CE39004 - DESIGN SESSIONAL PROJECT REPORT

Group-2

Department of Civil Engineering
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Residential Tower located at Guwahati

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1. Overview of earlier presentations incorporating suggestions
2. Design of whole structure including shear wall, foundation etc. in software
3. Plumbing and road network details
4. Quantity and cost estimation
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Design Basis Report

Problem Statement

- Building Location : Guwahati
- No. of floors: B+G+9
- No. of apartments on a floor: 4
- Area of each apartment: 143 m^2
- Height of the basement and ground floor: 2.7m
- Height of each floor: 3.4m
- Wall thickness:
  - exterior: 0.25m
  - interior: 0.15m
- Bearing Capacity: 20 ton/m^2

Materials considered

- M30 Grade concrete for RCC Structural elements
- Fe500 Steel for RCC Structural elements

Geometry

4 Appartments in each floor with 2 Hallways with stairs and lifts in each one.

Apartment Plan

Floor Plan

TOILET
MASTER BEDROOM
LIVING AREA
TOILET
KITCHEN
BEDROOM
BALCONY
MASTER BEDROOM
LIVING AREA
TOILET
KITCHEN
BALCONY
TOILET
TOILET
BEDROOM
TOILET
TOILET
BEDROOM
MASTER BEDROOM
LIVING AREA
KITCHEN
BALCONY
MASTER BEDROOM
TOILET
LIVING AREA
TOILET
KITCHEN
BALCONY
BEDROOM

Basement Plan

Beam-Column Layout

Isometric View

Front View

Side View

Back View

3D View of an apartment

3D View of a floor

Walk Through

STAAD PRO MODEL

Dead Load

LOAD DEFINITION AND VALUES AS PER IS:875 PART 1

Parameters used from IS 875 Part 1

Dead load	Value
Self weight factor	-1
Slab	4.875 kN/m^2
Exterior wall	14.8775 kN/m
Interior wall	8.9625 kN/m
Parapet wall	8.25 kN/m
Floor finish	1.1 kN/m^2
Roof finish	0.5 kN/m^2
Water tank load	10kN/m^2

Dead Load

Load & Definition
Definitions
Load Cases Details
7 : EQ +X
8 : EQ -X
9 : EQ +Z
10 : EQ -Z
1 : DL
SELFWEIGHT Y -1
YRANGE -2.7 30.6 FLOAD -4.875 GY
UNI GY -8.25 kN/m
PR GY -10 kN/m2
UNI GY -14.8775 kN/m
UNI GY -14.8775 kN/m
UNI GY -8.9265 kN/m
UNI GY -8.9265 kN/m
YRANGE -2.7 27.2 FLOAD -1.1 GY
YRANGE 30.6 30.6 FLOAD -0.5 GY
2 : LL
3 : WL +X
4 : WL -X
5 : WL +Z
6 : WL -Z
11 : 1.5 (DL+LL)
12 : 1.5 (DL + EQX)
13 : 1.5 (DL + EQZ)
New...
Add...
Edit ...
Delete...
Toggle Load
Assignment Method
Assign To Selected Beams/Plates
Use Cursor To Assign
Assign To View
Assign To Edit List

Live Load

LOAD DEFINITION AND VALUES AS PER IS:875 PART 2

Parameters used from IS 875 Part 2

Live Load	Value
Bedroom	2 kN/m2
Toilet & Bathroom	2 kN/m2
Kitchen	2 kN/m2
Dinning cum Living Room	3 kN/m2
Staircase	3 kN/m2
Common Space	3 kN/m2
Balcony	3 kN/m2

Live Load

Structure
Load & Definition
Definitions
Load Cases Details
7 : EQ +X
8 : EQ -X
9 : EQ +Z
10 : EQ -Z
1 : DL
2 : LL
YRANGE -2.7 27.2 FLOAD -3 GY
YRANGE 30.6 30.6 FLOAD -1.5 GY
3 : WL +X
4 : WL -X
5 : WL +Z
6 : WL -Z
11 : 1.5 (DL+LL)
12 : 1.5 (DL + EQX)
13 : 1.5 (DL + EQZ)
14 : 1.5 (DL - EQX)
15 : 1.5 (DL - EQZ)
16 : 1.2 (DL+LL+EQX)
17 : 1.2 (DL + LL - EQX)
18 : 1.2 (DL + LL + EQZ)
19 : 1.2 (DL + LL - EQZ)
20 : 0.9DL + 1.5 EQX
21 : 0.9DL - 1.5 EQX
New...
Add...
Edit ...
Delete...
Toggle Load
Assignment Method
Assign To Selected Beams/Plates
Use Cursor To Assign
Assign To View
Assign To Edit List

Wind Pressure & Design Forces

Reference: IS:875 PART 3

PARAMETERS USED FROM IS:875 PT.3

Wind Data	Value	Reference
Basic Wind Speed	50m/s	Fig 1, IS 875, Part 3
Wind Zone	5	Fig 1, IS 875, Part 3
Terrain Category	4	IS 875, Part 3, SEction 4.3.2.1

PARAMETERS USED FROM IS:875 PT.3

Design Factors	Value	Reference
Risk Coefficient Factor, K1	1	Table 1, IS 875, Part 3
Terrain and Height Factor, K2	Varies with height	Refer IS 875, Part 3
Topography Factor, K3	1	Refer IS 875, Part 3, Clause 6.3.3.1

PARAMETERS USED FROM IS:875 PT.3

Design Wind Parameters	Value	Reference
Design Wind Speed Vz= Vb*K1*K2*K3	50*K2	Clause 6.3, IS 875 Part 3
Pz= .6(Vz)^0.5	1500 Kz^2	Clause 7.2, IS 875, Part3

PARAMETERS USED FROM IS:875 PT.3

Height(m)	K2	Vz(m/s)	Pz(KN/m^2)
10	0.8	40	0.96
15	0.8	40	0.96
20	0.8	40	0.96
30	0.97	48.5	1.41135
50	1.1	55	1.815

Wind Load

Wind Load applied along X Direction

Wind Load applied along -X Direction

EARTHQUAKE LOAD

REFERENCE: IS:1893

Seismic Load (Reference- IS:1893)

Seismic Parameter	Value
Zone	V
Zone Factor	0.36
Importance Factor	1
Response reduction Factor	5
Soil Type	Medium
Structure Type	RC Frame Building

Earthquake Load

Seismic Load applied along X Direction

Seismic Load applied along -X Direction

Seismic Load calculation

Calculation of Self Weight

1. Slabs:

For Roof,
Dead Load= 4.875+0.5= 5.375 kN/m2
Total Area= 572-36= 532 m2
Hence, Load =5.375*532=2859.5 kN

For Apartments,
Dead Load= 4.875+1.1=5.975 kN/m2
Total Area=536 m2
Hence, Load = 5.975*536= 3202.6 KN

Total Load = 6062.1 KN

2. Beams(0.5m x 0.25m):

Length of beam per floor = 366 m
Length of beam in ground = 366 m
Total Floors=11
Total Load = unit weight x Total Length x Floor x C/S Area
=25 x 11 x 366x 0.5 x 0.25 kN=12851.25 kN

3. Columns:

Total columns= 67 (per floor)
Height of basement and ground floor = 2.7m
Height of other floors = 3.4m
Total weight =[25.8 x0.75 x 0.4 + 10.2x0.6x0.3] x25x67
=16039 KN

4. Walls:

Interior walls 0.15 m thick & 100 m per floor
Load = 24*.15*2.9*10*100= 10,440 KN

Exterior walls 0.25 m thick & 257 m per floor
Load= 24*0.25*2.9*10*257= 26830KN

Total Self Weight of Building= 71642.1 KN

Zone V

Zone Factor = 0.36(Z)
Importance Factor= 0.1
Response Reduction Factor= 5
Fundamental Natural Period, T= 0.075*(h^0.75)
= 0.075*(36^0.75)
=1.10227 s
From Software, T = 1.10227 s
h=36m
Sa/g= 0.907 (From Software)
Ah= (Z*I*Sa/g) / (2*RF) = .36*.1*0.907/ 2*5 = .0326
Base Shear, VB = Ah * W = .00326* 71642 = 2335.53KN
From Software= 2171.73 KN

Beam and Column Dimensions

Beam

- Depth = L/10 or L/12
  Depth = 6000/12 = 500mm

- As per the IS Code,
  W/D > 0.4 - 0.45
  W/D> 0.4
  W>200
  Let's consider W=250mm

Column

- **Group 1 : 750mmx400mm (upto 6floors)**

- **Group 2: 600mmx300mm (Above the 6th floor)**

Design of Beams

Bending moment Diagram

Dist. m	Fy kN	Mz kN-m
4.000006675	-46.303	-48.534
4.500007510	-63.723	-20.920
5.000008344	-78.565	14.760
5.500009179	-90.828	57.215
6.000010013	-100.514	105.158

Bending moment Mz= 105.158 KN-m

Max shear: Vmax = 100.514 KN

Maximum permissible deflection in beam
= min (L/250, 20 mm)
= min (6000/250, 20 mm)
= 20 mm
Hence beam deflection is under permissible limit.

Beam concrete design

struct_assn_thurs - Beam

Geometry | Property | Loading | Shear Bending | Deflection | Concrete Design

Beam no. = 1950 Design code : IS-456

Design Load

Mz Kn Met	Dist. Met	Load
74.41	3	11
-82.79	0	11
-105.16	6	11

Design Parameter

Fy(Mpa)	500
Fc(Mpa)	30
Depth(m)	0.5
Width(m)	0.25
Length(m)	6.000010013

Print | Close

MANUAL CALCULATIONS AND RESULTS:

For Flexure,

Assume clear cover = 30mm
Steel bar diameter = 12mm
Effective cover = d' = 30+6= 36mm
So, effective depth = d= D-d'= 500-36= 464mm
x/d= 1.22-[(1.22)^2 -(6.61Mu/fck bd^2)]^0.5
= 1.22- [(1.22)^2 -(6.61*105.158*10^6/ 30*250*464^2]^0.5
=.191444
(x/d) < (x/d)max = 0.46
Since it is an under reinforced section, beam is safe

Now, Pt = 41.38*(fck/fy)*(x/d)
=41.38*(30/500)*0.1914
=.475%
(Pt)min = (.85/100) = 0.17%
(Pt)max = 4%
(Pt)min < (Pt)provided < (Pt)max,
Hence beam is safe
So, Ast = Pt*b*d= (0.475/100)*250*464
= 551 mm^2

Ast (From Software) = 597.10 mm^2

For Shear,

Vmax = Vu= 100.514 KN

Tv = V/bd= 100.514/250*464 = 0.722N/mm^2

(Tc)max = 3.1 N/mm^2 (As per Table 20)

Tc= 0.578 N/mm^2

(By interpolation from Table 19, IS 456:2000)

Tu > Tc => Shear reinforcement should be provided

For V-Stirrups,

Vus= Vu-Vc = (Tv-Tc)*b*d

= (.722-.578)*250*464 = 16.7 KN

Taking 2- 8mm dia V-Stirrups. So, Asv = 100.53mm^2

Sv = (0.87fy*Asv*d / Vus) = 121mm

Provide 2legged- 8mm dia stirrups @ 120mm c/c

Design of Columns

Bending Moment Diagram

Concrete Design

A sc from Software =1561 mm^2

A sc from calculation = 1602 mm^2

MANUAL CALCULATIONS AND RESULTS:

Considering Beam No. 1866
My = 71.83 KNm
Mz = 60.93 KNm
Pu = 251.11 KN
L = 3.4m , b=300mm , D=600mm
Check for short column:
L eff = 0.65*L = 0.65*3.4 = 2.21 (Since both ends are restarined)
So, slenderness ratio, λ = L eff / b = 2.21/.3 = 7.367 < 12
Hence this is short column
Area of concrete = 180000mm^2
Assuming d’ = 30mm
d’ / D = 30/600 = 1/20 = 0.05

SP 16 Chart 47,

Pu / fck*b*D = (251.11*10^3)/(30*300*600)

= 0.05

P/fck = 0.89/30 = .03

Mu/fck*b*D^2 = 0.08

Mu y1 = 0.08*30*300*600^2

= 259.2 KNm

Now, d‘ / B = 30/300 = 0.1

Chart 48 SP 16:

Pu/ fck*b*D = 251.11*10^3 / 30*300*600

= 0.05

P/fck = 0.03

Mu z1 = .080*30*600*300^2

= 129.6 KN

Checking for

(Mu y / Mu y1) ^αn + (Mu z / Mu z1) ^αn ≤ 1

αn => P u2 = .45*fck*Ag + (0.75*fy-0.45*fck)*Asc

Asc = .89*300*600/100 = 1602mm^2

P u2= 0.45*30*600*300 +(.75*500-.45*30)*1602

= 3009.123KN

Alternatively referring to chart 63 of SP16,

P u2/Ag =16.8

P u2= 16.8*600*300=3024 KN

Pu/Pu2 = 251.11/3009.123= 0.083

Pu /Pu2 < 1 => αn=1

=> Mu y/Mu y1 + Mu z/ Mu z1 ≤ 1

71.83/259.2 + 60.93/129.6 ≤ 1

0.747 ≤ 1

Hence OK

Load Combinations (IS:456)

Load combination	DL factor	LL factor	EL/WL factor
Seismic +X	0	0	1
Seismic -X	0	0	1
Seismic +Z	0	0	1
Seismic -Z	0	0	1
DL	1	0	0
LL	0	1	0
WL +X	0	0	1
WL -X	0	0	1
WL +Z	0	0	1
WL -Z	0	0	1
DL+LL	1.5	1.5	0
DL+LL+S +X	1.2	1.2	1.2
DL+LL+S -X	1.2	1.2	1.2
DL+LL+S +Z	1.2	1.2	1.2
DL+LL+S –Z	1.2	1.2	1.2

Validation of Reaction Forces

For Node Number 143 :

For Dead load -

1. Column weight above node = Height x Area xunit weight
   = [(25.8*.75*.4) + (3*3.4*.6*.3)] *25 = 239.4 KN
2. Beam weight above node = Area x Length/2 x No. Of beams x No. Of Floors x unit wt.
   = 0.5*.25*6/2 *2 *11 *25kN =206.25 KN
3. Wall weight above node = Height x Area x unit Weight
   =[(3.4 *9 + 2.7*2]*[0.25 x6 x2/2] x24 kN =1296 KN
4. Slab weight above node = 1/4 x Area of Beam x Thickness x unit wt. x No of Floor
   = 1/4 x(6x6) x0.195 x25 x11 kN =482.625 KN

Net reaction(Manual) = 2224.275 kN

Net reaction (software) = 2371.95 KN

For Node Number: 143

For Live load

We see that only the slab will have a live load

Live load of slab above node = 1/4 xArea of Room x Load xNo of Floor

= 1/4 x(6 x6) x(3x10 + 1.5x1) =283.5 KN

Net reaction(manual) =283.5 kN

Net reaction (software) = 285.553 KN

struct_assn_thurs - Support Reactions:

All | Summary | Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	Fx kN	Fy kN	Fz kN	Mx kN-m	My kN-m	Mz kN-m
143	1 DL	19.511	2371.995	18.745	17.396	-0.103	-16.896
	2 LL	3.900	285.553	3.858	3.689	-0.021	-3.331
	3 WL +X	-12.100	-34.002	-0.991	-1.444	0.068	21.493
	4 WL -X	7.119	33.445	0.602	0.710	0.062	-19.658
	5 WL +Z	-1.772	-59.131	-19.254	-29.401	-0.054	1.450
	6 WL -Z	2.588	58.619	14.121	27.210	-0.115	-3.307
	7 EQ +X	-21.930	-148.425	-1.804	-1.997	-0.168	62.471
	8 EQ -X	21.930	148.425	1.804	1.997	0.168	-62.471
	9 EQ +Z	-2.568	-108.953	-16.021	-31.472	0.086	2.877
	10 EQ -Z	2.568	108.953	16.021	31.472	-0.086	-2.877
	11 1.5 (DL+L	35.117	3986.322	33.904	31.626	-0.187	-30.340
	12 1.5 (DL +	-3.628	3335.356	25.411	23.098	-0.407	68.362
	13 1.5 (DL +	25.414	3394.564	4.086	-21.115	-0.027	-21.029
	14 1.5 (DL -	62.162	3780.630	30.823	29.089	0.097	-119.050
	15 1.5 (DL -	33.119	3721.422	52.149	73.301	-0.283	-29.659
	16 1.2 (DL+L	1.777	3010.948	24.959	22.905	-0.351	50.693
	17 1.2 (DL +	54.409	3367.167	29.288	27.697	0.052	-99.237

Slab Design Manual

M30, Fe500

fck=30 N/mm^2, fy= 500 N/mm^2

γc = 25KN/mm^2

Span/Effective depth = 35

=> 6000/d = 35

d= 172 mm

Let us consider 12 dia bars,

Therefore thickness of slab, h = d+Φ/2+cover

=172+12/2+15=193mm

193mm is not appreciable

Let us consider h=195mm

so updated d=174mm

Step 2 (Load Calculation):

Dead Loads:

Self weight of Slab (DL) = (1*1*.195)*25

= 4.875 KN/m^2

Live Load (LL) = 3 KN/m^2

TL = 7.85 KN/m^2

Factored Design Load (W)= 1.5*TL

= 11.8125 KN/m^2

Step 3 (IS Code 456:2000, pg 90-91, Table 27)

Ly/Lx = 1

αx = 0.062

αy = 0.062

Mx= αx*W*Lx^2= .062*11.8125*6^2=26.3655 KNm

My= αy*W*Ly^2=.062*11.8125*6^2=26.3655 KNm

Step 4 (Checking depth requirement from moment)

Mu =0.138 fck*b*d^2

26.3655*10^6=.138*30*100*d^2

d= (26.3655*10^6/.138*30*1000)^0.5

= 80mm < 174mm(provided)

Hence OK

Step 5 (Checking for Shear)

Max design shear, V = 0.5W*Lx

=0.5*11.8125*6=35.4375 KN

Approx Avg. shear stress = V/bd

=35.4375*10^3/174*1000

=0.20 N/mm^2

Consider a very less % of reinforcement, referring to Table 19(Pg 73) of IS:456:2000,

The design shear strength,

Tc= 0.29N/mm^2 > 0.2N/mm^2

Hence OK

Step 6: Calculation of Area

(SP 16, Table 3, Pg 49)

Mu/bd^2 = 26.3655*10^6/1000*174^2 = 0.87

Pt= 0.207 (Table 3, SP16, Pg 49)

As = Pt*bd/100 = .207*1000*174/100= 360mm^2/m width

Our both spans are same

For main reinforcements, using 12 dia, a s = pi*12^2/4 =113mm^2

No of rods within 1000mm = 360/113 = 4 no.

Let us adopt a spacing = 280mm c/c

Check spacing ≯ 3d or 300mm [whichever is less]

280 ≯ 300 Hence OK

Actual As provided =>

(As) provided =4*113=452 mm^2 within 1000mm width

For distribution reinforcements using 12 dia as well

(Same calculations as above)

Design of Stair Case

Thickness of waist slab = 150mm
Rise =150mm, Tread=300mm
Self weight of waist slab = .15*25*(R^2+T^2)/T
= 4.19 KN/m^2
Self wt. of steps = 25*(1/2)*.15 = 1.875 KN/m^2
Floor finish = .05*20 = 1 KN/m^2
LL = 3 KN/m^2
fck = 30N/mm^2
fy = 500N/ mm^2

R = 150mm
T = 300mm
G = 250mm
N = 50mm

Considering 12 steps,
Considering 1m = 1000mm strip
One way slab, Span/eff. depth =25
12*250/d = 25
d= 120mm
Let us assume that we will use 12 dia bars
w = d+cover+ϕ/2 =120+20+6= 146 mm
Not feasible
Take w= 150mm
d=124mm

FT= Tiles finish= 15mm
Self wt. Tiles= 23.5KN/m^3
B= (G^2+R^2)^0.5 = 292mm
Dead load = [{WB+(1/2)RT}*25 + {FT*23.5}(1/4)
=[{.15*.292 + (1/2)*150*.3}25 +
{.015*.3*23.5}]*(1/.25)
=7.053KN/m^2
Live Load =3 KN/m^2
Factored design load= 1.5*7.055= 15.08 KN/m^2

Step 3: Checking (Mu,d req)

Design moment = M= WL^2/10=15.08*(3)^2/10 = 13.572 KNm

But Mu= .138 fck*b*d^2

if d=129mm remaining (Mu)avail = 68.89 KNm > 13.572 KNm

Hence OK

Step 4: Calculation of reinforcement

Mu/bd^2 = 13.572*10^6/1000*124^2 = 0.88

Mu = .36 fck bx(d-.42x)

Solving above eqn

x/d = 1.2 -((1.2)^2 -(6.61Mu/fck*b*d^2))^0.5

=1.2-((1.2)^2 -(6.61*.88)30)^0.5 = 0.084

x=0.084d = 10.416mm

Lever arm =z=d-.42x =119.625mm

Mu = .82 fy*As*x^2

13.522*10^6 =.87*500*As*119.625

As=400 mm^2/m width

Width of staircase considered 1.5m

Total As(req) =1.5*400=600mm^2
We considered 12 dia bars
so Total no of bars= 600/(pi/4)*12^2 = 6 bars
Distribution reinforcement =.12*1000*1.5/100
=180 mm^2/m width

Assuming 8 dia to be used
as=(pi/4)*8^2 = 50mm^2
Total steel required = 180*3750=675 mm^2 lover 3750mm length
Total no of rods= 675/50 = 14 number
Spacing b/w bars = 3750/14 = 270mm c/c

Design of Shear wall

Shear walls provide large strength and stiffness to buildings in the direction of their orientation, which significantly reduces the lateral sway of the building and thereby reduces damage to the structure and its contents.

Deflection before adding the shear wall

	Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	839	26 1.5 DL + 1	**24.848**	-9.101	-9.580	28.143	0.000	-0.000	-0.001
Min X	792	27 1.5 DL -1.	**-24.803**	-9.100	-9.533	28.087	0.000	0.000	0.001
Max Y	748	10 EQ -Z	0.000	**0.798**	-26.835	26.847	-0.000	0.000	-0.000
Min Y	739	11 1.5 (DL+L	0.029	**-22.780**	-2.690	22.939	-0.003	0.000	-0.000
Max Z	782	22 0,9 DL + 1	-0.007	-5.690	**41.346**	41.736	0.001	-0.000	-0.000
Min Z	852	31 1.5 DL - 1.	-4.618	-4.241	**-46.998**	47.414	-0.001	0.000	0.000
Max rX	640	30 1.5 DL + 1	-4.241	-9.435	34.337	35.861	**0.002**	0.000	-0.000
Min rX	739	11 1.5 (DL+L	0.029	-22.780	-2.690	22.939	**-0.003**	0.000	-0.000
Max rY	851	14 1.5 (DL -	-23.853	-5.185	-6.649	25.299	-0.000	**0.000**	0.001
Min rY	785	24 1.5DL + 1.	23.999	-5.026	1.932	24.596	0.000	**-0.000**	-0.001
Max rZ	845	11 1.5 (DL+L	-0.124	-12.291	-3.152	12.689	-0.000	-0.000	**0.001**
Min rZ	789	11 1.5 (DL+L	0.202	-12.293	-3.103	12.680	-0.000	0.000	**-0.001**
Max Rs	810	29 1.5 DL - 1.	4.871	-19.647	-45.935	**50.197**	-0.002	-0.000	-0.001

Maximum Drift Calculation

From the table from the slide before

Max deflection (δ) = 50.197 mm

Height of building (h) = 36 mm

Max drift = δ/h

= 50.197/36000

= 0.139%

As per Cl. 11.1 (IS 1893:2002), Max drift ≤ 0.4%

No shear wall is required in the building against deflection. Still, shear walls are provided adjacent to lifts and a few other places.

Shear wall Placement

Deflection after adding the shear wall

			Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	851	26 1.5 DL + 1	**24.440**	-5.683	-6.157	25.836	0.000	-0.000	-0.001
Min X	785	27 1.5 DL -1.	**-23.843**	-5.505	-3.756	24.757	0.000	0.000	0.001
Max Y	790	10 EQ -Z	-0.034	**1.339**	-15.267	15.325	-0.000	0.000	0.000
Min Y	739	11 1.5 (DL+L	-0.088	**-21.063**	-0.470	21.068	-0.005	0.000	-0.000
Max Z	786	28 1.5 DL + 1	5.029	-5.854	**24.314**	25.509	0.000	0.000	-0.000
Min Z	850	29 1.5 DL - 1.	4.764	-6.530	**-26.109**	27.331	-0.002	0.000	0.000
Max rX	831	11 1.5 (DL+L	-0.137	-15.509	-0.805	15.530	**0.002**	0.000	-0.000
Min rX	810	11 1.5 (DL+L	-0.140	-20.630	-0.454	20.635	**-0.005**	0.000	-0.000
Max rY	785	29 1.5 DL - 1.	4.549	-4.363	-20.557	21.501	0.000	**0.000**	0.000
Min rY	801	14 1.5 (DL -	-22.677	-2.166	-0.544	22.786	0.001	**-0.000**	0.000
Max rZ	825	11 1.5 (DL+L	-0.417	-8.358	-0.707	8.398	0.000	0.000	**0.002**
Min rZ	806	11 1.5 (DL+L	-0.372	-8.315	-0.150	8.324	0.000	0.000	**-0.002**
Max Rs	837	29 1.5 DL - 1.	4.878	-17.726	-25.256	**31.239**	-0.000	0.000	0.001

Max deflection after adding – 31.239 mm

Foundation Design

Using IS 2911-2010 Part 1

We have for node 143,
Load= 3986.222KN
pile capacity =750 KN
Assume group efficiency of 90%
Number of piles, n= 3986.222/(750*0.9)=5.90

Provide 6 piles
Now centre to centre distance between pile=3D
take D= 500mm
c/c distance=1.5m
Now edge distance=0.5m

Choosing 2 rowsx 3 Columns arrangement:
Thus length of pile cap = (1.5*2)+(0.5*2)= 4m
Width of pile cap= (1.5*1) + (0.5*2)= 2.5m

PILE ARRANGEMENT

Column Dimensions

Column Shape : Rectangular

Column Length - X (Pl) : 0.750 m

Column Width - Z (Pw) : 0.400 m

Pedestal

Include Pedestal? No

Pedestal Shape : N/A

Pedestal Height (Ph) : N/A

Pedestal Length - X (Pl) : N/A

Pedestal Width - Z (Pw) : N/A

Pile Cap Geometrical Data

Pile Cap Length P_{CL} = 4.000 m

Pile Cap Width P_{CW} = 2.500 m

Initial Pile Cap Thickness t_I = 0.300 m

Pile Geometrical Data

Pile spacing P_s = 1.500 m

Pile Edge distance **e** = 0.500 m

Pile Diameter d_p = 0.500 m

Pile Capacities

Axial Capacity P_P = 500.000 kN

Lateral Capacity P_L = 100.000 kN

Uplift Capacity P_U = 300.000 kN

Material Properties

Concrete f'_c = 25000.004 kN/m^2

Reinforcement f_y = 415000.070 kN/m^2

Concrete Cover

Bottom Clear Cover CC_B = 0.050 m

Side Clear Cover CC_S = 0.050 m

Pile in Pile Cap PC_P = 0.075 m

Reaction: Fx=0.15 Fy=242.49 Fz=-20.57 Mx=-36.11 My=0.01 Mz=-0.27

Reaction: Fx=2.14 Fy=-112.02 Fz=-16.64 Mx=-32.77 My=-0.02 Mz=-1.57

Reaction: Fx=1.23 Fy=-160.29 Fz=-23.62 Mx=-38.96 My=0.07 Mz=-1.12

Reaction: Fx=0.80 Fy=57.50 Fz=-17.61 Mx=-33.67 My=0.01 Mz=-1.40

Reaction: Fx=1.42 Fy=-107.73 Fz=-18.04 Mx=-33.78 My=-0.01 Mz=-1.58

Reaction: Fx=-1.87 Fy=236.15 Fz=-18.07 Mx=-33.80 My=-0.03 Mz=1.48

Load Comb 101 : 1.000 x DL+1.000 x DL+1.000 x DL+1.000 x DL+1.000 x DL+1.000 x DL+1.000 x DL+1.000 x D

Load Comb 102 : 0.800 x DL+0.800 x DL+0.800 x DL+0.800 x DL+0.800 x DL+0.800 x DL+0.800 x DL+0.800 x D

Load Comb 201 : 1.500 x DL+1.500 x DL+1.500 x DL+1.500 x DL+1.500 x DL+1.500 x DL+1.500 x DL+1.500 x D

Load Comb 202 : 0.960 x DL+0.960 x DL+0.960 x DL+0.960 x DL+0.960 x DL+0.960 x DL+0.960 x DL+0.960 x D

Load Comb 203 : 1.200 x DL+1.200 x DL+1.200 x DL+1.200 x DL+1.200 x DL+1.200 x DL+1.200 x DL+1.200 x D

Load Comb 204 : 0.900 x DL+0.900 x DL+0.900 x DL+0.900 x DL+0.900 x DL+0.900 x DL+0.900 x DL+0.900 x D

Loading applied at top of cap

Load Case	F_x (kN)	F_y (kN)	F_z (kN)	M_x (kNm)	M_y (kNm)	M_z (kNm)
101	-19.246	-2656.479	-17.081	-18.159	0.000	20.250
102	-15.397	-2125.183	-13.665	-14.527	0.000	16.200
201	-28.869	-3984.718	-25.621	-27.238	0.000	30.374
202	-18.476	-2550.220	-16.398	-17.433	0.000	19.440
203	-23.095	-3187.774	-20.497	-21.791	0.000	24.299
204	-17.322	-2390.831	-15.373	-16.343	0.000	18.225

PILE CAP DESIGN CALCULATION

Pile Reactions

Total pile number **N** = 6

	Arrangement		Reaction		
Pile No.	X (m)	Y (m)	Axial (kN)	Lateral (kN)	Uplift (kN)
1	-1.500	-0.750	-692.987	8.135	0.000
2	-1.500	0.750	-674.411	8.135	0.000
3	0.000	-0.750	-686.175	8.135	0.000
4	0.000	0.750	-667.598	8.135	0.000
5	1.500	-0.750	-679.362	8.135	0.000
6	1.500	0.750	-660.786	8.135	0.000

Reinforcement Calculation

Maximum bar size allowed along length # 32

Maximum bar size allowed along width # 20

Bending Moment At Critical Section = -1538.295 kNm (Along Length)

Bending Moment At Critical Section = -1132.168 kNm (Along Width)

Pile Cap Thickness **t** = 1.219 m

Selected bar size along length # 12

Selected bar size along width # 12

Selected bar spacing along length = 66.33 mm

Selected bar spacing along width = 77.76 mm

Pile Cap Thickness Check

Calculated Thickness (t) = 1.219 m

Check for Moment (Along Length)

Critical load case for thickness is reported only when required thickness is more than the given minimum thickness

Critical Load Case : 201

Pile No.	Moment along x_1-x_1 (kNm)	Moment along x_2-x_2 (kNm)
1	-779.597	0.000
2	-758.698	0.000
3	0.000	0.000
4	0.000	0.000
5	0.000	-764.269
6	0.000	-743.370

Effective Depth(d_{eff}) = $h_{cap} - (p_{id} + cc + 0.5 \times d_b)$ = 1.088 m

Depth of neutral axis for balanced section(x_u) $\frac{700 \times d_{eff}}{1100 + 0.87 \times f_y}$ = 0.521 m

As Per IS 456 2000 ANNEX G,G-1.1 C

Ultimate moment of resistance(M_{ulim}) = $0.36 \times f_c \times b \times X_u \times (d_{eff} - 0.416 \times X_u)$ = 10217.168 kNm

We observed M_u <= M_{ulim} hence singly reinforced and under reinforced section can be used

Check for One Way Shear (Along Length)

Pile No.	Shear Force x_1-x_1(kN)	Shear Force x_2-x_2(kN)
1	-241.535	0.000
2	-235.137	0.000
3	0.000	0.000
4	0.000	0.000
5	0.000	-236.842
6	0.000	-230.444
TOTAL	-476.672	-467.287

Design Shear Force for One-Way Action V_u = -476.672 kN

As Per IS 456 2000 ANNEX B,B-5.1 and Clause No 34.2.4.2

Design Shear Stress (T_v) = $\frac{V_u}{B \times d}$ = -175.247 kN/m^2

Allowable Shear Stress (T_c) = $\frac{0.85 \times \sqrt{0.8 \times f_c}}{6 \times \beta} \times \left(\sqrt{1 + 5 \times \beta} - 1\right)$ = 276.952 kN/m^2

Where Beta = $\max\left(\frac{0.8 \times f_c}{6.89 \times .pt}, 1\right)$ = 21.590

and percentage of steel required (p_t) = $\frac{A_{st}}{B \times d} \times 100$ = 0.134

Here $T_v <= T_c$ Hence safe

Check for Two Way Shear (Along Length)

Pile No.	Two-way Shear at column face (kN)
1	-420.792
2	-409.646
3	-213.353
4	-207.646
5	-412.617
6	-401.471
TOTAL	-2065.526

Design Two-Way Shear force = -2065.526 kN

As Per IS 456 2000 Clause 31.6.2.1

Two Way Shear Stress(T_v) = $\frac{V_t}{b_o \times d_{eff}}$ = -285.397 kN/m^2

Where,perimeter of critical section(b_0) = $2 \times (b + h + 2 \times d)$ = 6.652 m

As Per IS 456 2000 Clause 31.6.3.1

Allowable shear stress = $K_s \times T_c$ = 1250.000 kN/m^2

Where,k_s = $\min[(0.5 + \beta), 1]$ = 1.000

Ratio of shorter to longer dimension(B_c) = 0.533

and,T_c = $0.25 \times \sqrt{f_c} \times b \times d$ = 1250.000 kN/m^2

$T_v < K_s T_c$ hence Safe

Calculation of Maximum Bar Size

Along Length

Selected maximum bar size = 32 mm

Bar diameter corresponding to max bar size(d_b) =32.000 mm

As Per IS 456 2000 Clause No 26.2.1

Development Length(l_d) = $\frac{0.87 \times d_b \times f_y}{4 \times \tau_{bd}}$ = 1.289 m

Allowable Length(l_{db}) = $0.5 \times (B - b) - C_s$ = 1.575 m

$l_{db} > l_d$ hence, safe

Selection of Bottom and Top Reinforcement

Top reinforcement is provided same as bottom reinforcement

Along Length

Critical Load Case : 201

As Per IS 456 2000 Clause 26.5.2.1

Minimum Area of Steel (A_{stmin}) = $0.12\% \times B \times h_{cap}$ = 3627.000 mm2

As Per IS 456 2000 ANNEX G,G-1.1 b

Area of steel required (A_{sq}) = $0.5 \times \left(\frac{f_c}{f_y}\right) \times \left(1 - \sqrt{1 - \frac{4.5977 \times M_u}{f_c \times b \times d \times d}}\right) \times b \times d$ = 4093.596 mm2

Area of steel provided (A_{st}) = = 4093.596 mm2

$A_{stmin} <= A_{st}$ Steel area is accepted

Minimum spacing allowed (S_{min}) = 40 + d_b = 52 mm

Selected spacing (S) = 66.33 mm

S_{min} <= S <= 450 mm and selected bar size < selected maximum bar size... The reinforcement is accepted.

Similarly all the calculations are done along width too for the pile cap design

Pile Design

D=0.5m

Q design = 750KN

Taking Factor of Safety=2.5 as per Cl.B-5 (IS 2911 (Part 1/Sec 1) : 2010)

Q ultimate = 2.5*750= 1875KN

C = qu/2 = 20ton/m^2/2 = 10ton/m^2 = 99.64 KN/m^2

q tip = c*Nc= 10*9=90 ton/m^2= 896.76 KN/m^2

Q tip= q tip*(pi/4)*D^2

=896.76*(3.14/4)*0.5^2= 175.98 KN

Q friction= Q ult - Q tip

= 1875-175.98= 1699.02 KN

Q friction = α*C*(pi)*D*L

1324.02 = .45*99.64*3.14*.5*L

=> L= 24.135 m

[α obtained from Fig.2
IS 2911 (Part 1/Sec 1) : 2010]

Therefore length of Pile is taken as 25m

Calculation of Maximum Bar Size

Along Length

Selected maximum bar size = 32 mm

Bar diameter corresponding to max bar size(d_b) =32.000 mm

As Per IS 456 2000 Clause No 26.2.1

Development Length(l_d) = $\frac{0.87 \times d_b \times f_y}{4 \times \tau_{bd}}$ = 1.289 m

Allowable Length(l_{db}) = $0.5 \times (B - b) - C_s$ = 1.575 m

$l_{db} > l_d$ hence, safe

Selection of Bottom and Top Reinforcement

Top reinforcement is provided same as bottom reinforcement

Along Length

Critical Load Case : 201

As Per IS 456 2000 Clause 26.5.2.1

Minimum Area of Steel (A_{stmin}) = $0.12\% \times B \times h_{cap}$ = 3627.000 mm2

As Per IS 456 2000 ANNEX G,G-1.1 b

Area of steel required (A_{sq}) = $0.5 \times \left(\frac{f_c}{f_y}\right) \times \left(1 - \sqrt{1 - \frac{4.5977 \times M_u}{f_c \times b \times d \times d}}\right) \times b \times d$ = 4090.068 mm2

Area of steel provided (A_{st}) = = 4090.068 mm2

$A_{stmin} <= A_{st}$ Steel area is accepted

Minimum spacing allowed (S_{min}) = 40 + d_b = 52 mm

Selected spacing (S) = 66.33 mm

S_{min} <= S <= 450 mm and selected bar size < selected maximum bar size... The reinforcement is accepted.

Reinforcement Detailings

Beam Reinforcement Detailing

Column Reinforcement Detailing

Beam-Column Junction Reinforcement detailing

Stair Case Reinforcement Detailing

slab reinforcement detailing

Foundation Detailing

0.75 m
0.40 m
1.50 m
2.50 m
4.00 m

WATER SUPPLY DESIGN

Water demand as per National Building Code 1983 = 135 LPCD
There are 4 Apartments on each floor, hence,
Total Population = 10*4*5 = 200 (assuming average family size to be 5)
Considering Water Demand as 140 LPCD
Total water demand = 140*200= 28,000 litres/day
Average rate of supply = 28,000 litres/day
Hourly Demand = 1166.67 litres/hr = 1200 litres/hr
(Considering water losses in pipes)
Assuming the pump works for 8 hours (4 h morning + 4 h evening) per day to fill overhead tank,
Rate of pumping = 28,000/8 = ≈ 3500 L per hour

OVERHEAD TANK CAPACITY

According to SP35 Table 15,
Capacity of storage reservoir = maximum deficit + maximum surplus
= 2.40 + 7.0a = 9.4a
= (9.4*2*1200) = 22,560 L

According to SP35 Table 16,
Storage of overhead tank according to number of people = 70 Litres per person x Number of inhabitants = 70 x 200 = 14,000 L

According to SP35 Table 17,
Minimum storage needed for flushing
= number of floors x (270 L for 1 toilet + (180 L per secondary toilet(s)))
= 10 floors x ((270 x 8) L + (180 x 0) L)
= 21,600 L

As per clause 5.4.2.3 of SP35,
Maximum Storage=0.5 x Daily Supply
=0.5 x 28000
=14000 L
Storage of Overload tank=Max(22560 , 14000, 21600, 14000)=22,560 L

Required Storage Capacity of overhead tanks = 22,560 L

OVERHEAD TANK DESIGN

Dimensions of Storage Tank on roof Diameter= 2 m, Depth = 1.8 m
Volume of a Single Storage Tank (VT) = (3.14 x 2x2 x 1.8 x 1000/4) =
5,652 L
Number of tanks required on Roof = Required Storage / VT
= 22,560 / 5652
= 3.99

Provide 4 tank with each block having 1 tank.

DESIGN OF WATER SUPPLY PIPES FROM TANKS(FTA) REF- SP35

The sanitary fixtures in each apartment consist of the following:

a) 1 sink and 1 tap in the kitchen

b) 1 overhead flushing tank for water-closet and tap in each watercloset room;

c) 1 shower, a tap and 1 wash basin in each bathroom

Sl No.	**Description**	**No of Fixtures per Flat**
1	Kitchen Sink	2
2	Kitchen Tap	2
3	Ablution Tank	2
4	Supply to overhead flushing tank	1
5	Shower	2
6	Tap	2
7	Wash Basin	2
	Total	13

DESIGN OF SEWERAGE PIPE SYSTEM

Two pipe system is provided.
Soil pipes are connected to the Building drain while waste pipes are connected using trapped gully.
MVP = Main Ventilation Pipe 50 mm
MWP = Main Waste Pipe 100 mm
MSP = Main Soil Pipe 100 mm

From Horizontal Branches for each floor Table 52 SP35 (Not including sewerage pipes)

MWP - 100 mm pipe diameter is connected for branching
MSP - 75 mm pipe diameter is connected for branching

Waste Appliances used Table52 SP35

Pipe type	Diameter
Main	35 mm
Susidiary	25mm
Distribution	15mm

Waste Appliances used Table 54 SP35

Waste Appliance	Internal Diameter (mm)
Wash Basin	30
Domestic Sink	40
Bathrooms(Showers)	40
Urinal Floor Trap	65

DESIGN OF SEWER PIPE

We have total water demand calculated in previous section= 28,000 L/day
Now assume 90% of total consumption to reach sewer
Here, peak factor=3(since population <20,000)
>Total storage=28,000*0.9*3 L/day = 75,600 L/day
= 875 cm^3/s
Now using Mannings' formula $Q= AR^{2/3}\sqrt{s}/ n$
Take slope of main sewer pipe= 1/1000
Mannings' coefficient=0.012
On Calculating we get, d=13.668 cm= 136mm

Provide 140 mm diameter pipe for the sewer pipe connecting the pipes from sewage from all the flats to the municipal sewarage system.

Plumbing network

TOILET
TOILET
TOILET
TOILET

Electrical Wiring Network

GRAPHIC SYMBOL	DESCRIPTION
	LIGHT
	FAN
S	SWITCH
	SOCKET

ROAD NETWORK

Cost estimation

Materials	Quantity		Standard Price(Rs.)		Price (Rs.)
Concrete	2181.4	m3	4000	per m3	8725972
Steel	136.5	tons	85000	per tonn	11602500
Bricks	1741390	pieces	8	per piece	14998484
Tiles	76100.85	sqft	50	per sqft	3805042.5
Plastering	96068	sqft	90	per sqft	8646120
Painting Exterior	99587.7	sqft	35	per sqft	3485569.5
Painting Interior	38750.08	sqft	120	per sqft	4650009.6
Putty	12255	Kg	50	per kg	612750
Primer	1200	liters	200	per liter	240000
				Total Cost	52164203
				Cost per sq.m	10819.824

Thank You