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DATE	REV. NO.	DESCRIPTION	DESIGNED	CHECKED	APPROVED
08.05.2023	0	ISSUED FOR APPROVAL	RAKE	NKK	NKK

REVISIONS

CLIENT : M/S BUILT TECH ASSOCIATES

PROJECT : PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE

JOB No. A23001

TOTAL NO. OF PAGES

TITLE : STRUCTURAL DESIGN DOCUMENT

	NAME	SIGN	DATE
DSGN	RAKE	RAKE	08.05.2023
CHKD	NKK	NKK	08.05.2023
APPD	NKK	NKK	08.05.2023

DESIGN CALCULATIONS
(BASEMENT + STILT + 11 FLOORS ONLY)

CODE	IS
REV.	0

Doc. No. A210510-C-RB-01-DC-0001

RELEASED FOR ☐ PRELIMINARY ☑ APPROVAL ☐ CONSTRUCTION

PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REV	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN CALCULATIONS	DESIGNED		CHECKED		APPROVED	
CLIENT:	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

1.INTRODUCTION

Basement + Stilt+11 story Residential building has plan dimensions as per Floor plans.The building is located in seismic Zone- III on a site with Medium soil. The design wind speed is 50 m /sec for Chennai region. Design the building for seismic and wind loads as per IS: 1893(part-1)-2016 and IS: 875 (Part-3) - 2015.

2.STRUCTURAL SYSTEM

Conventional (Cast in-situ) RCC Framed structural system has been considered for the building.3-D Analysis has per been carried out in STAAD Pro As follows

1. RCC Framed Structure with Pile, Pile cap, Column, RC Walls, Beams & Floor slab (Slab Not Modeled)
2. Building is Designed with Extended Basement, Stilt and 11 Floors
3. 500 Diameter pile with pile length of 20m from Existing ground level. Pile cap with different depth and Raft slab of 400mm Thickness is considered and designed for uplift loads.
4. Retaining wall is considered Around the Extended Basement Boundary of 300mm Thickness for the basement of 3m from Finished Ground Level.
5. Extended Basement Roof Slab is designed for 15kN/m2 Live loading

3.MATERIALS

The Self weight of various elements are computed based on the Unit weight of materials as given below as IS:875 (Part-1) - 1987.

Table:1.Unit weight of materials(kN/m^3)

Steel	78.50
Plain cement concrete/Screed	24.00
Reinforced cement concrete(Dry)	25.00
Cement concrete solid block work	20.00
Water	9.81
Brick coba	18.00

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3.1.Concrete

Grade of Concrete for all structural items (Cast in-situ) considered as M25.

3.1.1.Exposure Category

As per the clause 8.2.2 of IS:456 -2000, the exposure category considered is moderate refer table-2.

Table:2.Nominal cover to reinforcement

(Environmental exposure condition)

Structural element	Minimum(As per is456-2000)	Considered for design
Pile, Pile cap	75	75
Column	40	40
Beam	30	30
Slab	20	20

3.2 Reinforcement

Reinforcement steel shall be of Grade Fe500 confirming to IS:1786 – 2008.

3.3.Structural steel

All Structural steel shall conform to IS: 2062 -2011. Pipes and tubes shall conform to IS: 1161- 2014.

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4.LOADS:

This load comprises of self - weight of all the elements in the structure. Additional weight of the elements which are not declared in the model are calculated as below and applied.

4.1.Dead loads(IS875-PART1-1987)

i)Wall load

Main wall load:

Wall thickness			= 230	mm
Height of wall			= 3000	mm
Self weight of wall	=	Wall thickness * Height *Unit weight of Brick		
	=	0.23 x 3 x 20	= 13.8	kN/m

Partition wall load:

Wall thickness			= 115	mm
Height of wall			= 3000	mm
Self weight of wall	=	Wall thickness * Height *Unit weight of Brick		
	=	0.115 x 3 x 20	= 6.9	kN/m

ii)Slab load:

Floor/Roof slab Load:

By self weight:

Thickness of slab			= 125	mm
Self weight of slab	=	Thickness of slab * Unit weight of RCC		
	=	0.125 x 25	= 3.125	kN/m^2

By Finishing:

Thickness of finishing			= 75	mm
Self weight of slab	=	Thickness of slab * Unit weight of PCC		
	=	0.075 x 24	= 1.8	kN/m^2
Total load	=	3.125 + 1.8	= 4.925	kN/m^2

4.2.Live load(IS875-PART2-1987)

Load on office/Tailoring eqip. room	= 2	kN/m^2
Load on Bathroom/balcony/staircase	= 3	kN/m^2
Load on roof	= 1.5	kN/m^2

4.3.Earthquake load(IS1893-PART1-2016)

Length of building	= 32	m
Width of building	= 26	m
Slab thickness	= 0.125	m
Wall thickness	= 0.23	m
Zone location	= Chennai	
Zone	= III	
Total height from founding Depth	= 44	m

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Design horizontal acceleration Spectrum:

$$A_h = \frac{Z\,I\,S_a}{2\,R\,g}$$

Zone factor Z = 0.16

Importance factor I = 1.2

Response reduction factor R = 5

T_a = RC MRF building = $0.075\,h^{0.75}$ = 1.2813

Sa/g = 1.06

Type of building = RC MRF building

Table.3)Design Acceleration co-efficient S_a/g

Rocky Hard soil	1+15T	T < 0.10 S
	2.5	0.10 s < T < 0.40 s
	1/T	0.40 s < T <4.0 s
	0.25	T > 4.00 s
Medium stiff soil	1+15T	T < 0.10 S
	2.5	0.10 s < T < 0.55 s
	1.36/T	0.55 s < T <4.0 s
	0.34	T > 4.00 s
Soft soil	1+15T	T < 0.10 S
	2.5	0.10 s < T < 0.67 s
	1.67/T	0.67 s < T <4.0 s
	0.42	T > 4.00 s

$$A_h = \frac{0.16 \times 1.2 \times 1.06}{2 \times 5} = 0.02$$

Total Self weight = 151440 kN

Base Shear = 3086 kN

From STAAD

X Base shear = 993 kN

Z Base shear = 1126 kN

Scale Factor

X Direction = 3.11

Z Direction = 2.74

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Note:

Design base shear VB will be evaluated by STAAD analysis is considering 100% Dead load and 25% of the live load .

4.4.Wind load(IS875-PART 3-2015)

Building location		=	Chennai	
Basic Wind Speed	V_b	=	50	m / sec
Greater horizontal dimension of the building	l	=	32	m
Lesser horizontal dimension of the building	w	=	26	m

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Maximum height of the building z / h = 36 m

Design life of structure = 50 Years,

Terrain Category = I

Terrain slope q = 0 °

Type of structure = All other building

Type of building = Non Circular

Maximum Frame (Bay) spacing = 9.6 m

Roof angle = 0 °

Design Calculation:

Design wind speed, $V_z = [V]_b \, k_1 \, k$ Where,

V_z = Design Wind Speed at height z, m/sec

k_1 = Probability factor (risk coefficient) (Ref: Cl:6.3.1, Table-1)

k_2 = Terrain roughness and height factor (Ref. Cl:6.3.2)

k_3 = Topography factor (Ref. Cl:6.3.3)

k_4 = Importance factor for the cyclonic region (Ref. Cl:6.3.4)

,Shall be taken 1.15 where the buidling within 200km from the coast (Ref: Cl: 6.6)

k_1 = 1

k_2 = 1

k_3 = 1

k_4 = 1

V_z = 50 x 1 x 1 x 1 = 50 m/s

Calculate design wind pressure,

Wind pressure, $P_z = [0.6 V]$ Ref: Cl:7.2

Where,

P_z = Wind pressure at height, z in N/m^2

V_z = Design wind speed at height, z in m/sec

P_z = 0.6 x 50 x 50 = 1500 N/m^2

Design wind pressure,

$$P_d = K_d \, K_a \, K_c \, P_z \geq [$$

Where,

P_d = Design wind pressure, N/m^2 Ref : Cl:7.2

K_d = Wind directionality factor Ref : Cl:7.2.1

(For Local pressure coefficient & cyclone affected region, Kd shall be 1.0 with respect to any shape)

K_a = Area averaging factor Ref : Cl7.2.2

K_c = Combination factor Ref : Cl:7.3.3.13

P_z = Wind pressure at height, N/m^2 Ref : Cl:7.2

K_d = 0.9

K_a = 0.8

K_c = 0.9

Tributary Area, = 832 Sqm

P_{d1} = 1500 x 0.9 x 0.8 x 0.9 = 972 kN/m^2

P_{d2} = 0.7 x 1500 = 1050 kN/m^2

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P_d = = 1050 kN/m^2

WIND LOAD CALCULATION BY FORCE COEFFICIENT METHOD

Building Location	=	Chennai	
V_b(regional wind speed)	=	50 m/s	
k_1(risk coefficient factor)	=	1	(cl:6.3.1)
k_3(topography factor)	=	1	(cl: 6.3.3)
k_4(cyclonic region)	=	1	(cl: 6.3.4)
K_d(wind directionality factor)	=	0.9	(cl: 7.2.1)
K_a(Area averaging factor)	=	0.8	(cl: 7.2.2)
Terrain category (k_2)	=	2	(cl:6.3.2.2)
Building dimension along - x dir.(a)	=	22.25 m	
Building dimension along - y dir.(b)	=	31.08 m	
Height of the building (h)	=	36 m	
a/b	=	0.7159	
h/b	=	1.1583	
cf (force coefficient)	=	1.25	(cl: 7.4)

Storey name	Total Height	k_2	Design wind speed (m/s) $V_z = V_b \times k_1 \times k_2 \times k_3 \times k_4$	Wind pressure $P_z = 0.6(V_z)^2$ kN/m^2	Design wind pressure $p_d = p_z \times k_d \times k_a$ (kN/m^2)
Floor 1	3	1.00	50	1.500	1.080
Floor 2	6	1.00	50	1.500	1.080
Floor 3	9	1.00	50	1.500	1.080
Floor 4	12	1.02	51	1.561	1.124
Floor 5	15	1.05	52.5	1.654	1.191
Floor 6	18	1.06	53.1	1.692	1.218
Floor 7	21	1.08	53.75	1.733	1.248
Floor 8	24	1.09	54.5	1.782	1.283
Floor 9	27	1.11	55.25	1.832	1.319
Floor 10	30	1.12	56	1.882	1.355
Floor 11	33	1.13	56.375	1.907	1.373
Floor 12	36	1.14	56.75	1.932	1.391

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5.LOAD COMBINATIONS:

The following load combinations are considered in scheme and detailed design stages in reference with IS 456:2000, IS 1893 (Part1): 2016 & IS 875 (Part5)

Load combinatoons used for foundation design

DL+SIDL+LL

DL+SIDL+LL±WL_x

DL+SIDL+LL±WL_z

DL+SIDL+LL+EQ

DL+SIDL±WL_x

DL+SIDL±WL_z

DL+SIDL±EQ

Table:4 Load combinations used for scheme and detail designs:(Limit state method)

Serviceability	Strength
DL+SIDL+LL+RLL	1.5DL+1.5SIDL+1.5LL+1.5RLL
DL+SIDL+0.8LL±0.8WL_x	1.5DL+1.5SIDL+1.5LL±1.5WL_x
DL+SIDL+0.8LL±0.8WL_z	1.5DL+1.5SIDL+1.5LL±1.5WL_z
DL+SIDL+0.8LL±0.8EQ	1.5DL+1.5SIDL+1.5LL±1.5EQ
DL+SIDL+±0.8WL_x	0.9DL+0.9SIDL±1.5WL_x
DL+SIDLL±0.8WL_z	0.9DL+0.9SIDL±1.5WL_z
DL+SIDLL±0.8EQ	0.9DL+0.9SIDL±1.5EQ
	1.2DL+1.2SIDL+1.2LL±1.2WL_x
	1.2DL+1.2SIDL+1.2LL±1.2WL_z
	1.2DL+1.2SIDL+1.2LL±1.2EQ

5.1.Deflection criteria:

Permissible deflections: IS 456:2000 (Clause 23.2 (b))

Max vertical deflection on beams and slab under vertical load:

Final deflection	≤	span/250	IS 456:2000 (Clause 23.2 (a))
Final deflection after erection of all loads&finishes	≤	span/350 (or)20mm whichever is less.	

Max storey drift due to lateral load:

Under wind load	≤	Height/500	IS 456:2000 (Clause 20.5)
Under earthquake load	≤	Height/250	IS 1893, part1 :2016 (Clause 7.11.1.1)

Long term deflections due to shrinkage and creep will be calculated as per Annexure-C of IS 456:2000.

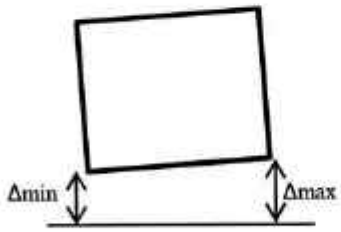
5.2.Crack width requirements:

Crack width for all structural members shall be limited to 0.3 mm.

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TITLE:	IS - 1893 Part-I Codal Provision	DESIGNED		CHECKED		APPROVED	
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		DATE	07.01.2023	DATE	07.01.23	DATE	07.01.23

A. Checking of Torsional Irregularity: IS:1893-Part-I :2016 Cl:7.1



Minimum Horizontal Displacement due to Earth quake load $\Delta min = 74.8$ mm

Maximum Horizontal Displacement due to Earth quake load $\Delta Max = 78.7$ mm

$\Delta min < 1.5\ \Delta Max$

Sl.No	Floor	Lateral Displacement at Min. end, Δ_1 (mm)	Lateral Displacement at Max. end, Δ_2 (mm)	Lateral Displacement at Middle, Δ_{Middle} (mm)	Lateral Displacement at Avg., Δ_{Avg} (mm)	Result
1	Stilt	0.00	0.00	0.00	0.00	Safe
2	1st	4.921	5.006	3.726	4.964	Safe
3	2nd	10.895	11.678	8.73	11.287	Safe
4	3rd	17.874	19.353	14.638	18.614	Safe
5	4th	25.334	27.594	21.033	26.464	Safe
6	5th	32.909	35.932	27.668	34.4205	Safe
7	6th	40.331	43.991	34.359	42.161	Safe
8	7th	47.395	51.501	40.944	49.448	Safe
9	8th	53.951	58.281	47.285	56.116	Safe
10	9th	59.895	64.233	53.263	62.064	Safe
11	10th	65.19	69.35	58.79	67.27	Safe
12	11th	69.897	73.734	63.837	71.8155	Safe
13	Roof	74.234	77.632	68.486	75.933	Safe

$\Delta_{Middle} < 1.2 \times \Delta_{Avg}$, the Floor will be Rigid Diapharam as per Cl:7.6.4

Stiffness at each floor will be a constant

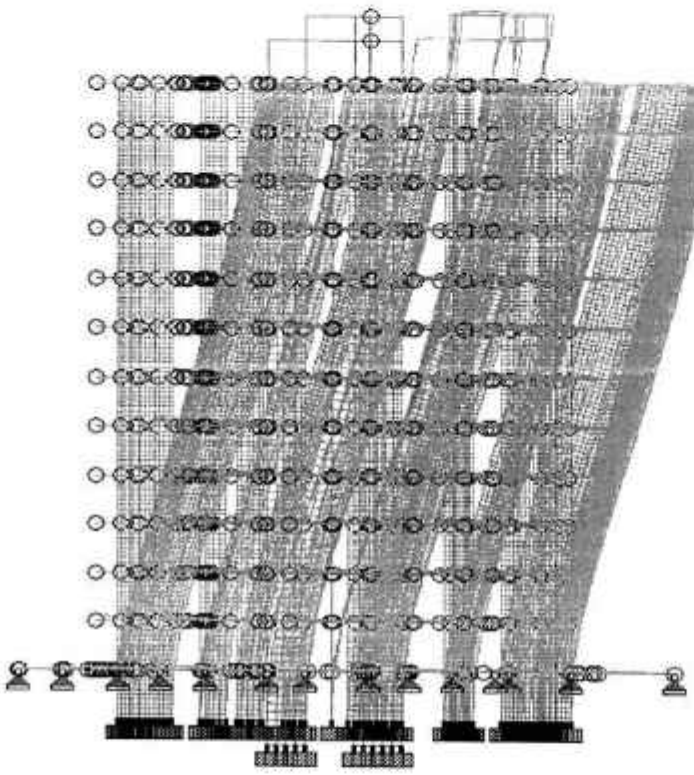
There is no change in the structure throughout the height of the building

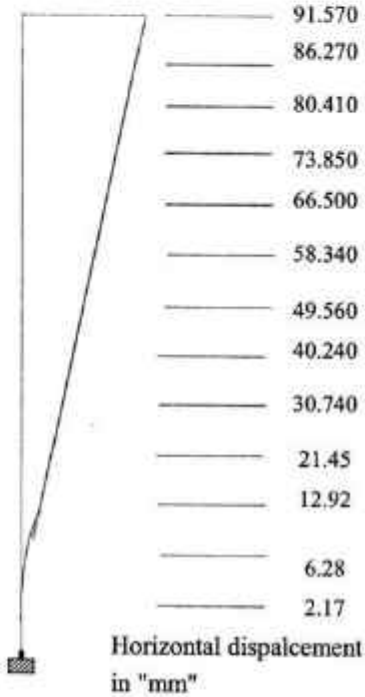
Hence, this building does not twist about its vertical axis, it is safe

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B. Sotrey Drifft:





Horizontal dispalcement in "mm"

Floor to Floor Height of the building = 3.0 m

Allowable storey drift h/250 = 12.00 mm

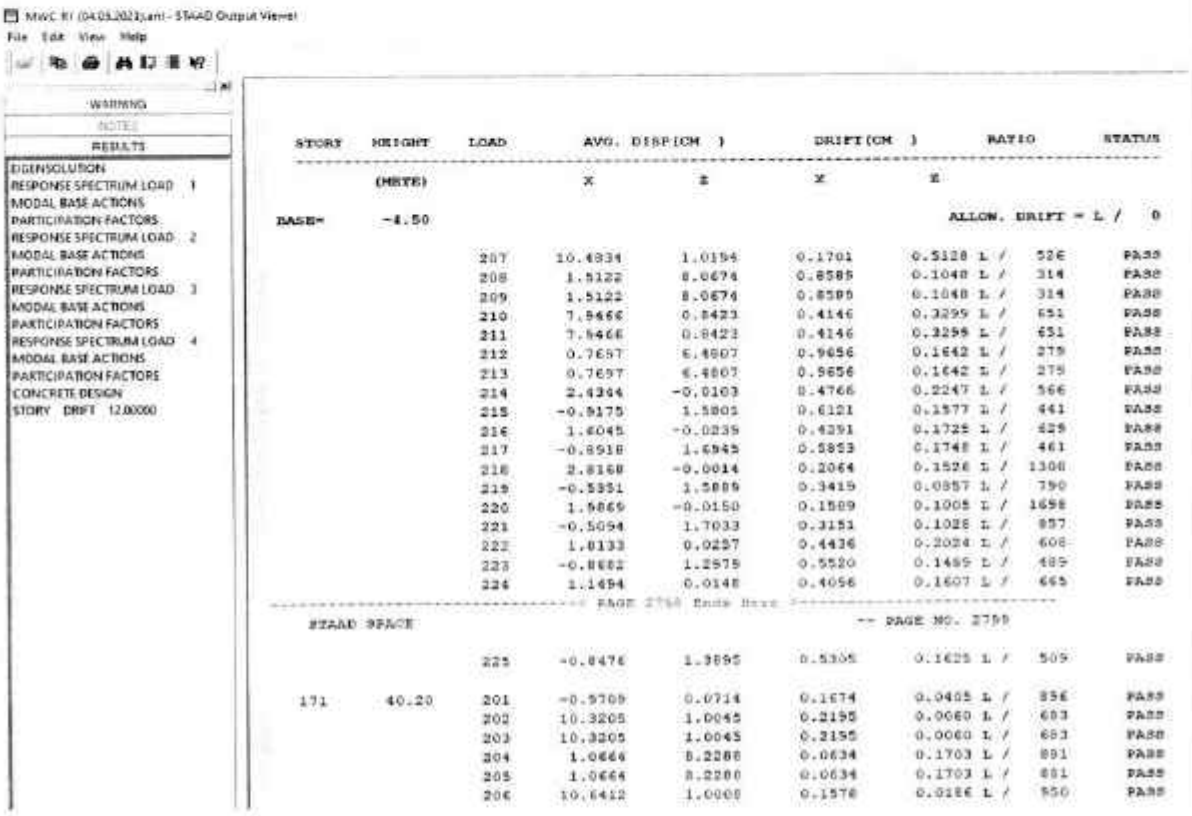
Sl.No.	Floor	Storey Drift (mm)	Allowable Drift (mm)	Result
1	Stilt	2.17	12.0	Safe
2	1st	4.11	12.0	Safe
3	2nd	6.64	12.0	Safe
4	3rd	8.53	12.0	Safe
5	4th	9.290	12.0	Safe
6	5th	9.500	12.0	Safe
7	6th	9.320	12.0	Safe
8	7th	8.780	12.0	Safe
9	8th	8.160	12.0	Safe
10	9th	7.350	12.0	Safe
11	10th	6.560	12.0	Safe
12	11th	5.860	12.0	Safe
13	Roof	5.300	12.0	Safe

Maximum Storey Drift = 9.5 mm < h/250 = 12.00 mm

Safe

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		DATE	07.01.2023	DATE	07.01.23	DATE	07.01.23



STORY	HEIGHT (METR)	LOAD	AVG. DISP(CM) X	AVG. DISP(CM) Z	DRIFT(CM) X	DRIFT(CM) Z	RATIO	STATUS
BASE=	-4.50					ALLOW. DRIFT = L /	0	
		207	10.4834	1.0194	0.1701	0.5128 L /	526	PASS
		208	1.5122	8.0674	0.8585	0.1048 L /	314	PASS
		209	1.5122	8.0674	0.8585	0.1048 L /	314	PASS
		210	7.9466	0.8423	0.4146	0.3299 L /	651	PASS
		211	7.9466	0.8423	0.4146	0.3299 L /	651	PASS
		212	0.7657	6.4807	0.9656	0.1642 L /	275	PASS
		213	0.7657	6.4807	0.9656	0.1642 L /	275	PASS
		214	2.4364	-0.0103	0.4766	0.2247 L /	566	PASS
		215	-0.9175	1.5805	0.6121	0.1577 L /	441	PASS
		216	1.6045	-0.0239	0.4291	0.1725 L /	429	PASS
		217	-0.8918	1.6945	0.5853	0.1748 L /	461	PASS
		218	2.8168	-0.0014	0.2064	0.1526 L /	1308	PASS
		219	-0.5351	1.5889	0.3419	0.0857 L /	790	PASS
		220	1.5869	-0.0150	0.1509	0.1005 L /	1698	PASS
		221	-0.5094	1.7033	0.3151	0.1026 L /	857	PASS
		222	1.8133	0.0257	0.4436	0.2024 L /	606	PASS
		223	-0.8682	1.2575	0.5520	0.1469 L /	489	PASS
		224	1.1494	0.0148	0.4056	0.1607 L /	665	PASS
			PAGE 2798 Ends Here					
STAAD SPACE					-- PAGE NO. 2799			
		225	-0.8476	1.3895	0.5305	0.1625 L /	509	PASS
171	40.20	201	-0.9709	0.0714	0.1674	0.0405 L /	856	PASS
		202	10.3205	1.0045	0.2195	0.0060 L /	683	PASS
		203	10.3205	1.0045	0.2195	0.0060 L /	683	PASS
		204	1.0666	8.2280	0.0634	0.1703 L /	891	PASS
		205	1.0666	8.2280	0.0634	0.1703 L /	891	PASS
		206	10.6412	1.0000	0.1578	0.0186 L /	550	PASS

C. Soft Storey Check:

Lateral stiffness is same for all the floors

Hence, Soft story is not occuring in this building

D. Model Participation Checks:

Modal Participation in X-dir, = 70.62 % > 65 % in 2nd Mode shape

Modal Participation in Z-dir, = 72.35 % > 65 % in 3rd Mode shape

Safe, Regular mode shapes in each principal directions

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PROJECT	RESIDENTIAL BUILDING AT WATERCANAL ROAD	DOCUMENT NO				REV	
		O21051-C-RB-01-DC-0001					
TITLE:	IS - 1893 Part-I Codal Provision	DESIGNED		CHECKED		APPROVED	
CLIENT	M/S BUILT TECH ASSOCIATES	CPA		NKK		NKK	
		DATE	07.01.2023	DATE	07.01.23	DATE	07.01.23

MWC 01 (04.05.2023).anl - STAAD Output Viewer

File Edit View Help

WARNING

RESULTS

EIGENSOLUTION
RESPONSE SPECTRUM LOAD 1
MODAL BASE ACTIONS
PARTICIPATION FACTORS
RESPONSE SPECTRUM LOAD 2
MODAL BASE ACTIONS
PARTICIPATION FACTORS
RESPONSE SPECTRUM LOAD 3
MODAL BASE ACTIONS
PARTICIPATION FACTORS
RESPONSE SPECTRUM LOAD 4
MODAL BASE ACTIONS
PARTICIPATION FACTORS
CONCRETE DESIGN
STORY DRIFT 12.00000

MODAL BASE ACTIONS FORCES IN KN LENGTH IN METE

MODE	PERIOD	FX	FY	FZ	MOMENTS ARE ABOUT THE ORIGIN MX	MY	MZ
1	2.725	1443.83	0.00	-522.51	-12890.56	-325.47	-35906.62
2	2.605	1828.34	0.00	590.68	14603.00	-56533.65	-46674.81
3	2.332	0.64	0.00	-46.65	-1160.09	277.63	-22.48
4	1.436	0.25	0.00	1.13	27.36	-6.26	-25.82
5	1.305	0.10	0.00	-0.11	4.08	4.08	8.13
6	1.132	0.36	0.00	-0.26	-109.00	-11.57	-11.74

MASS PARTICIPATION FACTORS IN PERCENT

BASE SHEAR IN KN

MODE	X	Y	Z	SUM-X	SUM-Y	SUM-Z	X	Y	Z
1	32.06	0.00	4.20	32.062	0.000	4.199	1443.83	0.00	0.00
2	38.62	0.00	4.03	70.683	0.000	8.230	1828.34	0.00	0.00
3	0.01	0.00	64.22	70.696	0.000	72.447	0.64	0.00	0.00
4	0.00	0.00	0.06	70.699	0.000	72.507	0.25	0.00	0.00
5	0.00	0.00	0.00	70.700	0.000	72.509	0.10	0.00	0.00
6	0.00	0.00	0.00	70.703	0.000	72.511	0.36	0.00	0.00
					TOTAL SRSS SHEAR		2325.70	0.00	0.00
					TOTAL 10PCT SHEAR		3272.17	0.00	0.00
					TOTAL ABS SHEAR		3273.52	0.00	0.00
					TOTAL CQC SHEAR		3133.35	0.00	0.00

E. Time Period Checks:

Time Period in 1st mode = 2.75 Sec

Sa/g = 0.495

$$Aw = \frac{0.16 \times 1.2 \times 0.49}{2 \times 5} = 0.01$$

Total Self weight 151440 kN

Base Shear 1425 kN < 3086 kN

From STAAD

X Base shear = 993 kN

Z Base shear = 1126 kN

Scale Factor

X Direction = 1.43 < 3.11 as per actual

Z Direction = 1.27 < 2.74 as per actual

Hence, the base shear as per the STAAD model time period is less than that of provided in STAAD model, it is safe

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		DOCUMENT NO	REV.	DATE
TITLE:		DESIGNED	CHECKED	SHEET
	Base raft Design			of

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Design of Raft

Grade of concrete	fck	=	25	N/mm^2		
Grade of steel	fy	=	500	N/mm^2		
Thickness of the base	t	=	400	mm		
Clear cover		=	50	mm		
Y dir Bending , Shear Force (stress)			B.M		Tv	
B.M @ Top face		=	74.78	kN-m	0.388	N/mm^2
B.M @Bottom face		=	112.17	kN-m	0.277	N/mm^2
X dir Bending , Shear Force (stress)						
B.M @ Top face		=	74.78	kN-m	0.388	N/mm^2
B.M @Bottom face		=	112.17	kN-m	0.277	N/mm^2
Modulus of elasticity of steel	E_s	=	200000	N/mm^2		
Modulus of elasticity of Concrete	Ec	=	25000	N/mm^2	Cl.6.2.3.1 IS 456:2000	
Long term Modulus of elasticity of Concrete	Ece	=	11905	N/mm^2	Annex C.4.1 IS 456:2000	
Modular Ratio	m	=	8			

Design

			Top Face		Bot Face		
Direction of Wall			X-Dir	Y-Dir	X-Dir	Y-Dir	
Factored Bending Moment	Mx,My	=	74.78	74.78	112.17	112.17	kN-m
Diameter of bar(mm)	Ø	=	12	12	12	12	mm
Effective Depth.	d	=	332	344	332	344	mm
Mu/bd^2		=	0.68	0.63	1.02	0.95	
Minimun Pt required		=	0.12	0.12	0.12	0.12	%
Minimun Ast required		=	480	480	480	480	mm^2
Ast required		=	537	514	819	788	mm^2
Spacing Required		=	211	220	153	154	mm
Spacing Provided		=	150	150	150	150	mm
Ast provided		=	754	754	754	754	mm^2
Pt Provided (single face)		=	0.19	0.19	0.19	0.19	%
Dist. From the point considered to the surface of the nearest longitudinal Bar	a_{cr}	=	84.14	84.14	84.14	84.14	mm
Distance from the compression face to the point at which the crack width is being calculated	a	=	400	400	400	400	mm
Depth of Neutral axis	x	=	57.54	58.67	57.54	58.67	mm
Cracked Moment of inertia	Icr	=	5.2E+08	5.6E+08	5.2.E+08	5.6.E+08	mm^4
Service stress in tension reinforcement	f_s	=	211.38	203.80	267.06	255.70	N/mm^2
Strain at the level considered,	e_1	=	0.0013	0.0012	0.0017	0.0015	
	e_m	=	0.0004	0.0003	0.0008	0.0006	
Design surface crack width	Wcr	=	0.075	0.063	0.16	0.126	mm Refer Annex F,
Permissible crack width		=	**0.3**	**0.3**	**0.3**	**0.3**	mm IS 456-2000
Result			SAFE	SAFE	SAFE	SAFE	

		DOCUMENT NO	REV.	DATE
TITLE:		DESIGNED	CHECKED	SHEET
	Base raft Design			of

Check for one way shear at "d" distance from face of column

Actual Shear stress (Refer analysis results)	Tv	=	0.388	0.388	0.277	0.277	N/mm^2
Allowable shear stress	Tc	=	**0.317**	**0.317**	**0.317**	**0.317**	N/mm^2 Table-19 of IS:456-2000
			PROVIDE SHEAR REINF.	PROVIDE SHEAR REINF.	SHEAR REINF. NOT REQ.	SHEAR REINF. NOT REQ.	
	Tv-Tc	=	**0.071**	**0.071**	**-0.040**	**-0.040**	
Assume 12 mm dia. at 150 c/c @ one direction			**4630**	**4630**	**NOT REQ.**	**NOT REQ.**	C /C

Spacing required at another direction shall be

Shear reinforcement is calculated as $\frac{0.87 * fy * Asv}{(Tv-Tc) * b}$

Provide 12 mm dia at 150 c/c @ other direction

Check for two way shear / Punching shear

Critical Factored Shear force at column V = 0 kN

Ks = 1

Shear resistance at d/2 distance from the column

Tc = Ks * 0.25 * Sqrt (fck) = 1.25 N/mm^2

= 2045 kN > 0 kN SAFE

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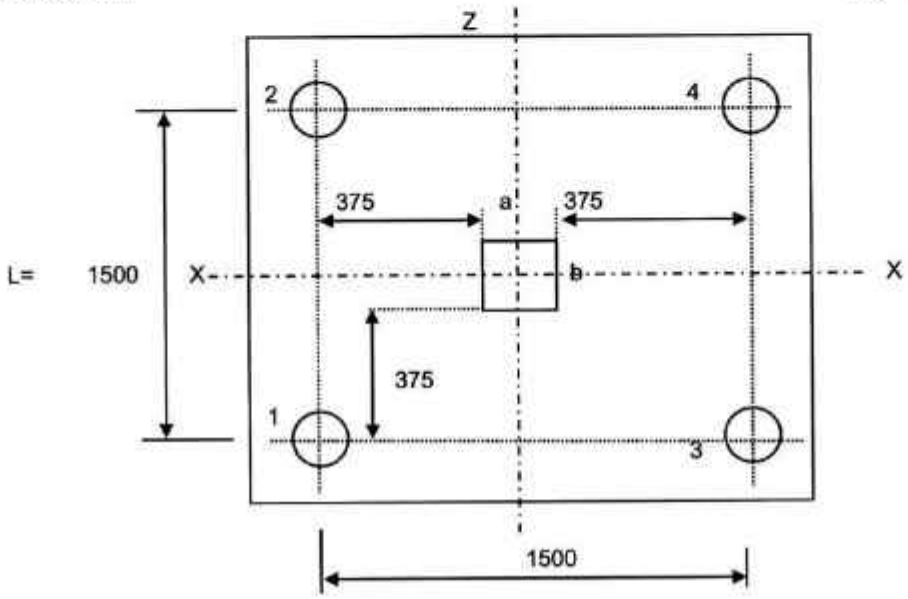
PROJECT:	DOCUMENT NO		DATE
	DESIGNED	CHECKED	SHEET OF

TITLE: Design Calculations - Pile Cap

Support-6:

1.Pile Layout

Diameter of Pile	h_p	= 500		mm
Spacing of Pile		= 3.00	D	
Size of Column		= 750	x 450	



Axial Load from Column	=	3590	kN

2.Forces in Pile

Pile No	Compression(kN)	Tension(kN)
1	821.00	200
2	821.00	200
3	821.00	200
4	821.00	200

3.Depth of Pile Cap

Bar size for effective depth		=	16	mm
Clear cover to reinforcement		=	75	mm
Total Depth of Pile Cap	D	=	900	mm
Effective depth	d	=	817	mm

4.Material Properties

Characteristic strength of Concrete	f_{cu}	=	25	N/mm^2
Characteristic strength of Steel	f_y	=	500	N/mm^2

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5.Check for Truss action

Shear span a_v in X-Direction		=	225	mm
Shear span a_v in Z-Direction		=	225.0	mm
a_v/d in X Direction		=	0.275	Truss action exists
a_v/d in Z Direction		=	0.275	Truss action exists

5.Tensile Forces

X-Direction

$$T_x = \frac{P}{24Ld}\left(3L^2 - a^2\right)$$ = 690.87 kN

Z-Direction

$$T_z = \frac{P}{24Ld}\left(3L^2 - b^2\right)$$ = 731.06 kN

PROJECT:	DOCUMENT NO		DATE
	DESIGNED	CHECKED	SHEET OF
TITLE: Design Calculations - Pile Cap			

6.TensileSteel

X-Direction

Area of Steel required		=	1588.20	mm^2
Breadth of pile cap	b	=	2200.00	mm^2
Percentage of steel	$p_{t,x}$	=	0.088	%
Minimum percentage of steel	$p_{t,min}$	=	0.180	%
Area of Steel provided		=	3235.32	mm^2

Z-Direction

Area of Steel required		=	1680.60	mm^2
Breadth of pile cap	b	=	2200.00	mm
Percentage of steel	$p_{t,z}$	=	0.094	%
Minimum percentage of steel	$p_{t,min}$	=	0.180	%
Area of Steel provided		=	3235.32	mm^2

7.Check for One WayShear

Max.Permissible shear stress	$v_{,max}$	=	4.0	N/mm^2

Section Parallel to Z axis

Critical section is at face of column

Shear force	V	=	1642.00	kN
Breadth at that section	b	=	2200.00	mm
Percentage of steel	p_t	=	0.180	%
Max.Permissible shear stress	$v_{,max}$	=	4.0	N/mm^2
Effective depth required to resist shear	d	=	186.6	< 817 Ok
Critical section is located at a distance av (from face of column)		=	226	mm
Shear Force at that section	V	=	1642.00	kN
Breadth at that section	b	=	2200.00	mm
Percentage of steel	p_t	=	0.180	%
Permissible shear stress	v	=	0.357	N/mm^2
Stress enhancement factor	2d/av	=	7.23	N/mm^2
Enhanced shear stress		=	2.580	N/mm^2 Ok
Effective depth required to resist shear	d	=	289.3	mm < 817 Ok

Section Parallel to X axis

Critical section is at face of column

Shear force	V	=	1642.00	kN
Breadth at that section	b	=	2200.0	mm
Percentage of steel	p_t	=	0.180	%
Max.Permissible shear stress	$v_{,max}$	=	4.0	N/mm^2
Effective depth required to resist shear		=	186.6	mm < 817 Ok
Critical section is located at a distance av (from face of column)		=	226.00	mm
Shear Force at that section	V	=	1642.00	kN
Breadth at that section	b	=	2200.0	mm
Percentage of steel	p_t	=	0.180	%
Permissible shear stress	v	=	0.357	N/mm^2
Stress enhancement factor	2d/av	=	7	
Enhanced shear stress		=	3	N/mm^2
Effective depth required to resist shear	d	=	289.27	mm < 817 Ok

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8.Check for Two WayShear(Punching Shear)

Critical section along periphery of Column

Max.Permissible shear stress	$v_{,max}$	=	4.0	N/mm^2

PROJECT:	DOCUMENT NO		DATE
	DESIGNED	CHECKED	SHEET OF
TITLE: Design Calculations - Pile Cap			

Perimeter at the section	u	=	3000	mm	
Shear Force	V	=	3284.0	kN	
Percentage of Steel in X-Direction	pt,x	=	0.180	%	
Percentage of Steel in Z-Direction	pt,z	=	0.180	%	
Max.Permissible shear stress	v	=	4.0	N/mm^2	
Effective depth required to resist shear	d	=	273.667	mm	< 817 Ok
Critical section at 20%pile diameter at inner side					
Max.Permissible shear stress	$v_{,max}$	=	4.0	N/mm^2	
Perimeter at the section	u	=	4400.0	mm	
Shear Force	V	=	3284.0	kN	
Percentage of Steel in X-Direction	pt,x	=	0.180	%	
Percentage of Steel in Z-Direction	pt,z	=	0.180	%	
Permissible shear stress	v	=	1.427	N/mm^2	
Effective depth required to resist shear	d	=	522.867	mm	< 817 Ok

9.Results

Depth of pile cap	d	=	817.00	mm
X-Direction				
Area of Steel		=	3235.32	mm^2
Diameter of rod		=	16.00	mm
Spacing for a width of 956.20 mm		=	59.42	mm
Provide 16 mm bars @ 100 mm c/c				
Z-Direction				
Area of Steel		=	3235.32	mm^2
Diameter of rod		=	16.00	mm2
Spacing for a width of 956.20 mm		=	59.42	mm
Provide 16 mm bars @ 100 mm c/c				
Total Depth of Pile Cap	D	=	900.00	mm
Top Steel and Side face Steel				
Area of Steel per direction		=	360.00	mm^2
Diameter of rod		=	16	mm
Spacing for a width of 1000 mm		=	558.51	mm
Provide 16 mm bars @ 100 mm c/c				

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PROJECT:	RESIDENTIAL BUILDING AT WATERCANAL ROAD	DOCUMENT NO				REV	
		O21051-C-RB-01-DC-0001					
TITLE:	IS - 1893 Part-I Codal Provision	DESIGNED		CHECKED		APPROVED	
CLIENT	M/S BUILT TECH ASSOCIATES	CPA		NKK		NKK	
		DATE	08.05.23	DATE	08.05.23	DATE	08.05.23

Design of RC Structural Walls:

Height of the wall	Lo	=	3000	mm
Thickness of the wall	h	=	230	mm
Characteristic Strength of concrete	fck	=	25	N/mm^2
Yield strength of the steel	fy	=	500	N/mm^2
Width of the wall	b	=	1200	mm
Effective height of the wall	Le	=	2250	mm
Clear Cover to Reinforcement,	c	=	40	mm
Assume Diameter of Main Reinforcement,	d	=	20	mm
Effective thickenss of wall	h_e	=	180	mm
	Le/h	=	$0.75L_o/h$	
		=	9.783 < 12	short braced

STAAD Result

Factored Bending Moment,	M	=	9	kNm
Factored Axial Force,	P	=	5354	kN
Factored Shear Force,	V	=	559	kN

Minimum accidental eccentricity

h/20 or 20 mm

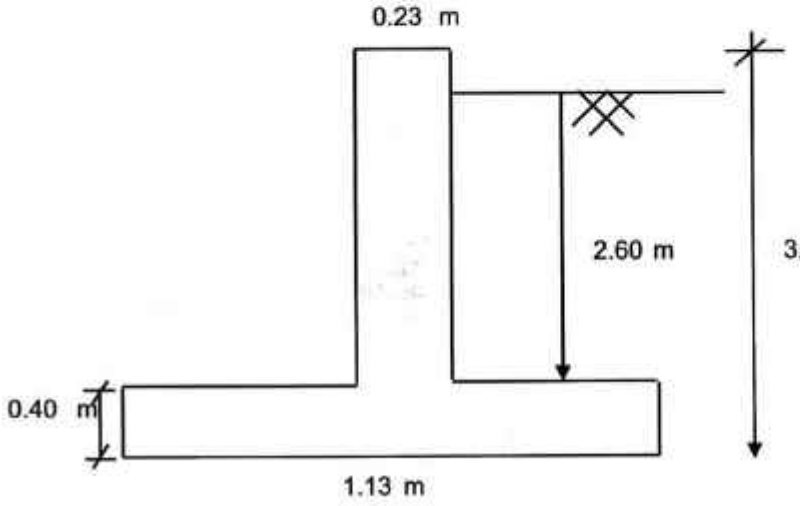
	e_{min}	=	11.5 mm	Adopt 20 mm
	emin	=	20 mm	
Eccentricity due to M (e_x)		=	M / P	
	ex	=	9 / 5354	
	ex	=	1.681 mm	
Additional eccentricity, ea		=	$L_e^2/2000h$	
		=	11.01 mm	
Total eccentricity,	e	=	12.7 mm	
Total Bending Moment, M		=	68 kNm	
	d'	=	50 mm	
	d'/h	=	0.217	
	$P/f_{ck}bh$	=	0.776	
	$M/f_{ck}bh^2$	=	0.043	
	p/f_{ck}	=	0.120	
Area of Reinforcement, Ast		=	pbh / 100	
		=	8280 mm^2	

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PROJECT:	DOCUMENT NO.		DATE
	DESIGNED	CHECKED	SHEET 1 OF 9
TITLE: DESIGN OF RETAINING WALL			

Design of retaining wall

Height of retaining wall	H	=	3.00	m
Height of soil below ground level	h_1	=	3.00	m
Thickness of stem at top	t	=	0.23	m
Safe bearing capacity	SBC	=	200	kN/m^2
Characteristic strength of concrete	fck	=	25	N/mm^2
Yield stress of steel	fy	=	500	N/mm^2
Co-efficient of friction	x	=	0.55	
Earth pressure co-efficient	Ka	=	0.33	
Surcharge load	q	=	4	kN/m^2
Lateral force above Ground level wind press (Reference)	p_d	=	0	kN/m^2
Unit weight of soil	γ	=	18	kN/m^3
Unit weight of concrete		=	25	kN/m^3
Length of Heel		=	0.45	m
Consider 1 m run of the wall				
Assume				
Base width	B	=	1.13	m
Thickness of base slab	d	=	0.40	m
Stem bottom thickness		=	0.23	m
Slope distance at the bottom of stem both side)		=	0.00	m
Height of soil above slab		=	2.60	m
Height of water above slab		=	0.00	m

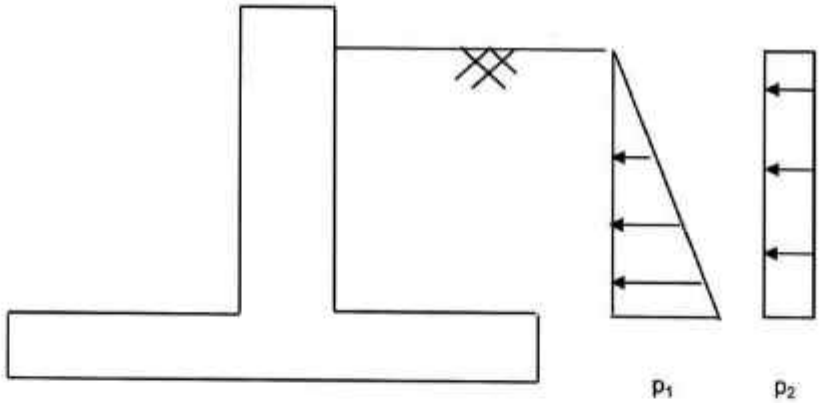


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PROJECT:	DOCUMENT NO		DATE
	DESIGNED	CHECKED	SHEET 2 OF 9
TITLE: DESIGN OF RETAINING WALL			

Calculation of pressure acting on wall

Pressure due to soil	p_1	=	(0.33 x 8 x 2.6)	
		=	6.864	kN/m^2
Pressure due to surcharge	p_2	=	0.33 x 4	
		=	1.32	kN/m^2
Pressure due to water	p3	=	0 * 10	
		=	0	kN/m^2
Pressure due to wind	p_3	=	$c_f \times A_e \times p_d$	
	c_f	=	1.2	
	A_e	=	1	m
		=	1.2 x 1 x0	
		=	0.000	kN/m^2



Check for overturning

Overturning moments

Moment due to wind	=	(0x0x(0+3))	
	=	0	kN-m/m
Moment due to earth pressure	=	0.33x8x2.6^3 /6	
	=	7.73	kN-m/m
Moment due to surcharge	=	(1.32x2.6x(2.6/ 2))	
	=	4.462	kN-m/m
Moment due to water table	=	0*10*0.5*0*(0/3)	
	=	0	
Total overturning moment	=	0+7.73+4.46+0	
	=	**12.2**	kN-m/m

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PROJECT:	DOCUMENT NO		DATE
	DESIGNED	CHECKED	SHEET 3 OF 9
TITLE: DESIGN OF RETAINING WALL			

Length of toe		=	0.45	m
Length of heel		=	0.45	m
Bottom thickness of stem		=	0.23	m
Thickness of base slab		=	0.40	m

Restoring moment

Taking moment about edge of the toe

Moment due to stem	M_1	=	0.23x2.6x 25 x0.565	
		=	8.447	kN-m
Lever arm 0.565 m				
Moment due to base slab	M_2	=	1.13x0.4x25x0.565	
		=	6.385	kN-m
Moment due to weight of soil	M_3	=	0.45x2.6x18x0.905	
		=	19.06	kN-m
Total restoring moment		=	**33.89**	kN-m

Factor of safety against overturning = Restoring moment/Overturinging moment

= **2.78** **Safe**

Check for sliding

Lateral forces on retaining wall:

Force due to wind		=	0	kN
Lateral load due to earth pressure		=	(1/2) x8x2.6^2 x0.33	
		=	8.92	kN
Lateral load due to surcharge		=	0.33x4x2.6	
		=	3.43	kN
Lateral load due to water		=	0* 10 * 0.5 * 0	
		=	0	
Total lateral force on wall		=	0+8.92+3.43+0	
	W_L	=	12.35	kN

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PROJECT	DOCUMENT NO		DATE
	DESIGNED	CHECKED	SHEET 4 OF 9

TITLE: DESIGN OF RETAINING WALL

Total vertical load due to retaining wall and soil:

Self weight of stem	W_1	=	0.23x2.6x25	
		=	14.95	kN
	W_2	=	(0x2.6x25)/ 2	
		=	0	kN
	W_3	=	(0x2.6x25)/ 2	
		=	0	kN
Self weight of base slab	W_2	=	1.13x0.4x25	
		=	11.30	kN
Self weight of soil	W_3	=	0.45x2.6x18	
		=	21.06	kN
	W_6	=	0.45x0x18	
		=	0	kN
Total vertical load	W_V	=	14.95+11.3+21.06	kN
		=	47.31	kN

Factor of safety against sliding = x^*W_V / W_L

= **2.11** **> 1.4 HENCE O.K.**

Sliding is not critical as retaining wall is attached with base slab and pile also.

Check for maximum & minimum stress

Force (kN)	Lever arm (m)	Moment (kN-m) about toe
Lateral load due to earth pressure		
= 8.92 ←	0.87	-7.73
Lateral load due to surcharge		
= 3.43 ←	1.3	-4.46
Lateral load due to water		
= 0.00 ←	0	0.00
Self weight of stem		
= 14.95 ↓	0.565	8.45
Self weight of base slab		
= 11.30 ↓	0.565	6.38
Weight of soil		
= 21.06 ↓	0.905	19.06

Total vertical load = 47.3 kN Total moment 21.7 kN-m

Eccentricity e = 0.106 m **<b/6 , OK**

Max stress = **65.5** **OK**

Min stress = **18.2** **OK**

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PROJECT	DOCUMENT NO		DATE
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TITLE: DESIGN OF RETAINING WALL			

Design of stem

Maximum bending moment due to soil & surcharge		=	12.2	KN-m
Factored bending moment	M_u	=	18.3	KN-m
Clear cover		=	40	mm
Effective depth (if 8 mm diameter bar is used)		=	230-40-4	
		=	186	mm
	M_u/bd^2	=	18.3x10^6/1000 x186^2	
		=	0.53	
Percentage of reinforcement	p_t	=	0.12	
Area of steel	Ast	=	pt x b x d /100	
		=	232	mm^2
Spacing of 8 mm diameter bar	S_v	=	217	mm

Provide 8 mm diameter bars at 200 mm C/C on soil face vertically

Minimum reinforcement on the inside face

If cantilever is propped then maximum positive (sagging) moment,	M_s	=	h_1^2*(p_1/33.54+9*p_2/128)	
		=	2.7	KN-m
	Ms/b*d^2	=	0.08	
Required % of steel,		=	0.02	
Minimum reinforcement is 0.12% and hence on each face 0.06%		=	0.06	
Minimum reinforcement %	p_t	=	0.06	
Minimum reinforcement	A_{st}	=	pt x b x d /100	
		=	112	mm^2
Spacing of steel reinforcement(8 mm diameter bars to be used)	S_v	=	450	mm

Provide 8 mm diameter bars at 250 mm C/C on inner face vertically

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PROJECT	DOCUMENT NO		DATE
	DESIGNED	CHECKED	SHEET 6 OF 9
TITLE: DESIGN OF RETAINING WALL			

Horizontal distribution reinforcement

Minimum reinforcement is 0.20% and hence on each face 0.10% p_t = 0.10

Use 8 mm diameter bar

Area of steel A_{st} = 0.1x230x 1000 /100

Area of steel reinforcement = 230 mm^2

Spacing of bars S_v = 219 mm C/C

Provide 8 mm diameter bars on both the faces at a spacing of 200 mm C/C horizontally

Check for shear

Shear force		=	12.36	KN
Factored shear force	V_u	=	18.53	KN
Nominal shear stress,	τ_v	=	0.10	N/mm^2
Maximum shear stress	$\tau_{c\,max}$	=	3.16 **Ok**	N/mm^3
For M30& p_t = 0.14 % , shear strength of concrete,	τ_c	=	0.28 **Safe**	N/mm^2

251 beta= 21.5 gama= 9.4 alpha= 4.47

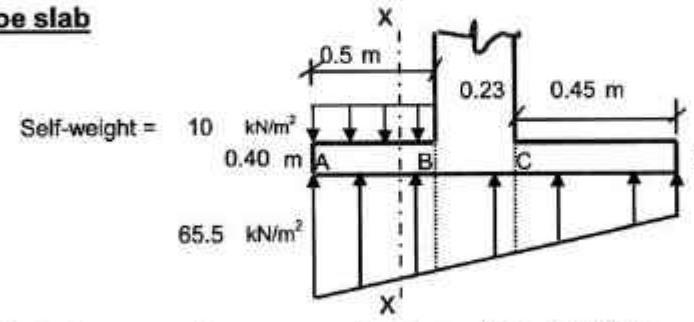
Check for deflection

Basic span value of span to effective depth ratio,	α =	10.00	
Modification factor for span larger than 10 m,	β =	1.00	
Modification factor depending on tension steel & stress,	γ =	1.44	
intermediate calculation pc= 0.11		1.24	
Modification factor depending on compression steel,	δ =	1.03	
Modification factor for flaged beam,	λ =		
Allowable L/d ratio,	$(L/d)_{Allowable}$ =	14.90	
Actual L/d ratio,	$(L/d)_{Actual}$ =	13.98	**Ok**

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PROJECT	DOCUMENT NO		DATE
	DESIGNED	CHECKED	SHEET 7 OF 9
TITLE: DESIGN OF RETAINING WALL			

Design of toe slab



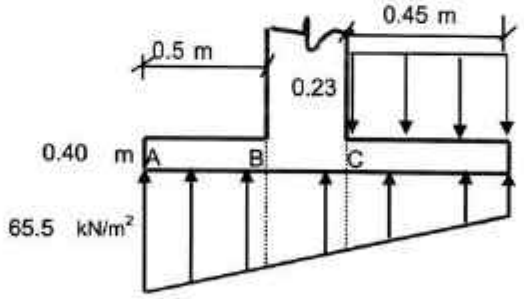
The toe is subjected to an upward pressure varying from 65.5 kN/m² to 46.7 kN/m².

The self-weight of toe		=	25x0.4	
		=	10	kN/m²
The upward water pressure		=	0	kN/m²
The net upward pressure at A		=	55.5	kN/m²
The net upward pressure at B		=	36.7	kN/m²
The net upward pressure at shear critical section X-X		=	51.5	kN/m²
The toe is treated as cantilever beam with critical section for shear at a distance of d = 0.355 m from face of stem B.				
Hence, length of cantilever from A to critical section for shear		=	0.45-0.355	
		=	0.10	m
Factored shear force at critical section, V_u		=	1.5x0.5x(55.5+51.5)x0.1	
		=	7.6	kN/m
Factored moment due to net upward pressure about B, M_u		=	0.45^2x (55.5+2x36.7)/6	
		=	6.5	kN-m/m
Required effective depth, d		=	43	mm
Provided effective depth is more than required.				
	M_u/bd^2	=	6.525x10^6 /(1000x355^2)	
		=	0.052	
Percentage of reinforcement	p_t	=	0.01 %	
Area of steel reinforcement	Ast	=	0.01x1000x355/100	
		=	42	mm²
Minimum % reinforcement for slab is 0.12% of gross section,	Ast	=	480	
Minimum steel		=	480	mm²
Use 10 mm diameter bars				
Area of one steel bar		=	3.14 x10^2 /4	
		=	79	mm²
Spacing of reinforcement		=	79 x 1000 /480	
		=	165	mm
Provide 10 mm Diameter bars with 150 mm C/C				
Check for shear				
Nominal shear stress,	τ_v	=	0.02	N/mm²
Maximum shear stress	$\tau_{c\,max}$	=	3.16 **Ok**	N/mm³
For M30& p_t = 0.15 % , shear strength of concrete,	τ_c	=	0.29 **Safe**	N/mm²

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PROJECT	DOCUMENT NO		DATE
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TITLE: DESIGN OF RETAINING WALL			

Design of heel slab



The toe is subjected to an upward pressure varying from 37.1 kN/m² to 18.2 kN/m².

The self-weight of toe, soil weight and surcharge		=	25x0.4+4+2.6x18	
		=	60.8	kN/m²
The upward water pressure		=	0	kN/m²
The net downward pressure at C		=	23.7	kN/m²
The net downward pressure at D		=	42.6	kN/m²
Provide clear cover,		=	40	mm
Effective depth, d		=	355	mm
Factored shear force at critical section, V_u		=	1.5x0.5x(23.7+42.6)x0.45	
		=	22.4	kN/m
Factored moment due to net upward pressure about B, M_u		=	0.45^2x (42.6+2x23.7)/6	
		=	4.56	KN-m/m
Required effective depth, d		=	36	mm
Provided effective depth is ok.				
	M_u/bd^2	=	4.56x10^6 /(1000x355^2)	
		=	0.036	
Percentage of reinforcement	p_t	=	0.01 %	
Area of steel reinforcement	Ast	=	0.01x1000x355/100	
		=	30	mm²
Minimum % reinforcement for slab is 0.12% of gross section,	Ast	=	480	
Minimum steel		=	480	mm²
Use 10 mm diameter bars				
Area of one steel bar		=	3.14 x10^2 /4	
		=	79	mm²
Spacing of reinforcement		=	79 x 1000 /480	
		=	165	mm

Provide 10 mm Diameter bars with 150 mm C/C

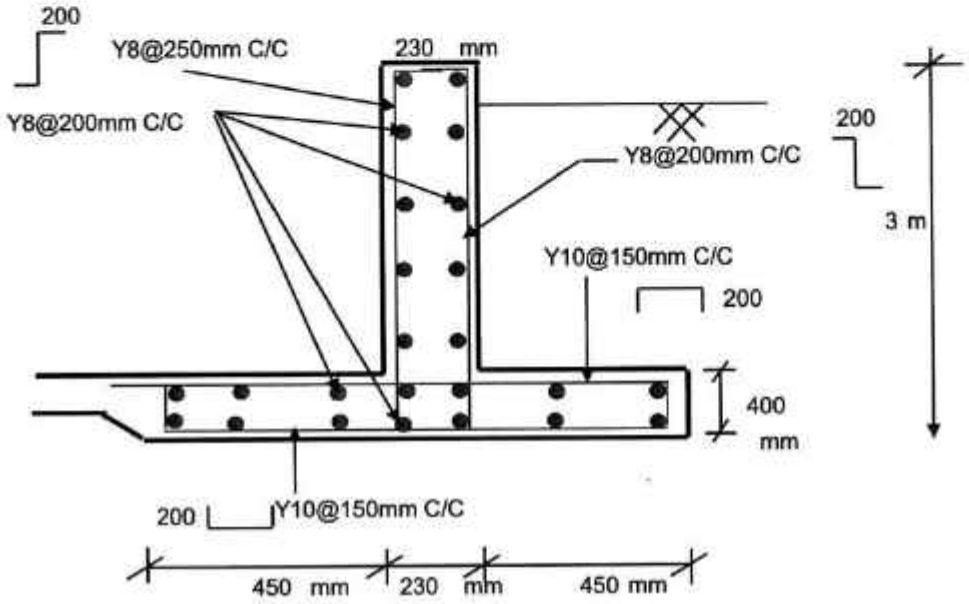
Check for shear

Nominal shear stress,	τ_v	=	0.06		N/mm²
Maximum shear stress	$\tau_{c max}$	=	3.16	Ok	N/mm³
For M30 & p_t = 0.15 % , shear strength of concrete,	τ_c	=	0.29	Safe	N/mm²

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PROJECT:	DOCUMENT NO		DATE
	DESIGNED	CHECKED	SHEET 9 OF 9
TITLE: DESIGN OF RETAINING WALL			

Reinforcement detail

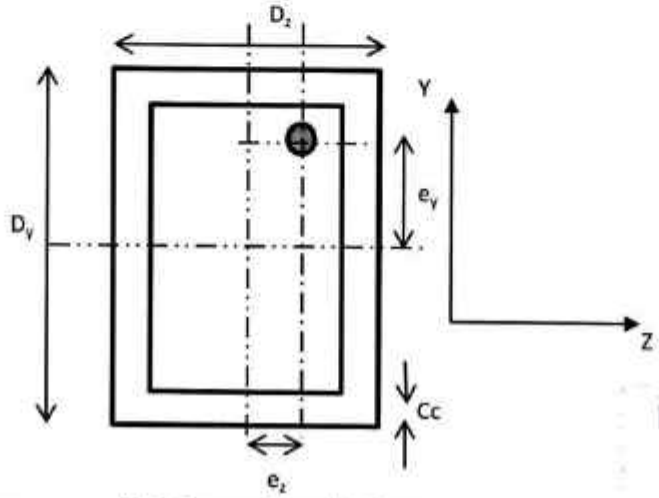


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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REV	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF COLUMN	DESIGNED		CHECKED		APPROVED	
CLIENT:	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

Input:

Depth of column D)		=	450	mm
Width of column (D_z)		=	750	mm
Unsupported length (L_y)		=	3000	mm
L_z			3000	mm
Column condition against sidesway		=	Restrained	
Multiplication factor of (k_y,k_z)	k_{y1}	=	0.8	
	k_{z1}	=	0.8	
Effective length (L_{ey})		=	2400	
Effective length (L_{ez})		=	2400	
Clear cover(cc)		=	50	mm



Pile, Pile cap **Fig:1 Cross section of column**

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Stadd.Pro

Analysis Result

Axial load (kN) P_u			5435	Compression
Moment (kNm)	M_{uy}	Bottom	5	Clock-wise
	M_{uz}	Bottom	22	Clock-wise

Axial load - [+ve= Compression , -ve=Tension]

Moment -[+ve = Clock-wise, -ve = Anti-clockwise]

Grade of Materials

Concrete (F_{ck}) N/mm^2	Steel (F_y) N/mm^2
25	500

Reinforcement Details

Longitudinal bar(mm)	25
Tie bar(mm)	8
Reinfircement %	2.5

Slenderness Ratio:

$$\text{About y-y} = \frac{L_{ey}}{Dy} = \frac{2400}{450} = 5.3333 < 12$$

$$\text{About z-z} = \frac{L_{ez}}{Dz} = \frac{2400}{750} = 3.2 < 12$$

Short-Column

Column is not slender about both axis

$$\text{Effective cover d'} = cc + \frac{\text{Dia of bar}}{2} = 50 + \frac{25}{2} = 62.5 \text{ mm}$$

$$\frac{d'}{D} = \frac{62.5}{450} = 0.14$$

$$\frac{d'}{B} = \frac{62.5}{750} = 0.08$$

PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REV	0
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CLIENT:	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

Minimum eccentricity:

e_min =Max of (((Unsupported length)/500+(Late

$$e_{min,y} = \frac{3000}{500} + \frac{450}{30} = 21 > 20 = 21 \text{ mm}$$

$$e_{min,z} = \frac{3000}{500} + \frac{750}{30} = 31 > 20 = 31 \text{ mm}$$

Additional Moment Calculation:

M_a = P_(u) e_(y) k_y

M_a = Additional moment along major and minor axis

e_a = Additional eccentricity along major and minor axis

ky = Reduction factor

e_a = $D_y/2000 \times (L_{ey}/i$

$$e_{ay} = \frac{750}{2000} \times \left(\frac{2400}{750}\right)^{\wedge 2} = 3.84 \text{ mm}$$

$$e_{az} = \frac{450}{2000} \times \left(\frac{2400}{450}\right)^{\wedge 2} = 6.4 \text{ mm}$$

k = Min of((P_uz/P_uz - P_u/P 26

Here,

P_{uz} = Maximum pure compression strength of the column

P_b = Axial strength corresponding to balance failure about the axis

P_u = Axial load

$P_{uz} = 0.45 f_{ck} \times D \times b + (0.75 \times f_y - 0.45 f_{ck}) A_s$ 44

$= (0.45 \times 450 \times 750) + (0.75 \times 500 - 0.45 \times 25)(0.025 \times 450 \times 750)$

= 6866.02 mm

IS456 1978 Table 6

P_b= (k_1+k_2 p/f_ck)f_ckBD

k_{y1} = 0.198

k_{y2} = 0.29

$P_{by} = (0.198 + 0.29 \times 0.025) \times 25 \times 450 \times 750$ = 1731.8 kN

$$k_y = \frac{6866.02}{6866.02} - \frac{5435}{1731.8}, \ 1 = 0.2787$$

k_{z1} = 0.211

k_{z2} = 0.493

$P_{bz} = (0.211 + 0.493 \times 0.025) \times 25 \times 450 \times 750$ = 1884.3 kN

$$k_z = \frac{6866.02}{6866.02} - \frac{5435}{1884.3}, \ 1 = 0.2873$$

$M_{ay} = 5435 \times 3.84 \times 0.2787$ = 5.82 kNm

$M_{az} = 5435 \times 6.4 \times 0.2873$ = 9.993 kNm

Moment Due to Eccentricity:

$M_e = P_u \times e_{min}$

$$M_{ey} = \frac{5435 \times 31}{1000} = 168.49 \text{ kNm}$$

$$M_{ez} = \frac{5435 \times 21}{1000} = 114.14 \text{ kNm}$$

Design moment:

M_d = **Max(M_e,M_u) + M_a**

M_{dy} = Max(5,168.485) + 5.82 = 174.305 kNm

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REV	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF COLUMN	DESIGNED		CHECKED		APPROVED	
CLIENT:	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

M_{dy} = Max(22,114.135) + 9.993 = 124.128 kNm

Uniaxial moment capacity about y-y axis:

$\frac{P_u}{F_{ck} * BD} = \frac{5435 \times 10^3}{25 \times 750 \times 450}$ = 0.64

$\frac{M_{uy}}{F_{ck} * bD^2}$ = = 0.09524

$M_uy1 = M_uy/(f_ck$

= 0.09524 x 25 x 750 x 450^2 = 362 kNm

Uniaxial moment capacity abouz-z axis:

$\frac{P_u}{F_{ck} * BD} = \frac{5435 \times 10^3}{25 \times 750 \times 450}$ = 0.64

$\frac{M_{uz}}{F_{ck} * DB^2}$ = = 0.080

$M_uz1 = M_uz/(f_ck$

= 0.08 x 25 x 450 x 750^2 = 506.25 kNm

Calculation of P_{uz}

p_t = 2.5 %

$\frac{P_u}{P_{uz}} = \frac{5435}{6866}$ = 0.7916

(P_u/P_{uz})	α^n
0.2	1
0.8	2
Other values	Interpolate

α^n = 1.99

$\frac{M_{dy}}{M_{uy1}} = \frac{174.305}{362}$ = 0.49

$\frac{M_{dz}}{M_{uz1}} = \frac{124.128}{506.25}$ = 0.25

Criteria for Biaxial Bending (Bresler's Formulation)

IS 456-2000
Cl 39.6
P.NO:71

$[Mdy/Muy1\,]\alpha^n+[Mdz/$

(0.49) ^ 1.99 + (0.25) ^ 1.99 = 0.31 ≤ 1

Check for Maximum and Minimum Reinforcement

IS 456-2000
Cl 26.5.3.1
P.No: 48

Maximum reinforcement = 4% of cross sectional area

$= \frac{4}{100} \times 450 \times 750$ = 13500 mm^2

Minimum reinforcement = 0.8% of cross sectional area

$= \frac{0.8}{100} \times 450 \times 750$ = 2700 mm^2

Provided reinforcement = 2.5% of cross sectional area

$= \frac{2.5}{100} \times 450 \times 750$ = 8437.5 mm^2

Codal provisions are satisfied

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REV	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF COLUMN	DESIGNED		CHECKED		APPROVED	
CLIENT:	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

Transverse reinforcement:

Lateral Ties

IS 456:2000 Clause 26.5.3.2 P.NO:49

1) Dia of tie bar sholuld be

$$> \frac{\text{Maximum longitudinal dia of bar}}{4} = \frac{25}{4} = 6.25 \text{ mm}$$

> 6mm = 6 mm

Take high value 8 mm

2) Pitch

= Least dimesnion fo compression member = 450 mm

(or)

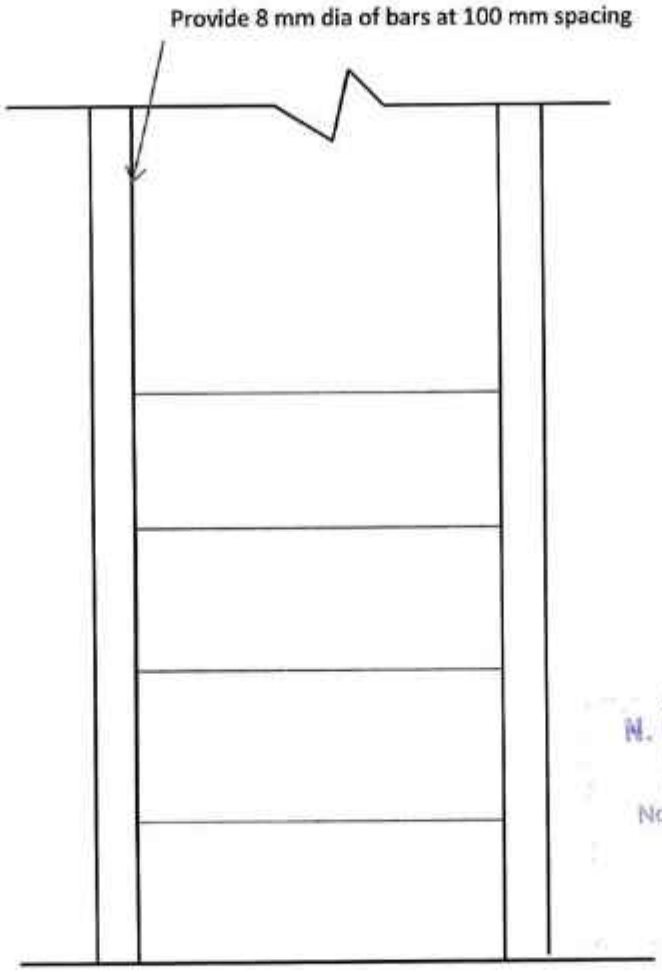
= 16*smallest dia of longitudinal bar = 400 mm

(or)

= 300mm = 300 mm

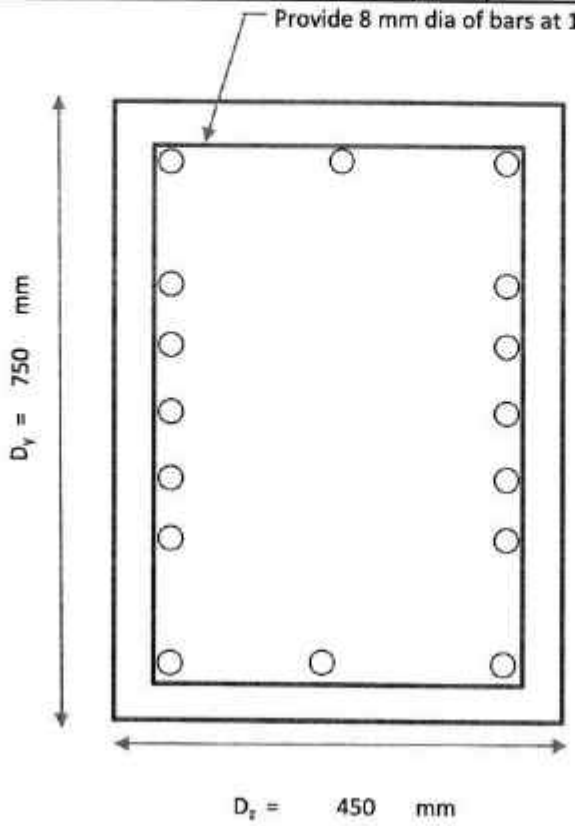
Take low value 100 mm

Provide 8 mm dia of bars at 100 mm spacing
Provide closer spacing on either side of the column
above and below beam / slab.



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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REV	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF COLUMN	DESIGNED		CHECKED		APPROVED	
CLIENT:	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023



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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REV	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF BEAM	DESIGNED		CHECKED		APPROVED	
CLIENT:	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

Span(L) = 7.2 m

Clear cover(C_c) = 30 mm

Modulus of elasticity of concrete (Ec) = $5000\sqrt{f_{ck}}$ (N/mm^2) = 25000

Modulus of elasticity of steel(E_s) (N/mm^2) = 200000

Type of beam = Fixed

Environment condition = Not Aggressive

Beam Numeration Details:-

Depth of Beam (D)mm	Width of Beam(b)mm
600	230

Grade of Concrete /Rft

Concrete(F_{ck}) N/mm^2	Steel (F_y) N/mm^2
25	500

Longitudinal Reinforcement Details:

Location	Left Support		Mid		Right Support	
	Dia(mm)	No	Dia(mm)	No	Dia(mm)	No
Top	20	2	20	2	20	2
Bottom	20	2	20	2	20	2

Transverse Reinforcement Details:

Location	Left Support	Mid	Right Support
Bar dia	8	8	8
No of Leg	2	2	2

ANALYSIS RESULT:

Stadd.pro

Location	Left Support	Mid	Right Support
BM at Top (kNm)	170	0	106
BM at Bottom (kNm)	0	214	0
SF (kN)	178	0	158
Torsion (kNm)	0	0	0

DESIGN CALCULATIONS:(For Top BM at Left support)

Equiavelent Bending moment:-

$M_e = M_u + M_t$

$M_t = T_u \frac{(1+D/b)}{1.7} = 0\left(\frac{1+ 600/230}{1.7}\right)$ = 0 kN.m

$M_e = 170 + 0$ = 170 kN.m

Limiting Moment:

M_u limit = 0.133*Fck*b*d*d

eff.depth = D-dia of bar x 0.5-cover = 560 mm

M_u limit = 0.133 x 25 x 230239.83 x 560 = 239.83 kNm

The Section is singly reinforced

$M_u < M_u$limit

Reinforcement At Tension Zone:

Singly Reinforced Beam:

IS456-2000

Annex-G

$$A_(st.required) = (bdf_ck)/[2f]_$$

$$= \frac{1000 \times 600 \times 25}{2 \times 500}\left[1-\left[1-\frac{4.598 \times 170 \times 10\text{^}6}{25 \times 1000 \times 560\text{^}2}\right]^{1/2}\right] = 594.2 \ mm^2$$

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		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF BEAM	DESIGNED		CHECKED		APPROVED	
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		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

Doubly Reinforced Beam:

SP.16 P.NO 10

$$A_{st\,Req} = A_{st.limit} + \Delta A_{st}$$

$$A_{st.limit} = \frac{p_t\ limit}{100} \times bd$$

$$P_{t.limit} = \frac{0.36\ F_{ck} * b * X_{umax}}{0.87 F_y * b * d} \times 100$$

$$\frac{Xu_{max}}{d} = \frac{0.0035}{0.0055 + \frac{0.87 * f_y}{E_s}} = 0.456$$

$$Pt_{limit} = \frac{0.36 \times 25 \times 230 \times 0.456 \times 560}{0.87 \times 500 \times 230 \times 560} \times 100 = 0.94\ \%$$

$$A_{st.limit} = \frac{0.94}{100} \times 230*560 = 0\ mm^2$$

$$\Delta A_{st} = \frac{M_u - M_u limit}{0.87 \times f_y \times (D-d')}$$

$$d' = 40\ mm$$

$$\Delta A_{st} = \frac{170 - 239.83}{0.87 \times 500 \times (560 - 40)} = 0\ mm^2$$

$$A_{st\,Req} = 0 + 0 = 0\ mm^2$$

Minimum reinforcement 26

IS456-2000

$$= \frac{0.85 \times bd}{Fy} = 218.96\ mm^2$$

IS13920 Cl.6.2.1

$$= \frac{0.24 \times \sqrt{Fck} * bd}{Fy} = 309.12\ mm^2$$

$A_{st\,pro}$ 44 = 595 mm^2

Maximum reinforcement:

IS456-2000

$A_{st\,max} = 0.04bD = 5520\ mm^2$

IS13920 Cl.6.2.2

$A_{st\,max} = 0.025bD = 3450\ mm^2$

$> A_{st\,pro}$ SAFE

Provide2No's of 20mm Dia bars

Reinforcement At Compression Zone:

$$A_{sc} = \frac{0.87 \times f_y \times \Delta A_{st}}{F_{sc} - 0.447 \times f_{ck}}$$

SP.16 P.NO 6

$$E_{sc} = \frac{0.0035 x (X_{umax} - d')}{X_{umax}}$$

$$X_{umax} = 0.456 \times d = 255.36$$

$$E_{sc} = \frac{0.0035 \times (255.36 - 40)}{255.36}$$

$$= 0.002951754$$

$$F_{sc} = 418.52\ N/mm^2$$

$$A_{sc} = \frac{0.87 \times 500 \times 0}{418.52 - 0.447 \times 30} = 0\ mm^2$$

Min A_{sc} = 309.12 mm^2

$A_{sc\,provided}$ = 310 mm^2

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Provide 2 No's of 20mm dia bars

Longitudinal reinforcement Details:

Top face:-

Zone	M_{eq}	M_{ulim}	Section	A_{streq}	$A_{st.pro}$	A_{screq}	$A_{st.pro}$	Status
Left	170	239.83	Singly	594.2	594.2	0	309.1	SAFE
Mid	0	239.83	Singly	0	309.1	0	309.1	SAFE
Right	106	239.83	Singly	367.5	367.5	0	309.1	SAFE

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Bottom face:-

Zone	M_{eq}	M_{ulim}	Section	A_{streq}	$A_{st.pro}$	A_{screq}	$A_{st.pro}$	Status
Left	0	239.83	Singly	0	594.2	0	309.1	SAFE
Mid	214	239.83	Singly U11	752.3	752.3	0	309.1	SAFE
Right	0	239.83	Singly	0	594.2	0	309.1	SAFE

SHEAR STRESS CALCULATION

Equiavelent Shear force:-

$$V_e = V_u + \frac{1.6T_u}{b} = 178 + \frac{1.6 \times 0}{0.23} = 178 \text{ kN}$$

$$\text{Shear Stress } \tau_v = \frac{V_e}{bd} = \frac{178 \times 1000}{230 \times 560} = 1.382 \text{ N/mm}^2$$

$$\text{Design Shear Stress } \tau_c = \frac{0.85\sqrt{(0.8\,Fck)}*(\sqrt{(1+5\beta)}-1)}{6\beta}$$

$$pt = \frac{100Ast}{bD} = \frac{100 \times 595}{600 \times 230} = 0.43116 \%$$

$$\beta = 0.8f_{ck}/6.89pt$$

(P.NO. 175)

$$= \frac{0.8 \times 30}{6.89 \times 0.44} = 8.08$$

$$\tau_c = \frac{0.85 \times\sqrt{(0.8x25)(\sqrt{(1+5x8.08)}-1))}}{6x8.08} = 0.426 \text{ N/mm}^2$$

IS456-2000 TABLE NO 20

τ_{cmax} = 3.1 /mm²

$\tau_c < \tau_v < \tau_{cmax}$

Provide shear reinforcement

$V_{us} = Vu - \tau c*bd = 123.12$ kN

$A_{sv} = 101 \text{ mm}^2$

$$S_v = \frac{0.87 F_y A_{sv} d}{V_{us}} = \frac{0.87 \times 500 \times 101 \times 560}{123.12 \times 1000} = 199.834 \text{ mm}$$

Minimumm and maximum spacing of striuups:

As per IS13920-2016:

At support:

IS13920-2016 CL.6.3.5

Maximum Spacing is minimum of

i) $\frac{d}{4} = \frac{560}{4} = 140$ mm

ii) $8\Phi = 8 \times 20 = 160$ mm

(Take diametre of smallest bar)

iii) 100mm = 100 mm

Take spacing as per IS3920 = 100 mm

At Mid:

Maximum Spacing is minimum of

i) $\frac{d}{2} = \frac{560}{2} = 280$ mm

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As per IS456-2000:

Maximum:

IS456-2000 CL.26.5.1.5

i) Max spacing of stirrups = 0.75d = 420 mm

ii) 300mm = 300 mm

Minimum:

$$A_{svmin} = \frac{0.4\ b\ S_v}{0.87\ F_y}$$

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$$S_v = \frac{(0.87 \times 500 \times 101)}{0.4 \times 230} = 477.6 \text{ mm}$$

S_{vprt} = 100 mm — SAFE

Provide T8 mm 2 Legged stirrups at 100 mm C/C

Transverse Reinforcement Details:

Zone	V_{eq}	τ_v	τ_c	τ_{cmax}	V_{us}	A_{sv}	S_v	Status
Left	178	1.3820	0.47	3.1	117.5	101	100	SAFE
Mid	0	0	0.47	3.1	60.54	101	280	SAFE
Right	158	1.22671	0.42589	3.1	103.2	101	100	SAFE

DEVELOPMENT LENGTH:

$$L_d = \frac{\phi \sigma_s}{\tau_{bd}} = \frac{20 \times 0.87 \times 500}{3} = 2900 \text{ mm}$$

Bar diametre ϕ = 20 mm

Bond stress τ_{bd} = 3 N/mm²

DEFLECTION :

$$\delta_L = \frac{1}{384} \times \frac{wl^4}{EI_{eff}}$$

$$w_{service} = \frac{12M_{eq}}{l^{2 \times 1.5}} = \frac{12 \times 170}{7.2 \times 7.2 \times 1.5} = 26.2346 \text{ kN}$$

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M} \times \frac{Z}{d}\left(1-\frac{a}{d}\right) \times \frac{b_w}{b}}$$

$I_r \le I_{eff} \le I_{gr}$

Cracking moment $M_r = \frac{F_{cr} \times I_{gr}}{y_t} = \frac{3.5 \times 4140000000}{300} = 48.3$ kNm

y_t = 300 mm

I_{gr} = 4140000000 mm⁴

Moduulus of rupture $F_{cr} = 0.7\sqrt{F_{ck}} = 3.5$ N/mm²

Modular ratio m = 8

$$\frac{b a^2}{2} = m\,Ast\,(d-a)$$

$$a^2 = \frac{2m\,Ast\,d - 2m\,Ast\,a}{b}$$

$$= \frac{5331200}{230} - \frac{9520}{230} * a$$

$$a^2 = 23180 - 41.3913043\,a$$

Neutral axis a = 133 mm

Lever arm $Z = d - \frac{a}{3} = 560 - \frac{133}{3} = 555.7$ mm

Moment at service load = 113.3 kNm

$$I_r = \frac{ba^3}{12} + m A_{st.pro} (D-a)^2 + (m-1)\,Asc.pro\,(a-d')^2$$

$$I_r = \frac{230x133x133x133}{3} + 8 \times 595 \times (600-133)^2$$

$$+ (8-1) \times 310 \times (133-40)^2$$

Critical moment of inertia I_r (mm^4) = 1101964179

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		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF BEAM	DESIGNED		CHECKED		APPROVED	
CLIENT:	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

$$I_{eff} = \frac{1101964179}{1.2 - \frac{48.3}{113.3} \times \frac{555.7}{560}\left(1-\frac{133}{560}\right) \times \frac{230}{230}} = 1255693776 \text{ mm}^4$$

$Ir \le Ieff \le Igr$ $\quad I_{eff}\ (mm^4) = 1255693777$

i)Deflection due to load

$$\delta_L = \frac{26.2345679012346 \times 7.2 \times 7.2 \times 7.2 \times}{384 \times 25000 \times 1255693777} = 5.85 \text{ mm}$$

ii)Deflection due to shrinkage:

$\delta_s = k_3 \times \Psi_{es} \times L^2$

Shrinkage curvature $\Psi_{es} = k_4 \frac{\varepsilon_{cs}}{D} = 0.322 \frac{0.0003}{600} = 0.161 \times 10^{\wedge -6}$

$\varepsilon_{cs} = 0.0003$

if, $0.25 \le P_t - P_c < 1.0$ $\quad k_4 = \frac{0.72(P_t-P_c)}{\sqrt{p_t}} = \frac{0.72 \times 0.2}{0.4472136} \le 1 = 0.322$

if, $P_t - P_c \ge 1.0$ $\quad k_4 = \frac{0.65(P_t-P_c)}{\sqrt{p_t}} = \frac{0.65 \times 0.2}{0.4472136} \le 1 = 0.196$

$P_t - P_c = 0.44 - 0.23 = 0.2$ %

$k_4 = 0.322$

$k_3 = 0.125$

$\delta_s = 0.125 \times 0.161 \times 10\wedge -6 \times 7.2 \times 7.2 \times 10\wedge 6 = 1.04328$ mm

iii)Deflection due to creep:

$$\delta_c = \delta_{1c} - \delta_1 = \frac{5}{384} \times \frac{wl^4}{E_{cc} I_{eff}} - \frac{5}{384} \times \frac{wl^4}{E_c I_{eff}}$$

$$= \frac{5}{384} \times \frac{wl^4}{I_{eff}} \left(\frac{1}{Ecc} - \frac{1}{Ec}\right), \quad Ecc = \frac{Ec}{1+\theta}, \quad \theta = 1.5$$

$$= \frac{5}{384} \times \frac{wl^4}{I_{eff}} \theta$$

$$= \frac{26.2345679012346 \times 7.2 \times 7.2 \times 7.2 \times}{384 \times 25000 \times 1255693777} \times 1.5 = 8.775 \text{ mm}$$

$\delta = \delta_l + \delta_s + \delta c = 15.67$ mm

Limiting Deflection:

IS 456-2000
CL 23.2

$$\delta_{max} = \frac{Span}{350}, \ 20$$

$= 20.57, \ 20$

$= 20$ mm

$\delta_{max} > \delta$ = SAFE

CRACK WIDTH CALCULATION

$$W_{cr} = \frac{3a_{cr} \times \varepsilon_m}{1 + \frac{2(a_{cr} - C_{min})}{h - x}}$$

$a_{cr} = [(0.5S]_2 + C^2_{min}]^{1/2}$

Cover to rft $C_{min} = 30$ mm

Spacing of bars $S = 100$ mm

$a_{cr} = 59$

$$\varepsilon_m = \varepsilon_1 - \frac{b(h-a)(g-a)}{3E_s A_s (d-a)}$$

$$\varepsilon_1 = \frac{Fs}{Es} \times \frac{(h-a)}{(g-a)}$$

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		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF BEAM	DESIGNED		CHECKED		APPROVED	
CLIENT:	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

$$F_s = \frac{m^*\text{Service moment}^*(d-a)}{I_r}$$

$$= \frac{8\text{x}113.34\text{x}10\text{^}6\,(560-133)}{1101964179} = 351.4 \quad N/mm^2$$

D = h = 600 mm

Crack width calculating distance from combression edje = g = 20 mm

$$\varepsilon_1 = \frac{351.4}{200000} \times \frac{(600-133)}{(20-133)} = -0.00726$$

$$\varepsilon_m = -0.01 - \frac{230(600-133)(20-133)}{3 \times 200000 \times 595(560-133)} = 0$$

$$W_{cr} = \frac{3 \times 59 \times 0}{1 + \frac{2(59-30)}{(600-133)}} = 0.000 \text{ mm}$$

IS456-2000 Limiting crack width = 0.3 mm

SAFE

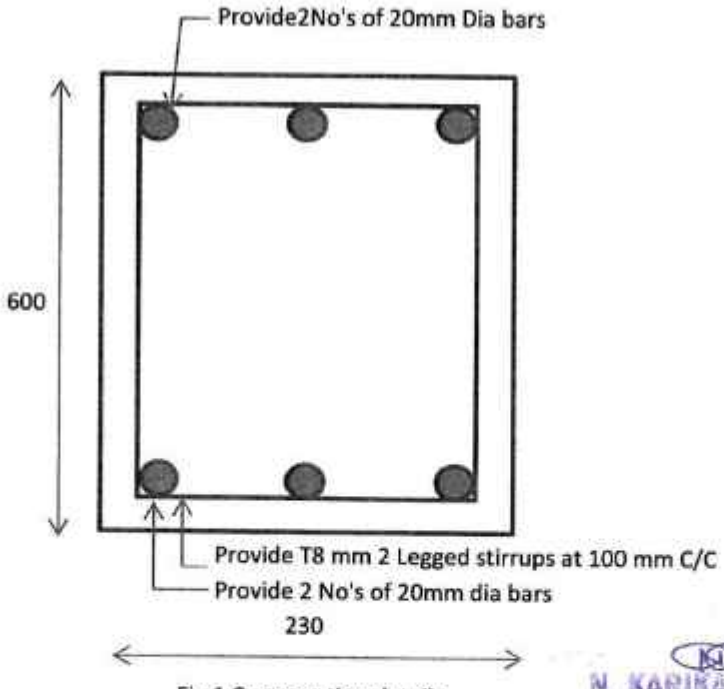


Fig.1 Cross section details

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		O22001-C-CB-01-DC-0001					
TITLE:	DESIGN OF SLAB	DESIGNED		CHECKED		APPROVED	
CLIENT	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

INPUT DETAILS:-

Grade of Concrete	F_{CK}	=	M25	
Grade of Steel	F_y	=	Fe500	
Cover to Main Reinforcement		=	20	mm
Thickness of the Slab		=	125	mm
Floor finishing thickness		=	75	
Assumed Dia of main Reinforcement		=	10	mm
Modulus of Elasticity OF steel	E_s	=	200000	N/mm^2
Support Condition		=	Fixed	
Creep Coefficient	θ	=	1.6	
crack width Distance	g	=	50	mm
Environment Condition		=	Moderate	

SIZES OF PANEL:-

Clear Span	L_y	=	3.020	m	(Longer Span)
	L_x	=	3.500	m	(Shorter Span)
Effective depth	dx	=	100	mm	= (125-20-(0.5x10)
	dy	=	90	mm	= (100-10)
Effective Span (As per IS456:2000 Cl 22.2 Pg34)	L_y	=	3.11	m	= (3.02+90)
	L_x	=	3.6	m	= (3.5+100)

SPAN RATIO:-

$$r = \frac{Ly}{Lx} = 1 < 2$$

Hence The Slab is Two Way Slab

LOADS ON SLAB:-

Self weight of Slab	= (0.125x25)	=	3.125	N/m^2
Floor Finish	= (0.075x24)	=	1.8	N/m^2
Super imposed dead load	=	=	2	
Live Load		=	5	N/m^2
Factored total load		w =	17.888	N/m^2
Boundary Condition		Two adjacent edges discontinuous		

REINFORCEMENT CALCULATION:

Moment Cofficient				Bending Moment in kN-m			
Short Span		Long Span		Short Span		Long Span	
α_x		α_y		$M_x = \alpha_x w l_x^2$		$My = \alpha_y w l_y^2$	
Support	Span	Support	Span	Support	Span	Support	Span
0.047	0.035	0.047	0.035	10.9	8.12	8.13	6.0553

CHECK FOR DEPTH:- b = 1000 mm

Mumax = 0.133Fck*bd^2

= 0.133 x 25 x 1000 x 100^2

dreq = 57.260 mm < $d_{provided}$ d = 100 mm

% of Reinforcement

% of steel , $P_t = f_ck/(2f_y)\,[1-\sqrt{((1-4.598M}$

Short Span	
Support	Span
$\frac{25}{2x500}\left(1-\sqrt{\frac{1-4.598x10.9}{25x1000x100^2}}\right)$ = 0.2646	$\frac{M25}{2x500}\left(1-\sqrt{\frac{1-4.598x8.12}{25x1000x100^2}}\right)$ = 0.194

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Long Span

Support	Span
25 / 2x500 (1- √(1-4.598x8.13 / 25x1000x90^2)) = 0.2425	M25 / 2x500 (1- √(1-4.598x6.055339106 / 25x1000x90^2)) = 0.17822

Ast required

Short Span				Long Span			
Support		Span		Support		Span	
0.26/100x1000x100		0.194/100x1000x100		0.24/100x1000x90		0.178/100x1000x90	
=	264.59	=	194	=	218.2626892	=	160.3974613

Design Required

Design Result along short Span					
At support	Provide	Y-	10	290	mm c/c
At span	Provide	Y-	10	400	mm c/c

Design Result along long Span					
At support	Provide	Y-	10	350	mm c/c
At span	Provide	Y-	10	480	mm c/c

Design Provided

Design Result along short Span					
At support	Provide	Y-	10	100	mm c/c
At span	Provide	Y-	10	100	mm c/c

Design Result along long Span					
At support	Provide	Y-	10	100	mm c/c
At span	Provide	Y-	10	100	mm c/c

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CHECK FOR DEFLECTION CONTROL:-

$A_{stprovided}$ = Spacing/3.14d^2

Short Span				Long Span			
Support		Span		Support		Span	
290/ 3.14 x 10 x 10		400/ 3.14 x 10 x 10		350/ 3.14 x 10 x 10		480/ 3.14 x 10 x 10	
=	785.4	=	785.4	=	785.4	=	785.398

Short Term Deflection

as per cl. C-2 p.no. 88 l_{eff} = $l_f/(1.2-M_r/M\ z/d\ (1-x/d)\ b_w/b)$

Reinforcement ratio	p	=	Ast/bd		
		=	785.4 / 1000 x 100	=	0.008
Modulus of elasticity	Ec	=	5000 √fck		
		=	5000 x √25	=	25000 KN/mm²
Modulur ratio	m	=	Es/Ec		
		=	200000 / 25000	=	8
Neutral axis factor	k	=	√(2pm+(pm)²)-pm		
		=	√(2x0.008x8) + (0.008x8)^2) - (0.008x8)	=	0.299
Lever arm factor	j	=	1-(k/3)		
		=	1-(0.299/3)	=	0.90
Lever arm	z	=	j d = 0.9x100	=	90
Neutral axis depth	x	=	k d = 0.299x100	=	29.9

PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REV	0
		O22001-C-CB-01-DC-0001					
TITLE:	DESIGN OF SLAB	DESIGNED		CHECKED		APPROVED	
CLIENT	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

Second moment of the area of the cracked section

I_{cr} = $(b\,(kd)^3/3) + (m\,Ast\,(d-kd)^2)$

$= \frac{1000 \times 0.299 \times 100\text{^}3}{3} + 8 \times 785.4 \times (100-0.299\times100)\text{^}2 = 39786007$ mm^4

Gross Moment of Inertia I_{gr} = $bD^3/12$

= 1000 x 100^3/12 = 83333333.33 mm^4

factor η = $z/d\,(1 - x/d)\,(bw/b)$ = 0.6305

Modulus of Rupture f_{cr} = $0.7\sqrt{fck}$ N/mm^2

= 0.7 x SQRT25 = 3.5

Distance from centroidal axis of gross section y_t = D/2 = 62.5 mm

Cracking Moment Mcr = $f_{cr} I_{gr}/y_t$ = 1

= $\frac{3.5 \times 83333333.33}{62.5}$ = 4.67 kNm/m

Effective second moment of area I_{eff} = $Icr/(1.2-(Mcr/M)\eta)$

= $\frac{39786007.3}{/10.9\times0.6305))}$ = 42786709 mm^4

Icr < Ieff < Igr The section is safe | 39786007.3 < 42786709 < 83333333.33 |

Short term deflection due to total load

$\delta_{D+L} = \frac{1\ M\ L^2}{384\ Ec\ Ieff}$

$= \frac{1 \times 14.4174740625 \times 3.5\text{^}2}{48 \times 25000 \times 42786709}$ = 0.5 mm

Short term deflection due to dead load alone

$\delta_D = \frac{5\ M_D\ L^2}{48\ Ec\ Ieff}$

$= \frac{1\text{x}\ 5.58 \times 3.5\text{^}2}{384\text{X}25000 \times 42786709}$ = 0.25 mm

Short term deflection due to live load alone $\delta_L = \delta_{D+L} - \delta_D$ = 0.20 mm

Deflection due to shrinkage: as per cl. C-3 p.no. 88

Ultimate shrinkage strain in cor Ecs from cl. 6.2.4 p.n 16 = 0.0003

Shrinkage constant k_4 from cl. C-3.1 p.n 88 = 0.064

Shrinkage constant k_3 = 0.125

Deflection due to shrinkage δsh = $k_3\,k_4\,Ecs\,L^2/D$

$= \frac{0.125\text{x}0.06 \times 0.0003\text{x}(3.6\text{x}1000)\text{^}2}{125.0}$ = 0.3 mm

Deflection due to creep: as per cl. C-4 p.no. 88

Modulus of elasticity due to creep, Ece = $Ec/(1+\theta)$

= 25000/(1+1.6) = 9615.4 N/mm^2

modulur ratio m = Es/Ece

= 200000/9615.38 = 20.8

Neutral axis factor k = $\sqrt{(2pm+(pm)^2)}-pm$

= $\sqrt{2\text{x}0.008\text{x}20.8)+(0.008\text{x}20.8)\text{^}2)}-(0.008\text{x}20.8)$ = 0.434

Lever arm factor = 1-(k/3)

j = 1-(0.434/3) = 0.855

Lever arm = j d = 85.5 mm

Neutral axis depth z = k d = 43.4 mm

Second moment of the area of the cracked section x

I_{cr} = $(b\,(kd)^3/3) + (m\,Ast\,(d-kd)^2)$

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REV	0
		O22001-C-CB-01-DC-0001					
TITLE:	DESIGN OF SLAB	DESIGNED		CHECKED		APPROVED	
CLIENT	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

$= \frac{(1000x(0.434x100)^3}{3} + 20.8x10.9x(100-0.434x100)^2 = 79583216$ mm^4

$\eta_{factor} = z/d(1-x/d)(bw/b)$

$= (85.5/100) \times (1-43.4/100) = 0.484$

Effective second moment of area, $I_{eff\ D+L} = Icr/(1.2-(Mcr/M)\eta)$

$= \frac{79583216}{(1.2-(4.67/10.9X0.484)} = 66319355$ mm^4

Icr and Ieff are less than Igr, Hence it is ok

Initial plus creep deflection due to permenent loads

$a_{i,cc} = (1/384)(M_D L^2/(EcI_{effD+L}))$

$= \frac{1}{384} \times \frac{5.58160770833333x10^6x3.6}{(3.5x1000)^2/(9615.38x66319355.3)} = 0.44$ mm

Short term deflection due to permanent loading

$a_i = (W_{DL}/W)\delta_{(D+L)}$

$= (4.925 / 17.8875) \times 0.2 = 0.2613$ mm

Deflection due to creep $= a_{cc} = a_{i,cc} - a_i = 0.18$ mm

$= L/250 = 14.4$ mm

Permissible final deflection due to all loads

Permissible deflection that occurs after construction

L/350 or 20mm which ever is less = 10.29 mm

Actual deflection due to all loads $= \delta_{D+L}+\delta_{sh}+a_{cc}$ = 0.88 mm

Actual deflection is less than Permissible deflection hence safe

Actual deflection that occurs after constructi $= \delta_L+\delta_{sh}+a_{cc}$ = 0.63 mm

Actual deflection is less than Permissible deflection hence safe

CHECK FOR CRACK WIDTH:-

$W_{cr} = \frac{3a_{cr} \times \varepsilon_m}{1 + \frac{2(a_{cr}-C_{min})}{h-a}}$

$a_{cr} = [[0.5S]^2 + C^2_{min}]^{1/2}$

Spacing of Ba = 20

$a_{cr} = 23$

$\varepsilon_m = \varepsilon_1 - \frac{b(h-a)(g-a)}{3E_s A_s (d-a)}$

$\varepsilon_1 = \frac{Fs}{Es} \times \frac{(h-a)}{(g-a)}$

$F_s = \frac{m \times Moment\ (d-a)}{I_r}$

$\frac{b\,a^2}{2} = m\ Ast\ (d-a)$

$a^2 = \frac{2m\ Ast\ d - 2m\ Ast\ a}{b}$

$= \frac{2 \times 20.8 \times 785.4 \times 100 - 220.8 \times 785.4a}{1000}$

$= \frac{653452.8 - 11007\ a}{1000}$

$a = 1$

$F_s = \frac{20.8 \times 10.9 \times (100-1)}{79583216} = 216.98$ N/mm^2

$\varepsilon_1 = \frac{216.98}{2.00E+05} \times \frac{(125-1)}{(50-1)} = 0.0028$

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$$\varepsilon_m = 0.0028 - \frac{1000 \times (125-1) \times (50-1)}{3 \times 200000 \times 785.4 \times (100-1)} = 0.00264$$

$$W_{cr} = \frac{3 \times 23 \times 0.00264}{1 + \frac{2 \times (23-20)}{125-1}} = 0.1345$$

Limiting Crack width = 0.200

safe

CHECK FOR SHEAR:-

Average effective depth = 95 mm

$V_U = W_U(0.5 \times L_x - d_{avg})$

$= 17.89 \times (0.5 \times 3.6 - (95 / 1000))$

$= 31$ kN

$\tau_v = V_u/bd = 0.32$

$(Pt)_{Provided} = 0.26$

Sp-24-126

$\beta = 0.8f_{ck}/6.89p_t$ (but Sould not less than 1)

$\beta = \frac{0.8 \times 25}{6.89 \times 0.26}$

$\beta = 11$

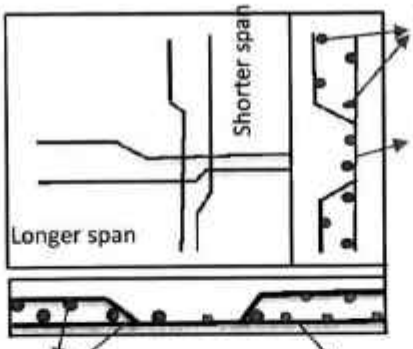
$\tau_c = 0.85*sqrt(0.8*fck)*(sqrt(1+5\beta)-1)/6\beta$

$= 0.85 \times SQRT(0.8x25) \times (SQRT(1+5x)-1)/6x11.16$

$= 1.19$ Safe

REINFORCEMENT DETAILS:-

Two way slab



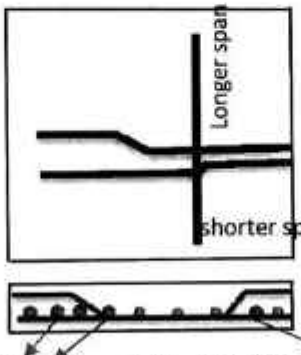
Provide 10 mm ϕ 290 mm c/c

Provide 10 mm ϕ 400 mm c/c

Provide 10 mm ϕ 350 mm c/c

Provide 10 mm ϕ 480 mm c/c

One way slab



Provide 10 mm ϕ 290 mm c/c

Provide 10 mm ϕ 350 mm c/c

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REVISION	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF STAIRCASE	DESIGNED		CHECKED		APPROVED	
CLIENT	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

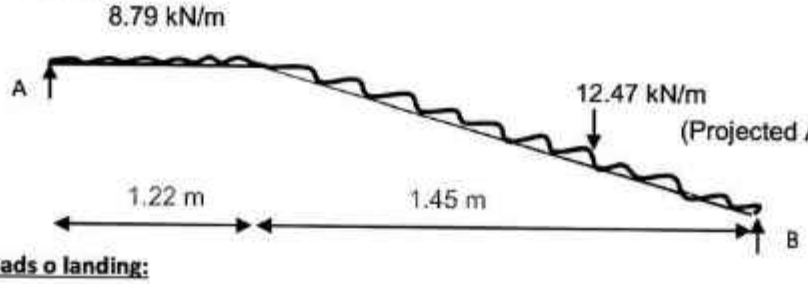
Input:

Unit Weight of concrete(RCC)		=	25	KN/m^3
Unit Weight of concrete(PCC)		=	24	KN/m^3
Unit Weight of Masonary work		=	20	KN/m^3
Tensile strength of steel		=	500	N/mm^2
Compressive strength of concrete		=	20	N/mm^2
Main bar Dia		=	12	mm
Distribution bar Dia		=	8	mm

Stair Case Details:

Tread		=	0.30	m
Riser		=	0.18	m
Thickness of slab		=	0.125	m
Efeective depth of slab	= 0.125 - 0.025 - 0.006	=	0.094	m
Thickness of finishing		=	0.05	
Clear cover		=	25	mm
X_{umax}/d		=	0.460	

A **Flight 1 Design:**



Loads o landing:

i)Dead load:

Self weight		=	3.75	kN/m^2
Finishing load		=	1.44	kN/m^2
Total load per m run	3.75 + 1.44	=	5.19	kN/m^2

ii)Live load:

building class		=	3	kN/m^2

IS 875 Part 2 Table 2 (Corridors, passages, lobbies, staircases)

UnFactored Loads on Landing slab		=	**8.79**	kN/m

Loads on Going (or) Horizontal portion on Projected Plan Area

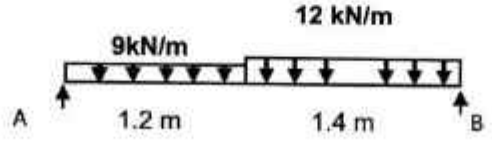
i)Dead load:

Self weight		=	3.618	kN/m^2
Finishing load		=	1.583	kN/m^2
Load due to Steps		=	2.188	kN/m^2
Total load per m run		=	**8.866**	kN/m^2

ii)Live load:

building class		=	3	kN/m^2
Unactored Loads on Going on Projected Plan Area		=	**12.47**	kN/m^2

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REVISION	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF STAIRCASE	DESIGNED		CHECKED		APPROVED	
CLIENT	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

As per Clause 33.1 (b) of IS : 456 -2000 , the effective span of stair is taken as = 2.67 m

Reaction at Support B = $\frac{+12.47x1.44817073170732x(0.7240853}{2.67}$ = 15.60 kN

Reaction at Support B = 13.17 kN

Shear Force is zero at = 15.6-12.47X = 0

X = 1.25 From B

Maximum B.M = (15.6x1.25) -12.47x(1.25)^2/2 = 9.76 kN-m

Design Bending Moment = = 14.65 kN-m

Check For Depth

d_{lim} = 74.0 mm

$d_{provided}$ = 94.0 mm

= SAFE

Reinforcement Details

$R=M_u/bd^2$ = 1.658

$(Pt)_{Required}$ = **0.427** %

Area of steel required ($A_{st)\ Required}$= = 401 mm^2

a_{st} = 113 mm^2

Spacing req = 282.149 mm

Spacing $_{pro}$ = 150 mm

Provide12mm bar at 150mm C/C

$(Pt)_{Provided}$ = = **0.802**

SAFE

Min steel required = **150** mm^2

Provide8mm bar at 325mm C/C

Check For Shear

24-126 $(Pt)_{Provided}$ = = 0.802

$\beta=0.8f_{ck}/6.89p_t$(but Sould not less than 1)

β = 2.895

Tc=0.85*sqrt(0.8*fck)*(sqrt(1+5β)-1)/6β Tc = 0.574 Nmm^2

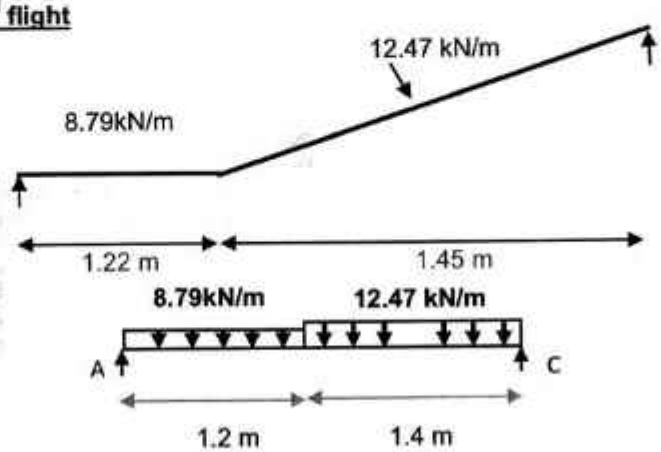
Design Shear Force (V_u) = **21.65** kN

T_v = 0.230 Nmm^2

Tv < Tc

SAFE

Design of 2nd flight



As per Clause 33.1 (b) of IS : 456 -2000 , the effective span of stair is taken as

= 1.21951219512195+1.44817073170732 = 2.67 m

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD,KORATTUR VILLAGE	DOCUMENT NO				REVISION	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF STAIRCASE	DESIGNED		CHECKED		APPROVED	
CLIENT	CLIENT : M/S BUILT TECH ASSOCIATES	RAKE		NKK		NKK	
		DATE	08.05.2023	DATE	08.05.2023	DATE	08.05.2023

	Reaction at Support C = $\frac{2/2)+12.47x1.44817073170732x(0.72408536585}{2.67}$	=	**15.60**	kN	
	Reaction at Support A =	=	**13.17**	kN	
	Shear Force is zero at = 15.6-12.47X	=	0		
		X =	1.25 From B		
	Maximum B.M = (15.6x1.25) -12.47x(1.25)^2/2	=	9.76	kN-m	
	Design Bending Moment	=	**14.65**		
	Check For Depth				
	d lim=	=	74.04	mm	
	d provided	=	94.00	mm	
			SAFE		
	Reinforcement Details				
	$R=M_u/bd^2$	=	1.658		
	$(Pt)_{Required}$	=	**0.427**	%	
	Area of steel required ($A_{st)\ Required}$=	=	401	mm^2	
	a_{st}	=	113	mm^2	
	Spacing req	=	282	mm	
	$Spacing_{pro}$	=	150	mm	
	Provide12mm bar at 150mm C/C				
	$(Pt)_{Provided}$ =	=	**0.802**		
			SAFE		
	Distribution steel				
	Min steel required	=	**150**	mm^2	
	Provide8mm bar at 325mm C/C				
	Check For Shear				
24-126	$(Pt)_{Provided}$ =	=	0.802		
	$\beta=0.8f_{ck}/6.89p_t$(but Sould not less than 1)				
		β =	1.000		
	Tc=0.85*sqrt(0.8*fck)*(sqrt(1+5β)-1)/6β	Tc =	0.821	Nmm^2	
	Design Shear Force (V_u)	=	**21.65**	kN	
	T_v	=	0.230	Nmm^2	
	Tv < Tc		**SAFE**		

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PROJECT:	RESIDENTIAL BUILDING AT WATERCANAL ROAD
TITLE:	RC WALL DESIGN
CLIENT	M/S BUILT TECH ASSOCIATES

DOCUMENT NO				REV	
021051-C-RB-01-DC-0001					
DESIGNED		CHECKED		APPROVED	
CPA		NKK		NKK	
DATE	08.05.23	DATE	08.05.23	DATE	08.05.23

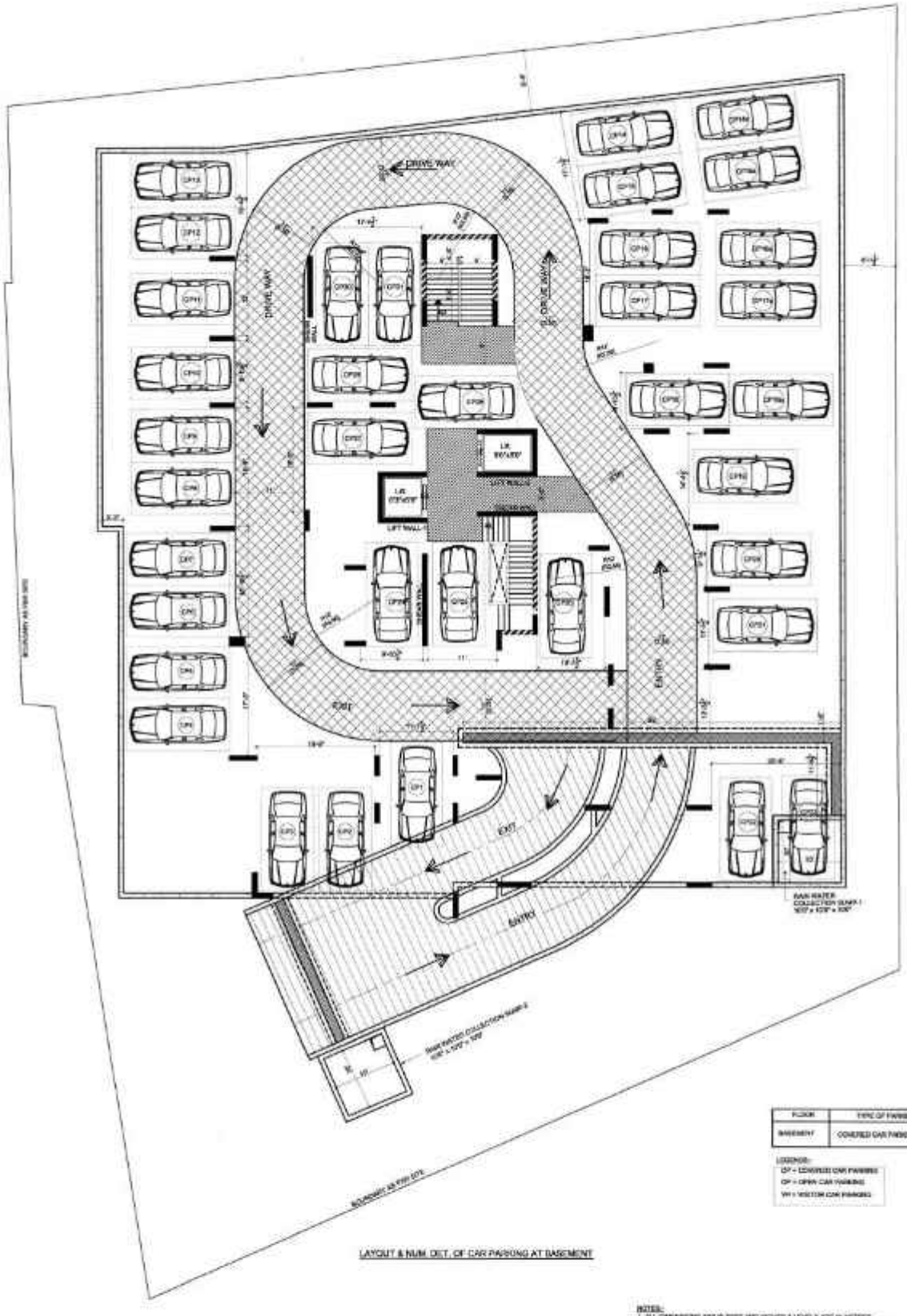
Sl.No	Wall ID	Load Case	Height of the wall Le	Thickness of wall h	Characteristic Strength of concrete fck	Yield strength of the steel fy	Width of the wall b	Effective height of the wall Le	Clear Cover to Reinforcement c	Assume Diameter of Main Reinforcement d	Effective thickness of wall h_e	Le/ k = 0.75Le / h	Check	Factored Bending Moment M	Factored Axial Force P	Factored Shear Force V	Minimum accidental eccentricity h/20	20	e_{min}	Eccentricity due to Bending Moment e_x	Additional eccentricity e_a	Total Eccentricity e_t	Total Bending Moment M_t	d'	d'/h	$P/f_{ck}bh$	$M/f_{ck}bh^2$	p/f_{ck}	A_{st}	% of Steel	Design Status
			mm	mm	N/mm2	N/mm2	mm	mm	mm	mm	mm			kNm	kN	kN	mm	mm	mm	mm	mm	mm	kNm	mm					mm2	%	
1	W1A	201	3000	230	25	500	1300	2250	40	20	180	9.78	ok	9	5354	559	11.5	20	11.5	1.78	11.01	12.77	68.3	50	0.2174	0.79	0.06	0.12	8100	2.35	Design ok
2		202	3000	230	25	500	1300	2250	40	20	180	9.78	ok	11	4275	435	11.5	20	11.5	2.50	11.01	13.51	57.7	50	0.2174	0.63	0.05	0.12	8100	2.35	Design ok
3		203	3000	230	25	500	1300	2250	40	20	180	9.78	ok	11	4275	435	11.5	20	11.5	2.50	11.01	13.51	57.7	50	0.2174	0.63	0.05	0.12	8100	2.35	Design ok
4		204	3000	230	25	500	1300	2250	40	20	180	9.78	ok	34	4271	445	11.5	20	11.5	8.04	11.01	19.04	81.3	50	0.2174	0.83	0.07	0.12	8100	2.35	Design ok
5		205	3000	230	25	500	1300	2250	40	20	180	9.78	ok	34	4271	445	11.5	20	11.5	8.04	11.01	19.04	81.3	50	0.2174	0.83	0.07	0.12	8100	2.35	Design ok
6		206	3000	230	25	500	1300	2250	40	20	180	9.78	ok	10	2480	248	11.5	20	11.5	3.94	11.01	14.95	37.1	50	0.2174	0.37	0.03	0.12	8100	2.35	Design ok
7		207	3000	230	25	500	1300	2250	40	20	180	9.78	ok	10	2480	248	11.5	20	11.5	3.94	11.01	14.95	37.1	50	0.2174	0.37	0.03	0.12	8100	2.35	Design ok
8		208	3000	230	25	500	1300	2250	40	20	180	9.78	ok	33	2476	250	11.5	20	11.5	12.49	11.01	24.50	60.7	50	0.2174	0.37	0.05	0.12	8100	2.35	Design ok
9		209	3000	230	25	500	1300	2250	40	20	180	9.78	ok	33	2476	250	11.5	20	11.5	12.49	11.01	24.50	60.7	50	0.2174	0.37	0.05	0.12	8100	2.35	Design ok
10		210	3000	230	25	500	1300	2250	40	20	180	9.78	ok	14	4088	418	11.5	20	11.5	3.48	11.01	14.50	59.3	50	0.2174	0.61	0.05	0.12	8100	2.35	Design ok
11		211	3000	230	25	500	1300	2250	40	20	180	9.78	ok	14	4088	418	11.5	20	11.5	3.48	11.01	14.50	59.3	50	0.2174	0.61	0.05	0.12	8100	2.35	Design ok
12		212	3000	230	25	500	1300	2250	40	20	180	9.78	ok	33	4085	424	11.5	20	11.5	8.12	11.01	19.63	78.1	50	0.2174	0.61	0.06	0.12	8100	2.35	Design ok
13		213	3000	230	25	500	1300	2250	40	20	180	9.78	ok	33	4085	424	11.5	20	11.5	8.12	11.01	19.63	78.1	50	0.2174	0.61	0.06	0.12	8100	2.35	Design ok
14		214	3000	230	25	500	1300	2250	40	20	180	9.78	ok	1	4431	460	11.5	20	11.5	0.02	11.01	11.30	51.8	50	0.2174	0.66	0.04	0.12	8100	2.35	Design ok
15		215	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-3	4528	472	11.5	20	11.5	-0.71	11.01	11.72	53.1	50	0.2174	0.67	0.04	0.12	8100	2.35	Design ok
16		216	3000	230	25	500	1300	2250	40	20	180	9.78	ok	2	4443	462	11.5	20	11.5	0.55	11.01	11.56	51.9	50	0.2174	0.66	0.04	0.12	8100	2.35	Design ok
17		217	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-3	4529	472	11.5	20	11.5	-0.72	11.01	11.72	53.1	50	0.2174	0.67	0.04	0.12	8100	2.35	Design ok
18		218	3000	230	25	500	1300	2250	40	20	180	9.78	ok	1	2636	273	11.5	20	11.5	0.19	11.01	11.20	30.3	50	0.2174	0.39	0.02	0.12	8100	2.35	Design ok
19		219	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-4	2733	285	11.5	20	11.5	-1.52	11.01	12.53	34.2	50	0.2174	0.40	0.03	0.12	8100	2.35	Design ok
20		220	3000	230	25	500	1300	2250	40	20	180	9.78	ok	2	2648	275	11.5	20	11.5	0.58	11.01	11.59	30.7	50	0.2174	0.39	0.03	0.12	8100	2.35	Design ok
21		221	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-4	2734	285	11.5	20	11.5	-1.52	11.01	12.53	34.3	50	0.2174	0.41	0.03	0.12	8100	2.35	Design ok
22		222	3000	230	25	500	1300	2250	40	20	180	9.78	ok	7	4212	438	11.5	20	11.5	1.63	11.01	12.64	53.2	50	0.2174	0.82	0.04	0.12	8100	2.35	Design ok
23		223	3000	230	25	500	1300	2250	40	20	180	9.78	ok	3	4293	448	11.5	20	11.5	0.73	11.01	11.74	50.6	50	0.2174	0.64	0.04	0.12	8100	2.35	Design ok
24		224	3000	230	25	500	1300	2250	40	20	180	9.78	ok	8	4222	440	11.5	20	11.5	1.82	11.01	12.83	54.5	50	0.2174	0.83	0.04	0.12	8100	2.35	Design ok
25		225	3000	230	25	500	1300	2250	40	20	180	9.78	ok	3	4291	448	11.5	20	11.5	0.73	11.01	11.74	50.4	50	0.2174	0.64	0.04	0.12	8100	2.35	Design ok
26	W1B	301	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-26.4	4833	102	11.5	20	11.5	-5.47	11.01	16.48	79.6	50	0.2174	0.72	0.07	0.1	6750	1.96	Design ok
27		202	3000	230	25	500	1300	2250	40	20	180	9.78	ok	1.6	3666	438	11.5	20	11.5	0.22	11.01	11.52	35.3	50	0.2174	0.45	0.03	0.1	6750	1.96	Design ok
28		203	3000	230	25	500	1300	2250	40	20	180	9.78	ok	1.6	3666	438	11.5	20	11.5	0.22	11.01	11.52	35.3	50	0.2174	0.45	0.03	0.1	6750	1.96	Design ok
29		204	3000	230	25	500	1300	2250	40	20	180	9.78	ok	52.9	3754	165	11.5	20	11.5	14.09	11.01	25.10	94.2	50	0.2174	0.58	0.08	0.1	6750	1.96	Design ok
30		205	3000	230	25	500	1300	2250	40	20	180	9.78	ok	52.9	3754	165	11.5	20	11.5	14.09	11.01	25.10	94.2	50	0.2174	0.58	0.08	0.1	6750	1.96	Design ok
31		206	3000	230	25	500	1300	2250	40	20	180	9.78	ok	7.4	1387	433	11.5	20	11.5	5.32	11.01	16.32	22.6	50	0.2174	0.21	0.02	0.1	6750	1.96	Design ok
32		207	3000	230	25	500	1300	2250	40	20	180	9.78	ok	7.4	1387	433	11.5	20	11.5	5.32	11.01	16.32	22.6	50	0.2174	0.21	0.02	0.1	6750	1.96	Design ok
33		208	3000	230	25	500	1300	2250	40	20	180	9.78	ok	58.7	3075	139	11.5	20	11.5	28.29	11.01	39.29	81.3	50	0.2174	0.31	0.07	0.1	6750	1.96	Design ok
34		209	3000	230	25	500	1300	2250	40	20	180	9.78	ok	58.7	3075	139	11.5	20	11.5	28.29	11.01	39.29	81.3	50	0.2174	0.31	0.07	0.1	6750	1.96	Design ok
35		210	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-8.3	2933	487	11.5	20	11.5	-2.82	11.01	13.82	40.6	50	0.2174	0.43	0.03	0.1	6750	1.96	Design ok
36		211	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-8.3	2933	487	11.5	20	11.5	-2.82	11.01	13.83	40.6	50	0.2174	0.43	0.03	0.1	6750	1.96	Design ok
37		212	3000	230	25	500	1300	2250	40	20	180	9.78	ok	32.8	3488	268	11.5	20	11.5	9.41	11.01	20.41	71.1	50	0.2174	0.52	0.06	0.1	6750	1.96	Design ok
38		213	3000	230	25	500	1300	2250	40	20	180	9.78	ok	32.8	3488	268	11.5	20	11.5	9.41	11.01	20.41	71.1	50	0.2174	0.52	0.06	0.1	6750	1.96	Design ok
39		214	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-15.3	4279	124	11.5	20	11.5	-1.57	11.01	14.57	62.3	50	0.2174	0.63	0.05	0.1	6750	1.96	Design ok
40		215	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-24.0	4233	10	11.5	20	11.5	-5.68	11.01	16.68	70.6	50	0.2174	0.63	0.06	0.1	6750	1.96	Design ok
41		216	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-13.7	4235	73	11.5	20	11.5	-3.23	11.01	14.24	60.3	50	0.2174	0.63	0.05	0.1	6750	1.96	Design ok
42		217	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-24.2	4233	10	11.5	20	11.5	-5.71	11.01	16.72	70.8	50	0.2174	0.63	0.06	0.1	6750	1.96	Design ok
43		218	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-9.5	2680	118	11.5	20	11.5	-2.64	11.01	14.65	38.1	50	0.2174	0.39	0.03	0.1	6750	1.96	Design ok
44		219	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-18.2	3533	4	11.5	20	11.5	-7.14	11.01	18.15	66.3	50	0.2174	0.38	0.04	0.1	6750	1.96	Design ok
45		220	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-7.9	2555	68	11.5	20	11.5	-3.09	11.01	14.09	36.0	50	0.2174	0.38	0.03	0.1	6750	1.96	Design ok
46		221	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-18.4	3554	6	11.5	20	11.5	-7.20	11.01	18.21	66.5	50	0.2174	0.38	0.04	0.1	6750	1.96	Design ok
47		222	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-21.7	3903	233	11.5	20	11.5	-5.57	11.01	16.57	64.7	50	0.2174	0.52	0.06	0.1	6750	1.96	Design ok
48		223	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-28.8	3969	144	11.5	20	11.5	-7.43	11.01	18.44	71.5	50	0.2174	0.52	0.08	0.1	6750	1.96	Design ok
49		224	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-20.5	3870	194	11.5	20	11.5	-5.29	11.01	16.30	63.1	50	0.2174	0.52	0.05	0.1	6750	1.96	Design ok
50		225	3000	230	25	500	1300	2250	40	20	180	9.78	ok	-28.9	3868	144	11.5	20	11.5	-7.42	11.01	18.43	71.5	50	0.2174	0.52	0.08	0.1	6750	1.96	Design ok

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PROJECT:	RESIDENTIAL BUILDING AT WATERCANAL ROAD	DOCUMENT NO		REV	
		023051-C-RB-01-DC-0001			
TITLE:	RC WALL DESIGN	DESIGNED	CHECKED	APPROVED	
		CPA	NKK	NKK	
CLIENT	M/S BUILT TECH ASSOCIATES	DATE 08.05.23	DATE 08.05.23	DATE 08.05.23	

51		201	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-26.4	4833	182	11.5	20	11.5	-5.43	11.01	16.48	79.6	50	0.2174	0.72	0.07	0.1	6750	1.96	Design ok
52		202	3000	230	25	500	1500	2250	40	20	180	9.78	ok	1.6	3066	438	11.5	20	11.5	0.32	11.01	11.52	55.3	50	0.2174	0.49	0.03	0.1	6750	1.96	Design ok
53		203	3000	230	25	500	1500	2250	40	20	180	9.78	ok	1.6	3066	438	11.5	20	11.5	0.32	11.01	11.52	55.3	50	0.2174	0.49	0.03	0.1	6750	1.96	Design ok
54		204	3000	230	25	500	1500	2250	40	20	180	9.78	ok	52.9	3754	165	11.5	20	11.5	14.08	11.01	25.10	94.2	50	0.2174	0.56	0.08	0.1	6750	1.96	Design ok
55		205	3000	230	25	500	1500	2250	40	20	180	9.78	ok	52.9	3754	165	11.5	20	11.5	14.08	11.01	25.10	94.2	50	0.2174	0.56	0.08	0.1	6750	1.96	Design ok
56		206	3000	230	25	500	1500	2250	40	20	180	9.78	ok	7.4	1387	433	11.5	20	11.5	5.32	11.01	16.32	22.6	50	0.2174	0.21	0.02	0.1	6750	1.96	Design ok
57		207	3000	230	25	500	1500	2250	40	20	180	9.78	ok	7.4	1387	433	11.5	20	11.5	5.32	11.01	16.32	22.6	50	0.2174	0.21	0.02	0.1	6750	1.96	Design ok
58		208	3000	230	25	500	1500	2250	40	20	180	9.78	ok	58.7	3075	159	11.5	20	11.5	28.29	11.01	39.29	83.5	50	0.2174	0.31	0.07	0.1	6750	1.96	Design ok
59		209	3000	230	25	500	1500	2250	40	20	180	9.78	ok	58.7	3075	159	11.5	20	11.5	28.29	11.01	39.29	83.5	50	0.2174	0.31	0.07	0.1	6750	1.96	Design ok
60		210	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-8.3	2935	487	11.5	20	11.5	-2.82	11.01	13.82	40.6	50	0.2174	0.43	0.03	0.1	6750	1.96	Design ok
61		211	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-8.3	2935	487	11.5	20	11.5	-2.82	11.01	13.82	40.6	50	0.2174	0.43	0.03	0.1	6750	1.96	Design ok
62		212	3000	230	25	500	1500	2250	40	20	180	9.78	ok	32.8	3485	208	11.5	20	11.5	9.41	11.01	20.41	71.1	50	0.2174	0.52	0.06	0.1	6750	1.96	Design ok
63	W1C	213	3000	230	25	500	1500	2250	40	20	180	9.78	ok	32.8	3485	208	11.5	20	11.5	9.41	11.01	20.41	71.1	50	0.2174	0.52	0.06	0.1	6750	1.96	Design ok
64		214	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-15.3	4279	124	11.5	20	11.5	-3.57	11.01	14.57	62.3	50	0.2174	0.63	0.05	0.1	6750	1.96	Design ok
65		215	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-24.0	4233	10	11.5	20	11.5	-5.53	11.01	14.24	60.3	50	0.2174	0.63	0.05	0.1	6750	1.96	Design ok
66		216	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-13.7	4233	73	11.5	20	11.5	-3.23	11.01	14.24	60.3	50	0.2174	0.63	0.05	0.1	6750	1.96	Design ok
67		217	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-24.2	4223	10	11.5	20	11.5	-5.71	11.01	16.72	70.8	50	0.2174	0.63	0.06	0.1	6750	1.96	Design ok
68		218	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-9.5	2400	118	11.5	20	11.5	-2.64	11.01	14.65	38.1	50	0.2174	0.39	0.03	0.1	6750	1.96	Design ok
69		213	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-18.2	2553	4	11.5	20	11.5	-7.14	11.01	18.15	46.5	50	0.2174	0.38	0.04	0.1	6750	1.96	Design ok
70		220	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-7.9	2555	68	11.5	20	11.5	-3.09	11.01	14.09	36.0	50	0.2174	0.38	0.03	0.1	6750	1.96	Design ok
71		221	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-18.4	2554	4	11.5	20	11.5	-7.20	11.01	18.21	46.5	50	0.2174	0.38	0.04	0.1	6750	1.96	Design ok
72		222	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-21.7	3005	235	11.5	20	11.5	-5.57	11.01	16.57	64.7	50	0.2174	0.57	0.06	0.1	6750	1.96	Design ok
73		223	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-28.8	3868	144	11.5	20	11.5	-7.43	11.01	18.44	71.3	50	0.2174	0.57	0.06	0.1	6750	1.96	Design ok
74		224	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-20.5	3870	194	11.5	20	11.5	-5.29	11.01	16.30	63.1	50	0.2174	0.57	0.05	0.1	6750	1.96	Design ok
75		225	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-28.9	3868	144	11.5	20	11.5	-7.45	11.01	18.47	71.5	50	0.2174	0.57	0.06	0.1	6750	1.96	Design ok
76		201	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-26.4	4833	182	11.5	20	11.5	-5.43	11.01	16.48	79.6	50	0.2174	0.72	0.07	0.1	6750	1.96	Design ok
77		202	3000	230	25	500	1500	2250	40	20	180	9.78	ok	1.6	3066	438	11.5	20	11.5	0.32	11.01	11.52	55.3	50	0.2174	0.49	0.03	0.1	6750	1.96	Design ok
78		203	3000	230	25	500	1500	2250	40	20	180	9.78	ok	1.6	3066	438	11.5	20	11.5	0.32	11.01	11.52	55.3	50	0.2174	0.49	0.03	0.1	6750	1.96	Design ok
79		204	3000	230	25	500	1500	2250	40	20	180	9.78	ok	52.9	3754	165	11.5	20	11.5	14.08	11.01	25.10	94.2	50	0.2174	0.56	0.08	0.1	6750	1.96	Design ok
80		205	3000	230	25	500	1500	2250	40	20	180	9.78	ok	52.9	3754	165	11.5	20	11.5	14.08	11.01	25.10	94.2	50	0.2174	0.56	0.08	0.1	6750	1.96	Design ok
81		206	3000	230	25	500	1500	2250	40	20	180	9.78	ok	7.4	1387	433	11.5	20	11.5	5.32	11.01	16.32	22.6	50	0.2174	0.21	0.02	0.1	6750	1.96	Design ok
82		207	3000	230	25	500	1500	2250	40	20	180	9.78	ok	7.4	1387	433	11.5	20	11.5	5.32	11.01	16.32	22.6	50	0.2174	0.21	0.02	0.1	6750	1.96	Design ok
83		208	3000	230	25	500	1500	2250	40	20	180	9.78	ok	58.7	3075	159	11.5	20	11.5	28.29	11.01	39.29	83.5	50	0.2174	0.31	0.07	0.1	6750	1.96	Design ok
84		209	3000	230	25	500	1500	2250	40	20	180	9.78	ok	58.7	3075	159	11.5	20	11.5	28.29	11.01	39.29	83.5	50	0.2174	0.31	0.07	0.1	6750	1.96	Design ok
85		210	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-8.3	2935	487	11.5	20	11.5	-2.82	11.01	13.82	40.6	50	0.2174	0.43	0.03	0.1	6750	1.96	Design ok
86		211	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-8.3	2935	487	11.5	20	11.5	-2.82	11.01	13.82	40.6	50	0.2174	0.43	0.03	0.1	6750	1.96	Design ok
87		212	3000	230	25	500	1500	2250	40	20	180	9.78	ok	32.8	3485	208	11.5	20	11.5	9.41	11.01	20.41	71.1	50	0.2174	0.52	0.06	0.1	6750	1.96	Design ok
88	W1D	213	3000	230	25	500	1500	2250	40	20	180	9.78	ok	32.8	3485	208	11.5	20	11.5	9.41	11.01	20.41	71.1	50	0.2174	0.52	0.06	0.1	6750	1.96	Design ok
89		214	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-15.3	4279	124	11.5	20	11.5	-3.57	11.01	14.57	62.3	50	0.2174	0.63	0.05	0.1	6750	1.96	Design ok
90		215	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-24.0	4233	10	11.5	20	11.5	-5.53	11.01	14.24	60.3	50	0.2174	0.63	0.05	0.1	6750	1.96	Design ok
91		216	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-13.7	4233	73	11.5	20	11.5	-3.23	11.01	14.24	60.3	50	0.2174	0.63	0.05	0.1	6750	1.96	Design ok
92		217	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-24.2	4223	10	11.5	20	11.5	-5.71	11.01	16.72	70.8	50	0.2174	0.63	0.06	0.1	6750	1.96	Design ok
93		218	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-9.5	2400	118	11.5	20	11.5	-2.64	11.01	14.65	38.1	50	0.2174	0.39	0.03	0.1	6750	1.96	Design ok
94		219	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-18.2	2553	4	11.5	20	11.5	-7.14	11.01	18.15	46.5	50	0.2174	0.38	0.04	0.1	6750	1.96	Design ok
95		220	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-7.9	2555	68	11.5	20	11.5	-3.09	11.01	14.09	36.0	50	0.2174	0.38	0.03	0.1	6750	1.96	Design ok
96		221	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-18.4	2554	4	11.5	20	11.5	-7.20	11.01	18.21	46.5	50	0.2174	0.38	0.04	0.1	6750	1.96	Design ok
97		222	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-21.7	3005	235	11.5	20	11.5	-5.57	11.01	16.57	64.7	50	0.2174	0.57	0.06	0.1	6750	1.96	Design ok
98		223	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-28.8	3868	144	11.5	20	11.5	-7.43	11.01	18.44	71.3	50	0.2174	0.57	0.06	0.1	6750	1.96	Design ok
99		224	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-20.5	3870	194	11.5	20	11.5	-5.29	11.01	16.30	63.1	50	0.2174	0.57	0.05	0.1	6750	1.96	Design ok
100		225	3000	230	25	500	1500	2250	40	20	180	9.78	ok	-28.9	3868	144	11.5	20	11.5	-7.45	11.01	18.47	71.5	50	0.2174	0.57	0.06	0.1	6750	1.96	Design ok

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FLOOR	TYPE OF PARKING	NOS
BASEMENT	COVERED CAR PARKING(CP)	51

LEGENDS:
CP = COVERED CAR PARKING
OP = OPEN CAR PARKING
VP = VISITOR CAR PARKING

LAYOUT & NUM. DET. OF CAR PARKING AT BASEMENT

NOTES:
1. ALL DIMENSIONS ARE IN FEET AND INCHES & LEVELS ARE IN METRES.

R	ISSUED FOR CONSTRUCTION	CPK	10.05.2023	CPK	10.05.2023	NKK	10.05.2023	NKK	10.05.2023
REV NO.	DESCRIPTION	DESIGNED		DRAWN		CHECKED		APPROVED	

SIVASANKARI MOBILE NO: 9894245632,
STRUCTURAL CONSULTANT E-MAIL ID: sivasankari.structures@gmail.com

CLIENT : M/S B.S.T TECH ASSOCIATES
CONSULTANT : SIVASANKARI, STRUCTURAL CONSULTANT

PROJECT: PROPOSED RESIDENTIAL BUILDING AT WATER CANAL ROAD ,CHENNAI.
CONCEPT ARCHITECT : SNA ASSOCIATES

SUPPLIER/ CONTRACTOR :

JOB No : 021051

	NAME	SIGN	DATE
DESIGN	CPK		10.05.2023
DRAWN	CPK		10.05.2023
CHKD	NKK		10.05.2023
APPD	NKK		10.05.2023

TITLE:
RESIDENTIAL BUILDING
LAYOUT & NUM. DET. OF CAR PARKING AT BASEMENT FLOOR

SECTION / ELEVATION MARKING
DRAWING NUMBER
SCALE 1:100

DRAWING No. 021051 - C - RB - 01 - NU- 0001

PROJECTION

SIZE A1 | REV 0

THIS ISSUED FOR: ☐ PRELIMINARY ☐ TENDER ☐ INFORMATION ☐ APPROVAL ☑ CONSTRUCTION

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STRUCTURAL STABILITY CERTIFICATE

Site located at Survey No: 613/2C, 610B/2B, T.S.No: 111/9, 110/21 Block No: 19, Ward No: E, Korattur Village, Ambattur Taluk within the limit of Greater Chennai Corporation.

It is Certified that, the Residential Building consisting of Basement floor + Stilt Floor + 10 Floors + 11 Floor Part with 63 Dwelling units is Promoting & holding by **M/s BUILT TECH ASSOCIATES,** 3-Dimensional structural Analysis has been carried out for the full Building using STAAD Pro. Loading and Load combinations is as per IS:875(Relevant Parts) – 2015. The Minimum Grade of concrete is used for All Structural Elements is M25. This Building is designed and detailed as per the following Bureau of Indian Standards IS: 456:2000, IS: 1893 (Part–1)- 2016 & IS:13920 -2016 as per the Analysis and design is **sound** and **safe** for the Residential usage.

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Zoning & Land Use Analysis

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

Property Documentation & Workflow Tools

Forms & Templates, Document Generator, Document Translation, Dictionary

Vastu & Property Management Tools

Home Vastu Shastra, Plot Vastu Shastra, Property Management

Verified RealEstate™

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

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