4.008	0.367	1.534	3.765	0.364
116	2.057	4.034	0.370	1.624	3.986	0.385
85	2.298	4.507	0.413	1.976	4.850	0.468
86	2.727	5.348	0.490	2.434	5.974	0.577
100	3.734	7.323	0.671	3.085	7.572	0.731
129	2.609	5.117	0.469	2.042	5.012	0.484
124	3.013	5.908	0.541	2.210	5.425	0.524
123	3.070	6.021	0.552	2.286	5.612	0.542
131	1.384	2.715	0.249	0.984	2.417	0.233
132	2.139	4.196	0.384	1.505	3.695	0.357
127	1.883	3.693	0.338	1.388	3.407	0.329
80	1.095	2.147	0.197	1.064	2.611	0.252
97	1.465	2.873	0.263	1.376	3.377	0.326
94	1.342	2.631	0.241	1.163	2.854	0.276
93	1.086	2.129	0.195	0.950	2.331	0.225
83	2.785	5.463	0.500	2.246	5.515	0.533
84	2.303	4.516	0.414	1.831	4.496	0.434
Σ	50.990	100.000	9.160	40.735	100.000	9.657

SECOND FLOOR EQX = 81.290 kN EQY = 63.772 kN
Column

Node No.	Base Shear					
	X - Direction EQX			Y - Direction EQY		
	Value (kN)	% Contribution		Value (kN)	% Contribution of	
		Floor	Total		Floor	Total

163	2.454	3.019	0.441	1.781	2.793	0.422
164	2.353	2.894	0.423	1.744	2.734	0.413
169	2.793	3.436	0.502	2.109	3.307	0.500
170	2.878	3.540	0.517	2.219	3.480	0.526
150	3.953	4.863	0.710	3.167	4.965	0.751
151	4.007	4.930	0.720	3.279	5.142	0.777
174	3.852	4.739	0.692	2.658	4.168	0.630
175	3.243	3.989	0.583	2.418	3.791	0.573
177	3.269	4.021	0.587	2.612	4.095	0.619
146	3.751	4.615	0.674	3.141	4.925	0.745
147	4.342	5.341	0.780	3.827	6.002	0.907
161	5.854	7.201	1.052	4.844	7.597	1.148
158	4.135	5.086	0.743	3.180	4.986	0.754
185	4.736	5.826	0.851	3.406	5.341	0.807
184	4.847	5.963	0.871	3.612	5.665	0.856
189	2.252	2.770	0.405	1.622	2.543	0.384
172	3.324	4.089	0.597	2.318	3.635	0.550
156	2.933	3.609	0.527	2.088	3.274	0.495
138	1.799	2.213	0.323	1.748	2.740	0.414
137	2.364	2.908	0.425	2.186	3.428	0.518
155	2.183	2.686	0.392	1.850	2.901	0.439
154	1.780	2.190	0.320	1.553	2.435	0.368
144	4.419	5.436	0.794	3.513	5.508	0.833
145	3.770	4.637	0.677	2.898	4.544	0.687
Σ	81.290	100.000	14.604	63.772	100.000	15.118

THIRD FLOOR EQX = 108.165 kN EQY = 83.085 kN
Column

Node No.	Base Shear					
	X - Direction EQX			Y - Direction EQY		
	Value (kN)	% Contribution		Value (kN)	% Contribution of	
		Floor	Total		Floor	Total
201	3.409	3.152	0.612	2.407	2.897	0.571
200	3.348	3.096	0.602	2.416	2.908	0.573
227	3.669	3.392	0.659	2.674	3.218	0.634
226	3.775	3.490	0.678	2.811	3.384	0.666
192	5.193	4.801	0.933	4.052	4.876	0.960
212	5.251	4.855	0.943	4.187	5.040	0.993
230	5.011	4.633	0.900	3.418	4.114	0.810
229	4.289	3.965	0.770	3.175	3.822	0.753

190	4.327	4.000	0.777	3.442	4.143	0.816
196	4.993	4.616	0.897	4.065	4.893	0.964
195	5.859	5.416	1.052	5.031	6.055	1.193
235	7.699	7.118	1.383	6.269	7.545	1.486
238	5.482	5.068	0.985	4.137	4.979	0.981
243	6.247	5.775	1.122	4.400	5.295	1.043
244	6.404	5.920	1.150	4.687	5.642	1.111
233	3.023	2.795	0.543	2.164	2.605	0.513
232	4.326	3.999	0.777	2.990	3.598	0.709
240	3.869	3.577	0.695	2.690	3.238	0.638
204	2.453	2.267	0.441	2.335	2.810	0.553
205	3.184	2.944	0.572	2.880	3.466	0.683
208	2.964	2.740	0.532	2.449	2.947	0.581
209	2.427	2.244	0.436	2.070	2.492	0.491
220	5.951	5.501	1.069	4.599	5.535	1.090
219	5.014	4.636	0.901	3.736	4.497	0.886
Σ	108.165	100.000	19.432	83.085	100.000	19.696

FOURTH FLOOR EQX = 129.688 kN EQY = 97.400 kN
Column

Node No.	Base Shear					
	X - Direction EQX			Y - Direction EQY		
	Value (kN)	% Contribution		Value (kN)	% Contribution of	
		Floor	Total		Floor	Total
260	4.152	3.202	0.746	2.863	2.940	0.679
259	4.208	3.245	0.756	2.966	3.045	0.703
254	4.441	3.424	0.798	3.129	3.213	0.742
253	4.512	3.479	0.811	3.249	3.336	0.770
248	6.151	4.743	1.105	4.687	4.812	1.111
247	6.210	4.789	1.116	4.837	4.966	1.147
288	5.845	4.507	1.050	3.967	4.073	0.941
287	5.144	3.967	0.924	3.782	3.883	0.897
285	5.126	3.952	0.921	4.044	4.152	0.959
252	5.739	4.425	1.031	4.529	4.650	1.074
251	7.384	5.694	1.327	6.143	6.307	1.456
293	9.124	7.036	1.639	7.267	7.461	1.723
296	6.583	5.076	1.183	4.865	4.995	1.153
270	7.492	5.777	1.346	5.158	5.296	1.223
271	7.594	5.855	1.364	5.430	5.575	1.287
291	3.611	2.785	0.649	2.560	2.628	0.607

290	5.104	3.936	0.917	3.497	3.590	0.829
298	4.566	3.521	0.820	3.116	3.199	0.739
263	2.984	2.301	0.536	2.765	2.838	0.655
264	3.880	2.992	0.697	3.420	3.511	0.811
267	3.650	2.815	0.656	2.935	3.014	0.696
268	2.967	2.288	0.533	2.460	2.526	0.583
279	7.502	5.785	1.348	5.608	5.758	1.329
278	5.717	4.408	1.027	4.122	4.232	0.977
Σ	129.688	100.000	23.298	97.400	100.000	23.090

TERRACE EQX = 141.482 kN EQY = 103.764 kN
Column

Node No.	Base Shear					
	X - Direction EQX			Y - Direction EQY		
	Value (kN)	% Contribution		Value (kN)	% Contribution of	
		Floor	Total		Floor	Total
310	4.829	3.413	0.867	3.237	3.120	0.767
309	4.163	2.942	0.748	2.849	2.746	0.675
304	5.034	3.558	0.904	3.407	3.284	0.808
303	5.176	3.658	0.930	3.576	3.446	0.848
350	6.967	4.924	1.252	5.192	5.003	1.231
349	7.069	4.996	1.270	5.379	5.184	1.275
335	6.307	4.458	1.133	4.271	4.116	1.013
334	5.064	3.579	0.910	3.696	3.562	0.876
332	5.495	3.884	0.987	4.315	4.158	1.023
354	5.663	4.003	1.017	4.305	4.149	1.021
353	8.945	6.322	1.607	7.191	6.930	1.705
340	10.664	7.538	1.916	8.329	8.027	1.975
343	6.777	4.790	1.217	4.897	4.719	1.161
320	7.609	5.378	1.367	5.113	4.927	1.212
321	9.035	6.386	1.623	6.327	6.097	1.500
338	3.661	2.588	0.658	2.579	2.486	0.611
337	5.234	3.700	0.940	3.553	3.424	0.842
317	5.134	3.629	0.922	3.441	3.317	0.816
313	3.080	2.177	0.553	2.782	2.681	0.660
314	4.002	2.828	0.719	3.432	3.307	0.814
345	3.686	2.605	0.662	2.879	2.775	0.683
346	3.109	2.197	0.559	2.511	2.420	0.595
329	9.149	6.467	1.644	6.599	6.360	1.564
328	5.631	3.980	1.012	3.903	3.761	0.925

Σ	141.482	100.000	25.417	103.764	100.000	24.598
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HEADRM EQX = 17.735 kN EQY = 12.031 kN

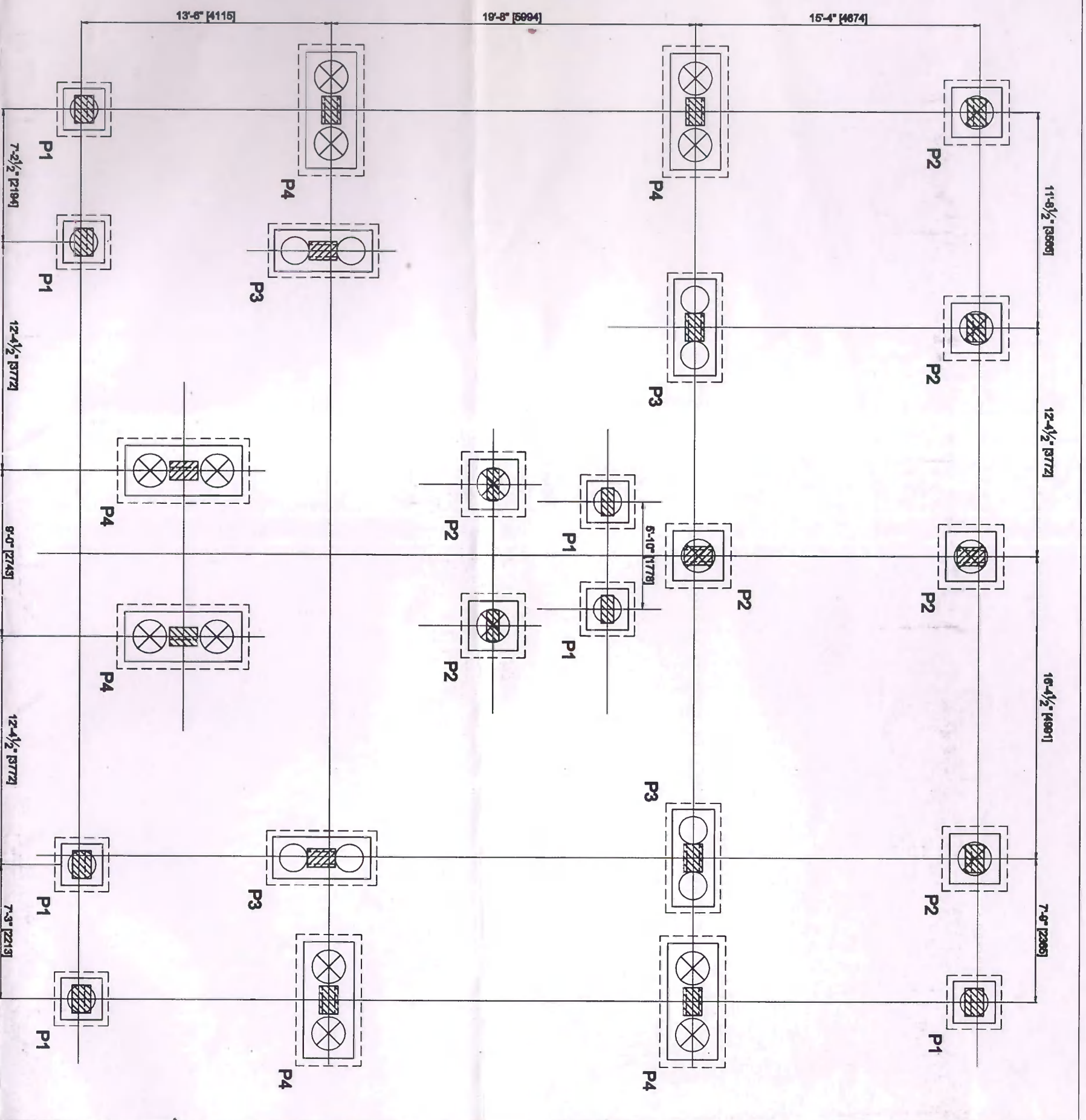
Column

Node No.	Base Shear					
	X - Direction EQX			Y - Direction EQY		
	Value (kN)	% Contribution		Value (kN)	% Contribution of	
		Floor	Total		Floor	Total
360	1.654	9.325	0.297	1.054	8.763	0.250
359	1.868	10.531	0.336	1.210	10.059	0.287
358	4.222	23.807	0.759	2.738	22.755	0.649
357	4.027	22.705	0.723	2.654	22.060	0.629
356	2.980	16.801	0.535	2.167	18.016	0.514
355	2.985	16.831	0.536	2.207	18.346	0.523
Σ	17.735	100.000	3.186	12.031	100.000	2.852

OHT EQX = 6.903 kN EQY = 4.558 kN

Column

Node No.	Base Shear					
	X - Direction EQX			Y - Direction EQY		
	Value (kN)	% Contribution		Value (kN)	% Contribution of	
		Floor	Total		Floor	Total
364	0.738	10.694	0.133	0.455	9.985	0.108
365	0.890	12.892	0.160	0.557	12.216	0.132
366	1.527	22.123	0.274	0.960	21.067	0.228
361	1.379	19.971	0.248	0.880	19.302	0.209
362	1.183	17.133	0.212	0.845	18.547	0.200
363	1.186	17.187	0.213	0.861	18.883	0.204
Σ	6.903	100.000	1.240	4.558	100.000	1.080



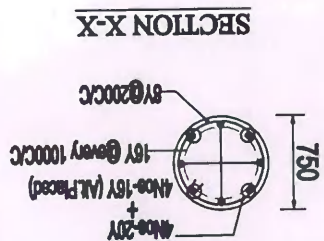
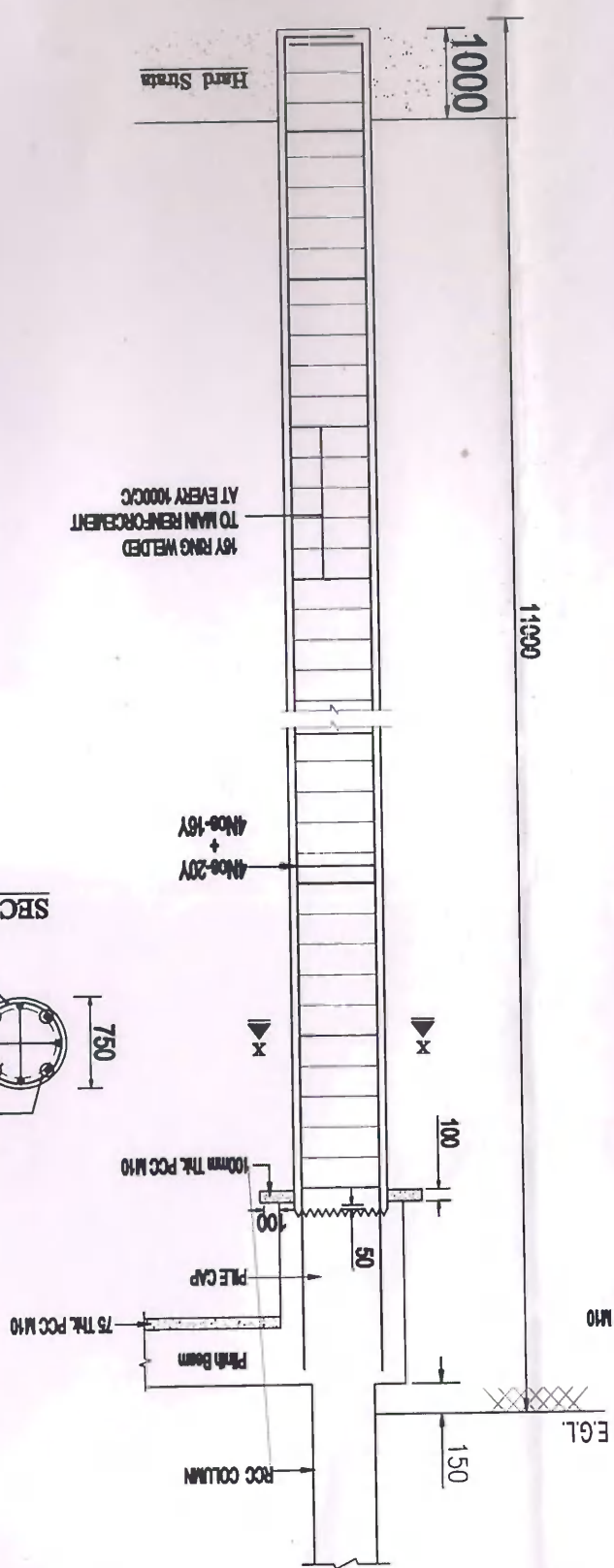
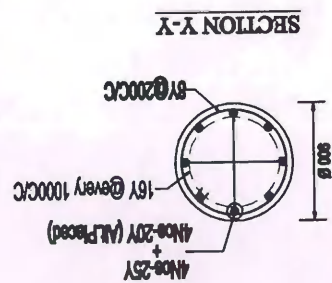
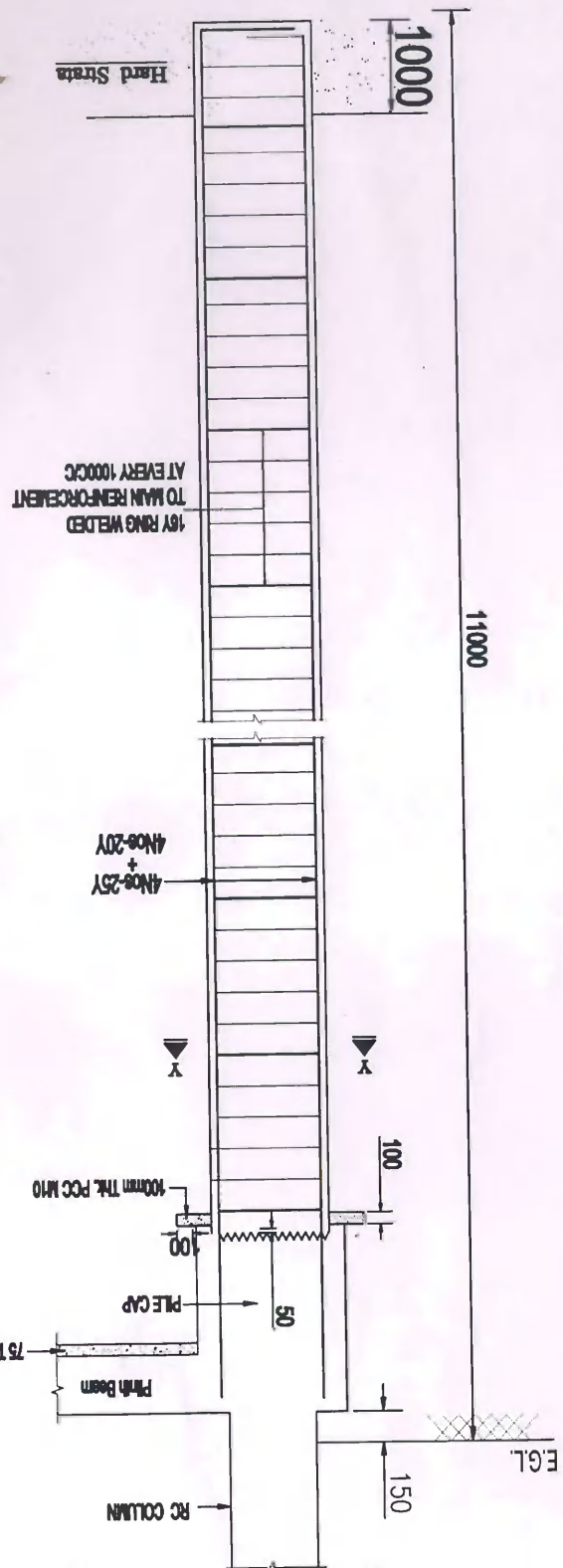
PILECAP LAYOUT

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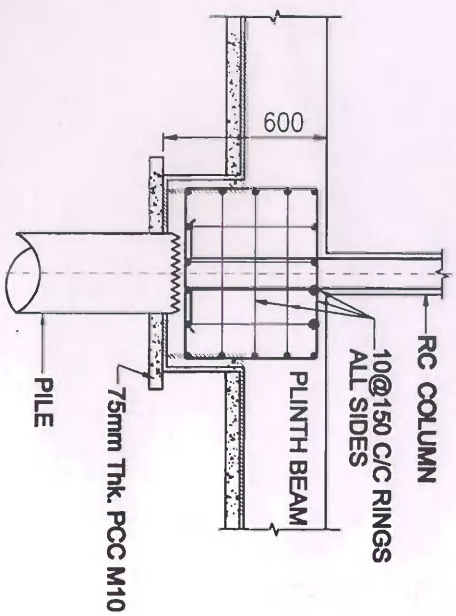
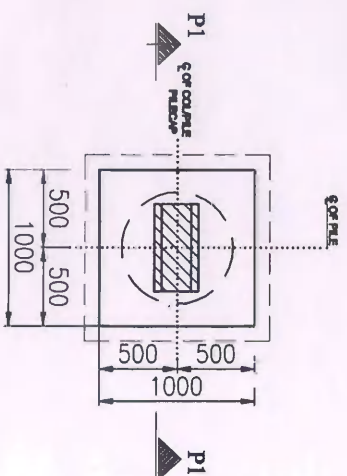
PILB DETAILS

9000 PILB DETAIL
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 LOAD CARRYING CAPACITY - 167T

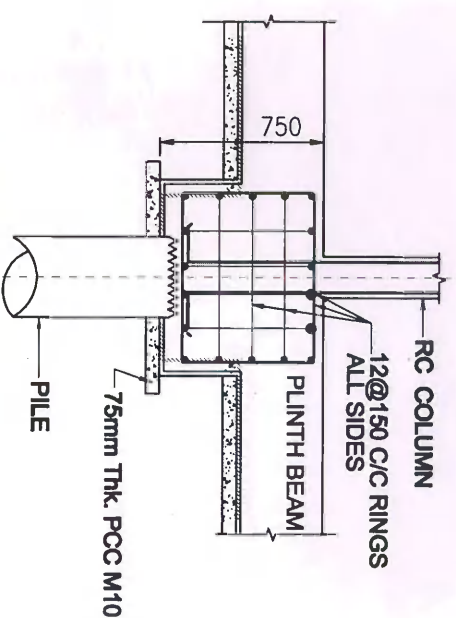
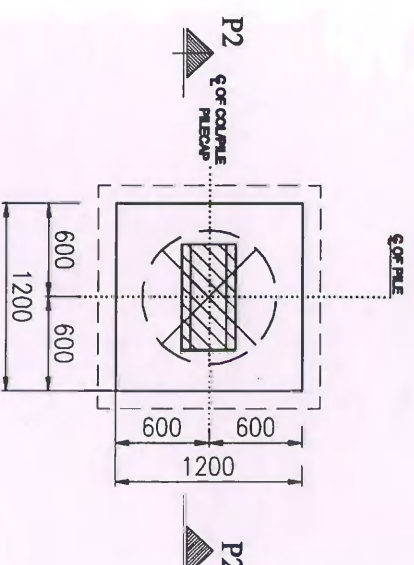
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 LOAD CARRYING CAPACITY - 118T



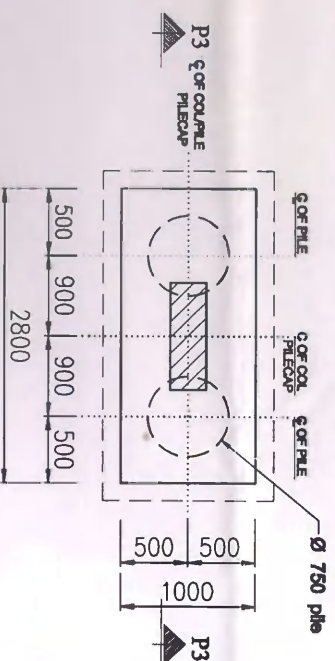
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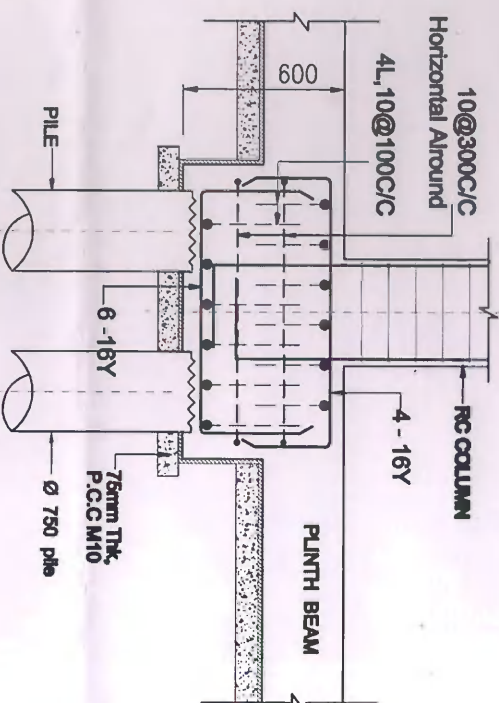
SECTION P1 - P1



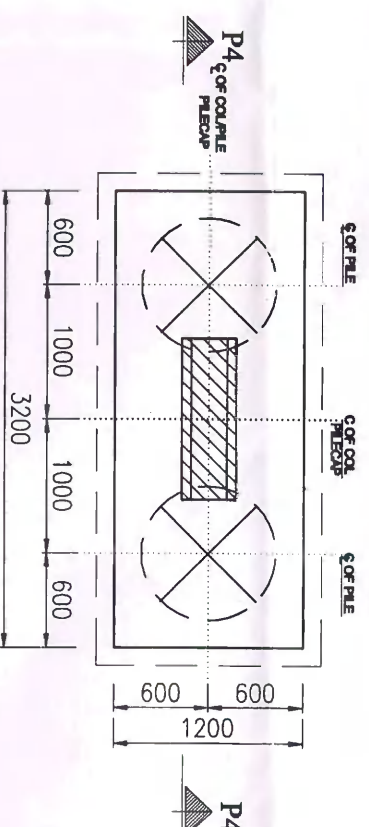
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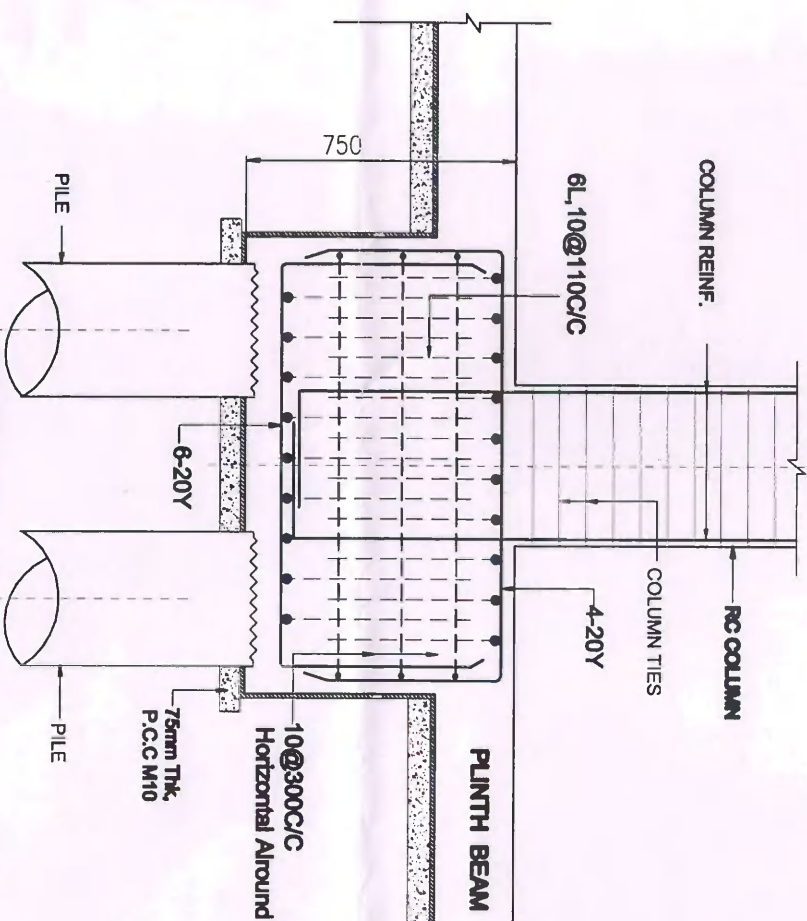
PLAN OF PILE CAP - P3



SECTION P3 - P3



PLAN OF PILE CAP - P4

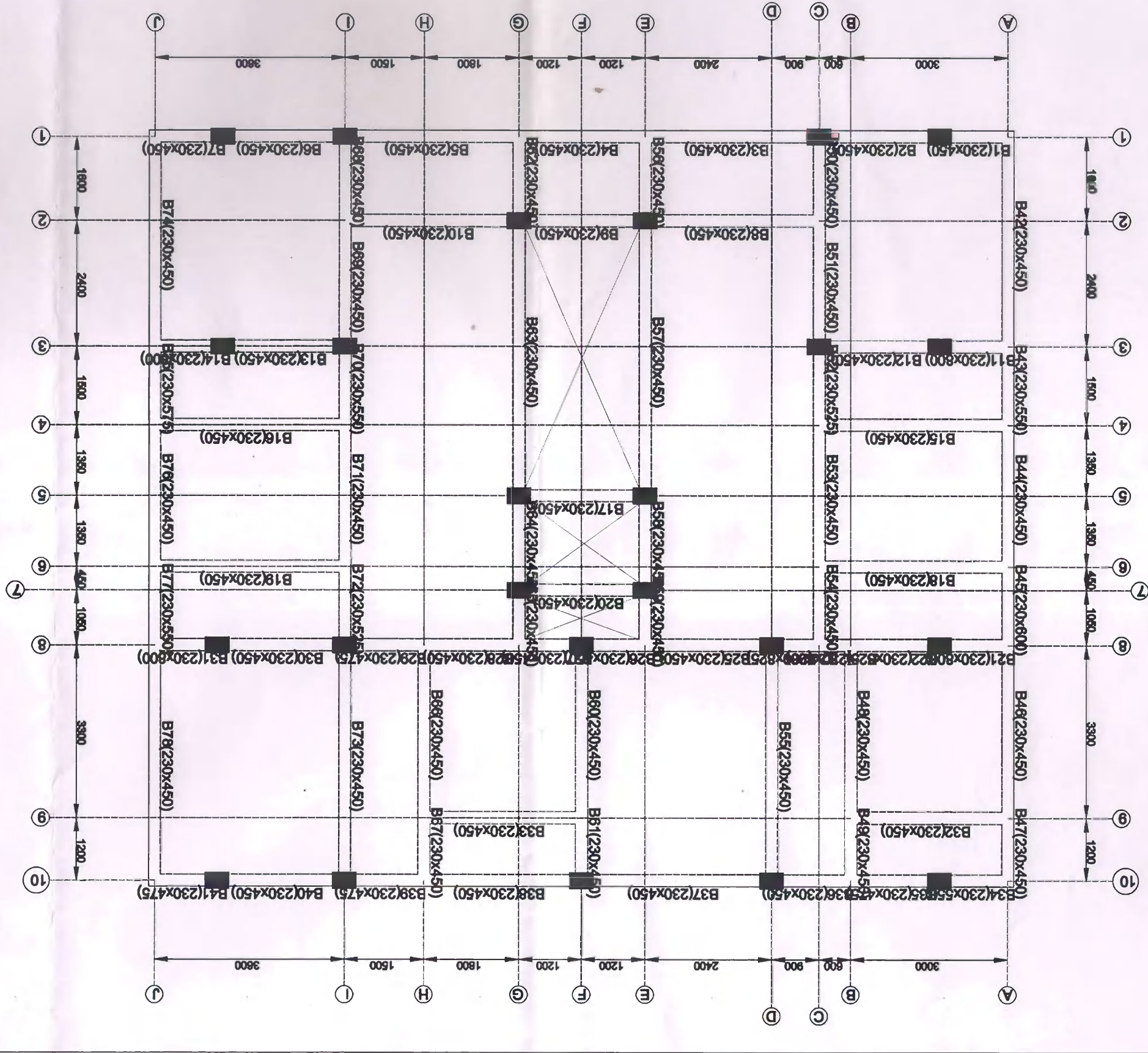


SECTION P4 - P4

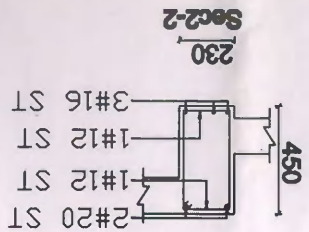
PILECAP DETAILS

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PROJECT : INDRNGR BEAM LAYOUT

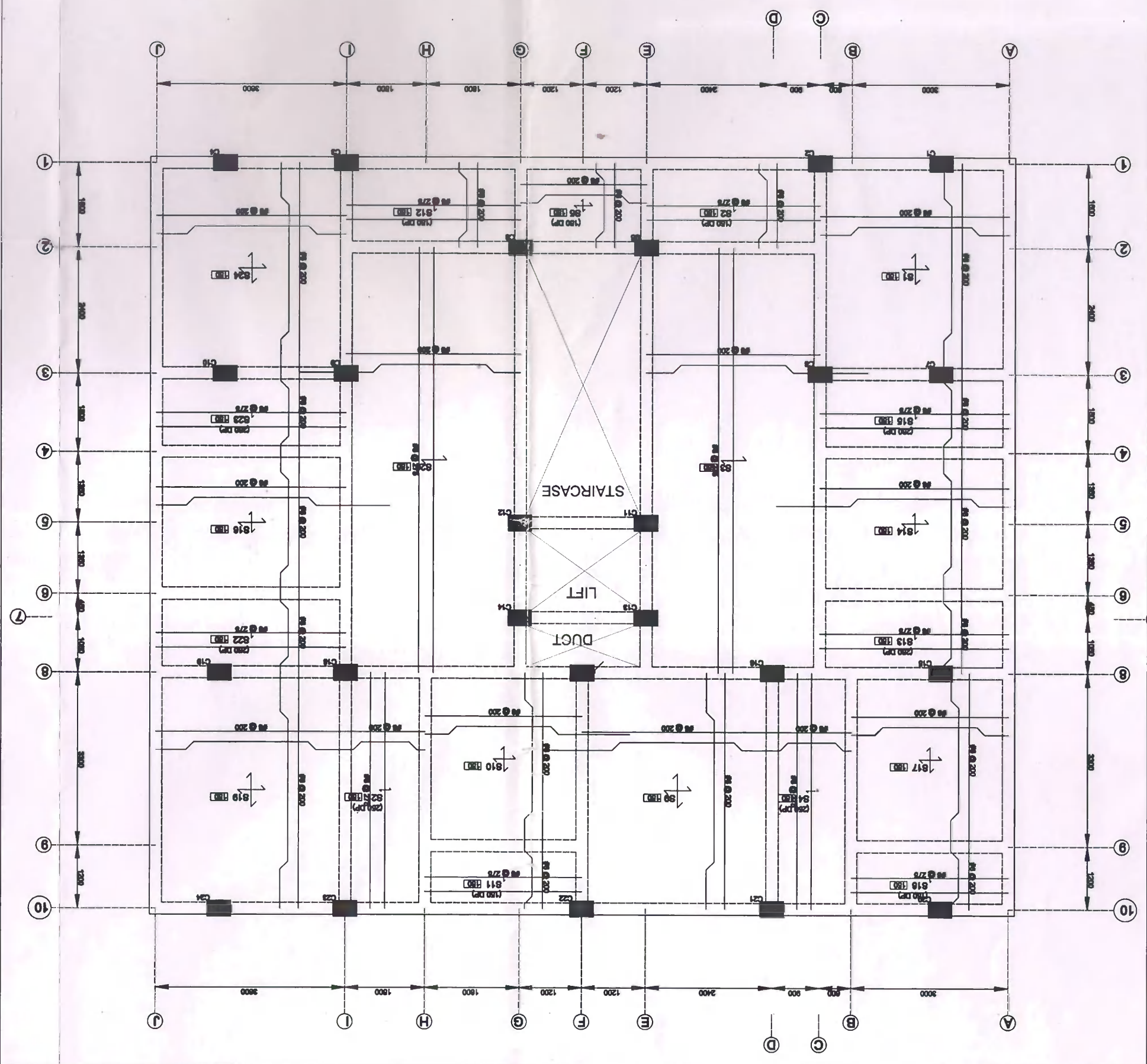


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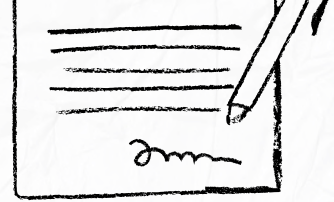
Shree

PROJECT : INDRNGR - DETAILS OF SLAB



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Sandhya



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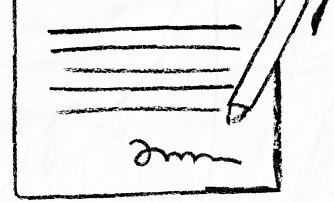
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