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Structural G+5 Residential Building in Addis Ababa

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Our greatest thanks from the depth of our heart is to GOD for endowing us with the courage, strength as well as health throughout our school time.

Next, it is our deepest gratitude and respect to our project co-coordinator Mr. Kibret for valuable advice, and approach.We also extend our thanks to all staff of civil engineering department.

Summary

This project deals about the structural analysis and design of G+5 Residential building considering all the external and internal effects according to EBCS 2015. The project is located in Addis Ababa. It includes wind load analysis The proper design for each structural component is also carried out. Slab analysis and design, staircase analysis and design is carried out manually. Frame analysis was carried out by using ETABS commercial software and lateral load analysis. Finally, the foundation for the structure was designed.

From the analysis, we observed that the structure is designed safely and economically under the exposed load. Based on the output from ETABS software it is selected that, isolated footing is the most compatible for the bearing capacity of the soil. In a view of these analysis and design producer, it is essential to use soft wares like ETABS in order to save time and accuracy. Finally, it is recommended that, it is appreciated to incorporate such kind of project for further experience of the student and it is better to come up with the design data which have the necessary information.

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1 Introduction

1.1 Background of the project

There are so many type of structure design based on its function such as Residential, Hotel, and Mixed use…..etc. structure design. In this case Residential building structure is design.

This project is G+5 Residential building allocated on Addis Ababa,. It is under terrain category IV and zone 2 it is not hazard to earth quake but analysis earth quake effect in the building. The total area of the site is about 166.808m^2.

The height of the building is +19 m. The geometry of the roof is flat roof type and the geometry of slab is solid slab. On the ground floor 3 .00m height long, it includes parking, store rooms, toilet are presented. First floor 3.00m height long, it includes office, living and dining room, kitchen, toilet, terrace is presented. Second floor 3.00m height long, it include dressing, Jacuzzi, toilet, terrace, are presented. Third floor includes TV room, 2childrenBed room, terrace, and toilet. Fourth and fifth floor are typical 3.00m long, it includes study room, family room, Bed room and toilet.

The limit state design method is used for the entire structure. The Ethiopian Building code of standards (EBCS_2015) is used in the design with some books and lecture note as a reference.

1.2 Objective

The main objective of this B.S.C project is to design G+5 residential building using EBCS 2015.that is safe & economical

Sub-objective

- To help the student in order to revise what have learnt in their time of study.
- To get more knowledge about engineering software like, ETABS v2016
- It develops the habit of working together or team.
- To understand more information about EBCS 2015.
- To identify natural hazards and make the building strong enough to sustain those hazards.

1.3 Design Specifications and Constants

1.3.1 Material used and Properties

The first step in design is to select construction materials which are capable of resisting the applied load. Considering the availability in market we select concrete and steel reinforcement as follows.

Concrete

Class I workmanship and ordinary loading condition is used.

Concrete grade C-25 is used because it can be used for construction in all areas including foundations. Domestic and commercial buildings are the type of buildings in which the C-25 concrete mix is applicable for.

Partial safety factor for concrete γc= 1.5

Characteristic strength, $fck = \frac{fcu}{1.25} = \frac{2.5}{1.25} = 20MPA$ Where fcu = 25Mpa

Design strength, $fcd = \frac{accfck}{\gamma c} = \frac{0.85*20}{1.5} = 11.33MPA$

Reinforcement

Steel S-400 Partial safety factor for concrete γs= 1.15

Characteristic strength, fyk=400Mpa

Design strength, $fyd = \frac{fyk}{\gamma s} = \frac{400}{1.5} = 347.83MPA$

Concrete cover

Cnom= Cmin + ΔCdev

Cmin = max of (Cmin, b, Cmin, dur + ΔCdur – ΔCdur, st - ΔCdur, add, 10mm)

Cmin = max of (10, 10+ 0 – 0-0, 10mm)

Cmin = 10mm

Cnom = 10+10 = 20mm

We take Ø=10mm for slab reinforcement and concrete cover of 20mm

Partial safety factors for load

1.35 for dead load

1.50 for live load

2 Roof Analysis and Deign

Roof is the upper part of a building that protects from any kind of weather. It is subjected to different kinds of loads such as wind load, its own self-weight, and the loads of the persons who go on the roof for maintenance and snow loads too. Type of roof on this project is flat roof.

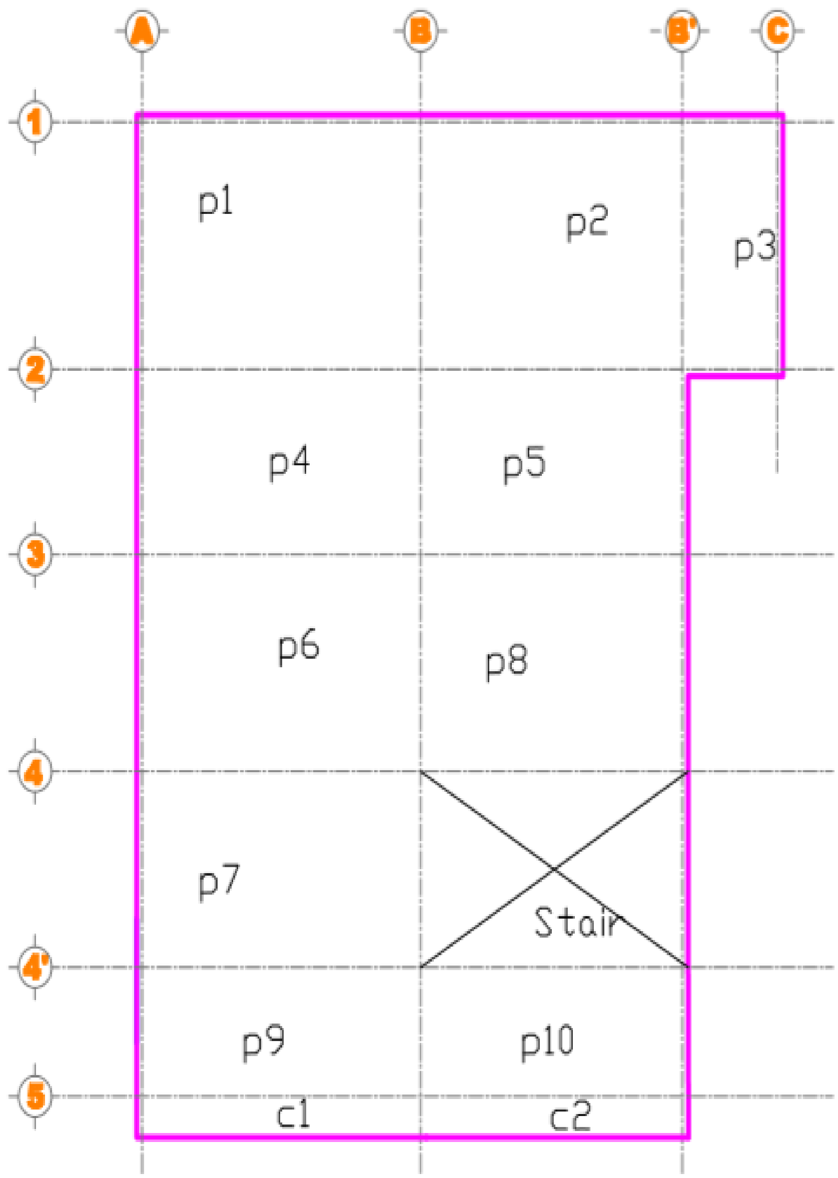


Figure 1 Wind Load Analysis of Roof

Wind produces dynamic loads on a structure at highly variable magnitudes. The variation in pressures at different locations on a building is complex to the point that pressures may become too analytically intensive for precise consideration in design. To simplify the complexity in analysis of wind load different codes provides specifications for wind load by considering basic static pressure zones on a buildings representative of peak loads that are likely to be experienced. Wind forces act directly or indirectly on the internal and external surface of structures. Wind loads fluctuate with time. A wind load produces static, dynamic and aerodynamic effects on structures.

The effect of wind load on the structure depends on many factors such as:

- Wind velocity direction
- The height of the s structure
- Topographic location of the structure
- Shape of the structure
- Terrain category
- The roughness of the surrounding

External wind pressure (Wex) is the wind pressure acting on the external surfaces of a structure and internal wind pressure(Wi) is the wind pressure acting on the internal surfaces of a structure.

$Wex = qref * Ce(Ze) * Cpe$

$Wi = qref * Ce(Zi) * Cpi$

Wnet = We-Wi = $q_p(\mathrm{Z})\mathrm{C_P}e - q_p(\mathrm{Z})\mathrm{C_P}\mathrm{i}$

Wnet = $q_p(\mathrm{Z})(\mathrm{C_{Pe}} - \mathrm{C_{Pi}})$

Where :Wnet=Net wind pres

sure load

We =External wind Pressure

Wi=Internal wind Pressure

$\mathrm{q_p}(Z)$= is the Peck velocity Pressure

Cpe=is the external pressure coefficient

Cpi= is the internal pressure coefficient

$qref$ = Reference mean wind pressure

Ce(z)= Exposure coefficient that takes into account the influence of terrain roughness.

Cpe = External wind pressure coefficient from Appendix A of EBCS - 1/19

Cpi = Internal wind pressure coefficient from ES- EN 1991-1-4:2015 section

Necessary information of the building

Maximum height of building(height of roof) above ground level =19m

Roof type _ flat roof

Location of the area –Addis Ababa

Altitude of the area above mean sea level =2355m

External wind Pressure on the roof

The wind pressure acting on the external surface of a structure is function of the peck velocity Pressure In order to determine the contact pressure on the outside of a structure or part of a structure, the peck velocity Pressure, q_p of the wind must be multiplied by an external pressure coefficient. The external pressure coefficient is accounts for the variation in dynamic pressure on different zones of the structure due to its geometry, area and proximity to other structures. Since our roof is flat roof.

Wex=q_p(Z)C_{Pe}……………………….…EBCS-1, 2015

Cpe- is the external pressure coefficient

Determination of q_p(Z) =Peck velocity Pressure

$q_p(Z) = [1 + 7I_v(z)] * 0.5 * \rho_{air} * V_m^2(z)$

Where: $I_v(z)$ - Turbulence intensity

ρair- Air density

V_m (z) -Mean wind velocity

Step 1: determination of Turbulence intensity ($I_v(z)$)

$I_v(z) = \frac{\sigma v}{V_m(Z)}$ for Zmin< Z <Zmax

$I_v(z) = I_v(Zmin)$ for Z <Zmax

Where:-V_m (z) -Mean wind velocity

σv − Standard deviation of Turbulence

from table 1below we take Z_0=1, Z_{min}=10

Category	Terrain classification	Zo(m)	Zmin
0	Sea or coastal area exposed to the open sea	0.003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0.01	2
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle height	0.05	4

III	Area with regular cover of vegetation or buildings or withIsolatedobstacles with separations of maximum 20 obstacle heights(suchas villages, suburban terrain, permanent forest)	0.3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1	10

Table 1Terrain Categories and Related Parameters

We use this formula $I_v(z)=\frac{\sigma v}{V_m(Z)}$, b/c Zmin< Z <Zmax 10<20<200

Step 2: determination of Mean wind velocity V_m (z)

$$V_m(z) = C_r(Z)C_o(Z)\, V_b$$

Where: $C_r(Z)$-Roughness factor

$C_o(Z)$- Terrian Topography

V_b- Basic wind velocity

$V_b = C_{dir}C_{season}V_{b,o}$,

Where, $\mathbf{C_{dir}}$ =Directional factor =1(ES EN 1991-1-4:2015 recommended)

C_{se}=Seasonal factor =1(ES EN 1991-1-4:2015 recommended)

$\mathbf{V_{b,o}}$=Fundamental value of basic wind velocity =22 m/sfor Ethiopia

Therefore,$\mathbf{V_b = C_{dir}C_{season}V_{b,o}}$=1*1*22m/s=22m/s

Step 3: determination of Roughness factor ($\mathbf{C_r(Z)}$)

The roughness coefficient Cr (z), accounts for the variability of mean wind velocity due to the height of the structure above ground level and the roughness of the terrain. It is defined by the logarithmic relationship:

$\mathbf{C_r(Z)}$= $\mathbf{K_r}$ln ($\mathbf{\frac{Z}{Zo}}$), for Zmin≤ Z ≤ Zmax

Where,$\mathbf{K_r}$-Terrain factor depending on the roughness length Z_0 calculated using$\mathbf{K_r = 0.19(\frac{Z_O}{Z_{O,II}})^{0.07}}$,

$Z_{o,II}$-Roughness length for category III=0.05m

$K_r = 0.19(\frac{Z_O}{Z_{O,II}})^{0.07}$=0.19(1/0.05)$^{0.07}$=0.2343

h=19m

C_r(Z)=0.2343ln(19/1)=0.69

Step 4: determination of Terrain orography ($C_o(Z)$):Based on EBCS-1, 2015 Art 3.8.4 (Page 58) defined by

Ct (z) = 1 + 2sΦ for 0.05 < Φ 0.3

Ct (z) = 1 + 0.6s for Φ > 0.3

Where: Φ - is the up wind slope H/Lu in the wind direction.

S -is the factor to be obtained by interpolation from the values of s = 1 at crest of hills, ridge or escarpment and the values of S = 0 at boundary of topography affected zone.

Ct (z) = 1 for Φ< 0.05

After finding terrain orography we can calculate Vm

$V_m(z) = C_r(Z)C_o(Z)$ V_b=0.69*1*22m/s=15.15m/s

Step 5: determination of Standard deviation of Turbulence (σv)

σv=$K_r V_b K_1$

Where, K_r-Terrian factor=0.2343from above

K_1-Turbulence Factor=1 recommended value

V_b-basic wind velocity=22m/s

$$\sigma v = 0.2343 * \frac{22m}{s} * 1 = 5.06m/s$$

$I_v(z)=\frac{\sigma v}{V_m(Z)} = \frac{5.06}{15.18} = 0.33$

Air density

Is affected by altitude and depends on the temperature and pressure to be expected in the region during windstorms. This is donated by ρair in kg/m3. A temperature of 20˚C has been selected as appropriate for Ethiopia and the variation of mean atmospheric pressure with altitude is given in table 3.1 of EBCS 1,2015 as below.

Table 2 Air density and its altitude (m) above sea level.

Site altitude above sea level (m)	Air density (kg/m^{3})
0	1.20
500	1.12
1000	1.06
1500	1.00

2000	0.94

Therefore the altitude of Adiss Ababa is 2355m which is greater than 2000m.So we take the maximum value of air density 0.94 from the above table.

Now we can calculate the peack velocity

$q_p(Z) = [1 + 7I_v(z)]*0.5* \rho_{air} * V^2_m (z)=(1+7*0.33)(0.5*0.94*(15.18)^2)$

$=358.5N/m^2$

$q_p(Z)=0.358KN/m^2$

<u>Calculation for external wind pressure coefficient (Cpe)</u>

<u>Geometric data of the building</u>

The height of building above the ground level is 18m.

The building roof is Flat roof from the given.

Width and length of the buildings is 14.88m and 11.6m respectively taken from floor plan.

Height of parapet of the roof=1m

14.88m<11.6m (h<b), and from EBCS-1 Section A.2.2it is stated that “for building whose height, h is less than b, shall be considered to be one part

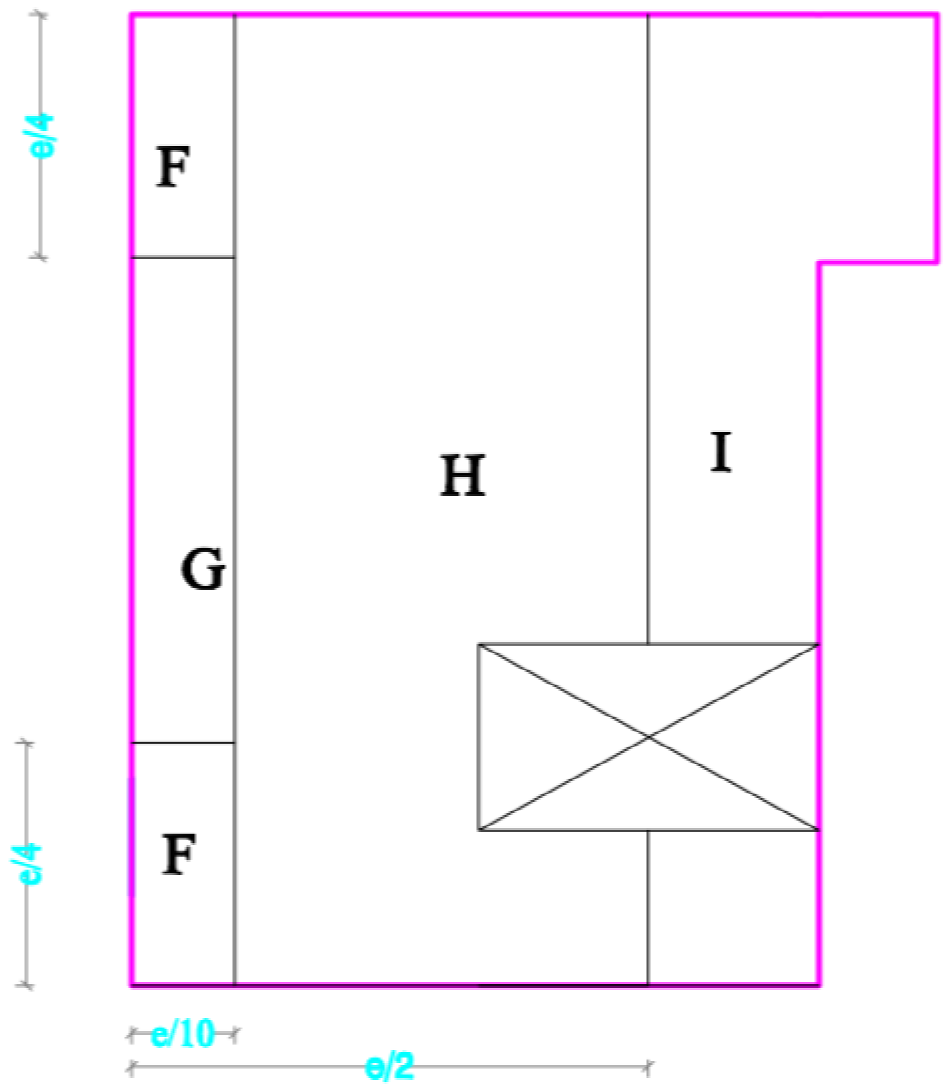


Figure 2 Area of wind zone

e= b or 2h whichever is the smaller

$$\text{Then, } e \leq \min\begin{cases} b = 14.88\text{m} \\ 2h = 2 * 11.6 = 23.2\text{m} \end{cases} \Rightarrow e = 14.88\text{m}$$

<u>Areas of each roof zone</u>

e/4 = 14.88/4 = 3.72m

e/10 = 14.88/10 = 1.488m

e/2 = 14.88/2 = 7.44m

b=14.88m

d=18.25m

Area F = 11.07m^2

Area G=11.07m^2

Area H= 81.55m^2

Area I= 36.11m^2

Determine external wind pressure coefficient

Height of parapet wall hp=1m

Height of the building h = 19m

For $\frac{hp}{h} = \frac{1}{19} = 0.0526$

Then using EBCS-1, 2015 Table 7.2 (Page 33) to determine external pressure coefficient as in a table, but for roofs with parapets, linear interpolation may be used for intermediate values of hp/h. Values of $C_{pe}$are given in Table 7.2 on EBCS-1 (Page 33), for different values of hp/h and for zone area above 10 m^2 and zone area below 1 m^2.For Zone F, G, H, I we use C_{pe}10 because they have area more than 10 m^2. The corresponding values are taken from the Table 7.2 using interpolation to determine the value of the intermediate values.

External wind pressure coefficients

Zone	F		G		H		I	
Area	11.07m^2		11.07m^2		81.55m^2		36.11m^2	
d/h	C_{pe},10	C_{pe},1	C_{pe},10	C_{pe},1	C_{pe},10	C_{pe},1	C_{pe},10	C_{pe},1
	-1.4	-2	-0.9	-1.6	-0.7	-1.2	+0.2	-0.2
Cpe	-1.4		-0.9		-0.7		0.2	

Table 3 The external pressure coefficient (Cpe) values.

Internal wind pressure coefficients

For closed building with internal partitions and opening windows the extreme values are

$$c_{pi} = 0.8 \text{ for pressure}$$

$$c_{pi} = -0.5 \text{ for suction}$$

Zone	F		G		H		I	
Area	11.07m^2		11.07m^2		81.55m^2		36.11m^2	
q_p(Zi)	0.358		0.358		0.358		0.358	
Cpi(+ve) for pressure	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Cpi(-ve) for suction	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5

Cpe	-1.4	0	-0.9	0	-0.7	0	0.2	-0.2
$W_{net}=q_p(Zi)*(Cpe-Cpi_{(-ve)})$	-0.322	0.179	-0.14	0.179	-0.07	0.179	0.25	-0.11
$W_{net}=q_p(Zi)*(Cpe-Cpi_{(+ve)})$	-0.787	-0.286	-0.608	-0.286	-0.537	-0.286	-0.215	-0.358

Table 4 Net wind pressure

Therefore $W_{net(+ve)}$= +0.179

$W_{net(-ve)}$= -0.787

Depth deetermination

- ✓ $\frac{l}{d} = K[11 + 1.5\sqrt{fck}\frac{\rho o}{\rho} + 3.2\sqrt{fck}\left(\frac{\rho o}{\rho} - 1\right)^{\frac{3}{2}}] \quad if\ \rho \leq \rho_o$
- ✓ $\frac{l}{d} = K[11 + 1.5\sqrt{fck}\frac{\rho o}{\rho - \rho'} + \frac{1}{12}\sqrt{fck}\sqrt{\frac{\rho'}{\rho}} \quad if\ \rho > \rho_o$

Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.

Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken. Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory.

Taking L/d=28.76 for end span

L/d = 33.2 for interior span from ES EN 1992:2015 9table 7.4N)

L/d = 8.85 for cantilever

Where L=effective length of two-way slab (the shorter length Lx)

d= effective depth but those value is for steel grade 500,

Depth of slab for roof

Dprovide=150mm

Panel	Ly	Lx	Ly/lx	Slab type	Support Type	L/d	D	Dcalc
P-1	5	3.6	1.4	Two way	End	28.76	125.2	150.2
P-2	4.7	3.6	1.3	Two way	End	28.76	125.2	150.2
P-3	3.6	1.7	2.2	one way	End	28.76	59.1	84.1

P-4	5	2.7	1.9	Two way	Interior	33.2	83.85	108.85
P-5	4.7	2.7	1.74	Two way	Interior	33.2	83.85	108.85
P-6	5	3.15	1.6	Two way	Interior	33.2	94.8	117.8
P-7	4.7	3.15	1.5	Two way	Interior	33.2	94.8	117.8
P-8	5	2.85	1.8	Two way	Interior	33.2	85.8	110.8
P-9	5	1.88	2.7	one way	Interior	33.2	56.62	81.62
P-10	4.7	1.88	2.5	one way	Interior	33.2	56.62	81.62
C-1	5	0.6	8.3	one way	Cantilever	8.85	67.79	92.79
C-2	4.7	0.6	7.8	one way	Cantilever	8.85	67.79	92.79

Table 5 Depth of slab for roof

2.1 Roof Load Combinations

Dead load, live load and wind loads on panels were converted to equivalent uniformly distributed loads. All loads are calculated depending on the service of the roof slabs and self weight. The design loads are factored according to the following formula.

Pd = 1.35LL+1.5LL+0.9WL

Where: Pd - design load

DL - total dead load on slab

LL - total live load on slab

WL - total wind load on slab

Live load

The live load of the roof is determined by assuming the roof is not accessible except for normal maintenance, repair, painting & minor repairs. So it is categorized in Category of H according to EBCS-1, 1995 (Page 49). Therefore, the recommended value ass it is set by the National Annex is Live load from code qk = 0.4KN/m^2 uniformly distributed load or Qk =1KN concentrated load (as our roof is flat).

Dead load(DL)

Usually the major part of the dead load is the weight of the structure itself. It will comprise the forces due to the static weights of the structure as well as attachment to the structures.

Wind load (W.L)

Action of the wind loads on structures is represented either as a wind pressure or as a wind force. The action of wind pressure on a structure is assumed to act normal to the surface except otherwise specified; e.g. for tangential friction forces. Wind pressure on the structure may be external wind pressure or internal wind pressure. The magnitude of the wind load depends on the roof shape, wind direction and location of the building. Appropriate fasteners and holding down bolts or anchors must be used.

STEP 1: Load determination

1. Dead Load

150mm RC solid slab=0.15*25 = 3.75 kN/m^2

30mm thick cement screed =0.03*23 = 0.69 kN/m^2

Ceramic tiles=0.02*21=0.42 kN/m^2

DL=4.86kN/m^2

2. Live Load

The roof is not accessible except for normal maintenance, repair, painting and minor repairs. The characteristic values, qk, and Qkare given in ES EN1991-1-1.

It is category H roof, hence

The distributed live load, qk=0.5KN/m^2

The concentrated live load, Qk=1KN

3. Wind Load

Wind Load (-ve) = -0.787KN/m^2

Wind Load (+ve) = 0.179KN/m^2

STEP 2: Design Load combination

Combination -1 Dead load + Live load

Pd= 1.35DL + 1.5LL = 1.35(4.86) + 1.5(0.5) = 7.311 KN/m^2

Combination -2 Dead load + Wind load

Pd = 1.35DL + 1.5WL= 1.35(4.86) + 1.5(-0.787) = 5.38 KN/m^2

Combination -3 Dead load - Wind load

Pd = 1.35DL - 1.50WL = 1.35(4.86) - 1.5(-0.787) = 7.74KN/m^2

Combination -4 Dead load + live load + Wind load

$Pd = 1.35DL + 1.50LL + 0.9WL= 1.35(4.86) + 1.5(0.5) + 0.9(-0.787) = 6.6027\ KN/m^2$

Combination -5 Dead load + live load - Wind load

$Pd = 1.35DL + 1.50LL + 0.9WL= 1.35(4.86) + 1.5(0.5) - 0.9(-0.787) = 8.02KN/m^2$

The maximum design load combination is $8.02KN/m^2$.

2.2 Moment and Shear force Analysis

Moment Calculation for two way slab using coefficient method

The first stage of design is to determine support and span moments for all panels.

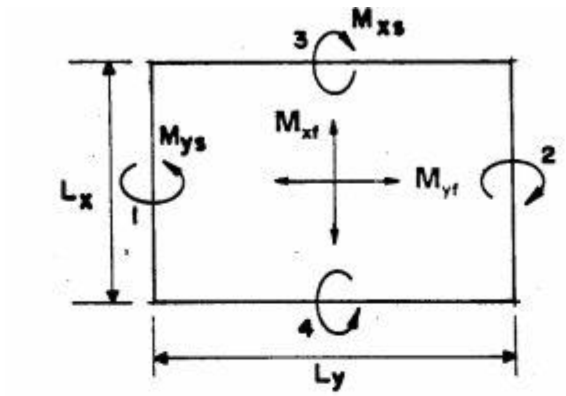
The support and span moments are calculated as

M_i=ßsiPdL_x^2ES-EN 1992

Where M_i=Design moment per unite width of reference

P_d=Uniformly Distributed Design Load

ßsi=Coefficient given in ES-EN



M_{XS}= ßsi$P_d L_x^2$

M_{XF}= ßsi$P_d L_x^2$

M_{YS}= ßsP_diL_x^2

M_{YF}= ßsi $P_d L_x^2$

- For panel 1

M_{XS}=ßsi$P_d L_x^2$	M_{YS}=ßsi$P_d L_x^2$
$M_{XS}=0.074*8*3.6^2$	$M_{YS}=0.045*8*3.6^2$
Mxs=7.67	M_{YS}=4.66

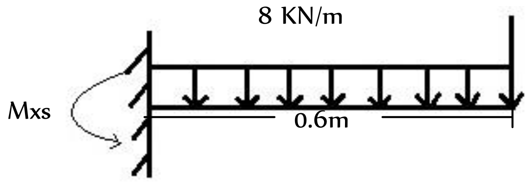
M_{XF}=ßsi$P_d L_x^2$	M_{YF}=ßsi$P_d L_x^2$
$M_{XF}=0.055*8*3.6^2$	$M_{YF}=0.03*8*3.6^2$

M_{XF}=5.7 M_{YF}= 3.52

Moment Calculation for one way & cantilever slabs.

Cantiliver1

Pd=11.3 KN/m



$$Mxs=8\ KN * (\frac{0.6m}{2})^2$$

=1.44KN.m

In a tabular form

Panel	Support Type	Ly	Lx	Ly/ lx	pd	Axs	αxf	αys	αyf	Mxs	Mxf	Mys	Myf
P-1	End	5	3.6	1.4	8KN	0.074	0.055	0.045	0.034	7.67	5.7	4.66	3.52
P-2	End	4. 7	3.6	1.3	8KN	0.063	0.047	0.037	0.028	6.53	4.87	3.84	2.9
P-3	End	3. 6	1.7	2.2	8KN					11.56			
P-4	Interior	5	2.7	1.9	8KN	0.065	0.049	0.037	0.028	3.79	2.85	2.15	1.63
P-5	Interior	4. 7	2.7	1.7 4	8KN	0.068	0.047	0.037	0.028	3.96	2.74	2.15	1.63
P-6	Interior	5	3.15	1.6	8KN	0.06	0.045	0.037	0.028	4.76	3.57	2.93	2.22
P-7	Interior	4. 7	3.15	1.5	8KN	0.058	0.043	0.037	0.028	4.6	3.41	2.93	1.81
P-8	Interior	5	2.85	1.8	8KN	0.064	0.048	0.037	0.028	4.15	1.94	2.4	

P-9	Interior	5	1.88	2.7	8KN							14.135	
P-10	Interior	4.7	1.88	2.5	8KN							14.135	
C-1	Cantilever	5	0.6	8.3	8KN	0.53						1.44	
C-2	Cantilever	4.7	0.6	7.8	8KN							1.44	

Table 6 Moments acting in Roof

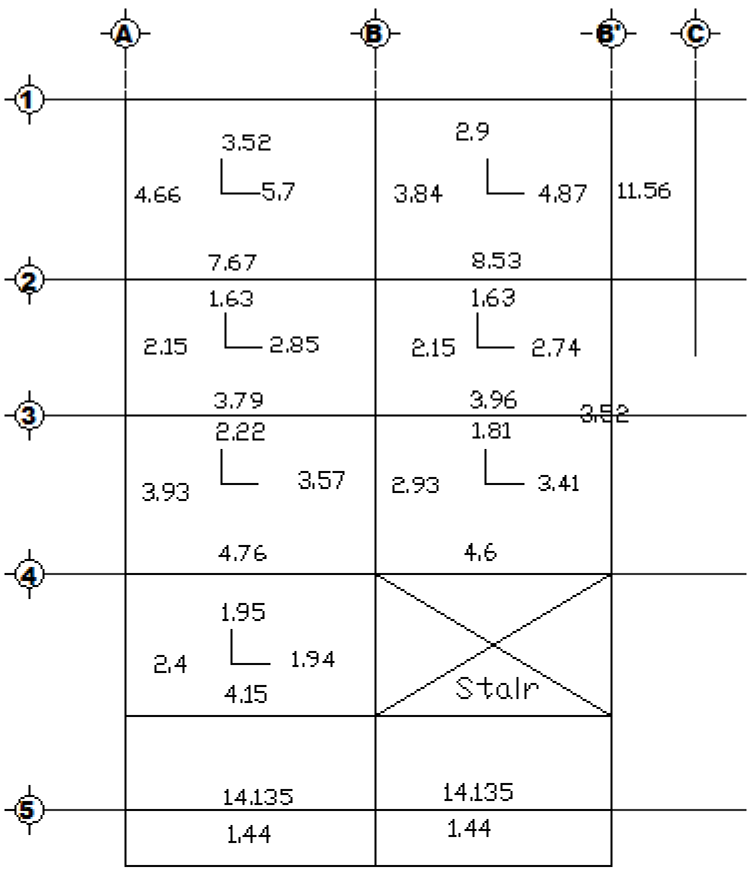


Figure 3 moment analysis for roof

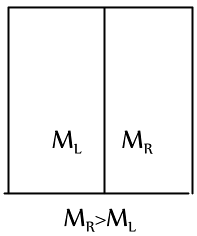
2.3 Adjustment of moments

Support moment adjustment

There are two cases.

Case (1) If $\frac{M_R-M_L}{M_R}*100<20\%$ then $Md=\frac{M_R+M_L}{2}, M_R>M_L$

Case (2) If $\frac{M_R-M_L}{M_R}*100>20\%$ then Distribute unbalanced moment $M_D=M_R-M_L$ based on relative stiffness.... $M_R>M_L$



$M_d=M_R-\frac{K_R}{K_R+K_L}*\Delta M$..............................Considering right

$M_d=M_R+\frac{K_R}{K_R+K_L}*\Delta M$..............................Considering Left

Note that If adjustment is between two-way and cantilever, then $M_d=M_{max}$

Span moment adjustment

* If the support moment is decreased while carrying on moment distribution of a balanced support moment the span moment Mxf and Myf are the increased to allow for the change of support moment. This increase is calculated being equal to the change of the support moment multiplied by the factor given in EBCS. If the support moment increased, no adjustment shall be made to the span moment.

Support adjustments between panel 1 and panel 2

$M_R=4.66$ (Panel), $M_L=3.84$(Panel 2)

$\frac{4.66-3.84}{4.66}*100=17.59<20\%$ [use average moment]

$\therefore Md=\frac{4.66+3.84}{2}=4.25KN.m$

For panel 1 & panel 4

$\frac{M_R-M_L}{M_R}*100<20\% = \frac{7.69-3.79}{7.69}*100 = 50.7\% > 20\%$

Use method 2

$K_{AB}=\frac{3}{4Lx}=\frac{3}{4*3.6}=0.21$

$K_{BC}=\frac{1}{Lx}=\frac{1}{2.7}=0.37$

SUPPORT MOMENT ADJUSTMENT ROOF

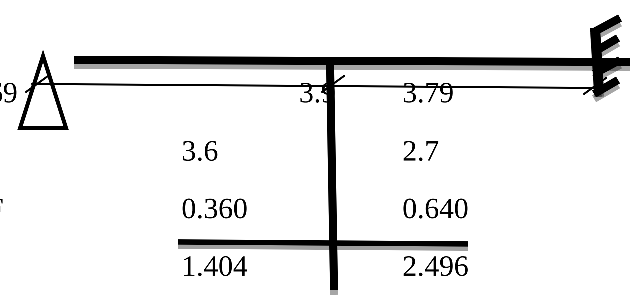
PANEL	MTD 1	MTD 2	MAX

P1&P2	4.25		
P2&P3		7.7	
P1&P4		6.29	
P4&P5		2.97	
P4&P6	3.46		
P6&P7	4.68		
P2&P5	2.67		
P6&P8	9.37		
P9&P10	14.13		
P5&P7	9.14		
P9&C1			14.13
P10&C2			14.1
C1&C2			1.44

$DF_{AB}=\frac{KAB}{KAB+KBC}=\frac{0.21}{0.21+0.37}=0.36$

$DF_{BC}=\frac{KBc}{KAB+KB\ C}=\frac{0.37}{0.21+0.37}=0.64$

B



7.69 3.5 3.79

A 3.6 2.7 *C*

DF 0.360 0.640

1.404 2.496

6.286 6.29

Span moment adjustment

PANEL	Mxs	Mxf	Madjust	Mxfadjusted=Mxs+ Mxf- Madjust
P1	7.67	5.7	10.97	2.40
P2	6.53	4.87	7.7	3.70
P4	2.15	2.85	4.96	0.04
P5	3.96	2.74	2.97	3.73
P6	4.76	3.57	3.46	4.87
P7	4.6	3.41	2.67	5.34
P8	4.15	1.94	2.67	3.42

PANEL	Msy	MSF	Madjust	Mxfadjusted
P1	4.66	3.52	4.96	3.22
P2	3.84	2.9	4.25	2.49
P4	3.79	1.63	2.97	2.45
P5	2.15	1.63	2.97	0.81
P6	2.93	2.22	4.68	0.47
P7	2.93	2.22	2.67	2.48
P8	2.4	1.81	2.67	1.54

Table 7 Span moment adjustment for roof

Between Cantilever and slab

M_R=14.135 M_L=1.44

Since p9 is cantilever and P1 is two-way slab take the maximum .

Thus M_d=14.135KN.m

2.3.1 Shear force analysis

Vsx=βvxPdLx

Vsy=βvyPdLx

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvxc	Bvxd	βvyc	βvyd	Vsxc	Vsxd	Vsyc	Vsyd
P1	EndSpan	3.6	5	1.39	8	0.52	0.35	0.4	0.26	14.98	10.08	11.52	7.488
P2	EndSpan	3.6	4.7	1.31	8	0.47	0.31	0.36		13.54	8.928	10.37	0
P3	EndSpan	1.7	3.6	2.12	8	0.59	0.38	0.36		8.02	5.168	4.90	0
P4	Interior	2.7	5	1.85	8	0.51		0.36	0.24	11.02	0	7.78	5.184
P5	Interior	2.7	4.7	1.74	8	0.49		0.36	0.24	10.58	0	7.78	5.184
P6	Interior	3.15	5	1.59	8	0.36		0.24		9.07	0	6.05	0
P7	Interior	3.15	4.7	1.49	8	0.47		0.36	0.24	11.84	0	9.07	6.048
p8	Interior	2.85	5	1.75	8	0.504		0.36	0.24	11.49	0	8.21	5.472
p9	Interior	1.88	5	2.66	8	0.52		0.36	0.24	7.82	0	5.41	3.6096
p10	Interior	1.88	4.7	2.50	8	0.52		0.36	0.24	7.82	0	5.41	3.6096
PANEL	TYPE	Lx	Ly	Ly/Lx	n (Pd)	WL							
C 1	Cantilever	0.6	5	8.33	8	4.80							
C2	Cantilever	0.6	4.7	7.83	8	4.80							

Table 8 Shear force analysis for roof

2.3.2 Reinforcement Calculation

panel	moment		Ast=Msd/Z*fyd	ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	4.96	215.00	10	350.00	350	ø	10	c/c	350
	M_{xf}=	4.25	215.00	10	350.00	350	ø	10	c/c	350
	M_{ys}=	8.3	215.00	10	350.00	350	ø	10	c/c	350
	M_{yf}=	3.22	215.00	10	350.00	350	ø	10	c/c	350
P-2	M_{xs} =	4.76	215.00	10	350.00	350	ø	10	c/c	350
	M_{xf}=	7.7	215.00	10	350.00	350	ø	10	c/c	350
	M_{ys}=	3.7	215.00	10	350.00	350	ø	10	c/c	350
	M_{yf}=	2.49	215.00	10	350.00	350	ø	10	c/c	350
P-3	M_{xs} =	3.79	215.00	10	350.00	350	ø	10	c/c	350
	M_{xf}=	7.7	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	8.73	215.00	10	365.12	215	ø	10	c/c	215
	M_{yf}=	8.73	215.00	10	365.12	350	ø	10	c/c	350
P-4	M_{xs} =	3.46	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	2.97	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	0.04	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	2.45	215.00	10	365.12	350	ø	10	c/c	350
P-5	M_{xs} =		215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	4.76	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	2.94	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	3.73	215.00	10	365.12	350	ø	10	c/c	350
P-6	M_{xs} =	0.81	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=		215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	3.46	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	4.68	215.00	10	365.12	350	ø	10	c/c	350
P-7	M_{xs} =	4.87	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	0.47	215.00	10	365.12	250	ø	10	c/c	250
	M_{ys}=	4.68	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	2.67	215.00	10	365.12	350	ø	10	c/c	350
P-8	M_{xs} =	5.34	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	2.48	215.00	10	365.12	350	ø	10	c/c	350

	M_{ys}=	9.14	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	2.67	215.00	10	365.12	350	ø	10	c/c	350
P-9	M_{xs} =	3.42	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	1.54	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	9.37	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	9.14	215.00	10	365.12	350	ø	10	c/c	350
P-10	M_{xs} =	14.13	300.91	10	260.87	250	ø	10	c/c	250
	M_{xf}=	14.13	300.91	10	260.87	250	ø	10	c/c	250
	M_{ys}=	9.14	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	3.46	215.00	10	365.12	350	ø	10	c/c	350
C1	M_{xs} =	14.13	215.00	10	260.87	250	ø	10	c/c	250
C2	M_{xs} =	14.13	300.91	10	260.87	250	ø	10	c/c	250

Table 9 Reinforcement calculation for roof

3 Slab Design

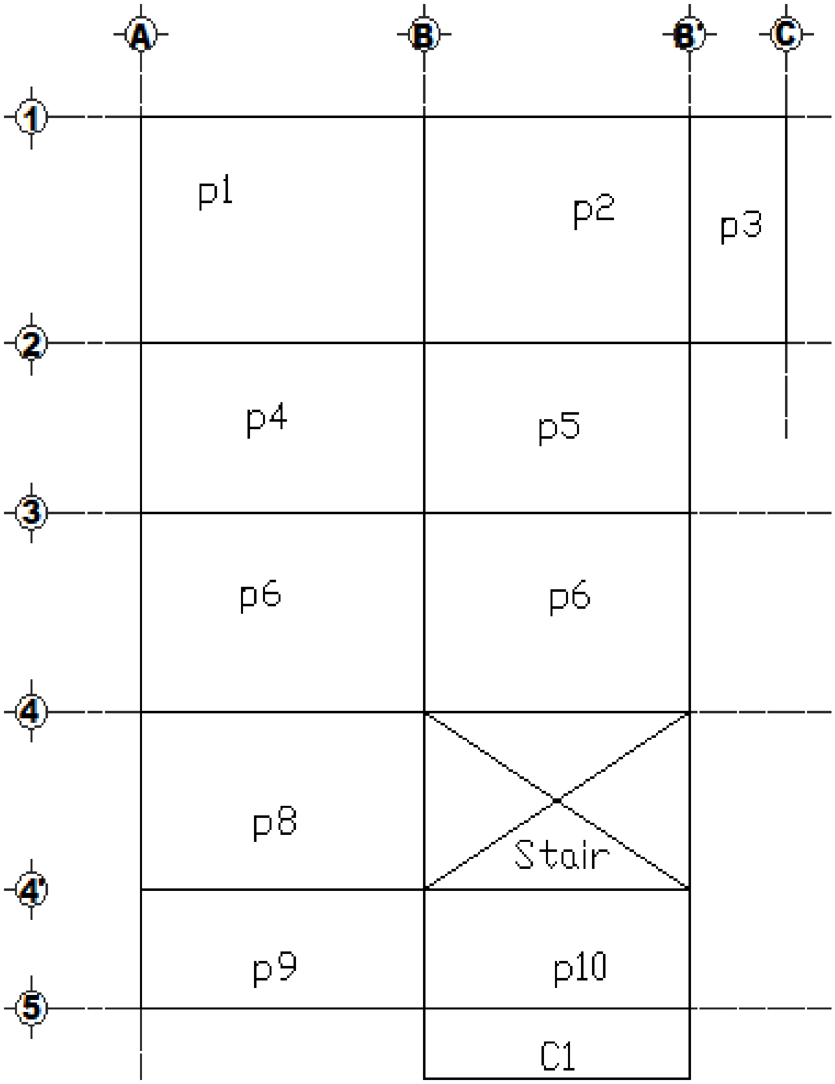


Figure 4First floor layout

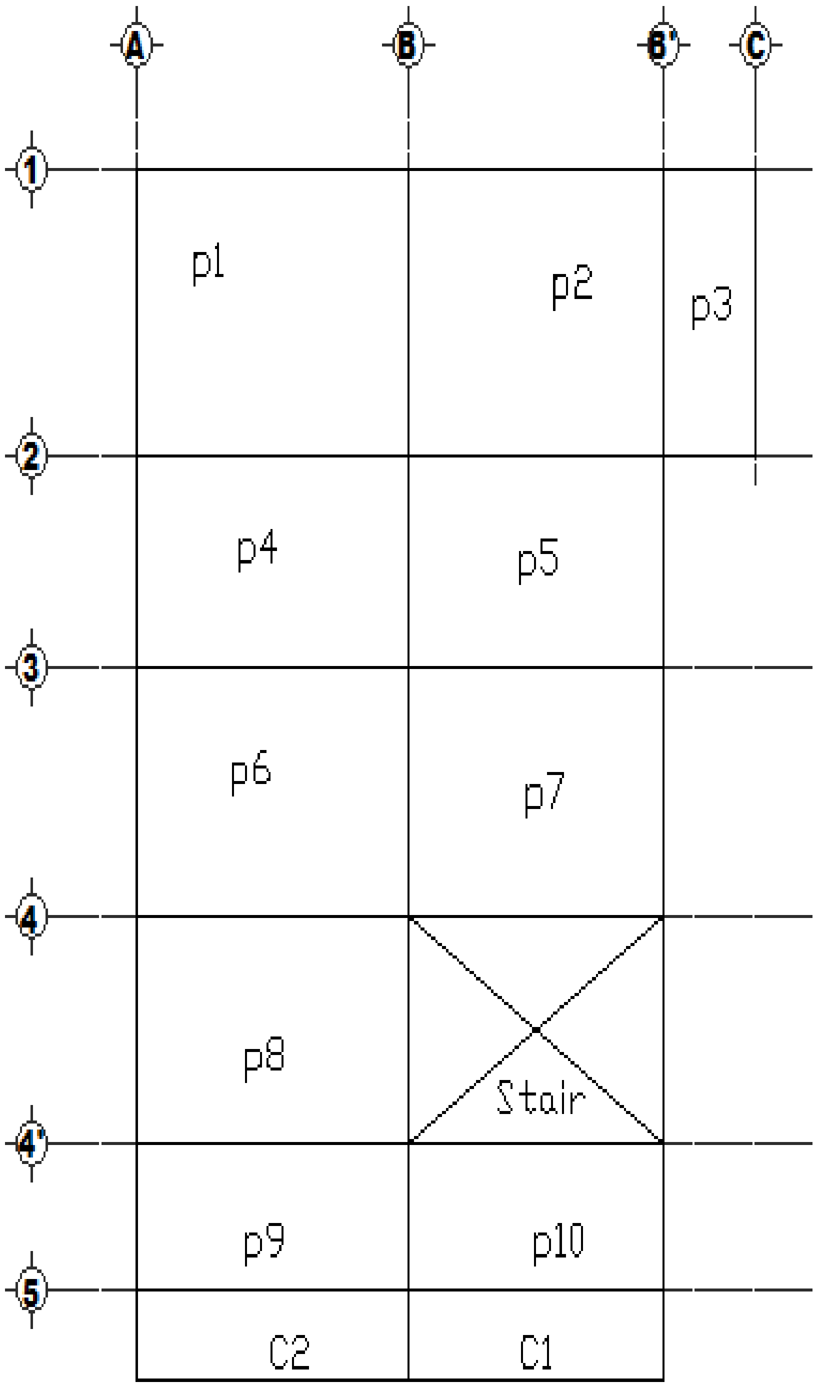


Figure 5 Second floor layout

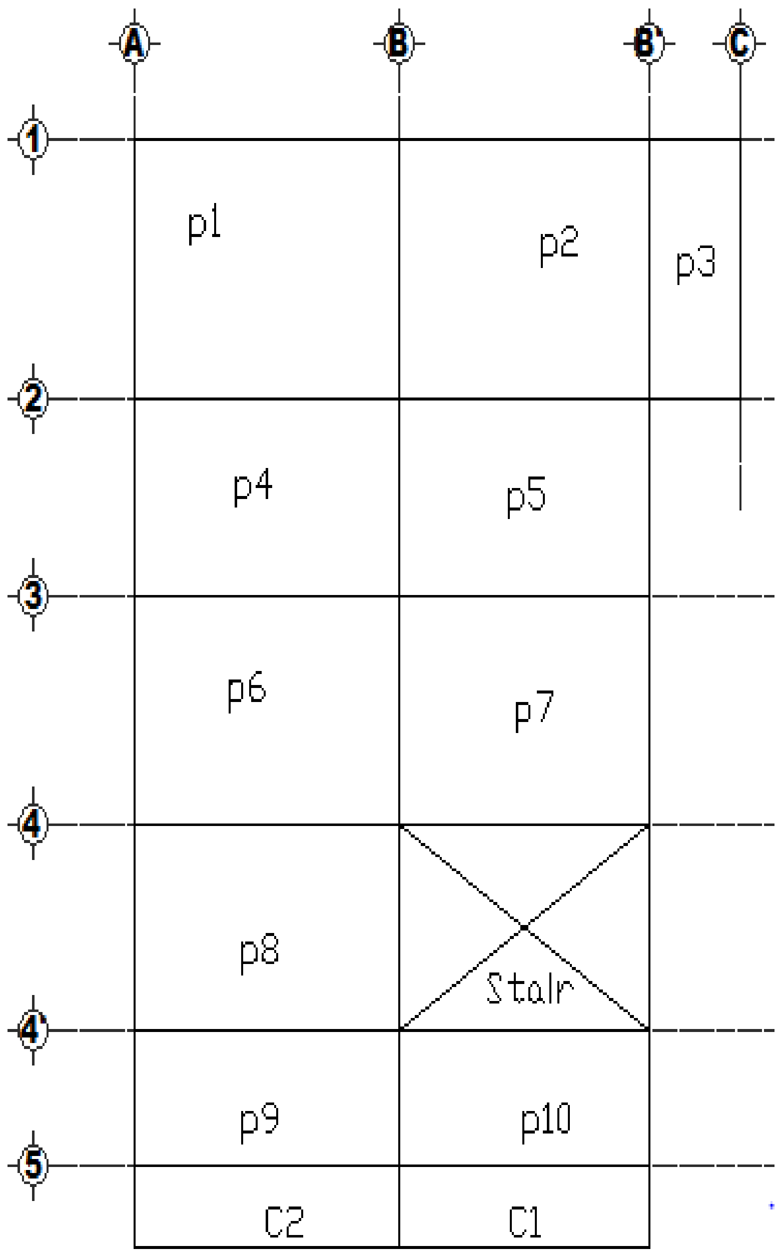


Figure 6 Third floor layout

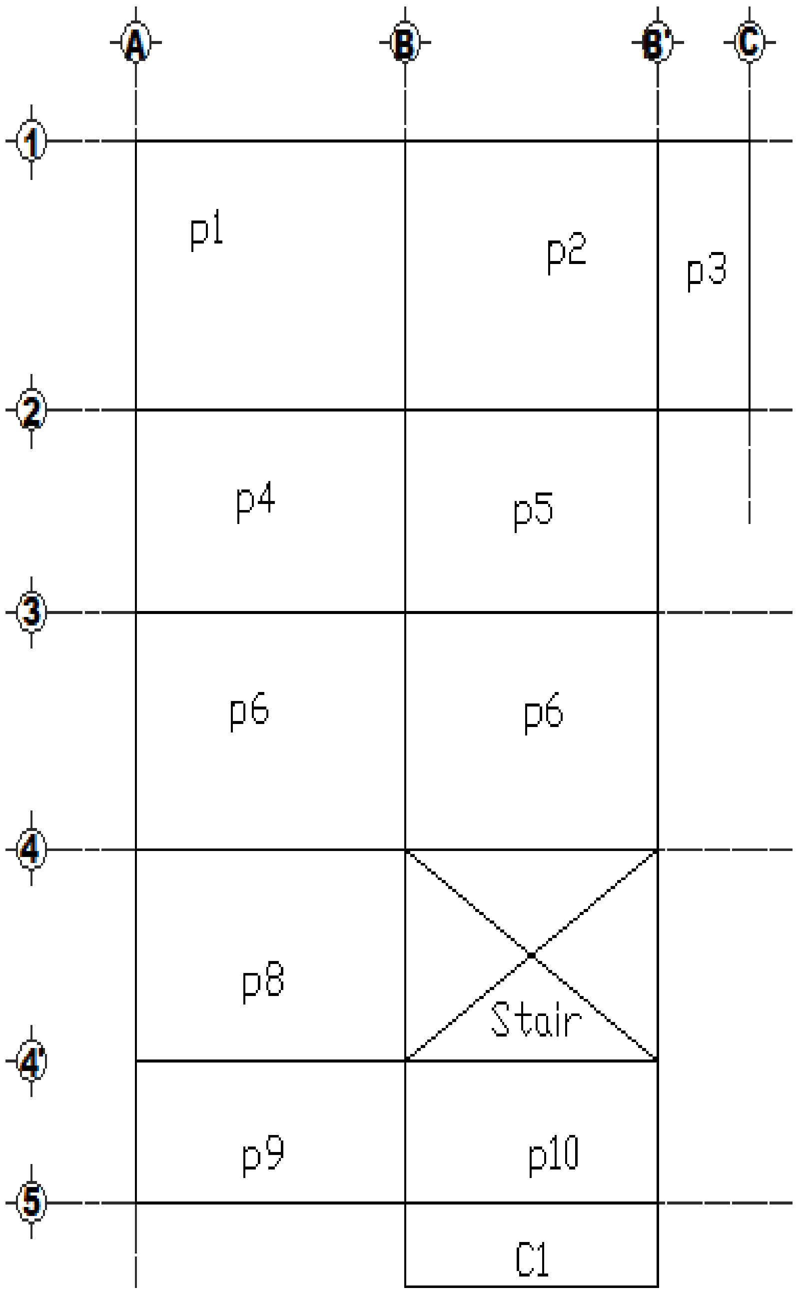


Figure 7 Fourth & Fifth floor layout

3.1 Overall process in design of slabs

- Concrete cover determination
- Depth determination
- Load calculation
- Moment analysis
- Moment adjustment for support & field moments
- Check depth for flexure
- Shear analysis (load transfer to beam)
- Calculate reinforcement for slab

3.2 Depth determination (Deflection criteria)

Concrete cover for slab

A durable structure should satisfy strength and serviceability requirements throughout its design working life. One of the main causes for poor durability is the corrosion of steel reinforcement. Good quality dense concrete and adequate cover are prime requirements in order to produce durable structures. Table 2.5 gives the details of water/cement ratio, minimum cement content for producing good quality concrete to satisfy various exposure classes and Cmin, dur which is the minimum cover to steel from durability considerations.

The minimum cover Cmin to steel should satisfy the code equation:
Cmin = Maximum {Cmin, b; Cmin,dur ; 10 mm}
i. For safe transmission of bond forces, the required cover is Cmin, b as given in Table 4.2 of ES-EN1992.
ii. For separated bars, Cmin, b ≥ bar diameter, φ.
iii. For bundled bars, Cmin, b ≥ equivalent bar diameter, φn.
$\varphi n = \varphi \sqrt{nb} \le 55$ mm. nb = Number of bars in the bundle. nb ≤ 4 for vertical bars in compression and for bars in a lapped joint, nb ≤ 3 for all other cases.
If the nominal maximum size of the aggregate is greater than 32 mm, Cmin, b shouldbe increased by 5 mm. The cover can be reduced if stainless steel bars are used.
For a design life of 50 years, minimum values of Cmin, dur for various classes of exposure are are given as follows in Table 4.4 N of ES-EN1992

X0=10mm, XC1=15mm, XC2/XC3=25mm, XC4=30mm, XD1/XS1=35mm, XD2/XS2=40mm, XD3/XS3=45mm

Exposure condition (according to ES EN 1992-1-1:2013 Table 4.1)

Structural class 2—XC1 – moderate humidity (concrete inside building with moderate or high air humidity)

Step 1: cover determination

<u>Cover for stirrup</u>

C nom=C min + C dev

C min → max (C min; bond or C min: dur or 10)

Concrete grade (according to Pr EN 1992-1-1:2013 ANNEX E (informative) Table E.1N)

Table 10 Recommended limiting values for composition and properties of concrete and minimum cover to steel for durability

Class designation	Maximum w/c ratio	Minimum strength class	Minimum cement content	Minimum air content	C_{dur} (mm)
X0	-	C12/15	-	-	10
XC1	0.65	C20/25	260	-	15(25)
XC2	0.60	C25/30	280	-	25(35)
XC3	0.55	C30/37	280	-	25(35)
XC4	0.50	C30/37	300	-	30(40)
XD1	0.55	C30/37	300	-	35(45)
XD2	0.55	C30/37	320	-	40(50)
XD3	0.45	C35/45	320	-	45(55)
XS1	0.50	C30/37	300	-	35(45)
XS2	0.45	C35/45	320	-	40(50)
XS3	0.45	C35/45	340	-	45(55)
XF1	0.55	C30/37	300	-	
XF2	0.55	C25/30	300	4.0	
XF3	0.50	C30/37	320	4.0	
XF4	0.45	C30/37	340	4.0	
XA1	0.55	C30/37	300	-	
XA2	0.50	C30/37	320	-	
XA3	0.45	C35/45	360	-	

For C-20/25 the exposure class is XC1 which implies dry or permanently wet.

For XC1 min C-20/25 Cmin; b requirements (according to PrEN 1992-1-1:2013 Table 4.2)

Arrangement of bar for separated use

Assume the diameter of the bar is Ø10mm

Cmin;dur requirements (according to EN 1992-1-1:2013 Table 4.4)

For structural class-2 (S2)

For exposure condition –Xc1

• use 10mm

$$C_{min} = \begin{cases} \textbf{Cmin; bond} = \mathbf{10} \\ \textbf{Cmin: dur} = \mathbf{10} \\ \mathbf{10} \end{cases}$$

C min=10

C nom=C min + C fire resistance

C nom =10+10=20mm

Concrete cover for longitudinal bar

Cnom=C min+ C dev

C min → max (C min; bond or C min: dur or 10)

C min → (Cmin; bond =10 or C min: dur=10 or 10)

C min=10

C nom=C min + C fire resistance

C dur (according to EBCS EN 1992-1-1:2013; 4.4.1.3)

The recommended value is 10mm because of cast in suite.

Cnom=Cmin +C fire resistance ==10+10=20mm

The governing C nominal is 20mm

- Depth determination

For panel 1

From EN 1992-1-1-2004 section 7.4.2 depth of the slab is determined

$$\frac{l}{d} = k(11 + 1.5\sqrt{fck} * \frac{\rho_0}{\rho} + 3.2\sqrt{fck} * \left(\frac{\rho_0}{\rho} - 1\right)^{\frac{3}{2}})\ if \rho <= \rho_0$$

$$\frac{l}{d} = k\left(11 + 1.5\sqrt{fck} * \frac{\rho^0}{\rho-\rho'} + \frac{1}{12}\sqrt{fck} * \sqrt{\frac{\rho'}{\rho}}\right) if \rho > \rho_0$$

fck = 25Mpa

Assume the concrete is lightly stressed ρ = 0.5%

$\rho_0 = \sqrt{fck} * 10^{-3} = \sqrt{20} * 10^{-3} = 0.4472\%$

$\rho^0 < \rho$So use

$$\frac{l}{d} = k(11 + 1.5\sqrt{fck} * \frac{\rho_0}{\rho-\rho'} + \frac{1}{12}\sqrt{fck} * \left(\sqrt{\frac{\rho'}{\rho_0}}\right)\ )$$

$$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.4472}{0.5-\rho\prime} + \frac{1}{12}\sqrt{fck} * \left(\sqrt{\frac{\rho\prime}{0.4472}}\right)\ )$$

For exterior slab K=1.3 as EN 1992-1-1 table 7.4

$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.4472}{0.5})$

$\frac{l}{d}$ = 17.7*k*1.25

$\frac{3600}{d}$ = 17.7*1.3*1.25=28.76

d = $\frac{3600}{28.76}$ = 125 mm

D = d + $\frac{Ø}{2}$ + cover

D = 125 + 10/2+ 20 = 150mm

D =150 mm

From Pr EN 1992-1-1-2004 section 7.4.2 depth of the slab is determined

For cantilever K=0.4 as EN 1992-1-1 table 7.4

$$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.4472}{0.5} + 3.2\sqrt{20} * \left(\frac{0}{0.5}\right)^{\frac{1}{2}})$$

$\frac{l}{d}$ = 17.7*k*1.25

$\frac{600}{d}$ = 17.7*0.4*1.25=8.85

d = $\frac{600}{8.85}$ = 67.8mm

D = d + $\frac{Ø}{2}$ + cover

D = 67.8 + 5+ 20 = 92.8mm

D =100 mm

From EN 1992-1-1-2004 section 7.4.2 depth of the slab is determined

For interior span slab K = 1.5 as EN 1992-1-1 table 7.4

$$\frac{l}{d} = k(11 + 1.5\sqrt{fck} * \frac{\rho_0}{\rho} + 3.2\sqrt{fck} * \left(\frac{\rho_0}{\rho} - 1\right)^{\frac{3}{2}})\ if \rho <= \rho_0$$

$$\frac{l}{d} = k\left(11 + 1.5\sqrt{fck} * \frac{\rho^0}{\rho - \rho'} + \frac{1}{12}\sqrt{fck} * \sqrt{\frac{\rho'}{\rho}}\right) if \rho > \rho_0$$

Fck = 30Mpa

Assume the concrete is lightly stressed ρ = 0.5%

$\rho_0 = \sqrt{fck} * 10^{-3} = \sqrt{20} * 10^{-3} = 0.448\%$

$\rho_0 > \rho$So use

$$\frac{l}{d} = k\left(11 + 1.5\sqrt{fck} * \frac{\rho^0}{\rho - \rho'} + \frac{1}{12}\sqrt{fck} * \sqrt{\frac{\rho'}{\rho}}\right) if\ \rho > \rho_0$$

$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.448}{0.5} + 0)$

$\frac{l}{d}$ = 17.7*k*1.25

$\frac{2700}{d}$ = 17.7*1.5*1.25=

d = $\frac{2700}{33.18}$ = 81.37 mm

D = d + $\frac{\emptyset}{2}$ + cover

D = 81.37 +10/2+ 20 = 106.37mm

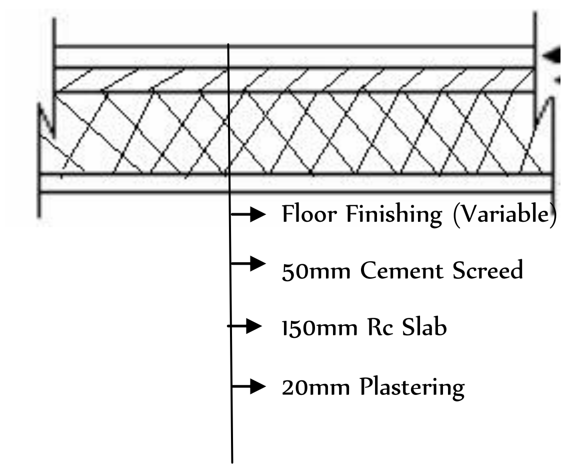
D =110mm

Take maximum depth for panels =150mm

3.2.1 Analysis and Design of Load

Load calculation

Sectional (detail) elevation of floor slabs



Room Functions and their finishing materials

No	Functions	Finishing	Thickness	Unit wt (kN/m^3)
1	Kitchen	Ceramic	2 cm	21
2	Office	PVC tile	2 cm	16
3	Terace	Marble	3 cm	27
4	Corridor	PVC tile	2 cm	16
5	Ch.B.room	Ceramic	2 cm	21
6	Tvroom	PVC tile	2 cm	16

7	Living room	PVC tile	2cm	16
8	Dinningroo m	PVC tile	2 cm	16
9	M.B. room	PVC tile	2cm	16
10	Study room	PVC tile	2 cm	16
11	G.B.room	Ceramic	2 cm	21
12	Toilet	Ceramic	2cm	21
13	Jacuzzi	Ceramic	2 cm	21
14	Family room	PVC tile	2 cm	16

Table 11Room Functions and their finishing materials

- Note: Unit weight for RC slabs ranges from 20-28 KN/m^3, we use the average; $\boldsymbol{\gamma}$=24 KN/m^3 for C-25 Concrete.

Live Load for different functions based on ES-EN 1990

Function	Catagory	Live Load
Kitchen	General	2 KN/m^2
Office	General	2 KN/m^2
Terace	General	2 KN/m^2
Corridor	Corridor	3 KN/m^2
C.B. room	Balconies	3 KN/m^2
TV room	General	2 KN/m^2
Study room	General	2 KN/m^2
Living room	General	2 KN/m^2
Dinning room	General	2 KN/m^2
Family room	General	2 KN/m^2
Toilet	General	2 KN/m^2
M.B. room	General	2 KN/m^2
Jacuzzi	General	2 KN/m^2

Table 12 Live Load for different functions

Dead Load Computation based on function rooms

Function	Material	Thickness (m)	Unit Weight, γ (KN/m^2)	γ_t (KN/m^2)	Total Dead Load (KN/m^2)
Kitchen G.B.room Toilet Jacuzzi	Ceramic	0.02	21	0.42	5.63
	Cement Screed	0.05	23	1.15	
	RC Slab	0.15	24	3.6	
	Plastering & Painting	0.02	23	0.46	
Office TV room Living room Dyning room Family room M.B. room	PVC tile	0.02	16	0.32	5.53
	Cement screed	0.05	23	1.15	
	RC slab	0.15	24	3.6	
	Plastering &painting	0.02	23	0.46	
Terace	Marble	0.03	27	0.81	5.53
	Cement screed	0.05	23	1.15	
	RC slab	0.15	24	3.6	
	Plastering &painting	0.02	23	0.46	

Table 13 Dead Load Computation

Design Dead Load and Live Loads for each Panel

- Since an individual panel might have different purpose (function) and finishing material,
- we might encounter different live load and dead load in a single panel. In such cases we used the maximum value as a governing dead load or live load for that panel.
- PD'=1.35D.L+1.5L.L

First Floor panels

panel	Function	Calculated dead load(KN/m^2)	Governing dead load (KN/m^2)	Live load (KN/m^2)	Governing live load (KN/m^2)	Pd'=1.35D.L+1.5L.L (KN/m^2)

P1	Office	5.53	6.02	2	2	11.13
	Terrace	6.02		2		
P2	Terrace	6.02	6.02	2	2	11.13
P3	Terrace	6.02	6.02	2	2	11.13
P4	Office	5.53	5.63	2	2	10.6
	Toilet	5.63		2		
P5	Corridor	5.53	5.63	5	5	15.1
	Kitchen	5.63		2		
P6	Living &Dinning room	5.53	5,53	2	2	10.47
P7	Corridor	5.53	5.63	2	2	15.1
	Kitchen	5.63		2		
P8	Living	5.53	5.53	2	2	10.47
P9	Living	5.53	5.53	2	2	10.47
P10	Terrace	6.02	6.02	2	2	11.13
C1	Terrace	6.02	6.02	2	2	11.13

Table 14 Design load

3.2.2 Design Moment Analysis

Moment Calculation for two way slab using coefficient method

The first stage of design is to determine support and span moments for all panels.

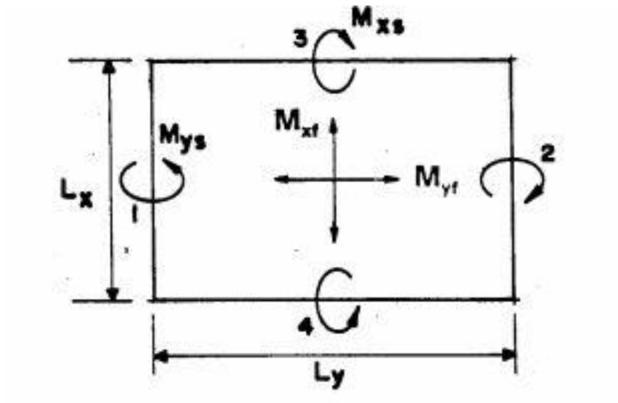
The support and span moments are calculated as

M_i=ßsiPdL$_x^2$ES-EN 1992

Where M_i=Design moment per unite width of reference

P_d=Uniformly Distributed Design Load

ßsi=Coefficient given in ES-EN



M_{XS}= ßsi$P_d L_x^2$ M_{YS}= ßsP_diL_x^2

M_{YF}= ßsi $P_d L_x^2$ M_{XF}= ßsi$P_d L_x^2$

For first floor panel 1

M_{XS}=ßsi$P_d L_x^2$ M_{YS}=ßsi$P_d L_x^2$

M_{XS}=0.074*13.3*3.6^2 M_{YS}=0.045*13.3*3.6^2

Mxs=12.76 M_{YS}=7.76

M_{XF}=ßsi$P_d L_x$ M_{YF}=ßsi$P_d L_x^2$

M_{XF}=0.055*13.3*3.6^2 M_{YF}=0.03*13.3*3.6^2

M_{XF}=9.48 M_{YF}= 5.17

Moment Calculation for one way & cantilever slabs.

Material Data

Thickness of HCB=200mm

Thickness plastering on two side=20mm

Unit weight of HCB=14KN/m^3

Unit weight of plastering=23 KN/m^3

Height of wall=3m

First Floor

Panel 3

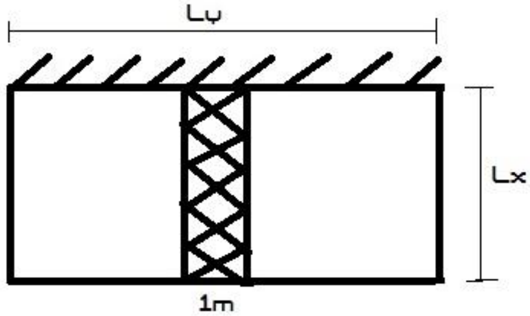
Load from external wall=(H_{pl}* t_{pl}*$\boldsymbol{\gamma}_{pl}$) + (H_{HCB} * t_{HCB}*$\boldsymbol{\gamma}_{HCB}$)

Where: H_{pl},t_{pl}&$\boldsymbol{\gamma}_{pl}$ =Height, Thickness, Unit Weight of Plastering Respectively.

H_{HCB} ,t_{HCB}&$\boldsymbol{\gamma}_{HCB}$= Height, Thickness, Unit Weight of HCB Respectively.

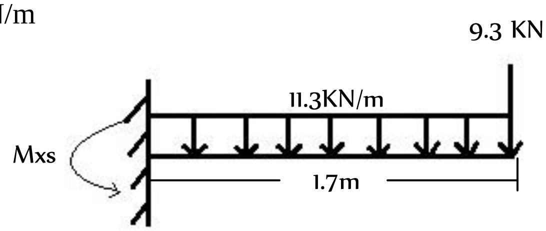
D.L $_{ex}$=(0.2m x 2.85m x 14 KN/m^3)+(0.02m x 2.85m x 23 KN/m^3) =9.3 KN/m

Taking 1m strip of the slab in the shorter direction



D.L$_{ex}$=9.3 kN

Pd=11.3 KN/m



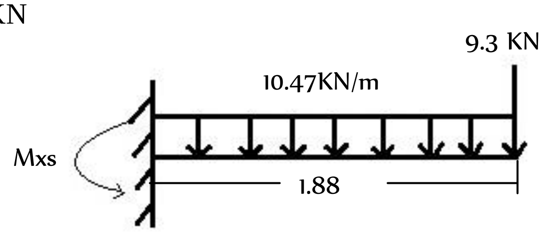
Mxs=9.3KN*1.7m + 15.627 KN *$(\frac{1.7m}{2})^2$

=31.89KN.m

Panel 9

Pd=10.47 KN/m

D.L$_{ex}$=9.3 KN



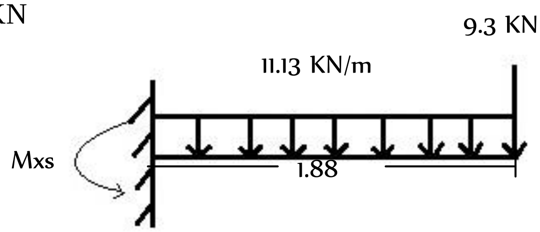
Mxs=9.3KN*1.88m + 10.47 KN *$(\frac{1.88m}{2})^2$

=36KN.m

Panel 10

Pd=11.13 KN/m

D.L_{ex}=9.3 KN



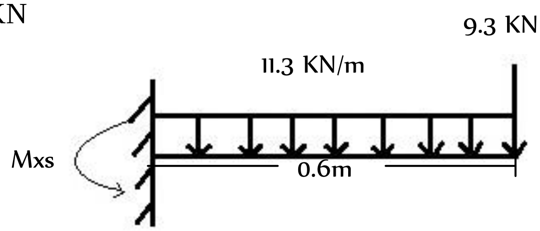
Mxs=9.3KN*1.88m + 11.13 KN *$\left(\frac{1.88m}{2}\right)^2$

=37.15KN.m

Cantiliver1

Pd=11.3 KN/m

D.L_{ex}=9.3 KN



Mxs=9.3KN*0.6m + 11.3 KN *$\left(\frac{0.6m}{2}\right)^2$ =7.6KN.m

In a tabular form

First floor

PANEL		Lx	Ly	Ly/Lx	n(pd)	αxs	αxf	αys	αyf	Mxs	Mxf	Mys	Myf
P1	End	3.6	5	1.39	13.3	0.074	0.055	0.045	0.03	12.76	9.48	7.76	5.17
P2	End	3.6	4.7	1.31	11.13	0.063	0.047	0.037	0.028	9.09	6.78	5.34	4.04
P4	Interior	2.7	5	1.85	12.6	0.065	0.049	0.037	0.028	5.97	4.50	3.40	2.57
P5	Interior	2.7	4.7	1.74	17.2	0.068	0.047	0.037	0.028	8.53	5.89	4.64	3.51
P6	Interior	3.15	5	1.59	10.47	0.06	0.045	0.037	0.028	6.23	4.67	3.84	2.91
P7	Interior	3.15	4.7	1.49	12.6	0.058	0.043	0.037	0.028	7.25	5.38	4.63	3.50
P8	Interior	2.85	5	1.75	10.47	0.064	0.048	0.037	0.028	5.44	4.08	3.15	2.38

Table 15 Moment analysis

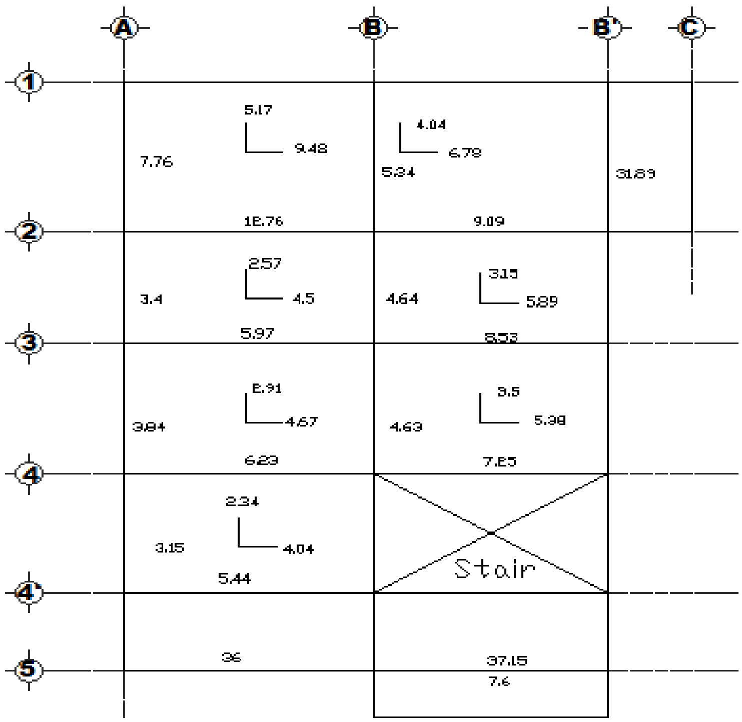


Figure 8First floor moment analysis

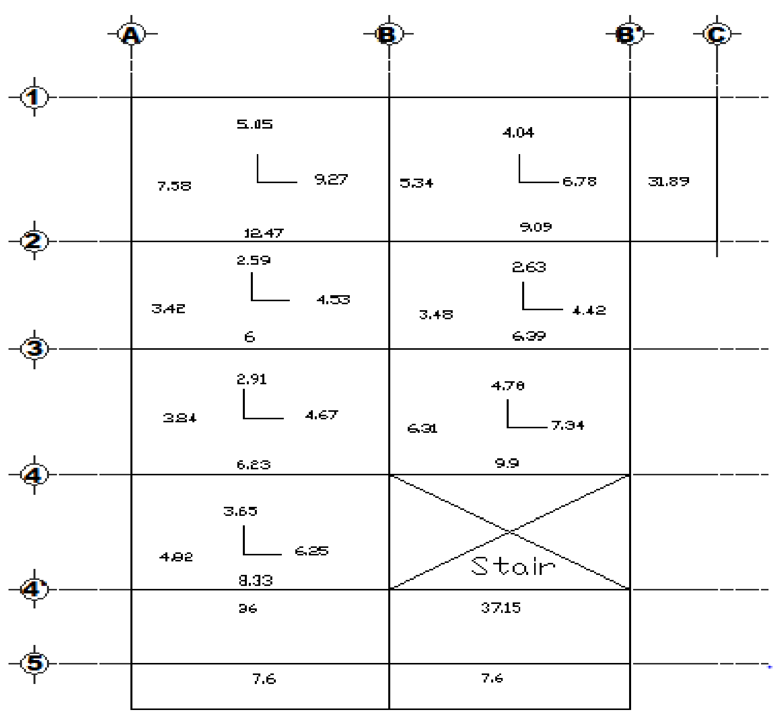


Figure 9 Second floor moment analysis

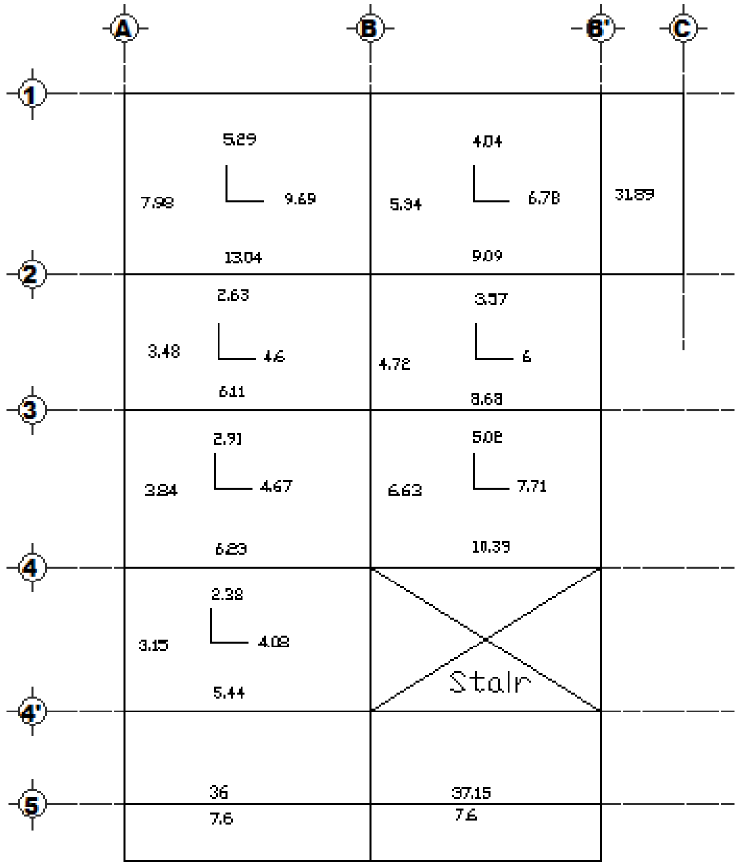


Figure 10 Third floor moment analysis

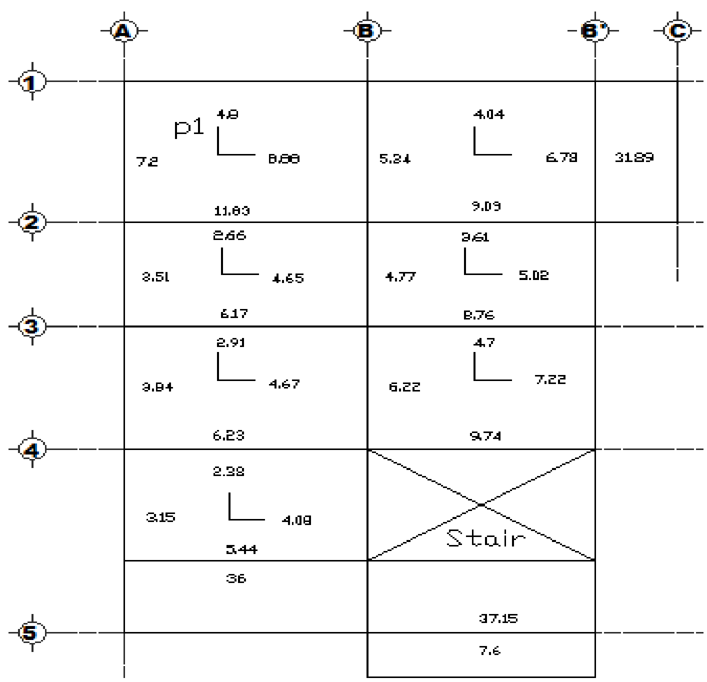


Figure 11Fourth &Fifth floor moment analysis

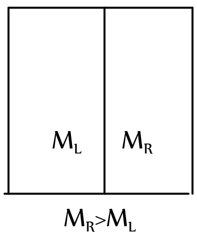
3.2.3 Adjustment of Moments

Support moment adjustment

There are two cases.

Case (1) If $\frac{M_R - M_L}{M_R} * 100 < 20\%$ then $Md = \frac{M_R + M_L}{2}$, $M_R > M_L$

Case (2) If $\frac{M_R - M_L}{M_R} * 100 > 20\%$ then Distribute unbalanced moment $M_D = M_R - M_L$ based on relative stiffness…. $M_R > M_L$



$M_d = M_R - \frac{K_R}{K_R + K_L} * \Delta M$……………………………..Considering right

$M_d = M_R + \frac{K_R}{K_R + K_L} * \Delta M$…………………………......Considering Left

Note that If adjustment is between two-way and cantilever, then $M_d = M_{max}$

Support moment adjustment for 1st floor

For panel 1 & panel 2

$\frac{MR-ML}{MR} * 100 < 20\% = \frac{12.76-9.09}{12.76} * 100 = 28.76\% > 20\%$

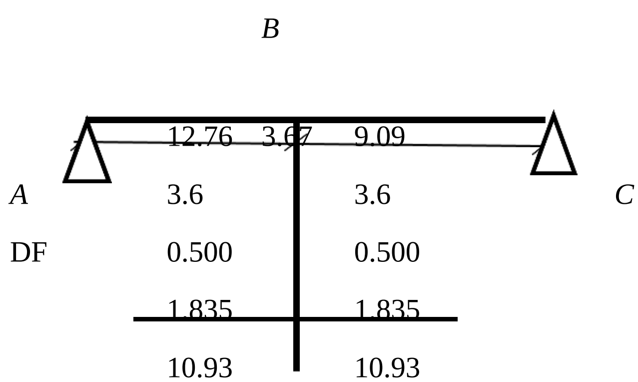
Use method 2

$KAB = \frac{3}{4Lx} = \frac{3}{4*3.6} = 0.63$

$KBC = \frac{3}{4Lx} = \frac{3}{4*3.6} = 0.63$

$DFAB = \frac{KAB}{KAB+KBC} = \frac{0.21}{0.21+0.21} = 0.5$

$DFBC = \frac{KBC}{KAB+KBC} = \frac{0.21}{0.21+0.21} = 0.5$



Support adjustments between panel 2 and panel 3

ML=11.8815(Panel 3) ML=6.439(Panel 2)

$\frac{11.8815-6.439}{11.8815}$*100=45.8>20% [use moment distribution]

$\therefore$Md=$11.8815 - \frac{1/4.8}{(1/4.8)+(1/4.8)} * (11.8815 - 6.439)$=9.16KN.m

Support adjustments between panel 3 and panel 4

ML=11.8815(Panel 3) ML=14.04(Panel 4)

$\frac{11.8815-14.04}{14.04}$*100=15.37<20% [use average moment]

$\therefore$Md=$\frac{11.8815+14.04}{2}$=12.96KN.m

Between Cantilever and slab

MR=23.89 ML=9.6

Since p4 is cantilever and p5 is two-way slab take the maximum .

Thus Md=23.89KN.m

Span moment adjustment

* If the support moment is decreased while carrying on moment distribution of a balanced support moment the span moment Mxf and Myf are the increased to allow for the change of support moment. This increase is calculated being equal to the change of the support moment multiplied by the factor given in EBCS. If the support moment increased, no adjustment shall be made to the span moment.

In atabular form

First Floor

Support moment adjustment

PANEL	MTD 1	MTD 2	MAX
P1&P2		6.55	
P2&P3		18.62	
P1&P4		10.32	
P4&P5		4.02	
P4&P6	6.1		
P6&P7	4.24		
P2&P5	8.81		

P6&P8	5.84		
P9&P10	36.58		
P8&P9	20.72		
P5&P7	7.89		
P9&C1			36
P10&C2			37.15
C1&C2			7.6

Table 16 Support moment adjustment

Span moment adjustment

PANEL	Mxs	Mxf	Madjust	Mxfadj	Mys	Myf	Madjust	Myfadj
P1	12.76	9.48	6.55	15.69	7.76	5.14	6.55	6.35
P2	9.09	6.78	18.62	-2.75	5.34	4.04	8.81	0.57
P4	5.97	4.5	10.32	0.15	3.4	2.57	4.02	1.95
P5	8.53	5.89	4.02	10.40	4.64	3.51	7.89	0.26
P6	6.23	4.67	6.1	4.80	3.85	2.91	4.24	2.52
P7	7.25	5.38	4.24	8.39	4.63	3.5	7.89	0.24
P8	5.44	4.08	5.84	3.68	3.15	2.31	4.5	0.96

Table 17 Span moment adjustment

3.2.4 Shear force analysis

Vsx=βvxPdLx

Vsy=βvyPdLx

First floor

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvxc	Bvxd	βvyc	βvyd	Vsxc	Vsxd	Vsyc	Vsyd
P1	End Span	3.6	5	1.39	13.3	0.52	0.35	0.4	0.26	24.90	16.758	19.15	12.4488
P2	End Span	3.6	4.7	1.31	11.13	0.47	0.31	0.36		18.83	12.42108	14.42	0
P3	End Span	1.7	3.6	2.12	1113	0.59	0.38	0.36		1116.34	718.998	681.16	0
P4	Interior	2.7	5	1.85	12.6	0.51		0.36	0.24	17.35	0	12.25	8.1648
P5	Interior	2.7	4.7	1.74	17.2	0.49		0.36	0.24	22.76	0	16.72	11.1456
P6	Interior	3.15	5	1.59	10.47	0.36		0.24		11.87	0	7.92	0
P7	Interior	3.15	4.7	1.49	12.68	0.47		0.36	0.24	18.77	0	14.38	9.58608
p8	Interior	2.85	5	1.75	10.47	0.504		0.36	0.24	15.04	0	10.74	7.16148
p9	Interior	1.88	5	2.66	10.47	0.52		0.36	0.24	10.24	0	7.09	4.724064
p10	Interior	1.88	4.7	2.50	11.13	0.52		0.36	0.24	10.88	0	7.53	5.021856
PANEL	TYPE	Lx	Ly	Ly/Lx	n (Pd)	WL							
C 1	Cantiliver	0.6	5	8.33	11.13	6.68							

Table18 Shear force analysis

3.2.5 Check depth for flexure

Cubic strength (fcu) = 25

γc=1.5

S-400

γs=1.15

εcu =3.5

b = 1000

Fck= 0.85*Fck/1.25 =25/1.25=20Mpa

Fcd= 0.85*Fck/γc = 0.85*20Mpa/1.5=11.33Mpa

Fyd= s/γs =400/1.15 =347.83Mpa

m= $\frac{fyd}{0.8*fcd}$=$\frac{347.83\text{Mpa}}{0.8*11.33}$=38.36

C1=$\frac{2.5}{m}$ = $\frac{2.5}{38.36}$ 0.0652

C2 =0.32*m^2*fcd = 0.32*38.36^2*11.33=5337

ρb = (((0.8*εcu)/(εcu+fyd(200))))*(Fcd*Fyd)

= (((0.8*3.5)/(3.5+347.83))))*(11.33*347.83)

= 0.01741Mpa

ρmax= 0.75*ρb = 0.75*0.01741

= 0.01306

Mmax = 37.15

d= $\sqrt{(Mmax/(0.8bfcd\rho\,\max m(1-(0.4m\rho\,\max))))}$

d = 101.132mm<Dused OK

3.2.6 Slab Reinforcement

For the given

Material Data, C-25, S-300

Effective depth, d=128mm

Width, b=1000mm

Moments calculated for each panels

Using design charts

Km=$\left(\sqrt{\frac{M}{b}}\right)$/d, Ks

As=Ks*M/d

To calculate spacing by selecting diameter of bar as

S=1000as/A   where-as=area of single bar

As=calculated area of steel

S=spacing

Compare the above result with minimum provision given by our code.

$As_{min}=\rho_{min}$*b*d=0.5/fyk*b*d=0.5*1000*128/300=213.33

$s_{max} \leq \begin{cases} 2D \\ 350mm \end{cases} = \begin{cases} 2*200mm \\ 350mm \end{cases}$=400mm

S_{max}=400

For First floor slab

Panel	moment		Ast=Msd/Z*fyd	ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	10.32	189.46	10	414.33	400	ø	10	c/c	400
	M_{xf}=	15.69	288.05	10	272.52	270	ø	10	c/c	270
	M_{ys}=	6.55	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	6.35	215.00	10	215.00	200	ø	10	c/c	200
P-2	M_{xs} =	8.81	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	9.32	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	6.55	215.00	10	365.12	350	ø	10	c/c	350

	M_{yf}=	0.57	215.00	10	365.12	350	ø	10	c/c	350
P-3	M_{xs} =	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	18.62	396.53	10	197.97	215	ø	10	c/c	215
	M_{yf}=	0	215.00	10	365.12	350	ø	10	c/c	350
P-4	M_{xs} =	6.1	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	0.4	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	4.02	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	1.95	215.00	10	365.12	350	ø	10	c/c	350
P-5	M_{xs} =	7.89	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	10.4	221.48	10	354.44	350	ø	10	c/c	350
	M_{ys}=	4.02	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	0.26	215.00	10	365.12	350	ø	10	c/c	350
P-6	M_{xs} =	5.84	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	4.8	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	4.24	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	2.52	215.00	10	365.12	350	ø	10	c/c	350
P-7	M_{xs} =	7.89	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	8.39	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	4.24	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	0.24	215.00	10	365.12	350	ø	10	c/c	350
P-8	M_{xs} =	20.72	441.25	10	177.90	215	ø	10	c/c	215
	M_{xf}=	3.68	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	0.96	215.00	10	365.12	350	ø	10	c/c	350
P-9	M_{xs} =	36	766.66	10	102.39	350	ø	10	c/c	350
	M_{xf}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	20.72	441.25	10	177.90	330	ø	10	c/c	330
	M_{yf}=	0.96	215.00	10	365.12	350	ø	10	c/c	350

P-10	M_{xs} =	37.15	791.15	10	99.22	215	ø	10	c/c	215
	M_{xf}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	35.58	757.71	10	103.60	215	ø	10	c/c	215
	M_{yf}=	0	215.00	10	365.12	350	ø	10	c/c	350
C1	M_{xs} =	7.6	215.00	10	365.12	350	ø	10	c/c	350

Table 19 Reinforcement calculation

4 Stair case Design

Stairs:- are structures which provide access passage to different floor levels one of the structural elements constructed with steps rising without a break from floor to floor or with steps rising to landing between floors with a serious of steps rising further from the landing to floor above. are sloping one-way spanning slab Stairs are different in their mode of planning. It can be straight flight with or without landing, quarter turn, half turn (also called doglegged or scissor type), open well (if well type opening exists), or it can be geometric stair (free standing stairs).

DESIGN PROCEDURE

1. Determination of depth for deflection: which is a function of design tensile strength of steel, effective span length of the shortest span in which more load is expected to transfer and support condition

2. Loading: which determines the total load in the stair and landing Analysis: determines moment and shear forces based on the analyzed moment

3. Check depth for flexure: this step helps to cross check the design depth as it is safe for flexure or not, if not revise the depth determined in step 1 and also the loads.

4. Reinforcement provision: using the computed moments, number and area of reinforcement bars determined.
5. Detailing: the arrangement of reinforcement

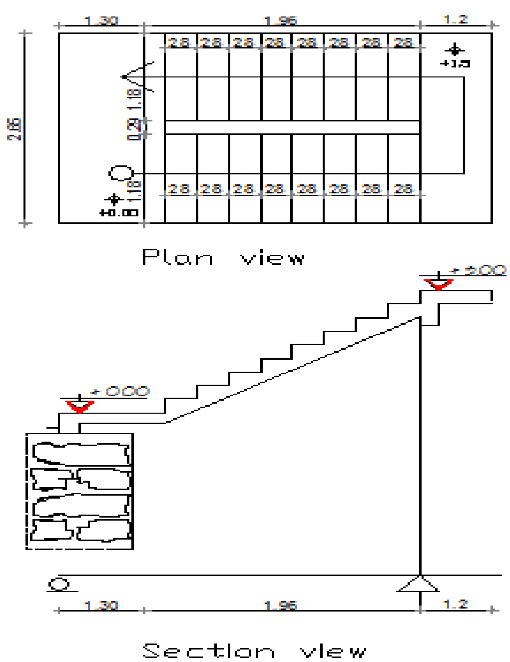


Figure 12 Stair case Layout

4.1 Depth determination

Height of Riser= $\frac{\text{Height of Stair}}{\text{No of Stair}} = \frac{1.5\text{m-0m}}{7} = 0.21\text{m}$

Therefore we use riser height 21 cm for the design.

Material

C-25

S-300

Depth for deflection for the inclined slab

From EN 1992-1-1-2004 section 7.4.2 depth of the slab is determined

$$\frac{l}{d} = k(11 + 1.5\sqrt{fck} * \frac{\rho_0}{\rho} + 3.2\sqrt{fck} * \left(\frac{\rho_0}{\rho} - 1\right)^{\frac{3}{2}})\ if\ \rho <= \rho_0$$

$$\frac{l}{d} = k\left(11 + 1.5\sqrt{fck} * \frac{\rho^0}{\rho-\rho'} + \frac{1}{12}\sqrt{fck} * \sqrt{\frac{\rho'}{\rho}}\right) if \rho > \rho_0$$

fck = 25Mpa

Assume the concrete is lightly stressed ρ = 0.5%

$$\rho_0 = \sqrt{fck} * 10^{-3} = \sqrt{20} * 10^{-3} = 0.4472\%$$

$\rho^0 < \rho$ So use

$$\frac{l}{d} = k(11 + 1.5\sqrt{fck} * \frac{\rho_0}{\rho-\rho'} + \frac{1}{12}\sqrt{fck} * \left(\sqrt{\frac{\rho'}{\rho_0}}\right)\ )$$

$$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.4472}{0.5-\rho'} + \frac{1}{12}\sqrt{fck} * \left(\sqrt{\frac{\rho'}{0.4472}}\right)\ )$$

For interior slab K=1.5 as EN 1992-1-1 table 7.4

$$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.4472}{0.5} + 3.2\sqrt{20} * \left(\frac{0.4472}{0.5} - 1\right)^{\frac{3}{2}})$$

$\frac{l}{d}$ = 20.52*k

$\frac{2850}{d}$ = 20.52*1.5

d = $\frac{2850}{30.78}$ = 92.6 mm

D = d + $\frac{Ø}{2}$ + cover

D = 92.6 + 5+ 20 = 117.6mm

D = 120 mm

Depth for deflection for the Landing

From EN 1992-1-1-2004 section 7.4.2 depth of the slab is determined

$$\frac{l}{d} = k(11 + 1.5\sqrt{fck} * \frac{\rho_0}{\rho} + 3.2\sqrt{fck} * \left(\frac{\rho_0}{\rho} - 1\right)^{\frac{3}{2}}) \; if \rho <= \rho_0$$

$$\frac{l}{d} = k\left(11 + 1.5\sqrt{fck} * \frac{\rho^0}{\rho - \rho'} + \frac{1}{12}\sqrt{fck} * \sqrt{\frac{\rho'}{\rho}}\right) if \rho > \rho_0$$

fck = 25Mpa

Assume the concrete is lightly stressed ρ = 0.5%

$\rho_0 = \sqrt{fck} * 10^{-3} = \sqrt{20} * 10^{-3} = 0.4472\%$

$\rho^0 < \rho$So use

$$\frac{l}{d} = k(11 + 1.5\sqrt{fck} * \frac{\rho_0}{\rho - \rho\prime} + \frac{1}{12}\sqrt{fck} * \left(\sqrt{\frac{\rho\prime}{\rho_0}}\right) \;)$$

$$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.4472}{0.5 - \rho\prime} + \frac{1}{12}\sqrt{fck} * \left(\sqrt{\frac{\rho\prime}{0.4472}}\right) \;)$$

For interior slab K=1.5 as EN 1992-1-1 table 7.4

$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.4472}{0.5} + 3.2\sqrt{20} * \left(\frac{0.4472}{0.5} - 1\right)^{\frac{3}{2}})$

$\frac{l}{d}$ = 20.52*k

$\frac{2850}{d}$ = 20.52*1.5

d = $\frac{2850}{20.52}$ = 117.6 mm

D = d + $\frac{\emptyset}{2}$ + cover

D = 117.6 + 5+ 20 = 117.6mm

D = 120 mm

d_{max}=max (81.5mm, 187.72mm)

d_{max}=187.72 mm

Overall depth

D=d+ cover+Ø/2, assume Ø14

=148.72 mm+15 mm+14mm/2=170.72 mm

Use D=180 mm

4.2 Load computation

Material Data

Unit weight of marble=27 KN/m^3

Unit weight of cement screed=23 KN/m^3

Unit weight of concrete=24 KN/m^3

Unit weight of plastering=23 KN/m^3

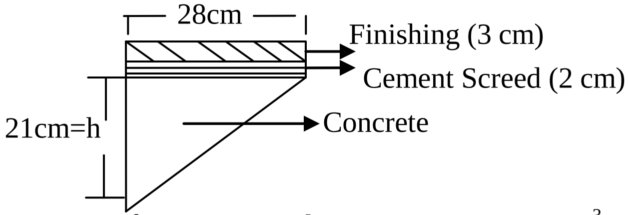
Thickness of cement screed=2cm

Thickness of plastering=2cm

Thickness of marble=3cm

Take 1m width strip

Step dead load



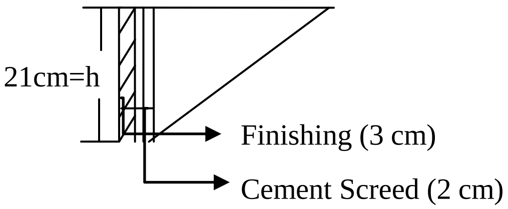
D.L of cement screed=$t_{sc}*\gamma_{sc}$=0.02*23 KN/m^3=0.46 KN/m

D.L of finishing=$t_{fin}*\gamma_{fin}$=0.03*27 KN/m^3=0.81 KN/m

D.L of Concrete=1/2*h*γ_{conc}=1/2*0.21m*24 KN/m^3 =2.52 KN/m

Therefore DL of step=0.46 KN/m + 0.81 KN/m + 2.52 KN/m=3.79 KN/m

Riser Dead Load



D.L of cement screed=$\frac{\text{No of riser (hcs*tcs*}\gamma\text{sc)}}{\text{Projected length (12*28 cm)}}$

$=\frac{8(0.21m*0.02m*23KN/m^3}{1.96}$

=0.394 KN/m

D.L of Finishing $=\dfrac{\text{No of riser (hcs*tcs*}\gamma\text{sc)}}{\text{Projected length (12*28 cm)}}$

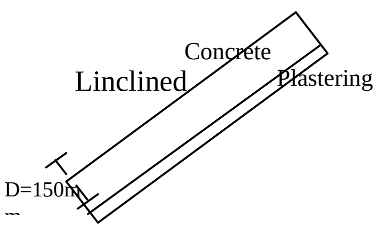
$=\dfrac{13(0.21m*0.03m*27KN/m^3}{1.96}$

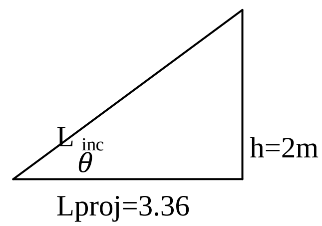
=0.694 KN/m

Therefore D.L of riser (21cm)= 0.394 KN/m + 0.694 KN/m

= 1.088 KN/m

Waist Dead Load





tan$\boldsymbol{\theta}$=2/3.36

$\theta = \tan^{-1}(\frac{1.5}{1.96})$=37.42°

Sin $\boldsymbol{\theta}$=1.5/L_{inc}

$L_{inc\backslash}$=1.5/sin $\boldsymbol{\theta}$ =1.5m/sin 37.42°

=3.29 m

D.L of concrete $=\dfrac{D*L\ inc*\gamma conc}{Lprojected}=\dfrac{0.15m*3.29m*24\ KN/m^3}{1.96\ m}$

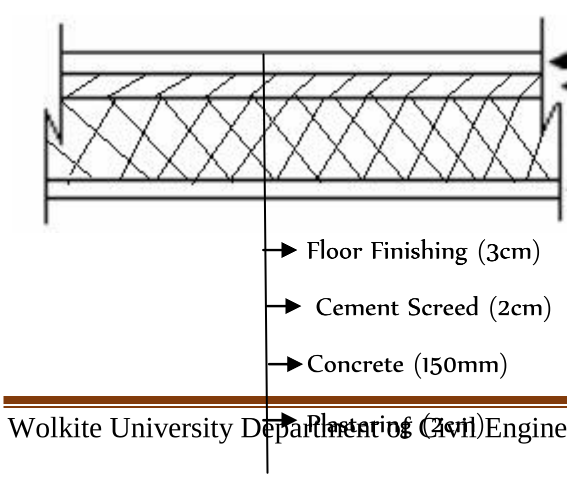
=6.04 KN/m

D.L of Plastering $=\dfrac{tpl*Linc*\gamma pl}{Lprojected}=\dfrac{0.02m*3.29m*23\ KN/m^3}{1.96\ m}$

=0.772 KN/m

Therefore D.L of waist = 6.04KN/m + 0.772KN.m=6.81 KN/m

Landing Dead Load



D.L of landing=D.L of finishing + D.L of cement screed + D.L of concrete + D.L of plastering

$=t_{fin}*\gamma_{fin}+t_{cs}*\gamma_{cs}+t_c\gamma_c+t_{pl}*\gamma_{pl}$

=0.03m*27 KN/m^3+0.02*23 KN/m^3 + 0.15m*24 KN/m^3 +0.02m*23 KN/m^3

=5.33 KN/m

Therefore D.L of Landing=5.33 KN/m

Total Dead Load and design load

For the inclined slab

Total D.L=D.L of Step+D.L of riser+D.L of waist

=3.79 KN/m+1.088KN/m+6.81 KN/m

=11.688 KN/m

Live load=3 KN/m^2*1m=3 KN/m

Design Load, pd=1.35 D.L + 1.5L.L

=1.35*11.688 KN/m + 1.5*5 KN/m

=23.28 KN/m

For the landing

D.L=5.33 KN/m

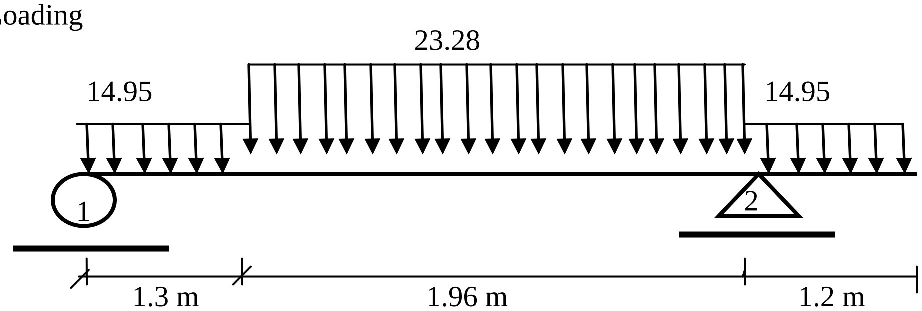
L.L=3 KN/m^2 * 1m=3KN/m

Deisgn load, pd=1.35 D.L + 1.5 L.L

=1.35*5.33 KN/m + 1.5*5 KN/m

=14.95 KN/m

Loading



4.3 Moment and Shear force calculation

Moment Analysis

+ΣM at 1=0

14.95*(1.3)*1.3/2+23.28*1.96m*(1.3+1.96/2)+14.95*1.2*(3.26+1.2/2)-R_2*3.26m=0

R_2*3.26m=185.9KN.m

R_2=57.02 KN

ΣFy=0 (+) ↓

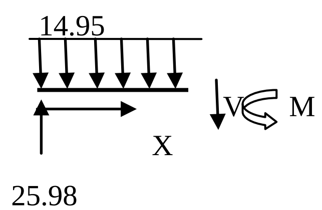
R_1+R_2=14.95 KN/m*1.3 m+23.28 KN/m*1.96 m+14.95 KN/m*1.2m

R_1+R_2=83 KN

R_1=25.98 KN

Analyzing using the method of section,

For Z=0, to Z=1.3m (Z in measured from support 1)



M(x)+14.95 KN/m * x^2/2=25.98x

M(x)=25.98x-7.475 x^2

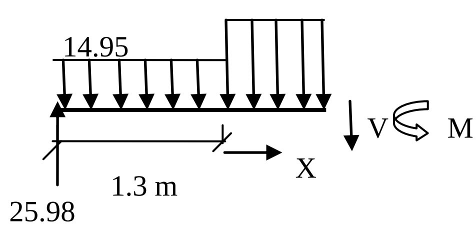
At Z=0, X=0

M(0)=0

At Z=1.3, x=1.3

M(1.3)=21.14KN.m

For Z=1.3 to Z=3.26m



M(x)+23.28 KN/m*x^2/2+14.95 KN/m*1.3 m*(x+0.83/2)=25.98 KN (x+1.3)

$M(x)=-11.64x^2+6.55x+21.14$

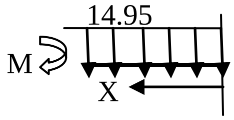
At Z=1.3m, x=0

M(0)=21.14KN.m

At Z=3.26m, X=1.96m

M(1.96)=-10.73KN.m

For Z=3.26 to Z=4.46m



$M(x)=-14.95\ x^2/2$

$=-7.475\ x^2$

At Z=4.46m, x=0

M(0)=0

At Z=3.26m, x=1.2m

M(1.2)=-10.764KN.m

Mmax (+ve) is b/n Z=1.3 to Z=3.26m

$M(x)=-11.64\ x^2+6.55x+21.14$

M is max at V=0, dm(x)/dx=0

$dM(x)/dx=d/dx(-11.64\ x^2+6.55x+21.14)=0$

=-22.92x+6.55=0,

Shear Force computation

V(x)=dM(x)/dx

For Z=0 to Z=1.3m

$M(x)=25.98x-7.475x^2$

dM(x)/dx=V(x)=-14.95x+25.98

at Z=0, x=0

V(0)=25.98 KN

At Z=1.3m, x=1.3m

V(1.3)=6.545 KN

For Z=1.3m to Z=3.26m

$M(x)=-11.64x^2+6.55x+21.14$

$dM(x)/dx=V(x)=-23.28x+6.55$

at Z=1.3m, x=0

V(0)=6.55 KN

At Z=3.26m, x=1.96m

V(1.96)= -39 KN

For Z=3.26m to Z=4.46m

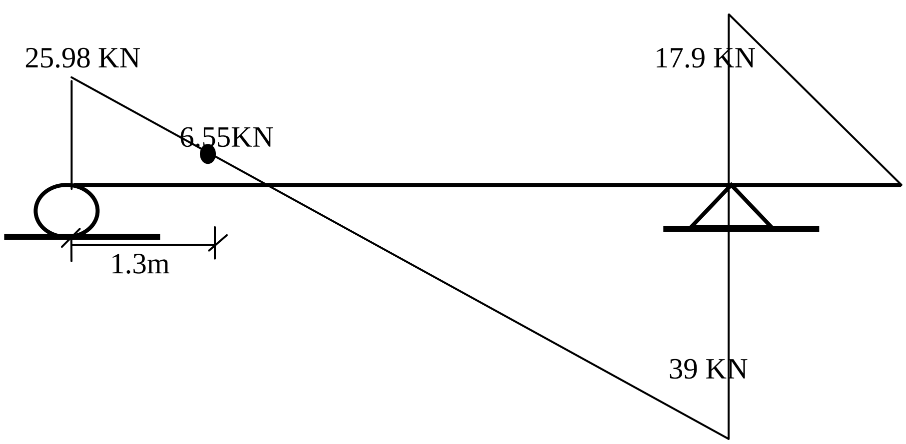
$M(x)= -7.475x^2$

$dM(x)/dx= -14.95x$

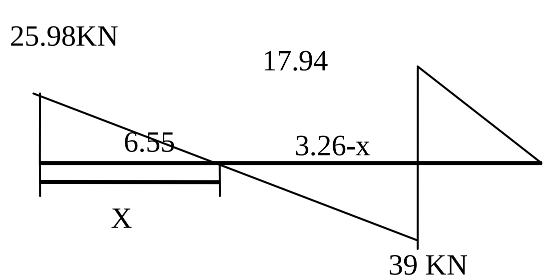
at Z=3.26m, x=1.2m

V(1.2)= -17.94 KN

Shear Force Diagram



Bending Moment Diagram



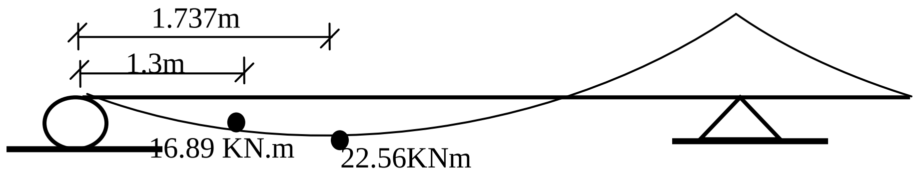
$\frac{25.98}{x} = \frac{39}{3.26-x}$

25.98(3.26-x)=39x

84.69-25.98x=39x

X=<u>1.737</u>

½*25.98*1.737=22.56



4.4 Check depth for flexure

Cubic strength (fcu) = 25

γc=1.5

S-400

γs=1.15

εcu =3.5

b = 1000

Fck= 0.85*Fck/1.25 =25/1.25=20Mpa

Fcd= 0.85*Fck/γc = 0.85*20Mpa/1.5=11.33Mpa

Fyd= s/γs =400/1.15 =347.83Mpa

m= $\frac{fyd}{0.8*fcd} = \frac{347.83\text{Mpa}}{0.8*11.33}$=38.36

C1=$\frac{2.5}{m} = \frac{2.5}{38.36}$ 0.0652

C2 =0.32*m^2*fcd = 0.32*38.36^2*11.33=5337

ρb = (((0.8*εcu)/(εcu+fyd(200))))*(Fcd*Fyd)

= (((0.8*3.5)/(3.5+347.83))))*(11.33*347.83)

= 0.01741Mpa

ρmax= 0.75*ρb = 0.75*0.01741

= 0.01306

Mmax = 22.56

d= $\sqrt{(Mmax/(0.8bfcd\rho \max m(1-(0.4m\rho \max))))}$

d = 78.81mm<Dused OK

4.5 Design reinforcement

Material Data, C-25,

S-400

Effective depth, d=98mm

Width, bs=1000mm

Using M = 15.49KN/m, calculate reinforcement,

μsd = $M/fcd*b*d^2$= $22.56*10^6/11.33*1000*170.72^2$=0.068

using μsd =0.068

Kz=0.91

Z=Kz*d =0.91*170.72 = 155.35mm

As=M/(fyd*z)

Fyd= fyk/γs =400/1.15 = 342.83

As = 15.49*10^6/(155.35*342.82)

= 423.59mm2

To calculate spacing by selecting diameter of bar as

S=1000as/As where:- as=area of single bar=$3.14*14^2/4$=153.9mm^2

As=calculated area of steel

S=spacing

S =1000∗153.9/423.59=363.32≈350mm

Compare the above result with minimum provision given by our code.

Asmin= 0.26*2.2Mpa/400Mpa*1000mm*170.72mm

= 244.13mm^2

With d = 180-15-7=148.72mm, for ϕ14main rebar

Asmin=244.13mm2<Ascal=423.59mm2 ok

$$\text{smax} \le \begin{cases} \text{2D400 Scal} = 2*180 = 360\text{mm} \\ 400\text{mm} \\ Scalc = 300mm \end{cases}$$

Smax = 350mm

Provide ϕ14c/c350mm

$A_{sprovided}$=1000*153.9/350=641.3mm^2

B) Transverse reinforcement

According to Eurocode 2 Part 1,1 - prEN 1992-1-1-2002 Section 9.3.1.1 the ratio of secondary to the main reinforcement shall be at least equal to 20% of the main reinforcement

$As,sv \geq 20\%\ As,cal$

As, ≥ 0.2* 423.59 =84.72mm^2<$Asmin$

Therefore take the minimum reinforcement

Using Ø8 main transverse rebar

S =ba$_s$ /As= 1000 ∗ 50.26/ 84.72=593.25mm

$$S_{max}=\begin{cases} 2D = 2 * 180 = 360 \\ 400 \\ Scalc = 593.25 \end{cases}$$

Smax= 360

As provided =1000*$\frac{153.9}{360}$ =641.3

Therefore provide ϕ8c/c 360mm

5 Lateral load calculation

Building structure are exposed to lateral loads of the earth quake and wind loads. The occurrence of these loads simultaneously on the structure is very rare and therefore, we will design the structure for the governing load among the two. Thus, the design process involves the determination of the two loads separately and designing for the maximum effect.

Earth quake analysis Using, Equivalent static (building code) analysis method This type of analysis is applied to buildings whose response is not significantly affected by contribution from higher modes vibration. These requirements are claimed to be satisfied by buildings which;

a) Meets the criteria for regularity in plan and elevation.

b) Have fundamental periods of vibration T1≤$\begin{cases} 4Tc \\ 2sec \end{cases}$

5.1 Determination of Center of Mass

The horizontal forces at each floor level, Fi, are distributed to lateral load resistive structural elements in proportion to their rigidities assuming rigid floor diaphragms. Center of mass (Xm, Ym): it is a point on a floor level where the whole floor mass and its inertial effects can be replaced using a lumped equivalent mass.

Xm = $\frac{\sum wixi}{\sum wi}$

Ym = $\frac{\sum wiyi}{\sum wi}$

Where:-Xm, and Ym are the coordinate of the point of application of Fi when the seismic action is parallel to the Y-direction and X-direction respectively.

Foundation

Column On	Width (m)	Depth (m)	Height (m)	Weight (KN)	Moment arm		Moment	
					X (m)	Y (m)	Mx=Wi*Yi (KNm)	My=Wi*Xi (KNm)
Axis-A1	0.3	0.3	2	5.4	0	0	0	0
Axis-A2	0.3	0.3	2	5.4	0	3.6	19.44	0
Axis-A3	0.3	0.3	2	5.4	0	2.7	14.58	0

Axis-A4	0.3	0.3	2	5.4	0	3.15	17.01	0
Axis-A5	0.3	0.3	2	5.4	0	2.95	15.93	0
Axis-A6	0.3	0.3	2	5.4	0	1.88	10.15	0
Axis-B1	0.3	0.3	2	5.4	5	0	0	27
Axis-B2	0.3	0.3	2	5.4	5	3.6	19.44	27
Axis-B3	0.3	0.3	2	5.4	5	2.7	14.58	27
Axis-B4	0.3	0.3	2	5.4	5	3.15	17.01	27
Axis-B5	0.3	0.3	2	5.4	5	2.95	15.93	27
Axis-B6	0.3	0.3	2	5.4	5	1.88	10.15	27
Axis-C1	0.3	0.3	2	5.4	4.7	0	0	23.38
Axis-C2	0.3	0.3	2	5.4	4.7	3.6	19.44	23.38
Axis-C3	0.3	0.3	2	5.4	4.7	2.7	14.58	23.38
Axis-C4	0.3	0.3	2	5.4	4.7	3.15	17.01	23.38
Axis-C5	0.3	0.3	2	5.4	4.7	2.95	15.93	23.38
Axis-C6	0.3	0.3	2	5.4	4.7	1.88	10.15	23.38
Axis-D1	0.3	0.3	2	5.4	1.7	0	0	9.18
Axis-D2	0.3	0.3	2	5.4	1.7	3.6	19.44	9.18
Axis-D3	0.3	0.3	2	5.4	1.7	2.7	14.58	9.18
Axis-D4	0.3	0.3	2	5.4	1.7	3.15	17.01	9.18
Axis-D5	0.3	0.3	2	5.4	1.7	2.95	15.93	9.18
Axis-D6	0.3	0.3	2	5.4	1.7	1.88	10.15	9.18
Total weight of Footing column				129.6			308.44	369.36

$Xm = \frac{\sum wixi}{\sum wi} = \frac{369.36}{129.6} = 2.85$ $Ym = \frac{\sum wiyi}{\sum wi} = \frac{308.44}{129.6} = 2.37$

Table 20 center of mass calculation for foundation

6 Wind Load Analysis

This calculation presents the automatically generated lateral wind loads for load pattern WLP according to EUROCODE1 2005, as calculated by ETABS.

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = IV

Wind Direction = 0;90 degrees

Basic Wind Velocity, V_b [EC 4.2(2)] $V_b = 22 \frac{\text{meter}}{\text{sec}}$

Windward Coefficient, $C_{p,wind}$ $C_{p,wind} = 0.8$

Leeward Coefficient, $C_{p,lee}$ $C_{p,lee} = 0.5$

Air Density, ρ $\rho = 1.25$

Top Story = Roof floor

Bottom Story = Base

Include Parapet = No

Factors and Coefficients

Structural Factor, cs<c_{d}> [EC 6.2(1)] $c_s c_d = 1$

Elevation, z_0 $z_0 = 1$

Minimum Elevation, z_{min} $z_{min} = 10$

Maximum Elevation, z_{max} $z_{max} = 200$

Turbulence Factor, k_l [EC 4.4(1)] $k_l = 1$

Orography Factor, c_o [EC 4.3.3] $c_o = 1$

Turbulence Intensity, I_v [EC 4.4(1)]

$$I_v = \frac{k_l}{c_o(z)\ln(\frac{z}{z_0})} \text{ for} z_{min} \leq z \leq z_{max}$$

$$= I_v(z_{min}) \text{ for } z z_{min}$$

Terrain Factor, k_r [EC 4.3.2(1) Eq. 4.5] $k_r = 0.19(^{\frac{z_0}{0.05}}$ $k_r = 0.234329$

Roughness Factor, <c_{r}>(z) [EC 4.3.2(1) Eq. 4.4]

$$c_r(z) = k_r \ln(\frac{z}{z_0}) \text{ for} z_{min} \leq z \leq z_{max}$$

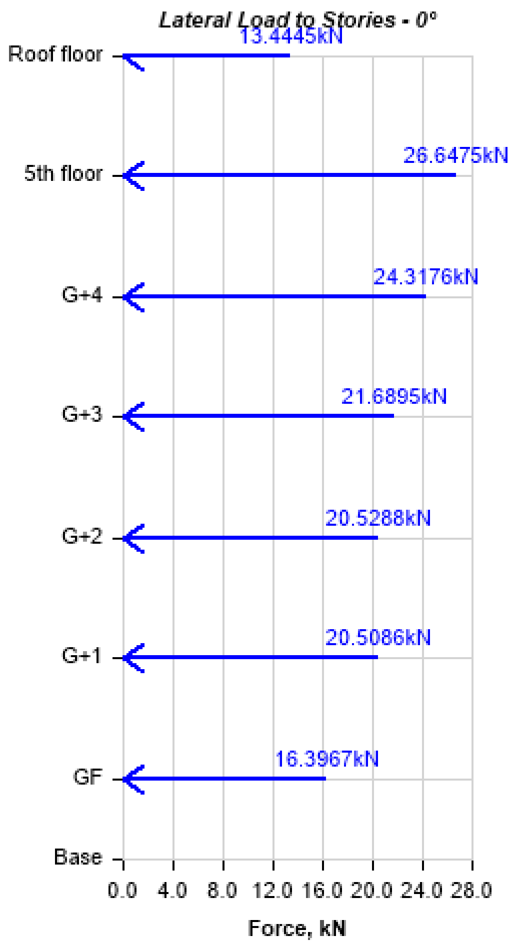
$$= c_r(z_{min}) \text{ for } zz_{min}$$

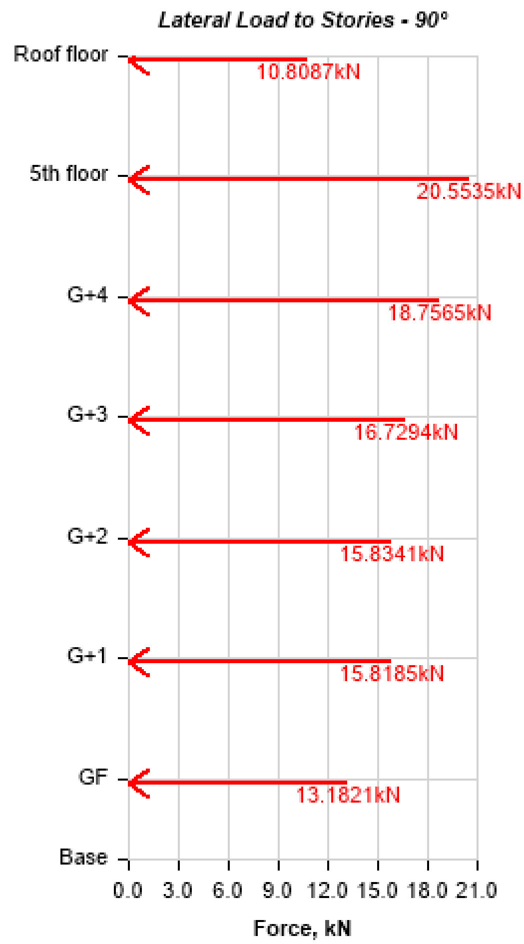
Lateral Loading

Peak Velocity Pressure, <$q_{p>(z)}$ [EC 4.5(1) Eq. 4.8] $q_p(z) = [1 + 7I_v(z)]\frac{1}{2}\rho[c_r(z)c_o(z)v_b]^2$

Wind Pressure, w [EC 5.2(1) Eq. 5.1] $w = q_p(z)c_s c_d(c_{p,wind} + c_{p,lee})$

Applied Story Forces





Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	13.4445	0
5th floor	17	26.6475	0
G+4	14	24.3176	0
G+3	11	21.6895	0
G+2	8	20.5288	0
G+1	5	20.5086	0
GF	2	16.3967	0
Base	0	0	0

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	0	10.8087
5th floor	17	0	20.5535
G+4	14	0	18.7565
G+3	11	0	16.7294
G+2	8	0	15.8341
G+1	5	0	15.8185
GF	2	0	13.1821
Base	0	0	0

7 Earthquake Analysis

7.1 Determination of Seismic Load

This calculation presents the automatically generated lateral seismic loads for load pattern EQXR according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = X + Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075m$
Structure Height Above Base, H	$H = 20$ m

Factors and Coefficients

Country =

Design Ground Acceleration, a_g	$a_g = 0.9g$
Ground Type [EC Table 3.1] = C	
Soil Factor, S [EC Table 3.2]	$S = 0$
Constant Acceleration Period Limit, T_B [EC Table 3.2]	$T_B = 0$ sec
Constant Acceleration Period Limit, T_C [EC Table 3.2]	$T_C = 0$ sec
Constant Displacement Period Limit, T_D [EC Table 3.2]	$T_D = 0$ sec
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 2.34$

Seismic Response

Spectral Response Acceleration, $Sd(T_1)$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})] \text{ for } T \leq T_B$$

$$= a_g S \frac{2.5}{q} \text{ for} T_B \leq T \leq T_C$$

$$= a_g S \frac{2.5}{q}[\frac{T_C}{T}] \geq \beta a_g \text{ for} T_C \leq T \leq T_D$$

$$= a_g S \frac{2.5}{q} [\frac{T_C T_D}{T^2}] \geq \beta a_g \text{ for} T_D \leq T$$

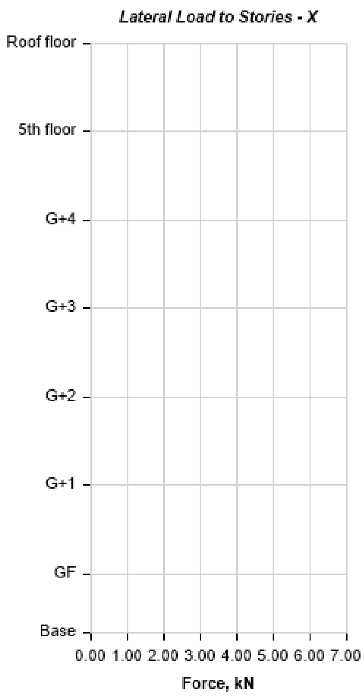
Equivalent Lateral Forces

Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X + Ecc. Y	2.466	4491.5788	0

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	0	0
5th floor	17	0	0
G+4	14	0	0
G+3	11	0	0
G+2	8	0	0
G+1	5	0	0
GF	2	0	0
Base	0	0	0

This calculation presents the automatically generated lateral seismic loads for load pattern EQXL according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075\text{m}$
Structure Height Above Base, H	$H = 20\text{ m}$
Factors and Coefficients	
Country =	
Design Ground Acceleration, a_g	$a_g = 0.4g$
Ground Type [EC Table 3.1] = B	
Soil Factor, S [EC Table 3.2]	$S = 1.2$
Constant Acceleration Period Limit, T_B [EC Table 3.2]	$T_B = 0.15\text{ sec}$
Constant Acceleration Period Limit, T_C [EC Table 3.2]	$T_C = 0.5\text{ sec}$
Constant Displacement Period Limit, T_D [EC Table 3.2]	$T_D = 2\text{ sec}$
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0.2$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 2$

Seismic Response

Spectral Response Acceleration, Sd($T_{1)}$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})] \text{ for } T \leq T_B$$

$$= a_g S\frac{2.5}{q} \text{ for} T_B \leq T \leq T_C$$

$$= a_g S\frac{2.5}{q}[\frac{T_C}{T}] \geq \beta a_g \text{ for} T_C \leq T \leq T_D$$

$$= a_g S\frac{2.5}{q}[\frac{T_C T_D}{T^2}] \geq \beta a_g \text{ for} T_D \leq T$$

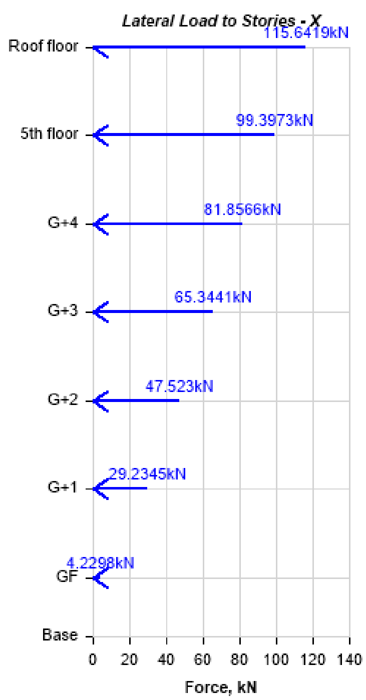
Equivalent Lateral Forces

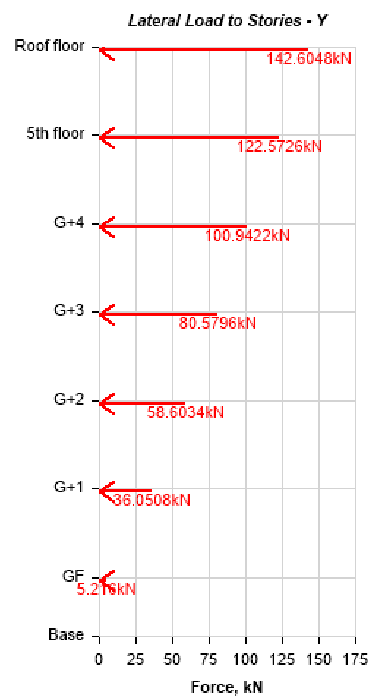
Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X	2.466	4491.5788	443.2272
Y	2.221	4491.5788	546.5692
X + Ecc. Y	2.466	4491.5788	443.2272
Y + Ecc. X	2.221	4491.5788	546.5692
X - Ecc. Y	2.466	4491.5788	443.2272
Y - Ecc. X	2.221	4491.5788	546.5692

Applied Story Forces





Story	Elevation	X-Dir	Y-Dir
	M	kN	kN
Roof floor	20	115.6419	0
5th floor	17	99.3973	0
G+4	14	81.8566	0
G+3	11	65.3441	0
G+2	8	47.523	0
G+1	5	29.2345	0
GF	2	4.2298	0
Base	0	0	0

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	0	142.6048
5th floor	17	0	122.5726
G+4	14	0	100.9422
G+3	11	0	80.5796
G+2	8	0	58.6034
G+1	5	0	36.0508
GF	2	0	5.216
Base	0	0	0

This calculation presents the automatically generated lateral seismic loads for load pattern -EQXR according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = X - Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075\mathrm{m}$
Structure Height Above Base, H	$H = 20\ \mathrm{m}$
Factors and Coefficients	
Country =	
Design Ground Acceleration, a_g	$a_g = 0.9g$
Ground Type [EC Table 3.1] = C	
Soil Factor, S [EC Table 3.2]	$S = 0$
Constant Acceleration Period Limit, T_B [EC Table 3.2]	$T_B = 0$ sec
Constant Acceleration Period Limit, T_C [EC Table 3.2]	$T_C = 0$ sec
Constant Displacement Period Limit, T_D [EC Table 3.2]	$T_D = 0$ sec
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 2.34$

Seismic Response

Spectral Response Acceleration, Sd($T_{1)}$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})] \text{ for } T \leq T_B$$

$$= a_g S\frac{2.5}{q} \text{ for} T_B \leq T \leq T_C$$

$$= a_g S\frac{2.5}{q}[\frac{T_C}{T}] \geq \beta a_g \text{ for} T_C \leq T \leq T_D$$

$$= a_g S\frac{2.5}{q}[\frac{T_C T_D}{T^2}] \geq \beta a_g \text{ for} T_D \leq T$$

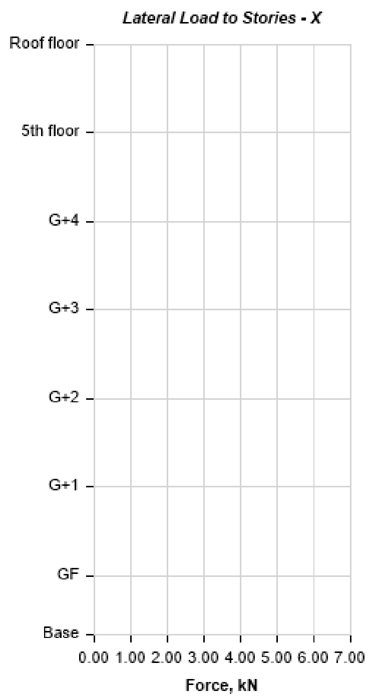
Equivalent Lateral Forces

Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X - Ecc. Y	2.466	4491.5788	0

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	0	0
5th floor	17	0	0
G+4	14	0	0
G+3	11	0	0
G+2	8	0	0
G+1	5	0	0
GF	2	0	0
Base	0	0	0

This calculation presents the automatically generated lateral seismic loads for load pattern -EQXL according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075\text{m}$
Structure Height Above Base, H	$H = 20$ m

Factors and Coefficients

Country =

Design Ground Acceleration, a_g	$a_g = 0.4g$
Ground Type [EC Table 3.1] = B	
Soil Factor, S [EC Table 3.2]	$S = 1.2$
Constant Acceleration Period Limit, T_B [EC Table 3.2]	$T_B = 0.15$ sec
Constant Acceleration Period Limit, T_C [EC Table 3.2]	$T_C = 0.5$ sec
Constant Displacement Period Limit, T_D [EC Table 3.2]	$T_D = 2$ sec
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0.2$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 2$

Seismic Response

Spectral Response Acceleration, $Sd(T_1)$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})] \text{ for } T \le T_B$$

$$= a_g S\frac{2.5}{q} \text{ for} T_B \le T \le T_C$$

$$= a_g S\frac{2.5}{q}[\frac{T_C}{T}] \ge \beta a_g \text{ for} T_C \le T \le T_D$$

$$= a_g S\frac{2.5}{q}[\frac{T_C T_D}{T^2}] \ge \beta a_g \text{ for} T_D \le T$$

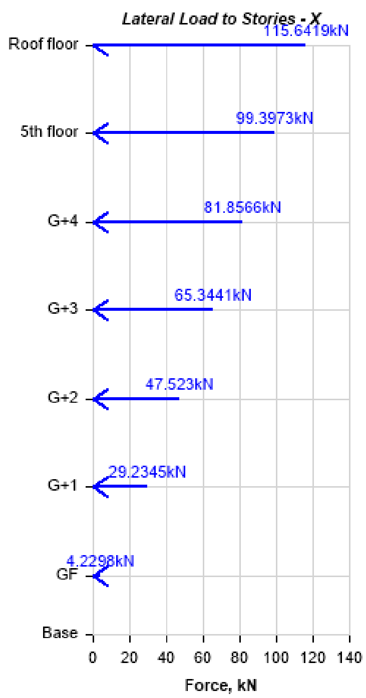
Equivalent Lateral Forces

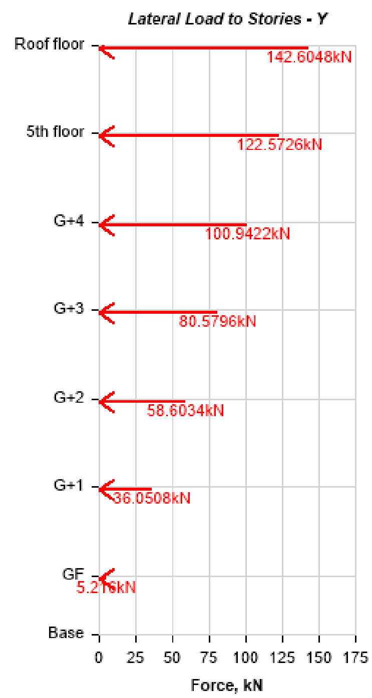
Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X	2.466	4491.5788	443.2272
Y	2.221	4491.5788	546.5692
X + Ecc. Y	2.466	4491.5788	443.2272
Y + Ecc. X	2.221	4491.5788	546.5692
X - Ecc. Y	2.466	4491.5788	443.2272
Y - Ecc. X	2.221	4491.5788	546.5692

Applied Story Forces





Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	115.6419	0
5th floor	17	99.3973	0
G+4	14	81.8566	0
G+3	11	65.3441	0
G+2	8	47.523	0
G+1	5	29.2345	0
GF	2	4.2298	0
Base	0	0	0

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	0	142.6048
5th floor	17	0	122.5726
G+4	14	0	100.9422
G+3	11	0	80.5796
G+2	8	0	58.6034
G+1	5	0	36.0508
GF	2	0	5.216
Base	0	0	0

This calculation presents the automatically generated lateral seismic loads for load pattern EQYR according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = Y + Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075m$
Structure Height Above Base, H	$H = 20$ m

Factors and Coefficients

Country =

Design Ground Acceleration, a_g	$a_g = 0.9g$

Ground Type [EC Table 3.1] = C

Soil Factor, S [EC Table 3.2]	$S = 0$
Constant Acceleration Period Limit, T_B [EC Table 3.2]	$T_B = 0$ sec
Constant Acceleration Period Limit, T_C [EC Table 3.2]	$T_C = 0$ sec
Constant Displacement Period Limit, T_D [EC Table 3.2]	$T_D = 0$ sec
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 2.34$

Seismic Response

Spectral Response Acceleration, $Sd(T_1)$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})] \text{ for } T \leq T_B$$

$$= a_g S\frac{2.5}{q} \text{ for} T_B \leq T \leq T_C$$

$$= a_g S\frac{2.5}{q}[\frac{T_C}{T}] \geq \beta a_g \text{ for} T_C \leq T \leq T_D$$

$$= a_g S \frac{2.5}{q} [\frac{T_C T_D}{T^2}] \geq \beta a_g \text{ for} T_D \leq T$$

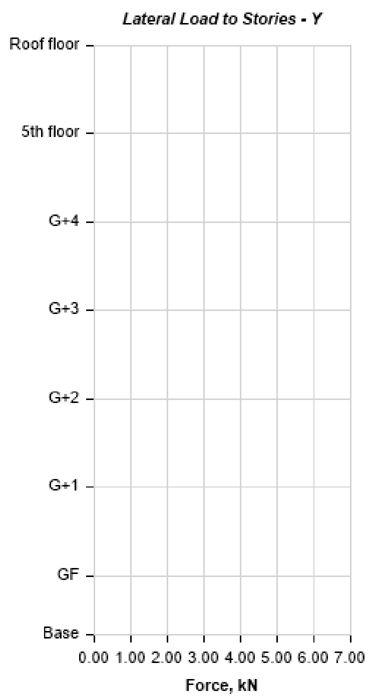
Equivalent Lateral Forces

Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
Y + Ecc. X	2.221	4491.5788	0

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	0	0
5th floor	17	0	0
G+4	14	0	0
G+3	11	0	0
G+2	8	0	0
G+1	5	0	0
GF	2	0	0
Base	0	0	0

This calculation presents the automatically generated lateral seismic loads for load pattern EQYL according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075m$
Structure Height Above Base, H	$H = 20$ m

Factors and Coefficients

Country =

Design Ground Acceleration, a_g	$a_g = 0.4g$

Ground Type [EC Table 3.1] = B

Soil Factor, S [EC Table 3.2]	$S = 1.2$
Constant Acceleration Period Limit, T_B [EC Table 3.2]	$T_B = 0.15$ sec
Constant Acceleration Period Limit, T_C [EC Table 3.2]	$T_C = 0.5$ sec
Constant Displacement Period Limit, T_D [EC Table 3.2]	$T_D = 2$ sec
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0.2$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 2$

Seismic Response

Spectral Response Acceleration, $Sd(T_1)$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})] \text{ for } T \leq T_B$$

$$= a_g S\frac{2.5}{q} \text{ for} T_B \leq T \leq T_C$$

$$= a_g S\frac{2.5}{q}[\frac{T_C}{T}] \geq \beta a_g \text{ for} T_C \leq T \leq T_D$$

$$= a_g S \frac{2.5}{q} [\frac{T_C T_D}{T^2}] \geq \beta a_g \text{ for} T_D \leq T$$

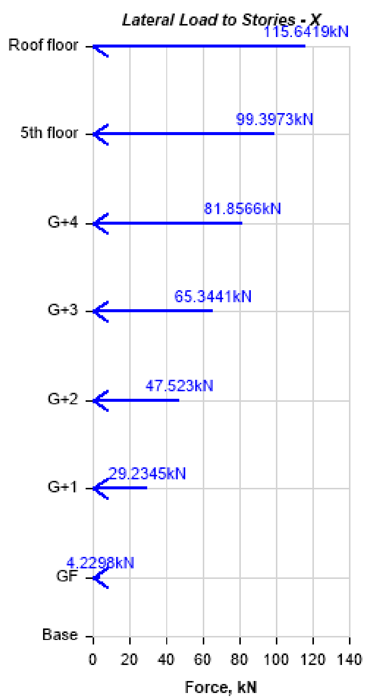
Equivalent Lateral Forces

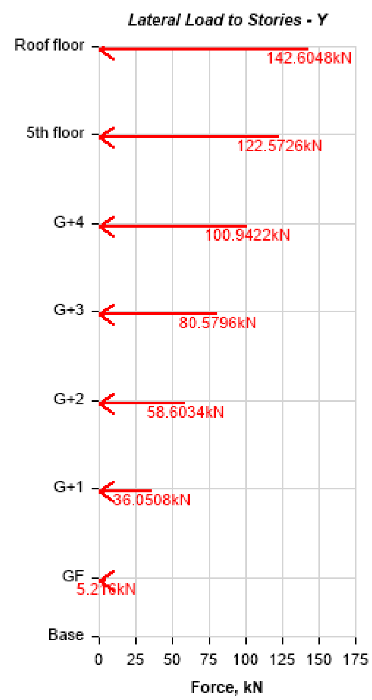
Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X	2.466	4491.5788	443.2272
Y	2.221	4491.5788	546.5692
X + Ecc. Y	2.466	4491.5788	443.2272
Y + Ecc. X	2.221	4491.5788	546.5692
X - Ecc. Y	2.466	4491.5788	443.2272
Y - Ecc. X	2.221	4491.5788	546.5692

Applied Story Forces





Story	Elevation	X-Dir	Y-Dir
	M	kN	kN
Roof floor	20	115.6419	0
5th floor	17	99.3973	0
G+4	14	81.8566	0
G+3	11	65.3441	0
G+2	8	47.523	0
G+1	5	29.2345	0
GF	2	4.2298	0
Base	0	0	0

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	0	142.6048
5th floor	17	0	122.5726
G+4	14	0	100.9422
G+3	11	0	80.5796
G+2	8	0	58.6034
G+1	5	0	36.0508
GF	2	0	5.216
Base	0	0	0

This calculation presents the automatically generated lateral seismic loads for load pattern -EQYR according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = Y - Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075\text{m}$
Structure Height Above Base, H	$H = 20\ \text{m}$

Factors and Coefficients

Country =

Design Ground Acceleration, a_g	$a_g = 0.9g$

Ground Type [EC Table 3.1] = C

Soil Factor, S [EC Table 3.2]	$S = 1.15$
Constant Acceleration Period Limit, T_B [EC Table 3.2]	$T_B = 0.2\ \text{sec}$
Constant Acceleration Period Limit, T_C [EC Table 3.2]	$T_C = 0.6\ \text{sec}$
Constant Displacement Period Limit, T_D [EC Table 3.2]	$T_D = 2\ \text{sec}$
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0.2$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 2.34$

Seismic Response

Spectral Response Acceleration, Sd($T_{1)}$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})] \text{ for } T \le T_B$$

$$= a_g S\frac{2.5}{q} \text{ for} T_B \le T \le T_C$$

$$= a_g S\frac{2.5}{q}[\frac{T_C}{T}] \ge \beta a_g \text{ for} T_C \le T \le T_D$$

$$= a_g S \frac{2.5}{q}\left[\frac{T_C T_D}{T^2}\right] \geq \beta a_g \text{ for } T_D \leq T$$

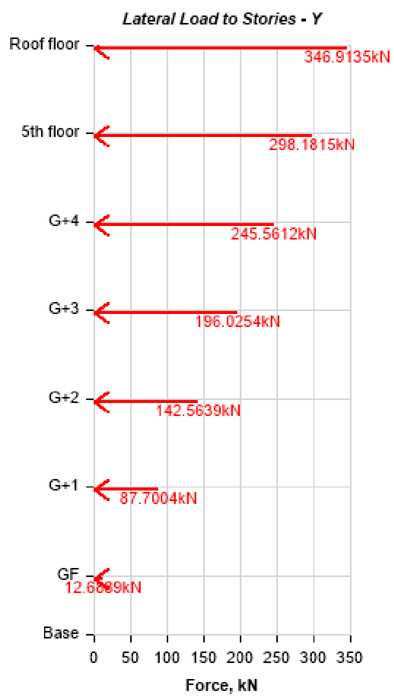
Equivalent Lateral Forces

Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
Y - Ecc. X	2.221	4491.5788	1329.6348

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	0	346.9135
5th floor	17	0	298.1815
G+4	14	0	245.5612
G+3	11	0	196.0254
G+2	8	0	142.5639
G+1	5	0	87.7004
GF	2	0	12.6889
Base	0	0	0

This calculation presents the automatically generated lateral seismic loads for load pattern -EQYL according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2] $C_t = 0.075\text{m}$

Structure Height Above Base, H $H = 20\ \text{m}$

Factors and Coefficients

Country =

Design Ground Acceleration, a_g $a_g = 0.4\text{g}$

Ground Type [EC Table 3.1] = B

Soil Factor, S [EC Table 3.2] $S = 1.2$

Constant Acceleration Period Limit, T_B [EC Table 3.2] $T_B = 0.15\ \text{sec}$

Constant Acceleration Period Limit, T_C [EC Table 3.2] $T_C = 0.5\ \text{sec}$

Constant Displacement Period Limit, T_D [EC Table 3.2] $T_D = 2\ \text{sec}$

Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$

Behavior Factor, q [EC 3.2.2.5(3)] $q = 2$

Seismic Response

Spectral Response Acceleration, $Sd(T_1)$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})]\ \text{for}\ T \le T_B$$

$$= a_g S\frac{2.5}{q}\ \text{for} T_B \le T \le T_C$$

$$= a_g S\frac{2.5}{q}[\frac{T_C}{T}] \ge \beta a_g\ \text{for} T_C \le T \le T_D$$

$$= a_g S \frac{2.5}{q} [\frac{T_C T_D}{T^2}] \geq \beta a_g \text{ for} T_D \leq T$$

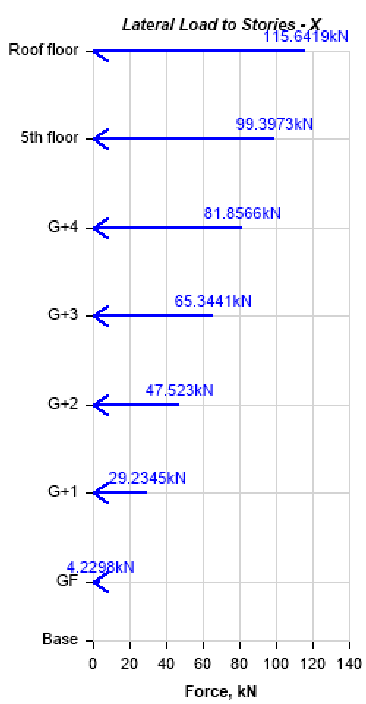
Equivalent Lateral Forces

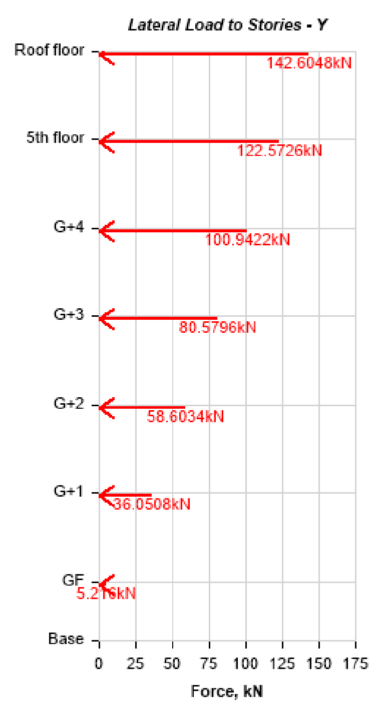
Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X	2.466	4491.5788	443.2272
Y	2.221	4491.5788	546.5692
X + Ecc. Y	2.466	4491.5788	443.2272
Y + Ecc. X	2.221	4491.5788	546.5692
X - Ecc. Y	2.466	4491.5788	443.2272
Y - Ecc. X	2.221	4491.5788	546.5692

Applied Story Forces





Story	Elevation	X-Dir	Y-Dir
	M	kN	kN
Roof floor	20	115.6419	0
5th floor	17	99.3973	0
G+4	14	81.8566	0
G+3	11	65.3441	0
G+2	8	47.523	0
G+1	5	29.2345	0
GF	2	4.2298	0
Base	0	0	0

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof floor	20	0	142.6048
5th floor	17	0	122.5726
G+4	14	0	100.9422
G+3	11	0	80.5796
G+2	8	0	58.6034
G+1	5	0	36.0508
GF	2	0	5.216
Base	0	0	0

8 Frame Analysis

8.1 Procedure for Modeling for 3D frame Analysis Using ETABS V2016

Step 1: Plot Grid Coordinates - Plot Grid Coordinates that represent the given structural design.

Step 2: Define Material - We define two types of material those are C-30, C-37 Concrete and S-400 Rebar with their material Properties.

Therefore, we used ETABS software V.2016.

8.1.1 Material specification to input Etabs

C-25: -Material type: concrete:

Symmetry type: Isotropic

Modulus of Elasticity: 29E3 Mpa

Poisson's ratio: 0.2

Modulus (G):10356491

Coeff. Thermal Expansion: 9.9E-06

Unit Weight:25 KN/m^3

S-400: - Material type: Rebar:

Unit Weight: 77 KN/m^3.

Modulus of Elasticity: 200E+09

Poisson's ratio: 0.3

Shear Modulus (G):76903069

Coeff. Thermal Expansion: 1.17E-05

Minimum Yield Stress, Fy: 440Mpa

Minimum Tensile Stress, Fu: 400.000 Mpa

Directional Symmetry type: uniaxial

Step 3: Define Frame Section - We define three types of Frame Section those are

Square Column (30*30 cm), and top tie beam (30*25cm), intermidate beam(30*30) and Grade beam (35*35cm)

These frame section has the C-30 material and S-400 rebar defined in step 1.

Step 4: Draw the different Structural Members

Using the grid System Draw the structural Members with their Defined Frame Section

Properties.

It includes assignment of Restraints (fixed Joint)

Step 5: Assign Loads

First, Introduce Live Load on Definition of load Pattern then: -

We use Load Combination: Serviceability = D. L+L.L

Combo 1 = 1.35D. L+1.5L.L

Combo 2 = 0.75 (combo 1) + EQx1

Combo 3 = 0.75 (combo 1) + (-EQx1)

Combo 4 = 0.75 (combo 1) + EQx2

Combo 5 = 0.75 (combo 1) + (-EQx2)

Combo 6 = 0.75(combo 1) + EQY1

Combo 7 = 0.75(combo 1) + (-EQY1)

Combo 8 = 0.75(combo 1) + EQY2

Combo 9 = 0.75(combo 1) +(- EQY2)

Combo 10 = DL + LL

Combo 11 = Envelope (Combo 2 up to Combo 5)

Combo 12 = Envelope (Combo 6 up to Combo 9)

Where: -

DL and LL is dead load and live load of the building.

EQX is earth quick along X- axis

EQY is earth quick along Y- axis

Next we assigned Fi (Story shear) as a joint load on each frame Joint. Note that We analyzed the 3D frame separately for each earthquake coming from X and Y Direction. Finally, we imposed the un-factored distributed load transferred from slab and wall on the beam members.

Step 6: Analysis

After checking for errors we run the analysis.

Finally, as shown below we determined the moments for major and minor Axis and shear Force

8.2 Analysis output of ETABS Beam Shear force diagram

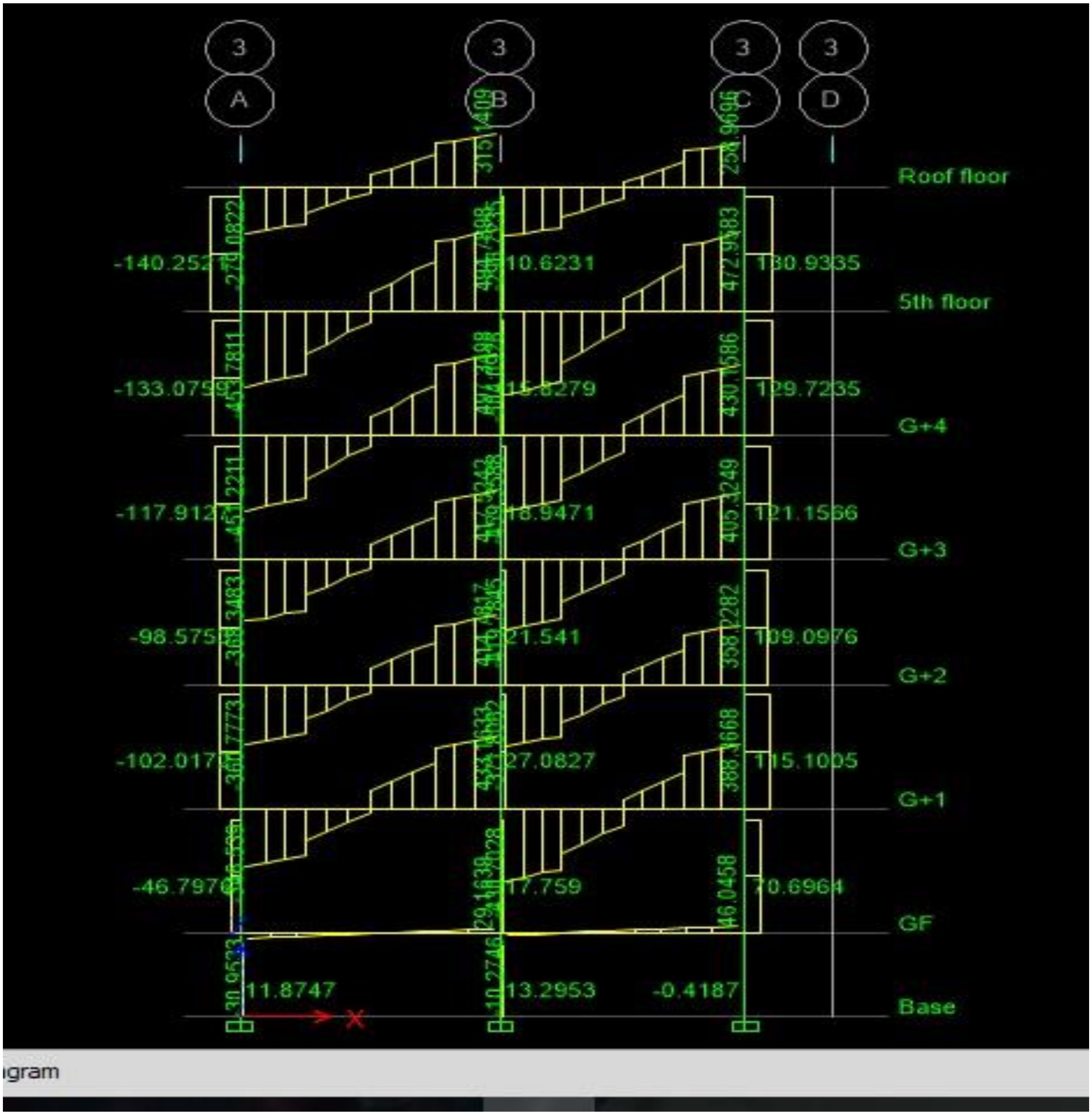


Figure 13 Shear at Axis 3

8.3 Analysis output of ETABS Beam Bending Moment diagram

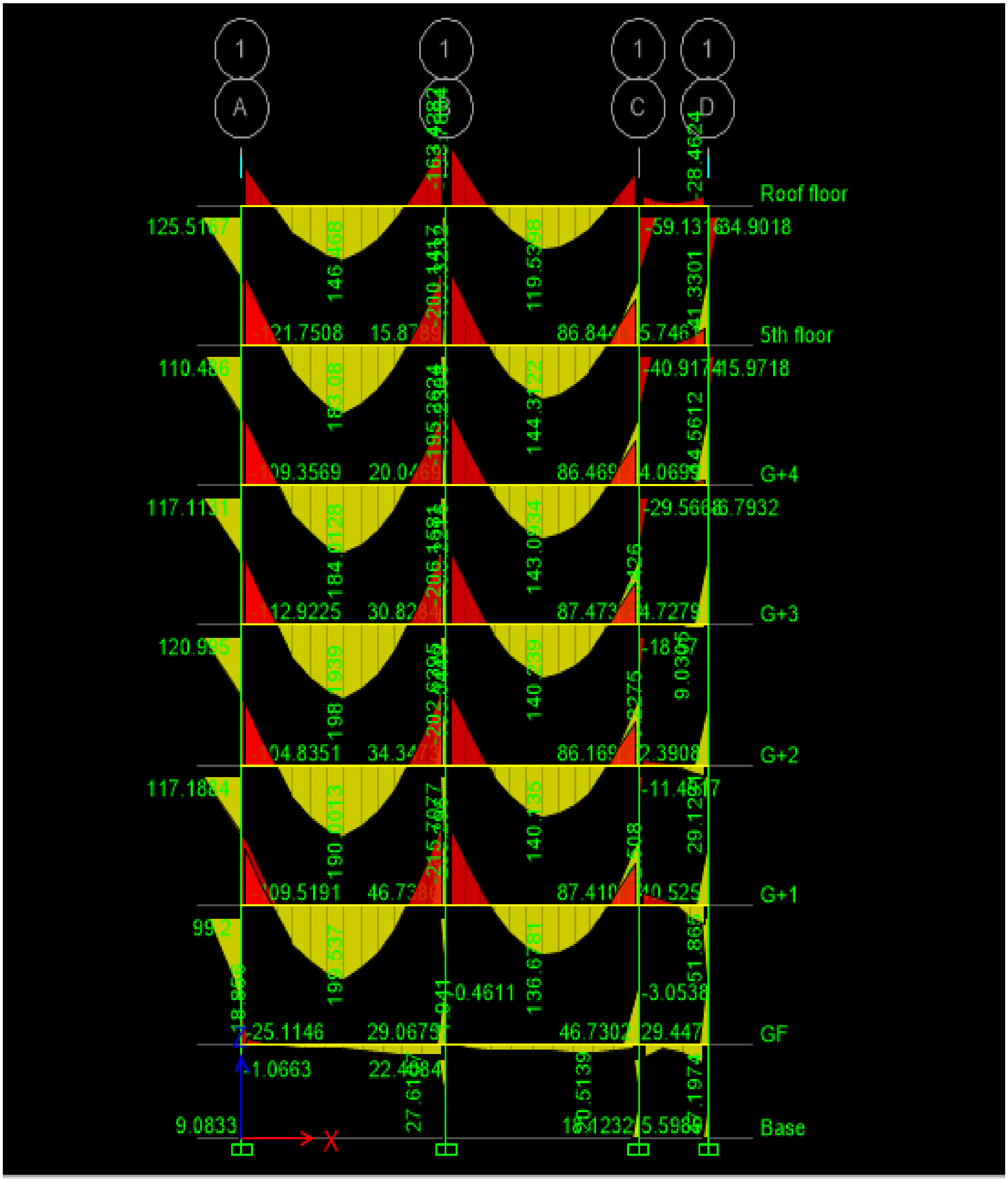


Figure 14 Moment at Axis 1

8.4 Analysis output of ETABS Column Axial force and Moments

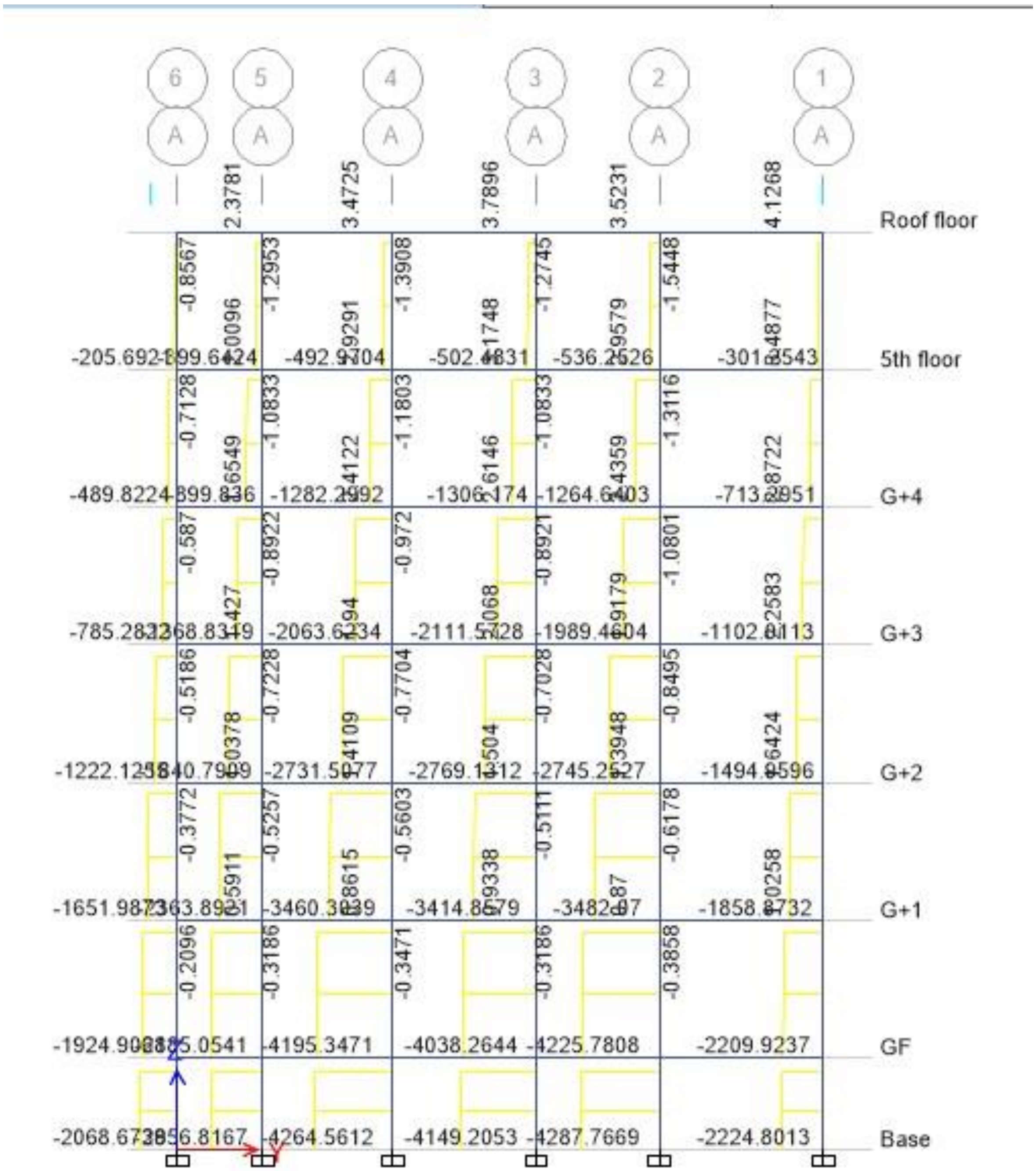


Figure 15 Axial force at Axis A

8.5 Analysis output of ETABS 3D Deflection Analysis

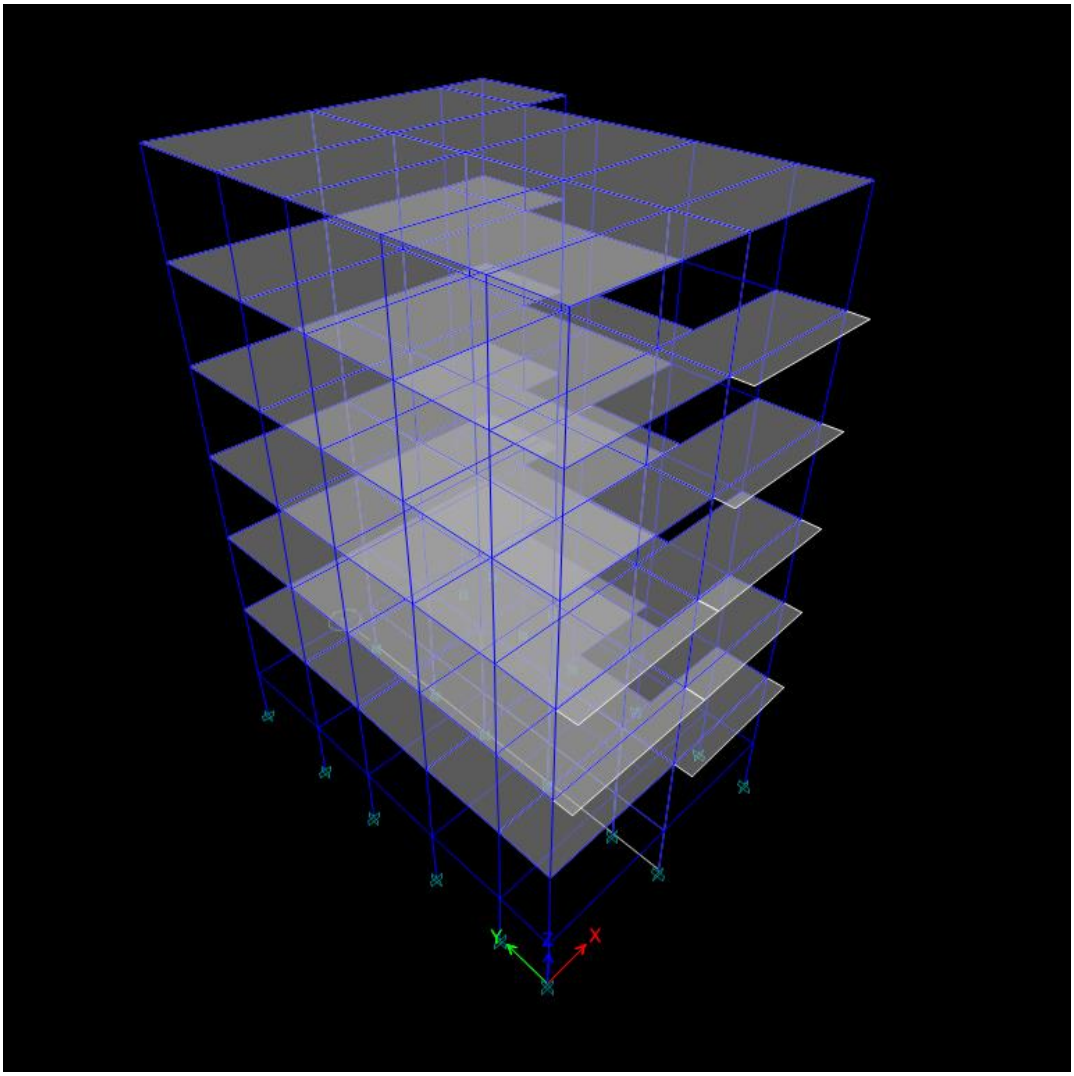


Figure 16 Analysis output of ETABS 3D Deflection Analysis

9 Design of Beams

Beams are flexural members which are used to transfer the loads from slab to columns. Basically beams should be designed for flexure (moment). Furthermore, it is essential to check and design the beam sections for torsion and shear. Beams may be designed for flexural moment depending on the magnitude of the moment and the X- sectional dimensions. On the other hand, the beam can be singly reinforced, doubly reinforced section. Doubly reinforced cross section Incase when the dimension of the section is limited the concrete may be subjected to higher compression stress. Thus additional steel bars are placed in the compression zone of the section.

Types of beam

There are many types of beams that are classified according to different bases. Some of the bases for classification are:

- Based on support condition
- Based on geometry
- Based on equilibrium condition

Material data:

- Grade of concrete C-25
- Steel grade S-400
- Use concrete cover C = 30mm

Cross sections of beams are

- Top tie beam =250mm*300mm
- Intermediate beam= 300*300mm
- Grade beam =350*350mm

Design constants

- Concrete Partial safety factor for concrete c = 1.5
- Fcu = 25 Mpa
- fck = 0.8*fcu = 0.8*25 = 20Mpa
- fcd=0.85*fck/1.5 = 0.85*20/1.5 = 11.33 Mpa
- fctd =1.03 Mpa
- Steel Partial safety factor for steel s = 1.15
- fyk =400Mpa

- fyd = fyk/1.15 = 400/1.15 = 347.82Mpa
- c = 0.0035
- s = Fyd/E = 347.82/200000 = 0.00174

Concrete cover for Beam

A durable structure should satisfy strength and serviceability requirements throughout its design working life. One of the main causes for poor durability is the corrosion of steel reinforcement. Good quality dense concrete and adequate cover are prime requirements in order to produce durable structures. Table 2.5 gives the details of water/cement ratio, minimum cement content for producing good quality concrete to satisfy various exposure classes and Cmin, dur which is the minimum cover to steel from durability considerations.

The minimum cover Cmin to steel should satisfay the code equation:

Cmin= Maximum {Cmin, b; Cmin, dur; 10mm}

i. For safe transmission of bond forces, the required cover is Cmin, b as given in Table 4.2 of ES-EN2005.
ii. For separated bars, Cmin, b ≥ bar diameter, φ.
iii. For bundled bars, Cmin, b ≥ equivalent bar diameter, φn.
φn = φ √nb ≤ 55 mm. nb = Number of bars in the bundle. nb ≤ 4 for vertical bars in compression and for bars in a lapped joint, nb ≤ 3 for all other cases.
If the nominal maximum size of the aggregate is greater than 32 mm, Cmin, b shouldbe increased by 5 mm. The cover can be reduced if stainless steel bars are used.
For a design life of 50 years, minimum values of Cmin, dur for various classes of exposure are are given as follows in Table 4.4 N of ES-EN2005

X0=10mm, XC1=15mm, XC2/XC3=25mm, XC4=30mm, XD1/XS1=35mm, XD2/XS2=40mm, XD3/XS3=45mm

Exposure condition (according to ES EN 1992-1-1:2013 Table 4.1)

Structural class 2—XC1 – moderate humidity (concrete inside building with moderate or high air humidity)

Step 1: cover determination

<u>Cover for stirrup</u>

C nom=C min + C dev

C min → max (C min; bond or C min: dur or 10)

Table 21 Recommended limiting values for composition and properties of concrete and minimum cover to steel for durability

Class Designation	Maximum w/c ratio	Minimum Strength class	Minimum Cement content	Minimum Air content(%)	cdur (mm)
X0	-	C12/15	-	-	10
XC1	0.65	C20/25	260	-	15(25)
XC2	0.60	C25/30	280	-	25(35)
XC3	0.55	C30/37	280	-	25(35)
XC4	0.50	C30/37	300	-	30(40)
XD1	0.55	C30/37	300	-	35(45)
XD2	0.55	C30/37	320	-	40(50)
XD3	0.45	C35/45	320	-	45(55)
XS1	0.50	C30/37	300	-	35(45)
XS2	0.45	C35/45	320	-	40(50)
XS3	0.45	C35/45	340	-	45(55)
XF1	0.55	C30/37	300	-	
XF2	0.55	C25/30	300	4.0	
XF3	0.50	C30/37	320	4.0	
XF4	0.45	C30/37	340	4.0	
XA1	0.55	C30/37	300	-	
XA2	0.50	C30/37	320	-	
XA3	0.45	C35/45	360	-	

For C-20/25 the exposure class is XC1 which implies dry or permanently wet.

For XC1 min C-20/25 Cmin; b requirements

Arrangement of bar for separated use

Assume the diameter of the bar is Ø10mm

Cmin;dur requirements

For structural class-2 (S2)

For exposure condition –Xc1

• use 10mm

$$C_{min} = \begin{cases} \text{Cmin; bond } = 10 \\ \text{C min: dur} = 10 \\ 10 \end{cases}$$

C min=10

C nom=C min + C fire resistance

C nom =10+10=20mm

Concrete cover for longitudinal bar

Cnom=C min+ C dev

C min → max (C min; bond or C min: dur or 10)

C min → (Cmin; bond =10 or C min: dur=10 or 10)

C min=10

C nom=C min + C fire resistance

The recommended value is 10mm because of cast in suite.

Cnom=Cmin +C fire resistance ==10+10=20mm

The governing C nominal is 20mm

Depth, D=d+d'

Where: d' = cover + Φst +Φlong/ 2 $use\Phi st = 8mm and \Phi long = 16mm$

d' = cover + Φst + Φlong/ 2 = $20mm + 8mm + 16mm/ 2 = 36mm$

Therefore, we have taken ΔCdev = 10mm

Cnom=Cmin+ΔCdev =20mm+10mm=30mm

STEP 2: Check depth for deflection

l/d =K(11+1.5$\sqrt{fck}*\rho o/\rho$+3.2*$\sqrt{fck}(\rho o/\rho$-1)^(3/2))*F1*F2*F3 ….if$\rho<\rho o$ art.7.4.2.(7.16a)

l/d =K(11+1.5$\sqrt{fck}*\rho o/(\rho - \rho$')+1/12*$\sqrt{fck}\sqrt{(\rho o/\rho)}$)*F1*F2*F3 ….if$\rho>\rho o$ art.7.4.2.(7.16b)

Table 22 Basic ratios of span/effective depth for reinforced concrete members without

Structural System	K	Concrete highly stressed ρ = 1.5%	Concrete lightly stressed ρ = 0.5%
Simply supported beam, one – or twoway spanning simply supported slab	11	14	20
End span of continuous beam or oneway continuous slab or two-way spanning slab continuous over one long side	11.3	18	26
Interior span of beam or one-way or two-way spanning slab	11.5	20	30

Slab supported on columns without beams (flat slab) (based on longer span)	11.2	17	24
Cantilever	0.4	6	8

Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.

Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.

Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory

axial compression at axis 1 span A-B

Beam cross section 300*300

Taking L/d=26 for end span L/d = 30 for interior span from ES EN 1992:2015 9table 7.4N

Where L= effective length of the beam

d = effective depth but those value is for steel grade 500,

We must have to modify it. In our case, Modification factor =500/400=1.25

Therefore,

- End span, 26*1.25=32.5
- Interior span, 30*1.25=37.5
- Cantilever, 8*1.25=10

End span,d=L/26=5000/32.5=153.85

Interior span,d=L/30= 5000/37.5=133.34

$$Drequired = drequired + stirrup + \frac{Øbar}{2} = 153.85 + 8 + \frac{20}{2} = 170mm$$

Therefore, D provided=500>Drequired …ok

Step-3 check the whether the beam single or double reinforced

A beam should be treated as singly reinforced if K < 0.167

A beam should be treated as doubly reinforced if K>0.167

Where $K = \frac{Msd}{bd2fck}$=$\frac{199.5}{300*300*20} = 0.128$

Step-4 provide reinforcement

$$Z = \frac{d}{2}(1 + \sqrt{1 - 3.53K} = \frac{300}{2}(1 + \sqrt{1 - 3.53 * 0.128)} = 261$$

$$Ast, cal = \frac{Msd}{Zfyd} = \frac{69.5}{261 * 347.83} = 765.5$$

Step-5 check the minimum and maximum reinforcement

A_{st},min = max $\begin{cases} 0.26\frac{f_{ctm}}{fyk}bd = 0.26\frac{2.2}{400} * 300 * 300 = 128.7\ mm^2 \\ 0.0013bd = 0.0013 * 300 * 300 = 117\ mm^2 \end{cases}$

A_{st},min=128.7mm^2

A_{st},max=0.04bd=0.04*300*300=3600mm^2

Therefore A_{st},min ≤ A_{st},cal ≤A_{st},max

$$no\ of\ bar = \frac{Aspro}{Ast} = \frac{765.5}{200.96} = 3.8$$

Use 4 Ø 16

9.1 Longitudinal reinforcement

9.1.1 Cross-section design (Width and depth)

Depth for top tie beam

d=h-d'=300-36=264mm

Depth for intermediate tie beam

d=h-d'=300-36=264mm

Depth for Grade beam

d=h-d'=350-36=314mm

9.1.2 Design for flexure

Axis-A

Roof

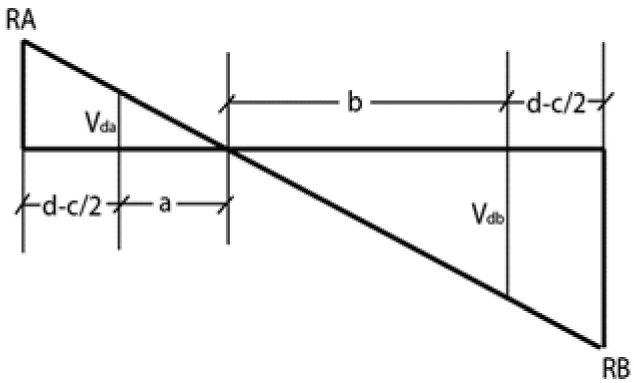
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	87	250	300	0.1933	234.5251	double	107.25	3000	1066.517	1066.517	5.30711	6Ø16
Span	1-2	12.61	250	300	0.0280	292.388	single	107.25	3000	123.9919	123.9919	0.616998	2Ø16
Support	2	61.72	250	300	0.1372	257.7331	single	107.25	3000	688.4836	688.4836	3.425973	5Ø16
Span	2-3	19.96	250	300	0.0444	287.7573	single	107.25	3000	199.4216	199.4216	0.992345	2Ø16
Support	3	58.53	250	300	0.1301	260.3153	single	107.25	3000	646.4229	646.4229	3.216675	5Ø16
Span	3-4	19.2	250	300	0.0427	288.2433	single	107.25	3000	191.5049	191.5049	0.95295	2Ø16
Support	4	50.09	250	300	0.1113	266.8722	single	107.25	3000	539.6169	539.6169	2.685195	4Ø16
Span	4-5	18.9	250	300	0.0420	288.4346	single	107.25	3000	188.3876	188.3876	0.937438	2Ø16
Support	5	43.4	250	300	0.0964	271.8191	single	107.25	3000	459.0369	459.0369	2.28422	3Ø16
Span	5-6	45.559	250	300	0.1012	270.2449	single	107.25	3000	484.6794	484.6794	2.41182	3Ø16

Table 23 Reinforcment design for flexure

Stirrup calculation for beam

Fck=20

Fyk=400 bw=300 d=125



Dw=0.08*$\frac{fyk\verb|^|0.5}{400}$

Dw =0.08*$\frac{20\verb|^|0.5}{400}$ =0.000894

ASW=$\frac{\pi d2}{4}$=$\frac{3.14*8\verb|^|2}{4}$= 50.24

$\cos\alpha$ =2.5

Fyd=$\frac{400}{1.15}$=347.8

Z=0.9*d=0.9*125=0.325

a =$\frac{RA*L}{RA+Rb}$ - (d+C$\frac{C}{2}$2)

a= $\frac{107.97*5}{107.97+104.22}$-0.325

= 2.219

b= L-(2*D+C$\frac{C}{2}$2+a)

= 5-(2*0.325+2.219) = 2.131

VE_{da}=(RA*a)$\frac{RA*a}{D+\frac{C}{2}+a}$= $\frac{107.97*2.219}{0.325+2.219}$= 94.178

VE_{db}=$\frac{RB*b}{D+\frac{C}{2}+b}$=$\frac{104.22*2.131}{0.325+2.131}$= 90.178

VE_d=Max (VE_{da},,VE_{db})

=Max(94.178,90.178)

VE_d=94.178

In tabular form

AXIS1		ROOF				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro
A-B	107.97	180.12	93.75	27.3	224.7	27.3
B-C	162.98	222.08	93.75	22.1	224.7	22.1
C-D	48.93	194.1735	93.75	25.3	224.7	25.3

Table 24 Stirrup for beam

9.2 Reinforcement detail for Beam

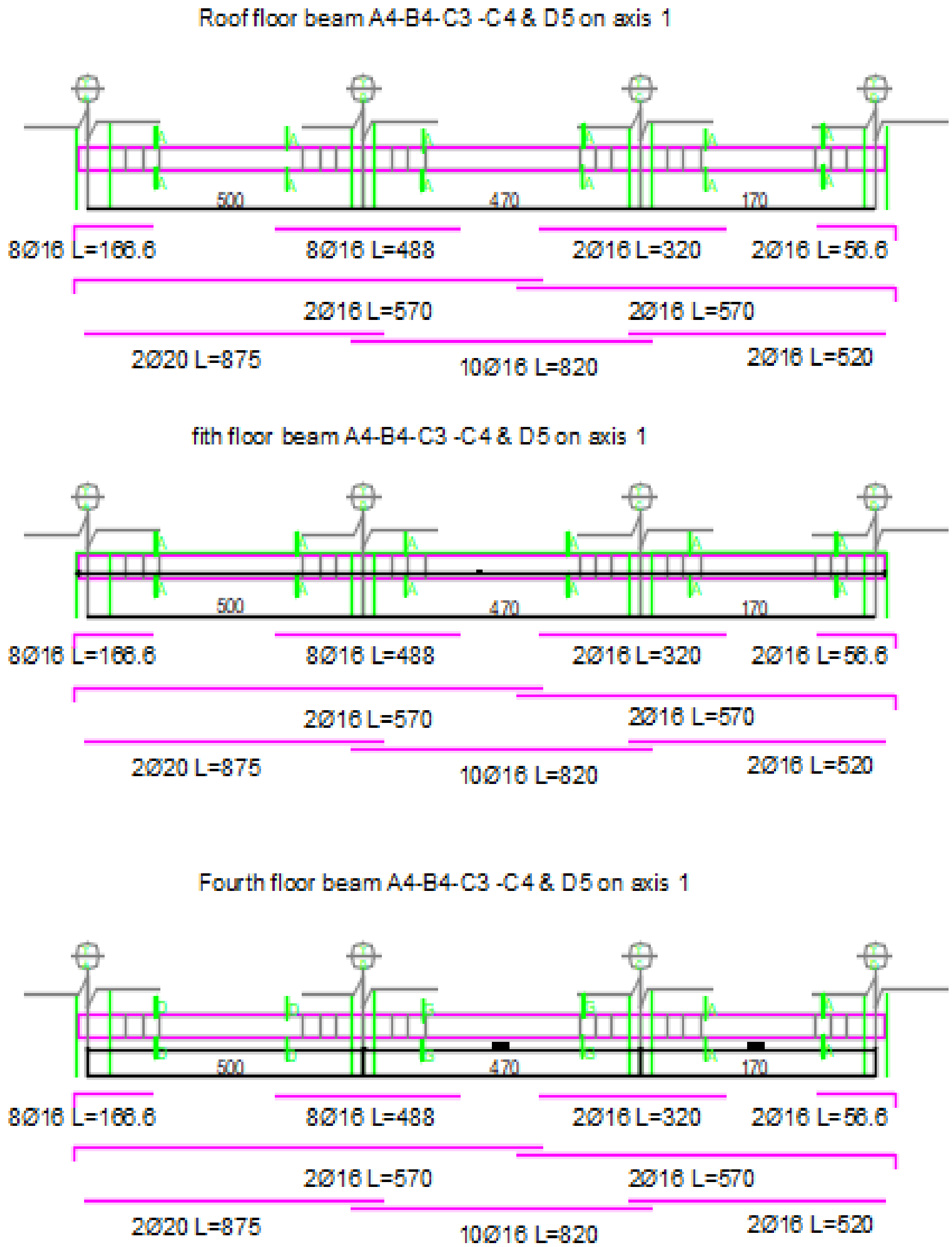
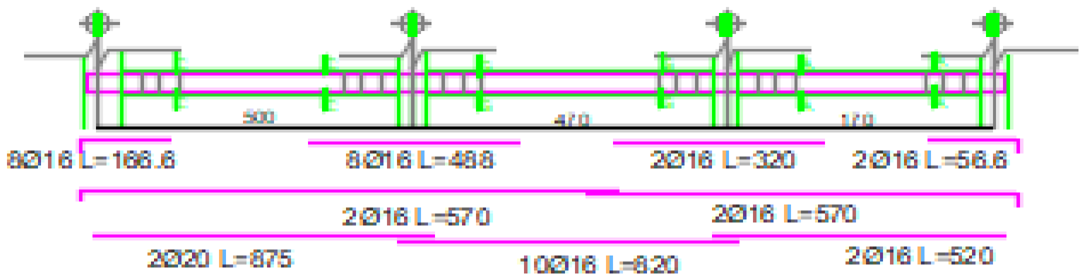
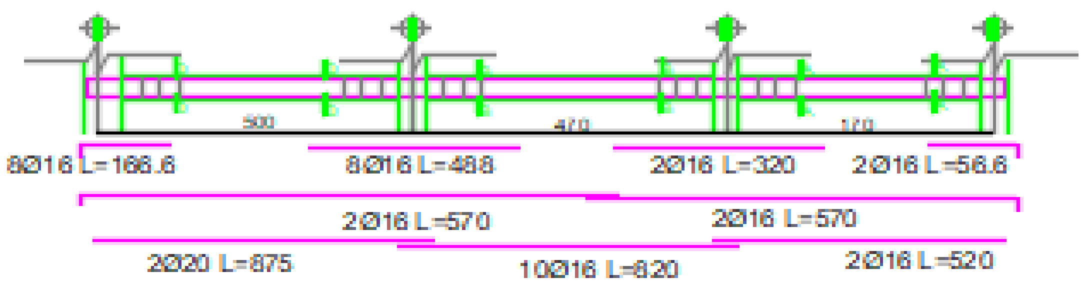


Figure 17 Reinforcement detail for roof, fifth & fourth floor

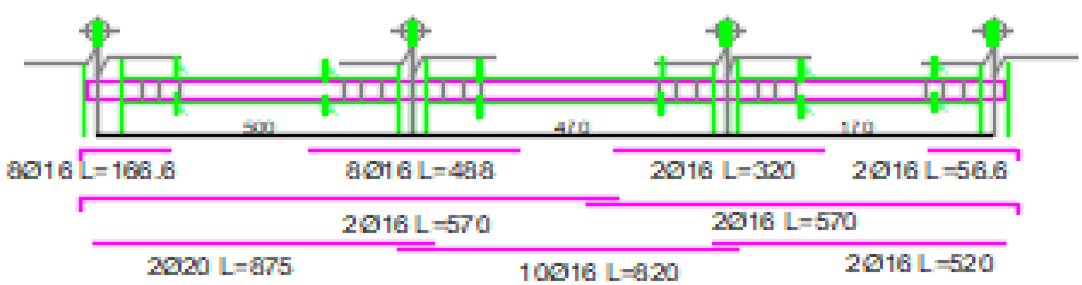
Third floor beam A4-B4-C3 -C4 & D5 on axis 1



Second floor beam A4-B4-C3 -C4 & D5 on axis 1



First floor beam A4-B4-C3 -C4 & D5 on axis 1



Ground floor beam A4-B4-C3 -C4 & D5 on axis 1

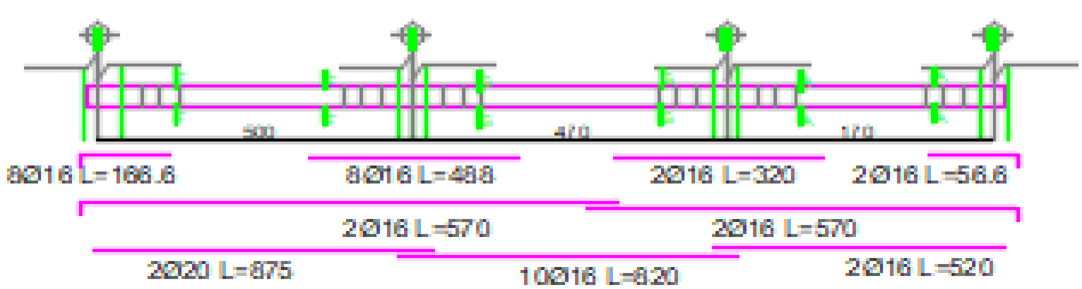


Figure 18 Reinforcement detail for third, second, first& ground floor

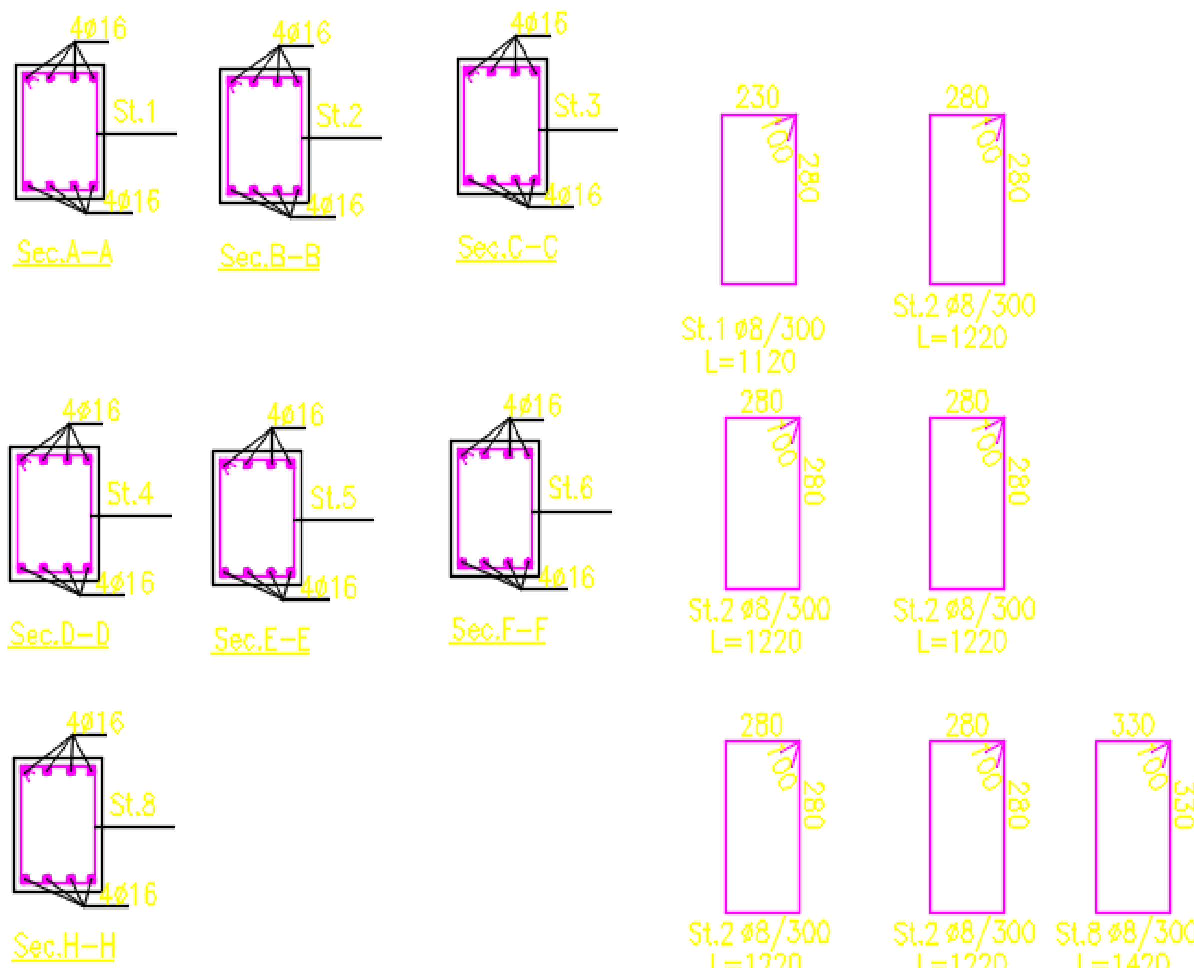


Figure 19 reinforcement detail for beam

10 Column Design

Column is a vertical structural member transmitting axial compression loads with or without moments. The cross sectional dimensions of a column are generally considerably less than its height. Column support mainly vertical loads from the floors and roof and transmit these loads to the foundation.

The strength of a column depends on many factors including the following:

- strength of the material
- Shape and size of the cross section
- Length
- The degree of positional and directional restraints at its end

Classification of columns

- Classification on the basis of geometry; rectangular, square, circular, L-shaped, T-shaped, etc. depending on the structural or architectural requirements.
- Classification on the basis of degree of slenderness; short column, slender column.
- Classification on the basis of loading: axially loaded column, columns under uni-axial bending or columns under biaxial bending.
- Classification on the basis of lateral reinforcement; tied columns, spiral columns.
- Classification on the basis of lateral stability is provided to the structure as a whole; braced or un braced column

Over all procedure in design of column

Step 1 Determination of concrete cover for column
Step 2 Determination of effective length of the column
Step 3 maximum and minimum reinforcement calculation
Step 4 Check slenderness
Step 5 reinforcement calculation

10.1 Design of column cross-section

Rectangular column sample calculation for 5th floor column

Given data

Top beam	depth(m)	width(m)	Length(m)
b1(x,x)	0.3	0.25	5
b2(x,x)	0.3	0.25	4.7
b1(y,y)	0.3	0.25	3.15
b2(y,y)	0.3	0.25	2.88

bottom beam	depth(m)	width(m)	length(m)
b3(x,x)	0.3	0.3	5
b4(x,x)	0.3	0.3	4.7
b3(y,y)	0.3	0.3	3.15
b4(y,y)	0.3	0.3	2.88
Column	0.3	0.3	3

Mtopx-x=145.64KNm

Mboty-y=145KNm

Mtopx-x=149.14KNm

Mboty-y=149.7KNm

NED=485.2KN

Concrete cover design

The concrete cover is the distance from the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface.

Cover Design for Bond

In order to transmit bond forces safely and to ensure adequate compaction of the

Concrete, the minimum cover should not be less than Cmin,

From table 4.2 bond requirement, arrangement of bars for separated minimum cover

Cmin, b= diameter of bar

Assume Φ20 longitudinal bar and Φ25 nominal maximum aggregate size;

Therefore; Cmin, b= 20mm.

If nominal maximum aggregate size is greater than 32mm Cmin, b should be increased by 5mm

Cover Design for Corrosion/Durability

The condition of exposure is assumed to be XC1

Exposure Class to determine Cmin, dur

ForXC1, member with slab geometry (position of reinforcement not affect by construction process the exposure class is reduced class by one

ForXC1 indicative strength class is C-20/25

Note: The recommended Structural Class (design working life of 50 years) is S4 for the Indicative concrete strengths given in Annex E and the recommended modifications to the Structural class is given in Table 4.3N. But based on the above table the exposure class is reducing by 1 and the structural class would be S3.

The value of minimum cover required for durability of reinforcement steel is determined by using ES EN 1992:2015 tables 4.4N.

Using structural class, S3 and XC1

First, the concrete cover

With:

$c_{min,b}$= 16 mm

$c_{min,dur}$= 10 mm

c_{min}= max (cmin,b; cmin,dur ; 10 mm) =max (16; 10; 10 mm) = 12 mm

Δc_{dev} = 16 mm.

ΔCdev (allowance in Design for Variation)

Note:The value of $\Delta c_{dev}$for use in a country may be found in its National Annex .The recommended values is 10 mm

We obtain from relation

cnom=cmin+Δcdev = 20 + 10 = 30 mm .

Cover Design for Fire

For the slab to sustain fire incident for 120 minutes the required cover and minimum height of the Section can be determined form Table 5.8 0f EN 1992-1-2:2015

Cfire = 20mm

Hs = 120mm

Governing Cover for Design

Governing cover accounting for

Corrosion

Bond/Durability

Fire

Cover = 36mm

Concrete C 20/25

Rebar S400

Fcd =0.85*fck/rc=0.85*20/1.5=11.33M

$$fyd = \frac{\text{fyk}}{\text{rs}} = \frac{400}{1.15} = 347.83\text{Mpa}$$

Determination of effective length of the column

Effective length is used to account for the shape of the deflection curve: it can also be defined as buckling length i.e. the length of pin-ended column with constant normal force, having the same cross section and buckling load.

Effective length (lo) for braced member

From EBCS EN 1992-1-1:2014 (5:15)

$$\text{Lo}= \sqrt{((l + \frac{k1}{0.45+k2})(l + \frac{k2}{0.45+k2})}$$

where, K1,K2-relative flexibilities of rotational restraint at both ends 1,2

l-length of column

K1-stiffness at end 1

K2-stiffness at end 2

Stiffness at each end (K) =column stiffness/Σbeam stiffness

$$\mathbf{K1}\ \text{x} - \text{x} = \frac{\text{Ic}/\text{Lc}}{2\left(\frac{Ib-1\text{T}}{\text{Lb}-1\text{T}} + \frac{\text{Ib}-2\text{T}}{\text{Lb}-2\text{T}}\right)}$$

Ic = $\frac{bh^3}{12} = \frac{300*300^3}{12}$ =6.75*10^8mm^4

IB-1T = $\frac{250*300^3}{12}$ = 3.906*10^8mm^4

K1y-y = $\frac{6.75*10^8/3}{2\left(\frac{3.906*10^8}{3.15}+\frac{3.906*10^8}{2.88}\right)}$

K1y-y= 0.43

K1x-x = $\frac{6.75*10^8/3}{2\left(\frac{3.906*10^8}{5}+\frac{3.906*10^8}{4.7}\right)}$

= 0.7

K2y-y = $\frac{6.75*10^8/3}{2\left(\frac{3.906*10^8}{3.15}+\frac{3.906*10^8}{2.88}\right)}$

= 0.43

K1x-x = $\frac{6.75*10^8/3}{2\left(\frac{3.906*10^8}{5}+\frac{3.906*10^8}{4.7}\right)}$= 0.7

Lo x-x = $\sqrt{((1+\frac{0.7}{0.45+0.43})(1+\frac{0.7}{0.45+0.7})}$

Lo x-x =2.23m

Lo y-y = $\sqrt{((1+\frac{0.43}{0.45+0.43})(1+\frac{0.43}{0.45+0.7})}$

Lo y-y=2.06m

10.2 Design of column Reinforcement

maximum and minimum reinforcement calculation

As,min=max$\begin{cases} 0.1 * \text{NED/fyd} = 0.1 * {485.2}/{347.8} = 139.5mm^2 \\ 0.002 * Ac = 0.002 * 300 * 300 = 180mm^2 \end{cases}$

As,min=180mm2

As,max=0.04∗Ac=0.04∗300*300=16000mm2

design action

The effect of imperfection taken from According to ES EN 1992: Art 5.2.7

$$eix\_xmax\begin{cases}\frac{Lo}{400}=\frac{2230}{400}=5.575mm, eix-x=20mm\\ \frac{h}{30}=\frac{300}{30}=10mm\\ 20mm\end{cases}$$

$$eiy\_ymax\begin{cases}\frac{Lo}{400}=\frac{2060}{400}=5.15mm, eiy\_y=20mm\\ \frac{h}{30}=\frac{300}{30}=10mm\\ 20mm\end{cases}$$

MED=max {MO2, Moe+M2, Mo1+0.5M2}

M01=min {|Mtop|,|Mbottom|+ei*NED}

M02=max {|Mtop|,|Mbottom|+ei*NED}

Moe=0.4*M01+0.6*M02≥0.4*M02

For first order moment M2=0

In the X-direction

M01x-x=min {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M01x-x=min {|145.64|,|149.14|+0.02*485.2}

min {242.68,246.18}

M01x-x=242.68KNm

M02x-x=max {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M02x-x=max {242.68,246.18}

M02x-x=246.18KNm

Moex-x=0.4*M01x-x+0.6*M02x-x≥0.4*M02x-x

Moex-x=0.4*242.78+0.6*246.18≥0.4*246.18

= 244.78 ≥ 98.472

Moex-x=244.78KNm

MEDx-x=246.18KNm

In the Y-direction

M01y-y=min {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M01y-y=min {|145.64|,|149.7|+0.02*485.2}

min {242.04,246.74}

M01y-y=242.04KNm

M02y-y=246.76KNm

Moey-y=0.4*M01y-y+0.6*M02y-y≥0.4*M02y-y

Moey-y=0.4*242.04+0.6*246.74≥0.4*246.74

= 244.86 ≥ 98.696

Moey-y=244.86KNm

MEDy-y=246.18KNm

Check slenderness

Slenderness ratio λ=$\frac{lo}{i}$where lo-effective length of column

i-is radius of gyration of the uncracked concrete

λx-x= $\frac{lox\_x}{i}$ =19.39

λy-y = $\frac{loy\_y}{i}$ =17.9

The minimum limiting value of slenderness is

λlim=20*A*B*C/$\sqrt{n}$

λlimx-x=20*A*B*C/$\sqrt{n}$

Where, A – 1/ (1 + 0.2φef), φef-effective creep ratio

B - √1 + 2ω,-Asfyd/Acfcd, mechanical reinforcement ratio

C = 1.7-rm

n=NEd/Acfcd, relative normal force

rm,moment ratio=Mo1x-x/Mo2x-x=96.07/103.31=0.93

Mo1, Mo2 are the first order end moments, |Mo2|≥|Mo1|

If φef is not known, A=0.7

If ω is not known, B=1.1

C=1.7-r_m

X- dxn

r_m= $\frac{\text{M01x\_x}}{\text{M02x\_x}} = \frac{242.68}{246.18}$=0.98

C = 1.7-0.98=0.72

Ac=$b*h$ = 300 ∗ 300 = 90000mm^2 for rectangular column

n=$\frac{\text{NEd}}{\text{Acfcd}} = \frac{485.2*1000}{90000*11.33}$=0.476

Therefore, slenderness limit (ƛlimx-x) =20*A*B*C/$\sqrt{n}$

ƛlimx-x = $\frac{20*0.7*1.1*0.72}{\sqrt{0.476}}$ =16.07

Y- dxn

A=0.7

B=1.1

C=1.7-(0.68)=0.72

n=$\frac{NEd}{Acfcd}$ = $\frac{485.2*1000}{90000*11.33}$=0.476

ƛlimy-y =20*A*B*C/$\sqrt{n}$

ƛlimy-y = $\frac{20*0.7*1.1*0.72}{\sqrt{0.476}}$ =16.07

Check slenderness

λx-x >λlimx-x, slender column otherwise short column

λy- y>λlimy-y, slender column otherwise short column

λx-x <λlimx-x= 19.39 >16.07, slender column

λy- y< λ limy-y= 17.9 > 16.07, slender column

vsd= $\frac{NEd}{Acfcd}$ = $\frac{485.2*1000}{90000*11.33}$ =0.476

μsdx-x= $\frac{MEDx\_x}{fcd*Ac*h}$= $\frac{246.18*10^6}{90000*11.33*300}$ =0.81

μsdy-y= $\frac{MEDx\_x}{fcd*Ac*h}$= $\frac{246.18*10^6}{90000*11.33*300}$ =0.81

Using biaxial rectangular chart number

ω=0.3

Longitudinal reinforcement

As,tot = $\frac{\omega*Ac*fcd}{fyd}$

As,tot=$\frac{0.3*90000*11.33}{347.8}$

As,tot=879.56 mm²

As min=180mm² and As max=3600mm²

As min <As<As max…………………………OK

Take Ø20mm, as=$\frac{3.14*20^2}{4}$=314mm²

Number of bar=As/as=879mm²/314mm²= 4bar

Provide 4 Ø20mm

For shear reinforcement

Diameter of bar = $Max\begin{cases}\frac{1}{4} * longitudinal\ bar\ diameter = \frac{20}{4} = 5 \\ 5mm \\ Diameter\ of\ stirrup = 8mm\end{cases}$

From above take Ø8mm

Spacing of shear reinforcement = $Min\begin{cases}20 * \text{Ølong} = 20 * 20\text{mm} = 400\text{mm} \\ Lesser\ dimention\ of\ column = 300mm \\ 400mm\end{cases}$

Finally, provideØ8 c/c 300mm

10.3 Reinforcement detail

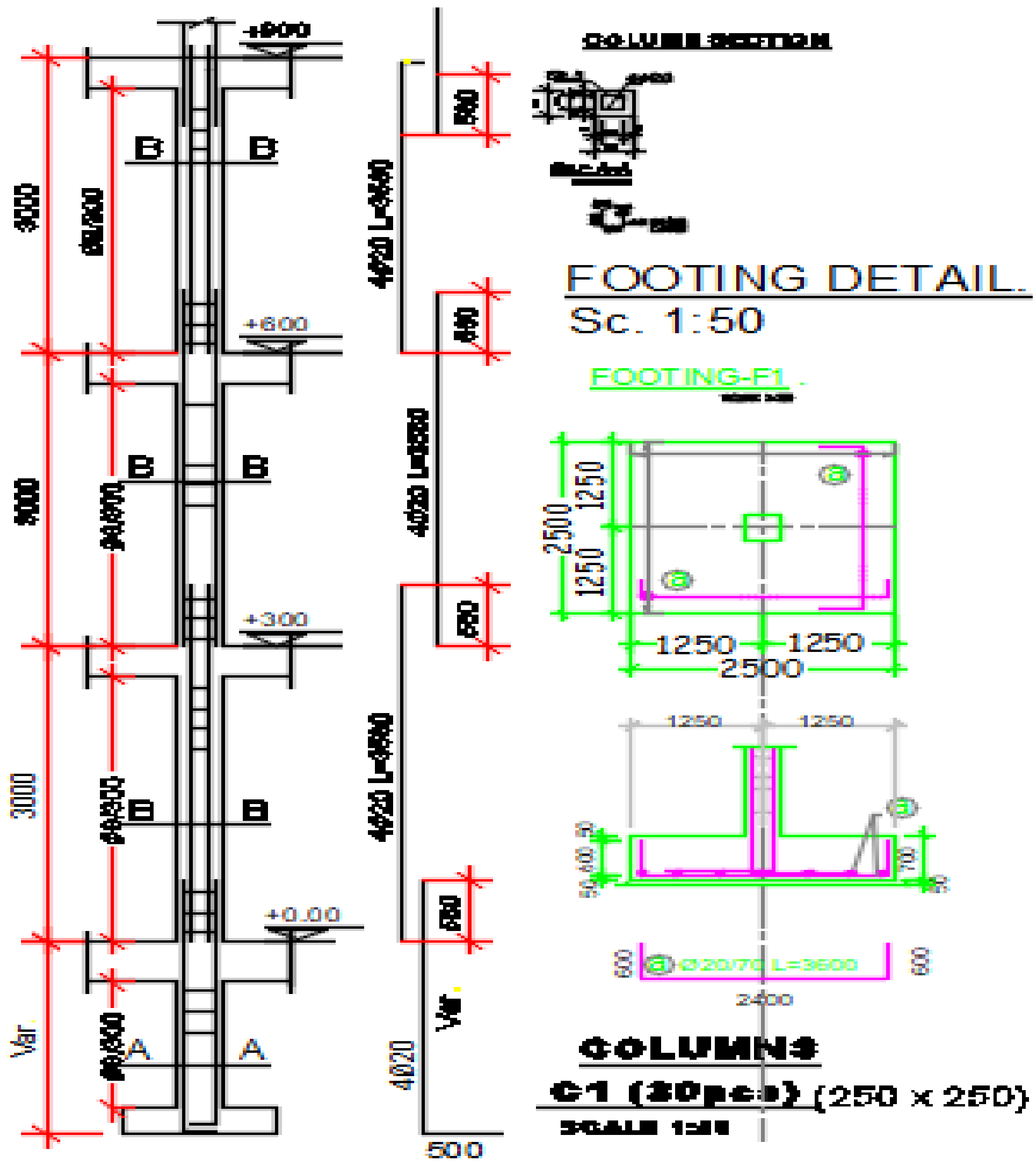


Figure 20 Reinforcement detail for column

11 Foundation Analysis and Design

The main purpose of footings and other foundation systems is to transfer column loads safely to the soil. Since, the soil bearing capacity is much lower than the concrete columns; the loads need to be transferred safely to the soil by using larger areas usually called shallow foundations. If the soil has low bearing capacity, or the applied loads are very large, it may be necessary to transfer the load to a deeper soil through the use of piles or caissons usually called deep foundation.

For the satisfactory design of foundations, it is important to have an understanding of thelocal geology and the type, thickness, parameters, properties and design bearing pressures of the soil or rock layers to which the foundation transfers the loads.

The Ultimate limit state design method(ULSD-method).

-Overall stability

-Bearing resistance failure

-Failure by sliding

Axially loaded footing at intersection of axis C-2(at Elevation-C) on structure two which has a compression force of 1271**.**69KN is selected as a representative sample for isolated footing design. From ETABS analysis output results the following loading data are obtained.

P=1271.69

M_Y=16.22 M_X=5.38

Material used

Steel

Steel grade, S – 500

Fyk = 400mpa, Fyd = 400/1.15 = 347.83MPa

Concrete

Concrete grade, C-30

Fck = 30 * 0.8= 24mpa, Fctk= 0.21Fck$^{2/3}$= 1.74MPa

11.1 Proportion of Area of a footing

$A=\frac{(N+W)}{\Upsilon soil}=\frac{1271.69+127.169}{150}=\underline{\mathbf{9.326}}$ where W=10%N

B*H*h =2.5*2.5*0.7

Area =2.5*2.5 =6.25m^2

Weight =6.25*0.7*25=109.4KN

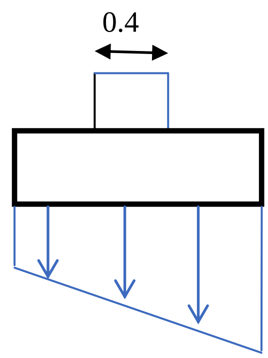
Actual contact pressure

δ=$\frac{P}{A}$ *$(1 \pm \frac{6ey}{B} \pm \frac{6eX}{B})$ where ex=$\frac{My}{P}$ =$\frac{16.22}{1271.69}$ =0.013

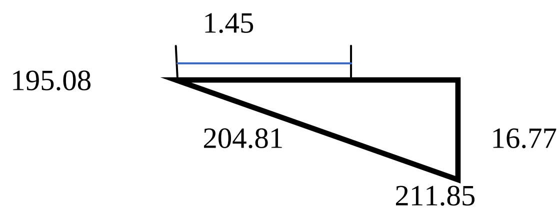
ex=$\frac{Mx}{P}$ =$\frac{5.38}{1271.69}$ =0.00423

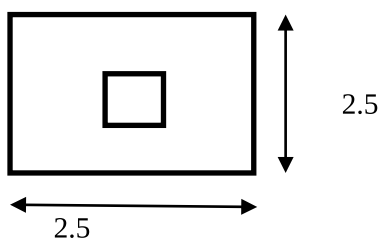
δ_{max}=$\frac{1271.69}{6.25}$ *$(1 + \frac{6(0.00423)}{2.5} + \frac{6(0.013)}{2.5})$ = $\underline{211.85}$

δ_{min}=$\frac{1271.69}{6.25}$ *$(1 - \frac{6(0.00423)}{2.5} - \frac{6(0.013)}{2.5})$ = $\underline{195.08}$



The column dimension is 40*40mm





By using similarity rule

$\frac{1.45}{x} = \frac{2.5}{16.77}$, x=9.73

<u>Punching shear resistance</u>

M_{xx}=204.81*$\frac{1.05^2}{2}$+(16.77)($\frac{1.05}{2}$)*$(1.05 * \frac{2}{3})$

= **<u>113.69</u>**

M_{yy}=($\frac{211.85+195.08}{2}$)*$\frac{1.05^2}{2}$= **<u>112.16</u>**

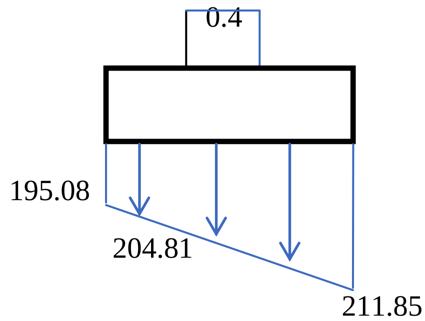
- Effective depth

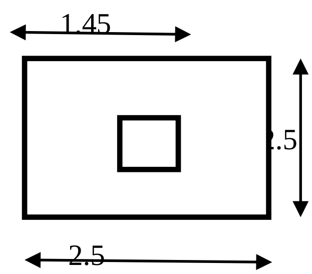
dx = h-c-o.5øbar=700-35-(0.5*24) = **653**

dy = h-c-1.5øbar =700-35-(1.5*24) = **629**

1. Vertical shear

Critical shear at 1.0d from face of column





Average pressure critical section

=195.08+($\frac{2.15}{2.5}$)*16.77

=209.5

- Design shear force

$V\varepsilon_d$= 209.5*0.35*2.5=183.31KN

$$K=1+\sqrt{\frac{200}{d}}= 1+\sqrt{\frac{200}{700}} = 1.535< 2.0$$

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d=36*24+700=1564

Asl =0mm^2

$\rho_1=\frac{Asl}{bd}$ =0

V_{RdC}=0

V_{min}=[$0.035K^{3/2}\sqrt{fck}$]bd

V_{min}=[$0.035* 1.535^{3/2}\sqrt{24}$]2500*700

V_{min}= 570.655KN

$V\varepsilon d<V_{min}$ **OK**

2. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700*629}{2}$ = 664.5

2d=1329

Control perimeter

U=2(400+400)+(2π*1329)=9950mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.329]+[2*0.4*1.329]+[π*1.329^2]

=7.835m^2

Average punching shear stress

Vεd=($\frac{195.08+211.85}{2}$)[(2.5*2.5)-7.835]= **322.5** KN

Punching shear stress

Vcd=$\frac{B\text{V}\varepsilon\text{d}}{Ud}$, B=1+K($\frac{\text{M}\varepsilon\text{d}}{\text{V}\varepsilon\text{d}}$)($\frac{U1}{W1}$)

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*664.5+16*664.5^2 +2*π*664.5*400

W1= $\underline{10038234.65\text{mm}^2}$

B=1+0.65($\frac{5.38*10^6}{322.5*10^3}$)($\frac{9950}{10038234.65}$)=1.01

$V_{\epsilon d}$=$\frac{1.01*322.5*10^3}{9950*700}$ =0.0467N/mm^2

Punching shear resistance

K=1+$\sqrt{\frac{200}{d}}$ =1+$\sqrt{\frac{200}{700}}$ =1.535<2

V_{Rdc}=V_{min}=0.035$K^{3/2}fck^{1/2}$

=0.035$(1.535)^{3/2}(24)^{1/2}$ =0.326

0.326 >Vεd(0.069) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Column perimeter u_o=2(400+400) =1600

$V\varepsilon_d=(\frac{BV\varepsilon d}{Uod})$ B=1+K$(\frac{M\varepsilon d}{V\varepsilon d})(\frac{Uo}{w1})$

W1=0.5*400^2+400*400

W1=240000mm^2

B=1+0.65$(\frac{5.38*10^6}{1271.6919*10^3})(\frac{1600}{240000})$

B=1.018

$V\varepsilon_d=(\frac{1.018*1271.6919*10^3}{1600*664.5})$ =1.22N/mm^2

Maximum shear resistance

$V_{ed.max}$=0.5[0.6(1- $\frac{fck}{250}$)]$\frac{fck}{1.5}$

=0.5[0.6(1- $\frac{24}{250}$)]($\frac{24}{1.5}$)

=4.34>$V\varepsilon_d$=1.22 **OK**

11.2 Reinforcement design for a footing

Main reinforcement-longitudinal bar

$$k = \frac{M_{xx}}{fck * b * d^2} = \frac{113.69 * 10\text{^}6}{24 * 2.5 * 629^2} = 0.0049$$

$$z = d\left(0.25 - \frac{k}{1.134}\right) = 0.7\left(0.25 - \frac{0.0049}{1.134}\right) = 0.246d$$

$$= 0.246d < 0.95d$$

$$\therefore z = 0.246$$

$$Asreq = \frac{Mxx}{fyd * z} = \frac{113.69 * 10\text{^}6}{347.83 * 0.246} = 79.22mm^2$$

Minimum and maximum area of reinforcement

$$Asmin = 0.26 * \left(\frac{fctm}{fyk}\right) * bd = 0.26 * \left(\frac{2.5}{400}\right) * 2500 * 629 = \mathbf{2555.3mm^2},$$

$$fctm = 0.3 * fck^{\frac{2}{3}} = 0.3 * 20^{\frac{2}{3}} = 2.5$$

$$Asmax = 0.04 * Ac = 0.04 * 2500 * 629 = \mathbf{62900mm^2}$$

Use the minimum of the two $\boldsymbol{Asmin = 2555.3mm^2}$

n=$\frac{As}{as}$,as=$\frac{3.14*24\text{^}2}{4}$ =452.16$\boldsymbol{mm^2}$

n=$\frac{2555.3}{4452.16}$= 5.65

Provide **6Ø24(As=2555.3mm^2)**

Main reinforcement-transverse bar

$$k = \frac{M_{yy}}{fck * b * d^2} = \frac{112.16}{24 * 2.5 * 629^2} = 0.00472 < Kbal = 0.167$$

There for compression reinforcement is not required.

$$z = d\left(0.25 - \frac{k}{1.134}\right) = d\left(0.25 - \frac{0.00472}{1.134}\right) = 0.245d < 0.95d$$

$$Asreq = \frac{Myy}{0.87fyk * z} = \frac{112.16 * 10^6}{0.87 * 400 * 0.245 * 629} = 2091mm^2$$

Minimum and maximum area of reinforcement

$$Asmax = 0.04 * Ac = 0.04 * 2.5 * 0.629 = \mathbf{62900mm^2}$$

Since $A_{s<}A_{min}$ use A_{smin}=$\mathbf{2555.3mm^2}$

n=$\frac{As}{as}$, as=$\frac{\Pi d^2}{4}$ =$\frac{3.14*24^2}{4}$ =452.16

n=$\frac{2555.3}{452.16}$ =5.65 Provide **5Ø24(As=2555.3mm^2)**

Reference

- Ethiopian Standard on European norm (ES EN 199p:2015)
- Sample Document from Mr. Mustefa (Foundation)
- Sample Document from Mr. Debissa (Slab)

Appendix

Appendix A : Load calculation for slab

Second Floor Panels

panel	Function	Calculated dead load(KN/m^2)	Governing dead load (KN/m^2)	Live load (KN/m^2)	Governing live load (KN/m^2)	Pd'=1.35D.L+1.5L.L (KN/m^2)
P1	Dressing room	5.53	6.02	2	2	11.13
	Terrace	6.02		2		
P2	Terrace	6.02	6.02	2	2	11.13
P3	Terrace	6.02	6.02	2	2	11.13
P4	Dressing room	5.63	5.63	2	2	10.6
	Jacuzi	5.63		2		
P5	Ch B room	5.53	5.53	5	5	10.47
	M B room	5.53		2		
P6	Master bed room	5.53	5,53	2	2	10.47
P7	Corridor	5.53	5.63	5	5	15.1
	Toilet	5.63		2		
P8	Master bed room	5.53	5.53	2	2	10.47
P9	Terrrace	6.02	6.02	2	2	11.13
P10	Terrace	6.02	6.02	2	2	11.13
C1	Terrace	6.02	6.02	2	2	11.13
C2	Terrace	6.02	6.02	2	2	11.13

Third Floor Panels

panel	Function	Calculated dead load(KN/m^2)	Governing dead load (KN/m^2)	Live load (KN/m^2)	Governing live load (KN/m^2)	Pd'=1.35D.L+1.5L.L (KN/m^2)
P1	Ch B room	5.63	6.02	2	2	11.13
	Toilet	5.63		2		
	Terrace	6.02		2		
P2	Terrace	6.02	6.02	2	2	11.13
P3	Terrace	6.02	6.02	2	2	11.13
P4	Toilet	5.63	5.63	2	2	10.6
	Ch B room	5.63		2		
P5	Ch B room	5.53	5.53	2	3	14.96
	Corridor	5.53		3		
P6	Tv room	5.53	5,53	2	2	10.47
P7	Corridor	5.53	5.53	5	5	14.96
	Ch B room	5.53		2		
P8	Tv room	5.53	5.53	2	2	10.47
P9	Terrrace	6.02	6.02	2	2	11.13
P10	Terrace	6.02	6.02	2	2	11.13
C1	Terrace	6.02	6.02	2	2	11.13
C2	Terrace	6.02	6.02	2	2	11.13

Fourth &Fifth Floor Panels

panel	Function	Calculated dead load(KN/m^2)	Governing dead load (KN/m^2)	Live load (KN/m^2)	Governing live load (KN/m^2)	Pd'=1.35D.L+1.5L.L (KN/m^2)
P1	Terrace	6.02	6.02	2	2	11.13
	Study Room	5.63		2		
P2	Terrace	6.02	6.02	2	2	11.13
P3	Terrace	6.02	6.02	2	2	11.13

P4	Study room	5.53	5.63	2	2	10.6
	Toilet	5.63		2		
P5	G B room	5.63	5.63	2	5	15.1
	Corridor	5.53		5		
P6	Family room	5.53	5,53	2	2	10.47
P7	Corridor	5.53	5.63	3	3	15.1
	G B room	5.63		2		
P8	Family room	5.53	5.53	2	2	10.47
P9	Family	5.53	5.53	2	2	10.47
P10	Terrace	6.02	6.02	2	2	11.13
C1	Terrace	6.02	6.02	2	2	11.13
C2	Terrace	6.02	6.02	2	2	11.13

Appendix B:Design Moment Analysis

Second floor

PANEL		Lx	Ly	Ly/Lx	N	αxs	αxf	αys	αyf	Mxs	Mxf	Mys	Myf
P1	End	3.6	5	1.39	13	0.074	0.055	0.045	0.03	12.47	9.27	7.58	5.05
P2	End	3.6	4.7	1.31	11.13	0.063	0.047	0.037	0.028	9.09	6.78	5.34	4.04
P4	Interior	2.7	5	1.85	12.67	0.065	0.049	0.037	0.028	6.00	4.53	3.42	2.59
P5	Interior	2.7	4.7	1.74	12.9	0.068	0.047	0.037	0.028	6.39	4.42	3.48	2.63
P6	Interior	3.15	5	1.59	10.47	0.06	0.045	0.037	0.028	6.23	4.67	3.84	2.91
P7	Interior	3.15	4.7	1.49	17.2	0.058	0.043	0.037	0.028	9.90	7.34	6.31	4.78
P8	Interior	2.85	5	1.75	16.03	0.064	0.048	0.037	0.028	8.33	6.25	4.82	3.65

Moment Calculation for one way & cantilever slabs.

PANEL	..	Lx	Ly	Ly/Lx	n(pd)	Mxs(KN)
P3	End	1.7	3.6	2.12	11.13	31.89
P9	End	1.88	5	2.66	10.47	36.00
P10	End	1.88	4.7	2.50	11.13	37.15
C1	Cantilever	0.6	4.7	7.83	11.13	7.60
C2	Cantilever	0.6	5	8.33	11.13	7.60

Third floor

PANEL		Lx	Ly	Ly/Lx	n(pd)	αxs	Axf	αys	αyf	Mxs	Mxf	Mys	Myf
P1	End	3.6	5	1.39	13.6	0.074	0.055	0.045	0.03	13.04	9.69	7.93	5.29
P2	End	3.6	4.7	1.31	11.13	0.063	0.047	0.037	0.028	9.09	6.78	5.34	4.04
P4	Interior	2.7	5	1.85	12.89	0.065	0.049	0.037	0.028	6.11	4.60	3.48	2.63
P5	Interior	2.7	4.7	1.74	17.5	0.068	0.047	0.037	0.028	8.68	6.00	4.72	3.57
P6	Interior	3.15	5	1.59	10.47	0.06	0.045	0.037	0.028	6.23	4.67	3.84	2.91
P7	Interior	3.15	4.7	1.49	18.06	0.058	0.043	0.037	0.028	10.39	7.71	6.63	5.02
P8	Interior	2.85	5	1.75	10.47	0.064	0.048	0.037	0.028	5.44	4.08	3.15	2.38

Moment Calculation for one way & cantilever slabs.

PANEL	..	Lx	Ly	Ly/Lx	n(pd)	Mxs(KN)
P3	End	1.7	3.6	2.12	16.17	31.89
P9	End	1.88	5	2.66	16.17	36.00
P10	End	1.88	4.7	2.50	16.17	37.15
C1	Cantilever	0.6	4.7	7.83	16.17	7.60
C2	Cantilever	0.6	5	8.33	11.13	7.60

Fourth & Fifth floor

PANEL		Lx	Ly	Ly/Lx	n(pd)	αxs	αxf	αys	αyf	Mxs	Mxf	Mys	Myf
P1	End	3.6	5	1.39	12.34	0.074	0.055	0.045	0.03	11.83	8.80	7.20	4.80
P2	End	3.6	4.7	1.31	11.13	0.063	0.047	0.037	0.028	9.09	6.78	5.34	4.04

P4	Interior	2.7	5	1.85	13.03	0.065	0.049	0.037	0.028	6.17	4.65	3.51	2.66
P5	Interior	2.7	4.7	1.74	17.67	0.068	0.047	0.037	0.028	8.76	6.05	4.77	3.61
P6	Interior	3.15	5	1.59	10.47	0.06	0.045	0.037	0.028	6.23	4.67	3.84	2.91
P7	Interior	3.15	4.7	1.49	16.93	0.058	0.043	0.037	0.028	9.74	7.22	6.22	4.70
P8	Interior	2.85	5	1.75	10.47	0.064	0.048	0.037	0.028	5.44	4.08	3.15	2.38

Moment Calculation for one way & cantilever slabs.

PANEL		Lx	Ly	Ly/Lx	n(pd)	Mxs(KN)
P3	End	1.7	3.6	2.12	11.13	31.89
P9	End	1.88	5	2.66	10.47	35.98
P10	End	1.88	4.7	2.50	11.13	37.15
C1	Cantilever	0.6	4.7	7.83	11.13	7.60

Appendix C: Moment adjustment

Second Floor

Support moment adjustment

PANEL	MTD 1	MTD 2	MAX
P1&P2		6.46	
P2&P3		18.62	
P1&P4		9.24	
P4&P5	3.45		
P4&P6	6.12		
P6&P7		5.08	
P6&P8		7.23	
P8&P9		25	
P10&C1			37.15
P2&P5		8.12	
P5&P7		8.01	

Span moment adjustment

PANEL	Mxs	Mxf	Madjust	Mxfadj	Mys	Myf	Madjust	Myfadj
P1	12.47	9.27	6.46	15.28	7.58	5.05	9.24	3.39
P2	9.09	6.78	18.62	-2.75	5.34	4.04	8.12	1.26
P4	6	4.53	9.24	1.29	3.42	2.59	5	1.01
P5	6.39	4.42	3.45	7.36	3.48	2.63	3.45	2.66
P6	6.23	4.67	6.12	4.78	3.84	2.91	6.5	0.25
P7	9.9	7.34	5.08	12.16	6.31	4.78	8.01	3.08
P8	8.33	6.25	7.23	7.35	4.82	3.65	7.23	1.24

Third floor

Support moment adjustment

PANEL	MTD 1	MTD 2	MAX
P1&P2		6.47	
P2&P3		18.62	
P1&P4		10.54	
P4&P5		4.1	
P4&P6	6.17		
P6&P7		5.34	
P6&P8	5.84		
P8&P9		23.85	
P9&C1			36
P10&C2			37.15
P2&P5	8.89		
P5&P7	9.54		

Span moment adjustment

PANEL	Mxs	Mxf	Madjust	Mxfadj	Mys	Myf	Madjust	Myfadj
P1	13.04	9.69	6.47	16.26	7.98	5.29	6.47	6.80
P2	9.09	6.78	8.89	6.98	5.34	4.04	8.89	0.49
P4	6.11	4.6	10.54	0.17	3.48	2.63	5.26	0.85
P5	8.68	6	4.1	10.58	4.72	3.57	4.1	4.19
P6	6.23	4.67	6.17	4.73	3.84	2.91	5.84	0.91
P7	10.39	7.71	5.34	12.76	6.63	5.02	5.34	6.31
P8	5.44	4.08	5.84	3.68	4.82	3.5	5.84	2.48

Fourth & Fifth floor

Support moment adjustment

PANEL	MTD 1	MTD 2	MAX
P1&P2		6.27	
P2&P3		18.62	
P1&P4		9	
P4&P5		4.23	
P4&P6	6.2		
P6&P7		5.03	
P6&P8	5.84		
P8&P9		23.85	
P10&C1			37.15
P2&P5	8.93		
P5&P7	9.25		

Span moment adjustment

PANEL	Mxs	Mxf	Madjust	Mxfadj	Mys	Myf	Madjust	Myfadj
P1	11.83	8.88	6.27	14.44	7.2	4.8	6.27	5.73
P2	9.09	6.78	13.4	2.47	5.34	4.04	8.93	0.45
P4	6.17	4.65	9	1.82	3.51	2.66	4.23	1.94
P5	8.76	6.05	4.23	10.58	4.72	3.61	4.23	4.10
P6	6.23	4.67	6.2	4.70	3.84	2.91	5.03	1.72
P7	9.74	7.22	5.03	11.93	6.2	4.7	9.25	1.65
P8	5.44	4.08	5.84	3.68	3.15	2.38	5.84	-0.31

Appendix D : Shear analysis

Second Floor

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvxc	βvxd	βvyc	βvyd	Vsxc	Vsxd	Vsyc	Vsyd
P1	End Span	3.6	5	1.39	13	0.52	0.35	0.4	0.26	24.34	16.38	18.72	12.168
P2	End Span	3.6	4.7	1.31	11.13	0.47	0.31	0.36	-	18.83	12.42108	14.42	0
P3	End Span	1.7	3.6	2.12	11.13	0.59	0.38	0.36	-	11.16	7.18998	6.81	0
P4	Interior	2.7	5	1.85	12.67	0.51	-	0.36	0.24	17.45	0	12.32	8.21016
P5	Interior	2.7	4.7	1.74	12.9	0.49	-	0.36	0.24	17.07	0	12.54	8.3592
P6	Interior	3.15	5	1.59	10.47	0.36	-	0.24	-	11.87	0	7.92	0
P7	Interior	3.15	4.7	1.49	17.2	0.47	-	0.36	0.24	25.46	0	19.50	13.0032
p8	Interior	2.85	5	1.75	10.47	0.504	-	0.36	0.24	15.04	0	10.74	7.16148
p9	Interior	1.88	5	2.66	11.13	0.52	-	0.36	0.24	10.88	0	7.53	5.021856
p10	Interior	1.88	4.7	2.50	11.13	0.52	-	0.36	0.24	10.88	0	7.53	5.021856
PANEL	TYPE	Lx	Ly	Ly/Lx	n (Pd)	WL	-	-	-	-	-	-	-
C 1	cantiliver	0.6	5	8.33	11.13	6.68	-	-	-	-	-	-	-
C2	cantiliver	0.6	4.7	7.83	11.13	6.68	-	-	-	-	-	-	-

Third floor

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvxc	βvxd	βvyc	βvyd	Vsxc	Vsxd	Vsyc	Vsyd
P1	End Span	3.6	5	1.39	13.6	0.52	0.35	0.4	0.26	25.46	17.136	19.58	12.7296
P2	End Span	3.6	4.7	1.31	11.13	0.47	0.31	0.36		18.83	12.42108	14.42	0
P3	End Span	1.7	3.6	2.12	11.13	0.59	0.38	0.36		11.16	7.18998	6.81	0
P4	Interior	2.7	5	1.85	12.89	0.51		0.36	0.24	17.75	0	12.53	8.35272
P5	Interior	2.7	4.7	1.74	17.5	0.49		0.36	0.24	23.15	0	17.01	11.34
P6	Interior	3.15	5	1.59	10.7	0.36		0.24		12.13	0	8.09	0
P7	Interior	3.15	4.7	1.49	18.06	0.47		0.36	0.24	26.74	0	20.48	13.65336
p8	Interior	2.85	5	1.75	10.47	0.504		0.36	0.24	15.04	0	10.74	7.16148
p9	Interior	1.88	5	2.66	11.13	0.52		0.36	0.24	10.88	0	7.53	5.021856
p10	Interior	1.88	4.7	2.50	11.13	0.52		0.36	0.24	10.88	0	7.53	5.021856
PANEL	TYPE	Lx	Ly	Ly/Lx	n (Pd)	WL							
C 1	cantiliver	0.6	5	8.33	11.13	6.68							
C2	cantiliver	0.6	4.7	7.83	11.13	6.68							

Fourth & Fifth floor

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvxc	βvxd	βvyc	βvyd	Vsxc	Vsxd	Vsyc	Vsyd
P1	End Span	3.6	5	1.39	12.34	0.52	0.35	0.4	0.26	23.10	15.5484	17.77	11.55024
P2	End Span	3.6	4.7	1.31	11.13	0.47	0.31	0.36		18.83	12.42108	14.42	0
P3	End Span	1.7	3.6	2.12	11.13	0.59	0.38	0.36		11.16	7.18998	6.81	0
P4	Interior	2.7	5	1.85	1303	0.51		0.36	0.24	1794.23	0	1266.52	844.344
P5	Interior	2.7	4.7	1.74	17.67	0.49		0.36	0.24	23.38	0	17.18	11.45016
P6	Interior	3.15	5	1.59	10.7	0.36		0.24		12.13	0	8.09	0

P7	Interior	3.15	4.7	1.49	16.93	0.47		0.36	0.24	25.06	0	19.20	12.79908
p8	Interior	2.85	5	1.75	10.47	0.504		0.36	0.24	15.04	0	10.74	7.16148
p9	Interior	1.88	5	2.66	10.47	0.52		0.36	0.24	10.24	0	7.09	4.724064
p10	Interior	1.88	4.7	2.50	11.13	0.52		0.36	0.24	10.88	0	7.53	5.021856
PANEL	TYPE	Lx	Ly	Ly/Lx	n (Pd)	WL							
C 1	Cantilever	0.6	5	8.33	11.13	6.68							
C2	Cantilever	0.6	4.7	7.83	11.13	6.68							

Appendix E :Reinforcement Calculation

For Second floor slab

panel	Moment		Ast=Msd/Z*fyd	ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	9.24	215.00	10	215.00	215	ø	10	c/c	215
	M_{xf}=	15.69	288.05	10	250.00	250	ø	10	c/c	250
	M_{ys}=	6.42	215.00	10	215.00	215	ø	10	c/c	215
	M_{yf}=	3.39	215.00	10	215.00	215	ø	10	c/c	215
P-2	M_{xs} =	8.12	215.00	10	215.00	215	ø	10	c/c	215
	M_{xf}=	9.41	215.00	10	215.00	215	ø	10	c/c	215
	M_{ys}=	18.62	396.53	10	350.00	350	ø	10	c/c	350
	M_{yf}=	1.26	215.00	10	215.00	215	ø	10	c/c	215
P-3	M_{xs} =	0	215.00	10	215.00	215	ø	10	c/c	215
	M_{xf}=	0	215.00	10	215.00	215	ø	10	c/c	215
	M_{ys}=	18.62	396.53	10	350.00	350	ø	10	c/c	350
	M_{yf}=	0	215.00	10	215.00	215	ø	10	c/c	215
P-4	M_{xs} =	6.12	215.00	10	215.00	215	ø	10	c/c	215
	M_{xf}=	1.29	215.00	10	215.00	215	ø	10	c/c	215
	M_{ys}=	3.45	215.00	10	215.00	215	ø	10	c/c	215
	M_{yf}=	1.01	215.00	10	215.00	215	ø	10	c/c	215
P-5	M_{xs} =	8.01	215.00	10	215.00	215	ø	10	c/c	215
	M_{xf}=	7.36	215.00	10	215.00	215	ø	10	c/c	215
	M_{ys}=	3.45	215.00	10	215.00	215	ø	10	c/c	215
	M_{yf}=	2.66	215.00	10	215.00	215	ø	10	c/c	215
P-6	M_{xs} =	7.23	215.00	10	215.00	215	ø	10	c/c	215
	M_{xf}=	4.78	215.00	10	215.00	215	ø	10	c/c	215
	M_{ys}=	5.08	215.00	10	215.00	215	ø	10	c/c	215
	M_{yf}=	0.25	215.00	10	215.00	215	ø	10	c/c	215

P-7	M_{xs} =	8.01	215.00	10	215.00	215	ø	10	c/c	215
	M_{xf}=	12.16	258.96	10	250.00	250	ø	10	c/c	250
	M_{ys}=	5.08	215.00	10	215.00	215	ø	10	c/c	215
	M_{yf}=	3.08	215.00	10	215.00	215	ø	10	c/c	215
P-8	M_{xs} =	25	532.40	10	215.00	215	ø	10	c/c	215
	M_{xf}=	7.35	215.00	10	215.00	215	ø	10	c/c	215
	M_{ys}=	0	215.00	10	215.00	215	ø	10	c/c	215
	M_{yf}=	1.24	215.00	10	215.00	215	ø	10	c/c	215
P-9	M_{xs} =	25	532.40	10	215.00	215	ø	10	c/c	215
	M_{xf}=	0	215.00	10	215.00	215	ø	10	c/c	215
	M_{ys}=	0	215.00	10	215.00	215	ø	10	c/c	215
	M_{yf}=	0.96	215.00	10	215.00	215	ø	10	c/c	215
P-10	M_{xs} =	37.15	791.15	10	215.00	215	ø	10	c/c	215
	M_{xf}=	0	215.00	10	215.00	215	ø	10	c/c	215
	M_{ys}=	0	215.00	10	215.00	215	ø	10	c/c	215
	M_{yf}=	0	215.00	10	215.00	215	ø	10	c/c	215
C1	M_{xs} =	7.6	215.00	10	215.00	215	ø	10	c/c	215
C2	M_{xs} =	7.6	215.00	10	365.12	200	ø	10	c/c	200

For Third floor slab

panel	Moment		Ast=Msd/Z*fyd	ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	10.54	193.50	10	405.68	400	ø	10	c/c	400
	M_{xf}=	16.26	298.51	10	262.97	250	ø	10	c/c	250
	M_{ys}=	6.47	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	6.8	215.00	10	215.00	215	ø	10	c/c	215
P-2	M_{xs} =	8.89	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	6.98	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	6.47	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	0.49	215.00	10	365.12	350	ø	10	c/c	350
P-3	M_{xs} =	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	18.62	396.53	10	197.97	215	ø	10	c/c	215
	M_{yf}=	0	215.00	10	365.12	350	ø	10	c/c	350
P-4	M_{xs} =	6.17	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	0.17	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	4.1	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	0.85	215.00	10	365.12	350	ø	10	c/c	350
P-5	M_{xs} =	9.54	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	10.58	225.31	10	348.41	350	ø	10	c/c	350
	M_{ys}=	4.1	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	0.91	215.00	10	365.12	350	ø	10	c/c	350
P-6	M_{xs} =	5.84	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	4.79	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	5.34	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	2.52	215.00	10	365.12	350	ø	10	c/c	350
P-7	M_{xs} =	9.54	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	12.76	271.74	10	288.88	250	ø	10	c/c	250

	M_{ys}=	5.34	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	6.31	215.00	10	365.12	350	ø	10	c/c	350
P-8	M_{xs} =	5.84	215.00	10	365.12	215	ø	10	c/c	215
	M_{xf}=	3.68	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	2.48	215.00	10	365.12	350	ø	10	c/c	350
P-9	M_{xs} =	23.85	507.91	10	154.55	215	ø	10	c/c	215
	M_{xf}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	0	215.00	10	365.12	350	ø	10	c/c	350
P-10	M_{xs} =	37.15	791.15	10	99.22	215	ø	10	c/c	215
	M_{xf}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	35.58	757.71	10	103.60	215	ø	10	c/c	215
	M_{yf}=	0	215.00	10	365.12	350	ø	10	c/c	350
C1	M_{xs} =	7.6	215.00	10	365.12	350	ø	10	c/c	350
C2	M_{xs} =	7.6	215.00	10	365.12	350	ø	10	c/c	350

For Fourth & Fifth floor slab

panel	moment		Ast=Msd/Z*fyd	ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	9	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	14.44	265.10	10	296.12	250	ø	10	c/c	250
	M_{ys}=	6.27	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	5.73	215.00	10	215.00	215	ø	10	c/c	215
P-2	M_{xs} =	8.93	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	2.47	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	18.62	396.53	10	197.97	215	ø	10	c/c	215
	M_{yf}=	0.45	215.00	10	365.12	350	ø	10	c/c	350
P-3	M_{xs} =	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	18.62	396.53	10	197.97	215	ø	10	c/c	215
	M_{yf}=	0	215.00	10	365.12	350	ø	10	c/c	350
P-4	M_{xs} =	6.2	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	1.82	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	4.23	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	1.94	215.00	10	365.12	350	ø	10	c/c	350
P-5	M_{xs} =	9.25	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	10.58	225.31	10	348.41	350	ø	10	c/c	350
	M_{ys}=	4.23	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	4.1	215.00	10	365.12	350	ø	10	c/c	350
P-6	M_{xs} =	5.84	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	4.7	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	5.03	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	1.72	215.00	10	365.12	350	ø	10	c/c	350
P-7	M_{xs} =	9.25	215.00	10	365.12	350	ø	10	c/c	350
	M_{xf}=	11.93	254.06	10	308.98	350	ø	10	c/c	350

	M_{ys}=	5.03	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	1.65	215.00	10	365.12	350	ø	10	c/c	350
P-8	M_{xs} =	5.84	215.00	10	365.12	215	ø	10	c/c	215
	M_{xf}=	3.68	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	0.28	215.00	10	365.12	350	ø	10	c/c	350
P-9	M_{xs} =	23.85	507.91	10	154.55	215	ø	10	c/c	215
	M_{xf}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	0	215.00	10	365.12	330	ø	10	c/c	330
	M_{yf}=	0	215.00	10	365.12	350	ø	10	c/c	350
P-10	M_{xs} =	37.15	791.15	10	99.22	215	ø	10	c/c	215
	M_{xf}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{ys}=	0	215.00	10	365.12	350	ø	10	c/c	350
	M_{yf}=	0	215.00	10	365.12	350	ø	10	c/c	350
C1	M_{xs} =	7.6	215.00	10	365.12	350	ø	10	c/c	350

Appendix F : BeamReinforcement

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	117.8	300	350	0.1603	290.3195	single	150.15	4200	1166.56	1166.56	5.804935	6Ø16
Span	1-2	117.11	300	350	0.1593	290.7587	single	150.15	4200	1157.975	1157.975	5.762216	6Ø16
Support	2	105.04	300	350	0.1429	298.1883	single	150.15	4200	1012.749	1012.749	5.039556	6Ø16
Span	2-3	182.54	300	350	0.2484	236.4525	double	150.15	4200	2219.484	2219.484	11.04441	12Ø16
Support	3	87.49	300	350	0.1190	308.2542	single	150.15	4200	815.9946	815.9946	4.060483	5Ø16
Span	3-4	182.23	300	350	0.2479	236.8224	double	150.15	4200	2212.254	2212.254	11.00843	12Ø16
Support	4	103.64	300	350	0.1410	299.0213	single	150.15	4200	996.4675	996.4675	4.958536	5Ø16
Span	4-5	182.6	300	350	0.2484	236.3806	double	150.15	4200	2220.888	2220.888	11.05139	12Ø16
Support	5	75.2	300	350	0.1023	314.8726	single	150.15	4200	686.627	686.627	3.416735	4Ø16
Span	5-6	3.77	300	350	0.0051	348.4085	single	150.15	4200	31.10932	150.15	0.747164	2Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	134.87	300	350	0.183	278.864	double	150.15	4200	1390.464	1390.46	6.919108	7Ø16
Span	1-2	120.9	300	350	0.164	288.325	single	150.15	4200	1205.539	1205.53	5.998902	6Ø16
support	2	125.36	300	350	0.170	285.393	double	150.15	4200	1262.855	1262.85	6.28411	7Ø16

span	2-3	181.2	300	350	0.246	238.035	double	150.15	4200	2188.537	2188.53	10.89041	11Ø16
support	3	93.13	300	350	0.126	305.104	single	150.15	4200	877.5647	877.564	4.366862	5Ø16
span	3-4	153.6	300	350	0.209	264.62	double	150.15	4200	1668.764	1668.76	8.303961	9Ø16
support	4	97.02	300	350	0.132	302.886	single	150.15	4200	920.9141	920.914	4.582574	5Ø16
Span	4-5	165.9	300	350	0.225	253.891	double	150.15	4200	1878.607	1878.60	9.348165	10Ø16
Support	5	98	300	350	0.133	302.322	single	150.15	4200	931.9541	931.954	4.63751	5Ø16
Span	5-6	1.4	300	350	0.002	349.410	single	150.15	4200	11.5194	150.15	0.747164	2Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	137	300	350	0.1864	277.3454	double	150.15	4200	1420.161	1420.161	7.066881	8Ø16
span	1-2	117.18	300	350	0.1594	290.7142	single	150.15	4200	1158.844	1158.844	5.766542	6Ø16
support	2	56.3	300	350	0.0766	324.4798	single	150.15	4200	498.8369	498.8369	2.48227	3Ø16
span	2-3	180.9	300	350	0.2461	238.3847	double	150.15	4200	2181.715	2181.715	10.85646	11Ø16
support	3	97.6	300	350	0.1328	302.5526	single	150.15	4200	927.442	927.442	4.615058	5Ø16
span	3-4	157.56	300	350	0.2144	261.3166	double	150.15	4200	1733.472	1733.472	8.625956	9Ø16
support	4	110.28	300	350	0.1500	295.0194	single	150.15	4200	1074.692	1074.692	5.347791	6Ø16
Span	4-5	183.3	300	350	0.2494	235.5361	double	150.15	4200	2237.395	2237.395	11.13354	12Ø16
Support	5	96.7	300	350	0.1316	303.0705	single	150.15	4200	917.3197	917.3197	4.564688	5Ø16
Span	5-6	6.36	300	350	0.0087	347.3066	single	150.15	4200	52.64801	150.15	0.747164	2Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	186.17	300	350	0.2533	231.9429	double	150.15	4200	2307.631	2307.631	11.48304	12Ø16
span	1-2	152.43	300	350	0.2074	265.5819	double	150.15	4200	1650.098	1650.098	8.211077	9Ø16
support	2	144.97	300	350	0.1972	271.4486	double	150.15	4200	1535.424	1535.424	7.640447	8Ø16
span	2-3	144.14	300	350	0.1961	272.0794	double	150.15	4200	1523.094	1523.094	7.579091	8Ø16
support	3	93.98	300	350	0.1279	304.6229	single	150.15	4200	886.9736	886.9736	4.413682	5Ø16
span	3-4	125.78	300	350	0.1711	285.1129	double	150.15	4200	1268.331	1268.331	6.311359	7Ø16
support	4	116.13	300	350	0.1580	291.3796	single	150.15	4200	1145.838	1145.838	5.70182	6Ø16
Span	4-5	91.04	300	350	0.1239	306.2804	single	150.15	4200	854.5765	854.5765	4.252471	5Ø16
Support	5	99.04	300	350	0.1347	301.7196	single	150.15	4200	943.7238	943.7238	4.696078	5Ø16
Span	5-6	3.307	300	350	0.0045	348.6047	single	150.15	4200	27.27337	150.15	0.747164	2Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	46.46	350	350	0.0542	332.3776	single	175.175	4900	401.8697	401.8697	1.99975	2Ø16
span	1-2	76.09	350	350	0.0887	320.0249	single	175.175	4900	683.5679	683.5679	3.401512	4Ø16
support	2	35.49	350	350	0.0414	336.7119	single	175.175	4900	303.0299	303.0299	1.507911	2Ø16
span	2-3	80.06	350	350	0.0934	318.2889	single	175.175	4900	723.1558	723.1558	3.598506	4Ø16

support	3	41.81	350	350	0.0488	334.2292	single	175.175	4900	359.6446	359.6446	1.789633	2Ø16
span	3-4	8.305	350	350	0.0097	346.9825	single	175.175	4900	68.81291	175.175	0.871691	2Ø16
support	4	30.6	350	350	0.0357	338.6069	single	175.175	4900	259.8146	259.8146	1.292867	2Ø16
Span	4-5	8.76	350	350	0.0102	346.8156	single	175.175	4900	72.61783	175.175	0.871691	2Ø16
Support	5	34.12	350	350	0.0398	337.245	single	175.175	4900	290.8716	290.8716	1.44741	2Ø16
Span	5-6	8.98	350	350	0.0105	346.7349	single	175.175	4900	74.4589	175.175	0.871691	2Ø16

Axis B

Roof

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	89	250	300	0.1978	232.4106	double	107.25	3000	1100.961	1100.961	5.478509	9Ø16
span	1-2	115	250	300	0.2556	196.9308	double	107.25	3000	1678.889	1678.889	8.354345	9Ø16
support	2	102.29	250	300	0.2273	216.6769	double	107.25	3000	1357.245	1357.245	6.753808	7Ø16
span	2-3	2.97	250	300	0.0066	298.2424	single	107.25	3000	28.63024	107.25	0.533688	2Ø16
support	3	30.94	250	300	0.0688	280.5339	single	107.25	3000	317.0829	317.0829	1.577841	2Ø16
span	3-4	92.18	250	300	0.2048	228.9318	double	107.25	3000	1157.626	1157.626	5.760482	6Ø16
support	4	103.4	250	300	0.2298	215.1913	double	107.25	3000	1381.446	1381.446	6.874231	7Ø16
Span	4-5	73.19	250	300	0.1626	247.8875	single	107.25	3000	848.8578	848.8578	4.224014	5Ø16
Support	5	27.24	250	300	0.0605	283.0118	single	107.25	3000	276.7199	276.7199	1.37699	2Ø16
Span	5-6	19.21	250	300	0.0427	288.2369	single	107.25	3000	191.6089	191.6089	0.953468	2Ø16

5^{th} floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	15.87	300	350	0.0216	343.1987	single	150.15	4200	132.9441	150.15	0.747164	2Ø16
span	1-2	188.8	300	350	0.2569	228.4384	double	150.15	4200	2376.133	2376.133	11.82391	12Ø16
support	2	156.98	300	350	0.2136	261.8093	double	150.15	4200	1723.84	1723.84	8.578028	9Ø16
span	2-3	66.08	300	350	0.0899	319.5882	single	150.15	4200	594.4526	594.4526	2.958064	3Ø16
support	3	70.38	300	350	0.0958	317.3843	single	150.15	4200	637.5316	637.5316	3.17243	4Ø16
span	3-4	158.89	300	350	0.2162	260.1759	double	150.15	4200	1755.769	1755.769	8.736908	9Ø16
support	4	172.52	300	350	0.2347	247.4581	double	150.15	4200	2004.359	2004.359	9.973921	10Ø16
Span	4-5	62.12	300	350	0.0845	321.5885	single	150.15	4200	555.3526	555.3526	2.763498	3Ø16
Support	5	104.88	300	350	0.1427	298.2838	single	150.15	4200	1010.883	1010.883	5.030269	6Ø16
Span	5-6	40.62	300	350	0.0553	332.0047	single	150.15	4200	351.7495	351.7495	1.750346	2Ø16

4^{th} floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	20.04	300	350	0.0273	341.3654	single	150.15	4200	168.7781	168.7781	0.839859	2Ø16
span	1-2	19.41	300	350	0.0264	341.6437	single	150.15	4200	163.339	163.339	0.812794	2Ø16
support	2	162.73	300	350	0.2214	256.7932	double	150.15	4200	1821.889	1821.889	9.065929	10Ø16

span	2-3	5.44	300	350	0.0074	347.6988	single	150.15	4200	44.98146	150.15	0.747164	2Ø16
support	3	73.67	300	350	0.1002	315.6747	single	150.15	4200	670.9478	670.9478	3.338713	4Ø16
span	3-4	162.44	300	350	0.2210	257.0535	double	150.15	4200	1816.801	1816.801	9.040608	10Ø16
support	4	184.52	300	350	0.2510	234.0354	double	150.15	4200	2266.729	2266.729	11.27951	12Ø16
Span	4-5	64.17	300	350	0.0873	320.5564	single	150.15	4200	575.5267	575.5267	2.863887	2Ø16
Support	5	108.97	300	350	0.1483	295.8194	single	150.15	4200	1059.054	1059.054	5.269975	6Ø16
Span	5-6	52.47	300	350	0.0714	326.3524	single	150.15	4200	462.2343	462.2343	2.300131	3Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	30.82	300	350	0.0419	336.5298	single	150.15	4200	263.2976	263.2976	1.310199	2Ø16
span	1-2	101.39	300	350	0.1379	300.3484	single	150.15	4200	970.5271	970.5271	4.829454	5Ø16
support	2	199.29	300	350	0.2711	211.2321	double	150.15	4200	2712.461	2712.461	13.49752	14Ø16
span	2-3	77.46	300	350	0.1054	313.6792	single	150.15	4200	709.953	709.953	3.532808	4Ø16
support	3	145.81	300	350	0.1984	270.8059	double	150.15	4200	1547.986	1547.986	7.702955	8Ø16
span	3-4	103.02	300	350	0.1402	299.3884	single	150.15	4200	989.2918	989.2918	4.92283	5Ø16
support	4	152.62	300	350	0.2076	265.4275	double	150.15	4200	1653.116	1653.116	8.226094	9Ø16
Span	4-5	89.55	300	350	0.1218	307.1124	single	150.15	4200	838.3128	838.3128	4.17154	5Ø16
Support	5	122.95	300	350	0.1673	286.9871	double	150.15	4200	1231.697	1231.697	6.129068	7Ø16
Span	5-6	8.96	300	350	0.0122	346.1933	single	150.15	4200	74.4093	150.15	0.747164	2Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	34.34	300	350	0.0467	334.9192	single	150.15	4200	294.78	294.78	1.466859	2Ø16
Span	1-2	105.45	300	350	0.1435	297.9433	single	150.15	4200	1017.538	1017.538	5.063387	6Ø16
support	2	42.47	300	350	0.0578	331.1357	single	150.15	4200	368.7347	368.7347	1.834866	2Ø16
Span	2-3	55.34	300	350	0.0753	324.9514	single	150.15	4200	489.6194	489.6194	2.436402	3Ø16
support	3	147.06	300	350	0.2001	269.8416	double	150.15	4200	1566.836	1566.836	7.796756	8Ø16
Span	3-4	110.11	300	350	0.1498	295.1235	single	150.15	4200	1072.657	1072.657	5.337664	6Ø16
support	4	164.82	300	350	0.2242	254.892	double	150.15	4200	1859.052	1859.052	9.250858	10Ø16
Span	4-5	95.045	300	350	0.1293	304.0173	single	150.15	4200	898.8119	898.8119	4.472591	5Ø16
Support	5	68.38	300	350	0.0930	318.4135	single	150.15	4200	617.4125	617.4125	3.072315	4Ø16
Span	5-6	9.45	300	350	0.0129	345.9826	single	150.15	4200	78.52634	150.15	0.747164	2Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	46.73	300	350	0.0636	329.1162	single	150.15	4200	408.2107	408.2107	2.031303	3Ø16
span	1-2	125.05	300	350	0.1701	285.5994	double	150.15	4200	1258.822	1258.822	6.264041	7Ø16
support	2	49.41	300	350	0.0672	327.832	single	150.15	4200	433.3127	433.3127	2.156214	3Ø16

span	2-3	68.43	300	350	0.0931	318.3879	single	150.15	4200	617.9137	617.9137	3.07481	4Ø16
support	3	36.97	300	350	0.0503	333.7052	single	150.15	4200	318.511	318.511	1.584947	2Ø16
span	3-4	108.76	300	350	0.1480	295.9472	single	150.15	4200	1056.557	1056.557	5.257548	6Ø16
support	4	116.53	300	350	0.1585	291.1266	single	150.15	4200	1150.784	1150.784	5.726432	6Ø16
Span	4-5	92.18	300	350	0.1254	305.6402	single	150.15	4200	867.0898	867.0898	4.314738	5Ø16
Support	5	45.16	300	350	0.0614	329.8635	single	150.15	4200	393.6022	393.6022	1.958609	2Ø16
Span	5-6	6.51	300	350	0.0089	347.2425	single	150.15	4200	53.89965	150.15	0.747164	2Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	29.06	350	350	0.0339	339.1992	single	175.175	4900	246.3081	246.3081	1.225657	2Ø16
span	1-2	22.4	350	350	0.0261	341.7363	single	175.175	4900	188.4494	188.4494	0.937746	2Ø16
support	2	30.33	350	350	0.0354	338.7109	single	175.175	4900	257.443	257.443	1.281066	2Ø16
span	2-3	21.97	350	350	0.0256	341.8988	single	175.175	4900	184.744	184.744	0.919307	2Ø16
support	3	40.4	350	350	0.0471	334.7865	single	175.175	4900	346.9376	346.9376	1.726401	2Ø16
span	3-4	17.3	350	350	0.0202	343.6534	single	175.175	4900	144.7316	175.175	0.871691	2Ø16
support	4	58.12	350	350	0.0678	327.6359	single	175.175	4900	510.0021	510.0021	2.537829	3Ø16
Span	4-5	29.54	350	350	0.0344	339.0148	single	175.175	4900	250.5127	250.5127	1.24658	2Ø16
Support	5	123.8	350	350	0.1444	297.5453	single	175.175	4900	1196.204	1196.204	5.95245	6Ø16
Span	5-6	29.6	350	350	0.0345	338.9917	single	175.175	4900	251.0386	251.0386	1.249197	2Ø16

Axis C

Roof floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	59.13	250	300	0.1314	259.8342	single	107.25	3000	654.2585	654.2585	3.255665	4Ø16
span	1-2	126.76	250	300	0.2817	161.2632	double	107.25	3000	2259.877	2259.877	11.24541	12Ø16
support	2	50.83	250	300	0.1130	266.3121	single	107.25	3000	548.7406	548.7406	2.730596	3Ø16
span	2-3	13.86	250	300	0.0308	291.6111	single	107.25	3000	136.646	136.646	0.679966	2Ø16
support	3	5.67	250	300	0.0126	296.6262	single	107.25	3000	54.95553	107.25	0.533688	2Ø16
span	3-4	49.48	250	300	0.1100	267.3319	single	107.25	3000	532.1287	532.1287	2.647934	3Ø16
support	4	76.82	250	300	0.1707	244.5583	double	107.25	3000	903.0874	903.0874	4.493866	5Ø16
Span	4-5	26.82	250	300	0.0596	283.2902	single	107.25	3000	272.1856	272.1856	1.354427	2Ø16
Support	5	55.23	250	300	0.1227	262.9243	single	107.25	3000	603.9238	603.9238	3.005194	4Ø16
Span	5-6	2.37	250	300	0.0053	298.5991	single	107.25	3000	22.81906	107.25	0.533688	2Ø16

5th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	86.84	300	350	0.1181	308.6124	single	150.15	4200	808.992	808.992	4.025637	5Ø16
Span	1-2	136.7	300	350	0.1860	277.5608	double	150.15	4200	1415.951	1415.951	7.045936	8Ø16

Support	2	69.92	300	350	0.0951	317.6216	single	150.15	4200	632.8914	632.8914	3.14934	4Ø16
Span	2-3	42.14	300	350	0.0573	331.2911	single	150.15	4200	365.698	365.698	1.819755	2Ø16
Support	3	102.89	300	350	0.1400	299.4652	single	150.15	4200	987.7899	987.7899	4.915356	5Ø16
Span	3-4	85.54	300	350	0.1164	309.3261	single	150.15	4200	795.0429	795.0429	3.956225	4Ø16
Support	4	89.74	300	350	0.1221	307.0066	single	150.15	4200	840.381	840.381	4.181832	5Ø16
Span	4-5	33	300	350	0.0449	335.5343	single	150.15	4200	282.758	282.758	1.407036	2Ø16
Support	5	37.47	300	350	0.0510	333.4733	single	150.15	4200	323.0431	323.0431	1.6075	2Ø16
Span	5-6	61.86	300	350	0.0842	321.7189	single	150.15	4200	552.8041	552.8041	2.750816	3Ø16

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	86.46	300	350	0.1176	308.8214	single	150.15	4200	804.9069	804.9069	4.005309	5Ø16
Span	1-2	144.73	300	350	0.1969	271.6314	double	150.15	4200	1531.851	1531.851	7.622664	8Ø16
Support	2	62.33	300	350	0.0848	321.4831	single	150.15	4200	557.4127	557.4127	2.773749	3Ø16
Span	2-3	31.46	300	350	0.0428	336.2382	single	150.15	4200	268.9983	268.9983	1.338566	2Ø16
Support	3	188.47	300	350	0.2564	228.8907	double	150.15	4200	2367.293	2367.293	11.77992	12Ø16
Span	3-4	94.47	300	350	0.1285	304.3446	single	150.15	4200	892.4134	892.4134	4.440752	5Ø16
Support	4	32.21	300	350	0.0438	335.8958	single	150.15	4200	275.6919	275.6919	1.371875	2Ø16
Span	4-5	39.33	300	350	0.0535	332.6078	single	150.15	4200	339.9612	339.9612	1.691686	2Ø16
Support	5	71.909	300	350	0.0978	316.5923	single	150.15	4200	653.0113	653.0113	3.249459	4Ø16

Span	5-6	54.72	300	350	0.0744	325.2551	single	150.15	4200	483.6819	483.6819	2.406856	3Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	87.47	300	350	0.1190	308.2652	single	150.15	4200	815.7788	815.7788	4.059409	5Ø16
Span	1-2	144.845	300	350	0.1971	271.5438	double	150.15	4200	1533.562	1533.562	7.631181	8Ø16
Support	2	64.95	300	350	0.0884	320.1618	single	150.15	4200	583.2403	583.2403	2.902271	3Ø16
Span	2-3	53.06	300	350	0.0722	326.0654	single	150.15	4200	467.8432	467.8432	2.328042	3Ø16
Support	3	179.7	300	350	0.2445	239.7621	double	150.15	4200	2154.793	2154.793	10.72249	11Ø16
Span	3-4	68.02	300	350	0.0925	318.598	single	150.15	4200	613.8064	613.8064	3.054371	4Ø16
Support	4	115.69	300	350	0.1574	291.6573	single	150.15	4200	1140.409	1140.409	5.674808	6Ø16
Span	4-5	31.78	300	350	0.0432	336.0922	single	150.15	4200	271.8525	271.8525	1.352769	2Ø16
Support	5	22	300	350	0.0299	340.4967	single	150.15	4200	185.758	185.758	0.924353	2Ø16
Span	5-6	47.2	300	350	0.0642	328.8917	single	150.15	4200	412.5978	412.5978	2.053134	3Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	86.16	300	350	0.1172	308.9862	single	150.15	4200	801.6863	801.6863	3.989283	4Ø16
Span	1-2	153.22	300	350	0.2085	264.9383	double	150.15	4200	1662.68	1662.68	8.273685	9Ø16

Support	2	53.2	300	350	0.0724	325.9972	single	150.15	4200	469.1758	469.1758	2.334672	3Ø16
Span	2-3	99.97	300	350	0.1360	301.1788	single	150.15	4200	954.2962	954.2962	4.748687	4Ø16
Support	3	155.88	300	350	0.2121	262.7363	double	150.15	4200	1705.722	1705.722	8.487868	9Ø16
Span	3-4	48.88	300	350	0.0665	328.0868	single	150.15	4200	428.3318	428.3318	2.131428	3Ø16
Support	4	100.18	300	350	0.1363	301.0563	single	150.15	4200	956.6898	956.6898	4.760598	5Ø16
Span	4-5	24.87	300	350	0.0338	339.2164	single	150.15	4200	210.7836	210.7836	1.048883	2Ø16
Support	5	87.7	300	350	0.1193	308.1382	single	150.15	4200	818.261	818.261	4.07176	5Ø16
Span	5-6	47.9	300	350	0.0652	328.5569	single	150.15	4200	419.1436	419.1436	2.085707	3Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	87.41	300	350	0.1189	308.2983	single	150.15	4200	815.1317	815.1317	4.056189	5Ø16
span	1-2	164.39	300	350	0.2237	255.2868	double	150.15	4200	1851.334	1851.334	9.212452	10Ø16
support	2	53.2	300	350	0.0724	325.9972	single	150.15	4200	469.1758	469.1758	2.334672	3Ø16
span	2-3	47.43	300	350	0.0645	328.7818	single	150.15	4200	414.747	414.747	2.063828	3Ø16
support	3	180.4	300	350	0.2454	238.9622	double	150.15	4200	2170.427	2170.427	10.80029	11Ø16
span	3-4	30.3	300	350	0.0412	336.7664	single	150.15	4200	258.6734	258.6734	1.287188	2Ø16
support	4	104.82	300	350	0.1426	298.3196	single	150.15	4200	1010.183	1010.183	5.026788	6Ø16
Span	4-5	12.53	300	350	0.0170	344.6527	single	150.15	4200	104.5219	150.15	0.747164	2Ø16
Support	5	122.9	300	350	0.1672	287.0199	double	150.15	4200	1231.056	1231.056	6.125875	7Ø16

Span	5-6	35.12	300	350	0.0478	334.5601	single	150.15	4200	301.7993	301.7993	1.501788	2Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	46.73	350	350	0.0545	332.2695	single	175.175	4900	404.3367	404.3367	2.012026	3Ø16
span	1-2	43.5	350	350	0.0507	333.5588	single	175.175	4900	374.9339	374.9339	1.865714	2Ø16
support	2	73.45	350	350	0.0857	321.1679	single	175.175	4900	657.5027	657.5027	3.271809	4Ø16
span	2-3	8.01	350	350	0.0093	347.0906	single	175.175	4900	66.34795	175.175	0.871691	2Ø16
support	3	79.12	350	350	0.0923	318.7019	single	175.175	4900	713.7391	713.7391	3.551648	4Ø16
span	3-4	7.12	350	350	0.0083	347.4163	single	175.175	4900	58.92067	175.175	0.871691	2Ø16
support	4	57.67	350	350	0.0673	327.8217	single	175.175	4900	505.7666	505.7666	2.516753	3Ø16
Span	4-5	9.24	350	350	0.0108	346.6394	single	175.175	4900	76.63582	175.175	0.871691	2Ø16
Support	5	22.9	350	350	0.0267	341.5472	single	175.175	4900	192.7625	192.7625	0.959208	2Ø16
Span	5-6	7.13	350	350	0.0083	347.4126	single	175.175	4900	59.00405	175.175	0.871691	2sØ16

Axis D

Roof

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	34.9	250	300	0.0776	277.8286	single	107.25	3000	361.1489	361.1489	1.797118	2Ø16
span	1-2	51.6	250	300	0.1147	265.7264	single	107.25	3000	558.281	558.281	2.77807	3Ø16
support	2	25.502	250	300	0.0567	284.16	single	107.25	3000	258.0175	258.0175	1.283925	2Ø16

5th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	45.74	300	350	0.0622	329.5879	single	150.15	4200	398.9907	398.9907	1.985424	2Ø16
Span	1-2	53.8	300	350	0.0732	325.7047	single	150.15	4200	474.8933	474.8933	2.363124	3Ø16
Support	2	49.88	300	350	0.0679	327.6056	single	150.15	4200	437.7367	437.7367	2.178228	3Ø16

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	44.06	300	350	0.0599	330.385	single	150.15	4200	383.4087	383.4087	1.907886	2Ø16
Span	1-2	64.05	300	350	0.0871	320.617	single	150.15	4200	574.3418	574.3418	2.857991	3Ø16
Support	2	68.3	300	350	0.0929	318.4546	single	150.15	4200	616.6108	616.6108	3.068326	4Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	44.72	300	350	0.0608	330.0723	single	150.15	4200	389.5207	389.5207	1.938299	2Ø16
Span	1-2	78.2	300	350	0.1064	313.2862	single	150.15	4200	717.6345	717.6345	3.571031	4Ø16
Support	2	66.4	300	350	0.0903	319.4253	single	150.15	4200	597.6358	597.6358	2.973904	3Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	42.39	300	350	0.0577	331.1734	single	150.15	4200	367.9983	367.9983	1.831202	2Ø16
Span	1-2	89.04	300	350	0.1211	307.396	single	150.15	4200	832.7695	832.7695	4.143956	5Ø16
Support	2	78.09	300	350	0.1062	313.3447	single	150.15	4200	716.4912	716.4912	3.565343	4Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	40.52	300	350	0.0551	332.0515	single	150.15	4200	350.8341	350.8341	1.745791	2Ø16
Span	1-2	102.43	300	350	0.1394	299.7367	single	150.15	4200	982.483	982.483	4.888948	5Ø16
Support	2	80.6	300	350	0.1097	312.004	single	150.15	4200	742.6989	742.6989	3.695755	4Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
Support	1	29.44	350	350	0.0343	339.0532	single	175.175	4900	249.6363	249.6363	1.242219	2Ø16
span	1-2	15.59	350	350	0.0182	344.2913	single	175.175	4900	130.1841	175.175	0.871691	2Ø16
support	2	41.61	350	350	0.0485	334.3084	single	175.175	4900	357.8395	357.8395	1.78065	2Ø16

Axis 1

Roof

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	112	250	300	0.2489	202.2685	double	107.25	3000	1591.943	1591.943	7.921691	8Ø16
Span	A-B	115	250	300	0.2556	196.9308	double	107.25	3000	1678.889	1678.889	8.354345	9Ø16
support	B	109.67	250	300	0.2437	206.0647	double	107.25	3000	1530.108	1530.108	7.613995	8Ø16
Span	B-C	119.53	250	300	0.2656	187.456	double	107.25	3000	1833.223	1833.223	9.122328	10Ø16
support	C	0	250	300	0.0000	300	single	107.25	3000	0	107.25	0.533688	2Ø16
span	C-D	0	250	300	0.0000	300	single	107.25	3000	0	107.25	0.533688	2Ø16
support	D	28.46	250	300	0.0632	282.1999	single	107.25	3000	289.9452	289.9452	1.442801	2Ø16

5th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	110.48	300	350	0.1503	294.8968	single	150.15	4200	1077.089	1077.089	5.359717	6Ø16
span	A-B	125	300	350	0.1701	285.6327	double	150.15	4200	1258.172	1258.172	6.260808	6Ø16
support	B	86.84	300	350	0.1181	308.6124	single	150.15	4200	808.992	808.992	4.025637	5Ø16
span	B-C	144.31	300	350	0.1963	271.9505	double	150.15	4200	1525.613	1525.613	7.591625	8Ø16
support	C	5.74	300	350	0.0078	347.571	single	150.15	4200	47.47951	150.15	0.747164	2Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	41.33	300	350	0.0562	331.6718	single	150.15	4200	358.257	358.257	1.782728	2Ø16

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	117.11	300	350	0.1593	290.7587	single	150.15	4200	1157.975	1157.975	5.762216	6Ø16
span	A-B	125	300	350	0.1701	285.6327	double	150.15	4200	1258.172	1258.172	6.260808	7Ø16
support	B	86.46	300	350	0.1176	308.8214	single	150.15	4200	804.9069	804.9069	4.005309	5Ø16
span	B-C	143	300	350	0.1946	272.9392	double	150.15	4200	1506.288	1506.288	7.495462	8Ø16
support	C	4.06	300	350	0.0055	348.2854	single	150.15	4200	33.51418	150.15	0.747164	2Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	4.56	300	350	0.0062	348.0731	single	150.15	4200	37.6645	150.15	0.747164	2Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	120.93	300	350	0.1645	288.3058	single	150.15	4200	1205.92	1205.92	6.000796	7Ø16
span	A-B	137.89	300	350	0.1876	276.7039	double	150.15	4200	1432.7	1432.7	7.129281	8Ø16
support	B	87.47	300	350	0.1190	308.2652	single	150.15	4200	815.7788	815.7788	4.059409	5Ø16
span	B-C	140.23	300	350	0.1908	274.9975	double	150.15	4200	1466.054	1466.054	7.295253	8Ø16
support	C	4.76	300	350	0.0065	347.9881	single	150.15	4200	39.32606	150.15	0.747164	2Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	9.03	300	350	0.0123	346.1632	single	150.15	4200	74.99714	150.15	0.747164	2Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	117.17	300	350	0.1594	290.7206	single	150.15	4200	1158.72	1158.72	5.765924	6Ø16
span	A-B	117	300	350	0.1592	290.8285	single	150.15	4200	1156.609	1156.609	5.755421	6Ø16
support	B	86.16	300	350	0.1172	308.9862	single	150.15	4200	801.6863	801.6863	3.989283	4Ø16
span	B-C	140.13	300	350	0.1907	275.071	double	150.15	4200	1464.617	1464.617	7.288103	8Ø16
support	C	2.39	300	350	0.0033	348.9927	single	150.15	4200	19.68881	150.15	0.747164	2Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	29.12	300	350	0.0396	337.302	single	150.15	4200	248.2049	248.2049	1.235096	2Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	109.51	300	350	0.1490	295.4903	single	150.15	4200	1065.488	1065.488	5.301989	6Ø16
span	A-B	118.34	300	350	0.1610	289.9746	single	150.15	4200	1173.301	1173.301	5.838481	6Ø16
support	B	78.9	300	350	0.1073	312.9135	single	150.15	4200	724.9209	724.9209	3.607289	4Ø16
span	B-C	136.67	300	350	0.1859	277.5823	double	150.15	4200	1415.531	1415.531	7.043844	8Ø16
support	C	10.52	300	350	0.0143	345.5218	single	150.15	4200	87.53427	150.15	0.747164	2Ø16
span	C-D	1.86	300	350	0.0025	349.2166	single	150.15	4200	15.31285	150.15	0.747164	2Ø16
support	D	29.12	300	350	0.0396	337.302	single	150.15	4200	248.2049	248.2049	1.235096	2Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	25.11	350	350	0.0293	340.7086	single	175.175	4900	211.8856	211.8856	1.054367	2Ø16
span	A-B	1.06	350	350	0.0012	349.6178	single	175.175	4900	8.716662	175.175	0.871691	2Ø16
support	B	46.73	350	350	0.0545	332.2695	single	175.175	4900	404.3367	404.3367	2.012026	3Ø16
span	B-C	20.51	350	350	0.0239	342.4493	single	175.175	4900	172.1897	175.175	0.871691	2Ø16
support	C	29.44	350	350	0.0343	339.0532	single	175.175	4900	249.6363	249.6363	1.242219	2Ø16
span	C-D	17.19	350	350	0.0200	343.6945	single	175.175	4900	143.7941	175.175	0.871691	2Ø16
support	D	3.05	350	350	0.0036	348.8979	single	175.175	4900	25.13271	175.175	0.871691	2Ø16

Axis 2

Roof

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	123.46	250	300	0.2744	176.6329	double	107.25	3000	2009.521	2009.521	9.999608	10Ø16
span	A-B	114.67	250	300	0.2548	197.5473	double	107.25	3000	1668.847	1668.847	8.304375	9Ø16
support	B	27.6	250	300	0.0613	282.7727	single	107.25	3000	280.614	280.614	1.396368	2Ø16
span	B-C	104.69	250	300	0.2326	213.4209	double	107.25	3000	1410.282	1410.282	7.017726	8Ø16
support	C	121	250	300	0.2689	183.8157	double	107.25	3000	1892.521	1892.521	9.417401	10Ø16
span	C-D	0	250	300	0.0000	300	single	107.25	3000	0	107.25	0.533688	2Ø16
support	D	67.29	250	300	0.1495	253.0695	single	107.25	3000	764.4492	764.4492	3.803987	4Ø16

5th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	117.89	300	350	0.1604	290.2621	single	150.15	4200	1167.682	1167.682	5.810519	6Ø16
span	A-B	114.59	300	350	0.1559	292.3487	single	150.15	4200	1126.895	1126.895	5.607558	6Ø16
support	B	86.84	300	350	0.1181	308.6124	single	150.15	4200	808.992	808.992	4.025637	5Ø16
span	B-C	119.6	300	350	0.1627	289.1658	single	150.15	4200	1189.11	1189.11	5.917148	6Ø16
support	C	4.08	300	350	0.0056	348.2769	single	150.15	4200	33.68009	150.15	0.747164	2Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16

support	D	1.73	300	350	0.0024	349.2715	single	150.15	4200	14.24036	150.15	0.747164	2Ø16

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	118.98	300	350	0.1619	289.5645	single	150.15	4200	1181.317	1181.317	5.878369	6Ø16
span	A-B	121	300	350	0.1646	288.2604	single	150.15	4200	1206.808	1206.808	6.005216	7Ø16
support	B	116.33	300	350	0.1583	291.2532	single	150.15	4200	1148.309	1148.309	5.714119	6Ø16
span	B-C	106.55	300	350	0.1450	297.2836	single	150.15	4200	1030.435	1030.435	5.12756	6Ø16
support	C	93.74	300	350	0.1275	304.759	single	150.15	4200	884.3134	884.3134	4.400445	5Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	20.68	300	350	0.0281	341.0823	single	150.15	4200	174.3128	174.3128	0.8674	2Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	120.93	300	350	0.1645	288.3058	single	150.15	4200	1205.92	1205.92	6.000796	7Ø16
span	A-B	137.89	300	350	0.1876	276.7039	double	150.15	4200	1432.7	1432.7	7.129281	8Ø16
support	B	87.47	300	350	0.1190	308.2652	single	150.15	4200	815.7788	815.7788	4.059409	5Ø16
span	B-C	140.23	300	350	0.1908	274.9975	double	150.15	4200	1466.054	1466.054	7.295253	8Ø16
support	C	4.76	300	350	0.0065	347.9881	single	150.15	4200	39.32606	150.15	0.747164	2Ø16

span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	9.03	300	350	0.0123	346.1632	single	150.15	4200	74.99714	150.15	0.747164	2Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	117.17	300	350	0.1594	290.7206	single	150.15	4200	1158.72	1158.72	5.765924	6Ø16
span	A-B	117	300	350	0.1592	290.8285	single	150.15	4200	1156.609	1156.609	5.755421	6Ø16
support	B	86.16	300	350	0.1172	308.9862	single	150.15	4200	801.6863	801.6863	3.989283	4Ø16
span	B-C	140.13	300	350	0.1907	275.071	double	150.15	4200	1464.617	1464.617	7.288103	8Ø16
support	C	2.39	300	350	0.0033	348.9927	single	150.15	4200	19.68881	150.15	0.747164	2Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	29.12	300	350	0.0396	337.302	single	150.15	4200	248.2049	248.2049	1.235096	2Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	109.51	300	350	0.1490	295.4903	single	150.15	4200	1065.488	1065.488	5.301989	6Ø16
span	A-B	108.55	300	350	0.1477	296.0748	single	150.15	4200	1054.062	1054.062	5.245134	6Ø16
support	B	78.9	300	350	0.1073	312.9135	single	150.15	4200	724.9209	724.9209	3.607289	4Ø16
span	B-C	136.67	300	350	0.1859	277.5823	double	150.15	4200	1415.531	1415.531	7.043844	8Ø16

support	C	9.5	300	350	0.0129	345.9611	single	150.15	4200	78.94673	150.15	0.747164	2Ø16
span	C-D	1.86	300	350	0.0025	349.2166	single	150.15	4200	15.31285	150.15	0.747164	2Ø16
support	D	29.12	300	350	0.0396	337.302	single	150.15	4200	248.2049	248.2049	1.235096	2Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	46.46	350	350	0.0542	332.3776	single	175.175	4900	401.8697	401.8697	1.99975	2Ø16
span	A-B	21.9	350	350	0.0255	341.9252	single	175.175	4900	184.1411	184.1411	0.916307	2Ø16
support	B	73.45	350	350	0.0857	321.1679	single	175.175	4900	657.5027	657.5027	3.271809	4Ø16
span	B-C	20.58	350	350	0.0240	342.423	single	175.175	4900	172.7907	175.175	0.871691	2Ø16
support	C	6.22	350	350	0.0073	347.745	single	175.175	4900	51.42418	175.175	0.871691	2Ø16
span	C-D	24.15	350	350	0.0282	341.0734	single	175.175	4900	203.5669	203.5669	1.012972	2Ø16
support	D	6.21	350	350	0.0072	347.7487	single	175.175	4900	51.34096	175.175	0.871691	2Ø16

Axis 3

Roof floor

Type	Loc	Moment	b(mm)	d(mm)	K	Z(mm)	Beam	As,min	As,max	As,cal	As,prov	No of	Remark

		(KNm)					type					bar	
support	A	109.61	250	300	0.2436	206.159	double	107.25	3000	1528.571	1528.571	7.606346	8Ø16
span	A-B	103.2	250	300	0.2293	215.4614	double	107.25	3000	1377.045	1377.045	6.852332	7Ø16
support	B	114.23	250	300	0.2538	198.3571	double	107.25	3000	1655.657	1655.657	8.238739	9Ø16
span	B-C	104.69	250	300	0.2326	213.4209	double	107.25	3000	1410.282	1410.282	7.017726	8Ø16
support	C	121	250	300	0.2689	183.8157	double	107.25	3000	1892.521	1892.521	9.417401	10Ø16
span	C-D	0	250	300	0.0000	300	single	107.25	3000	0	107.25	0.533688	2Ø16
support	D	67.29	250	300	0.1495	253.0695	single	107.25	3000	764.4492	764.4492	3.803987	4Ø16

5th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	117.89	300	350	0.1604	290.2621	single	150.15	4200	1167.682	1167.682	5.810519	8Ø16
span	A-B	114.59	300	350	0.1559	292.3487	single	150.15	4200	1126.895	1126.895	5.607558	8Ø16
support	B	86.84	300	350	0.1181	308.6124	single	150.15	4200	808.992	808.992	4.025637	5Ø16
span	B-C	119.6	300	350	0.1627	289.1658	single	150.15	4200	1189.11	1189.11	5.917148	6Ø16
support	C	4.08	300	350	0.0056	348.2769	single	150.15	4200	33.68009	150.15	0.747164	2Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	1.73	300	350	0.0024	349.2715	single	150.15	4200	14.24036	150.15	0.747164	2Ø16

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	118.98	300	350	0.1619	289.5645	single	150.15	4200	1181.317	1181.317	5.878369	6Ø16
span	A-B	121	300	350	0.1646	288.2604	single	150.15	4200	1206.808	1206.808	6.005216	7Ø16
support	B	116.33	300	350	0.1583	291.2532	single	150.15	4200	1148.309	1148.309	5.714119	6Ø16
span	B-C	106.55	300	350	0.1450	297.2836	single	150.15	4200	1030.435	1030.435	5.12756	6Ø16
support	C	93.74	300	350	0.1275	304.759	single	150.15	4200	884.3134	884.3134	4.400445	5Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	20.68	300	350	0.0281	341.0823	single	150.15	4200	174.3128	174.3128	0.8674	2Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	120.93	300	350	0.1645	288.3058	single	150.15	4200	1205.92	1205.92	6.000796	7Ø16
span	A-B	137.89	300	350	0.1876	276.7039	double	150.15	4200	1432.7	1432.7	7.129281	8Ø16
support	B	87.47	300	350	0.1190	308.2652	single	150.15	4200	815.7788	815.7788	4.059409	5Ø16
span	B-C	140.23	300	350	0.1908	274.9975	double	150.15	4200	1466.054	1466.054	7.295253	8Ø16
support	C	4.76	300	350	0.0065	347.9881	single	150.15	4200	39.32606	150.15	0.747164	2Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	9.03	300	350	0.0123	346.1632	single	150.15	4200	74.99714	150.15	0.747164	2Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	117.17	300	350	0.1594	290.7206	single	150.15	4200	1158.72	1158.72	5.765924	6Ø16
span	A-B	117	300	350	0.1592	290.8285	single	150.15	4200	1156.609	1156.609	5.755421	6Ø16
support	B	86.16	300	350	0.1172	308.9862	single	150.15	4200	801.6863	801.6863	3.989283	4Ø16
span	B-C	140.13	300	350	0.1907	275.071	double	150.15	4200	1464.617	1464.617	7.288103	8Ø16
support	C	2.39	300	350	0.0033	348.9927	single	150.15	4200	19.68881	150.15	0.747164	2Ø16
span	C-D	0	300	350	0.0000	350	single	150.15	4200	0	150.15	0.747164	2Ø16
support	D	29.12	300	350	0.0396	337.302	single	150.15	4200	248.2049	248.2049	1.235096	2Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	109.51	300	350	0.1490	295.4903	single	150.15	4200	1065.488	1065.488	5.301989	6Ø16
span	A-B	108.55	300	350	0.1477	296.0748	single	150.15	4200	1054.062	1054.062	5.245134	6Ø16
support	B	78.9	300	350	0.1073	312.9135	single	150.15	4200	724.9209	724.9209	3.607289	4Ø16
span	B-C	136.67	300	350	0.1859	277.5823	double	150.15	4200	1415.531	1415.531	7.043844	8Ø16
support	C	9.5	300	350	0.0129	345.9611	single	150.15	4200	78.94673	150.15	0.747164	2Ø16
span	C-D	1.86	300	350	0.0025	349.2166	single	150.15	4200	15.31285	150.15	0.747164	2Ø16
support	D	29.12	300	350	0.0396	337.302	single	150.15	4200	248.2049	248.2049	1.235096	2Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	46.46	350	350	0.0542	332.3776	single	175.175	4900	401.8697	401.8697	1.99975	2Ø16
span	A-B	21.9	350	350	0.0255	341.9252	single	175.175	4900	184.1411	184.1411	0.916307	2Ø16
support	B	73.45	350	350	0.0857	321.1679	single	175.175	4900	657.5027	657.5027	3.271809	4Ø16
span	B-C	20.58	350	350	0.0240	342.423	single	175.175	4900	172.7907	175.175	0.871691	2Ø16
support	C	6.22	350	350	0.0073	347.745	single	175.175	4900	51.42418	175.175	0.871691	2Ø16
span	C-D	24.15	350	350	0.0282	341.0734	single	175.175	4900	203.5669	203.5669	1.012972	2Ø16
support	D	6.21	350	350	0.0072	347.7487	single	175.175	4900	51.34096	175.175	0.871691	2Ø16

Axis 3

Roof

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	109.61	250	300	0.2436	206.159	double	107.25	3000	1528.571	1528.571	7.606346	8Ø16
span	A-B	103.56	250	300	0.2301	214.9743	double	107.25	3000	1384.979	1384.979	6.891817	7Ø16
support	B	104.5	250	300	0.2322	213.6848	double	107.25	3000	1405.985	1405.985	6.996341	7Ø16
span	B-C	109.35	250	300	0.2430	206.5661	double	107.25	3000	1521.94	1521.94	7.573348	8Ø16
support	C	114.7	250	300	0.2549	197.4916	double	107.25	3000	1669.755	1669.755	8.308891	9Ø16

5th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	108.4	300	350	0.1475	296.1659	single	150.15	4200	1052.282	1052.282	5.236276	6Ø16
span	A-B	113.17	300	350	0.1540	293.2353	single	150.15	4200	1109.566	1109.566	5.521325	6Ø16
support	B	128	300	350	0.1741	283.6201	double	150.15	4200	1297.51	1297.51	6.456559	7Ø16
span	B-C	131	300	350	0.1782	281.5696	double	150.15	4200	1337.591	1337.591	6.656006	7Ø16
support	C	145	300	350	0.1973	271.4257	double	150.15	4200	1535.871	1535.871	7.642673	8Ø16

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	118.41	300	350	0.1611	289.9298	single	150.15	4200	1174.176	1174.176	5.842836	6Ø16
span	A-B	143.9	300	350	0.1958	272.261	double	150.15	4200	1519.544	1519.544	7.561423	8Ø16
support	B	108.2	300	350	0.1472	296.2872	single	150.15	4200	1049.91	1049.91	5.224474	6Ø16
span	B-C	104	300	350	0.1415	298.8076	single	150.15	4200	1000.644	1000.644	4.979318	5Ø16
support	C	114.7	300	350	0.1561	292.2798	single	150.15	4200	1128.243	1128.243	5.614265	6Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	113.34	300	350	0.1542	293.1295	single	150.15	4200	1111.633	1111.633	5.531615	6Ø16
span	A-B	106.9	300	350	0.1454	297.0729	single	150.15	4200	1034.552	1034.552	5.148052	6Ø16
support	B	107.71	300	350	0.1465	296.5839	single	150.15	4200	1044.11	1044.11	5.195611	6Ø16
span	B-C	118.15	300	350	0.1607	290.0961	single	150.15	4200	1170.927	1170.927	5.826666	6Ø16
support	C	110.6	300	350	0.1505	294.8231	single	150.15	4200	1078.528	1078.528	5.366879	6Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	109.61	300	350	0.1491	295.4292	single	150.15	4200	1066.681	1066.681	5.307927	6Ø16
span	A-B	133.2	300	350	0.1812	280.0405	double	150.15	4200	1367.481	1367.481	6.804742	7Ø16
support	B	104.5	300	350	0.1422	298.5103	single	150.15	4200	1006.456	1006.456	5.008241	6Ø16
span	B-C	109.35	300	350	0.1488	295.5879	single	150.15	4200	1063.58	1063.58	5.292494	6Ø16
support	C	114.7	300	350	0.1561	292.2798	single	150.15	4200	1128.243	1128.243	5.614265	6Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	109.61	300	350	0.1491	295.4292	single	150.15	4200	1066.681	1066.681	5.307927	6Ø16
span	A-B	133.2	300	350	0.1812	280.0405	double	150.15	4200	1367.481	1367.481	6.804742	7Ø16

support	B	104.5	300	350	0.1422	298.5103	single	150.15	4200	1006.456	1006.456	5.008241	6Ø16
span	B-C	109.35	300	350	0.1488	295.5879	single	150.15	4200	1063.58	1063.58	5.292494	6Ø16
support	C	114.7	300	350	0.1561	292.2798	single	150.15	4200	1128.243	1128.243	5.614265	6Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	35.49	350	350	0.0414	336.7119	single	175.175	4900	303.0299	303.0299	1.507911	2Ø16
span	A-B	17.36	350	350	0.0202	343.631	single	175.175	4900	145.243	175.175	0.871691	2Ø16
support	B	79.12	350	350	0.0923	318.7019	single	175.175	4900	713.7391	713.7391	3.551648	4Ø16
span	B-C	10.26	350	350	0.0120	346.2644	single	175.175	4900	85.18778	175.175	0.871691	2Ø16
support	C	0	350	350	0.0000	350	single	175.175	4900	0	175.175	0.871691	2Ø16

Axis 4

Roof

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	118.5	250	300	0.2633	189.8089	double	107.25	3000	1794.897	1794.897	8.931614	9Ø16
span	A-B	111.2	250	300	0.2471	203.6022	double	107.25	3000	1570.218	1570.218	7.813587	8Ø16

support	B	127.3	250	300	0.2829	155.6169	double	107.25	3000	2351.849	2351.849	11.70307	12Ø16
span	B-C	108.79	250	300	0.2418	207.4331	double	107.25	3000	1507.817	1507.817	7.503071	8Ø16
support	C	121	250	300	0.2689	183.8157	double	107.25	3000	1892.521	1892.521	9.417401	10Ø16

5th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	187.6	300	350	0.2552	230.0651	double	150.15	4200	2344.336	2344.336	11.66569	12Ø16
span	A-B	151.62	300	350	0.2063	266.2372	double	150.15	4200	1637.29	1637.29	8.147343	9Ø16
support	B	149.14	300	350	0.2029	268.2148	double	150.15	4200	1598.635	1598.635	7.954991	8Ø16
span	B-C	184.4	300	350	0.2509	234.1847	double	150.15	4200	2263.811	2263.811	11.26498	12Ø16
support	C	83.9	300	350	0.1141	310.221	single	150.15	4200	777.5506	777.5506	3.869181	4Ø16

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	187.6	300	350	0.2552	230.0651	double	150.15	4200	2344.336	2344.336	11.66569	12Ø16
span	A-B	151.63	300	350	0.2063		double	150.15	4200	1637.448	1637.448	8.148127	9Ø16

support	B	149.14	300	350	0.2029	268.2148	double	150.15	4200	1598.635	1598.635	7.954991	8Ø16
span	B-C	184.4	300	350	0.2509	234.1847	double	150.15	4200	2263.811	2263.811	11.26498	12Ø16
support	C	83.9	300	350	0.1141	266.2291	single	150.15	4200	906.0335	906.0335	4.508526	5Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	165.06	300	350	0.2246	254.6707	double	150.15	4200	1863.377	1863.377	9.272376	10Ø16
span	A-B	101.01	300	350	0.1374	300.5711	single	150.15	4200	966.1731	966.1731	4.807788	5Ø16
support	B	115.69	300	350	0.1574	291.6573	single	150.15	4200	1140.409	1140.409	5.674808	6Ø16
span	B-C	137.52	300	350	0.1871	276.9711	double	150.15	4200	1427.478	1427.478	7.103292	8Ø16
support	C	54.15	300	350	0.0737	325.5338	single	150.15	4200	478.2337	478.2337	2.379746	3Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	183.19	300	350	0.2492	235.6696	double	150.15	4200	2234.786	2234.786	11.12055	12Ø16

span	A-B	133.2	300	350	0.1812	280.0405	double	150.15	4200	1367.481	1367.481	6.804742	7Ø16
support	B	100.18	300	350	0.1363	301.0563	single	150.15	4200	956.6898	956.6898	4.760598	5Ø16
span	B-C	126.86	300	350	0.1726	284.3893	double	150.15	4200	1282.476	1282.476	6.38175	7Ø16
support	C	48.98	300	350	0.0666	328.0387	single	150.15	4200	429.2709	429.2709	2.136101	3Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	145.81	300	350	0.1984	270.8059	double	150.15	4200	1547.986	1547.986	7.702955	8Ø16
span	A-B	144.02	300	350	0.1959	272.1703	double	150.15	4200	1521.318	1521.318	7.570253	8Ø16
support	B	104.82	300	350	0.1426	298.3196	single	150.15	4200	1010.183	1010.183	5.026788	6Ø16
span	B-C	128.14	300	350	0.1743	283.5253	double	150.15	4200	1299.364	1299.364	6.465783	7Ø16
support	C	30.3	300	350	0.0412	336.7664	single	150.15	4200	258.6734	258.6734	1.287188	2Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark

support	A	18.64	350	350	0.0217	343.1518	single	175.175	4900	156.17	175.175	0.871691	2Ø16
span	A-B	29.56	350	350	0.0345	339.0071	single	175.175	4900	250.688	250.688	1.247452	2Ø16
support	B	57.67	350	350	0.0673	327.8217	single	175.175	4900	505.7666	505.7666	2.516753	3Ø16
span	B-C	34.2	350	350	0.0399	337.2139	single	175.175	4900	291.5805	291.5805	1.450938	3Ø16
support	C	7.12	350	350	0.0083	347.4163	single	175.175	4900	58.92067	175.175	0.871691	2Ø16

Axis 5

Roof

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	114.5	250	300	0.2544	197.8618	double	107.25	3000	1663.725	1663.725	8.278884	9Ø16
span	A-B	108.56	250	300	0.2412	207.7855	double	107.25	3000	1502.078	1502.078	7.474513	8Ø16
support	B	122.2	250	300	0.2716	180.5238	double	107.25	3000	1946.143	1946.143	9.684229	10Ø16
span	B-C	116.7	250	300	0.2593	193.6171	double	107.25	3000	1732.866	1732.866	8.622941	9Ø16
support	C	106.24	250	300	0.2361	211.2261	double	107.25	3000	1446.033	1446.033	7.195627	8Ø16

5th floor

Type	Loc	Moment	b(mm)	d(mm)	K	Z(mm)	Beam	As,min	As,max	As,cal	As,prov	No of	Remark

		(KNm)					type					bar	
support	A	128.05	300	350	0.1742	283.5863	double	150.15	4200	1298.172	1298.172	6.459852	7Ø16
span	A-B	138.69	300	350	0.1887	276.1237	double	150.15	4200	1444.04	1444.04	7.185709	8Ø16
support	B	94.68	300	350	0.1288	304.2252	single	150.15	4200	894.7484	894.7484	4.452371	5Ø16
span	B-C	86.23	300	350	0.1173	308.9478	single	150.15	4200	802.4374	802.4374	3.993021	4Ø16
support	C	33	300	350	0.0449	335.5343	single	150.15	4200	282.758	282.758	1.407036	2Ø16

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	133.7	300	350	0.1819	279.6898	double	150.15	4200	1374.335	1374.335	6.838848	7Ø16
span	A-B	139.83	300	350	0.1902		double	150.15	4200	1460.312	1460.312	7.266682	8Ø16
support	B	71.9	300	350	0.0978	316.597	single	150.15	4200	652.92	652.92	3.249005	4Ø16
span	B-C	92.71	300	350	0.1261	305.3415	single	150.15	4200	872.9284	872.9284	4.343792	5Ø16
support	C	39.3	300	350	0.0535	275.2913	single	150.15	4200	410.4289	410.4289	2.042341	3Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	141.43	300	350	0.1924	274.1111	double	150.15	4200	1483.381	1483.381	7.381476	8Ø16
span	A-B	153.16	300	350	0.2084	264.9873	double	150.15	4200	1661.721	1661.721	8.268915	9Ø16
support	B	79.19	300	350	0.1077	312.7587	single	150.15	4200	727.9453	727.9453	3.622339	4Ø16
span	B-C	91.86	300	350	0.1250	305.8202	single	150.15	4200	863.5711	863.5711	4.297229	5Ø16
support	C	31.78	300	350	0.0432	336.0922	single	150.15	4200	271.8525	271.8525	1.352769	2Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	156.36	300	350	0.2127	262.333	double	150.15	4200	1713.605	1713.605	8.527093	9Ø16
span	A-B	185.98	300	350	0.2530	232.1878	double	150.15	4200	2302.845	2302.845	11.45922	12Ø16
support	B	75.87	300	350	0.1032	314.5198	single	150.15	4200	693.5214	693.5214	3.451042	4Ø16
span	B-C	82.84	300	350	0.1127	310.7962	single	150.15	4200	766.3059	766.3059	3.813226	4Ø16
support	C	24.87	300	350	0.0338	339.2164	single	150.15	4200	210.7836	210.7836	1.048883	2Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	123.3	300	350	0.1678	286.757	double	150.15	4200	1236.195	1236.195	6.151447	7Ø16
span	A-B	189.45	300	350	0.2578	227.5363	double	150.15	4200	2393.766	2393.766	11.91166	12Ø16
support	B	78.2	300	350	0.1064	313.2862	single	150.15	4200	717.6345	717.6345	3.571031	4Ø16
span	B-C	82.98	300	350	0.1129	310.7204	single	150.15	4200	767.7884	767.7884	3.820603	4Ø16
support	C	12.53	300	350	0.0170	344.6527	single	150.15	4200	104.5219	150.15	0.747164	2Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	25.99	350	350	0.0303	340.3735	single	175.175	4900	219.5272	219.5272	1.092393	2Ø16
span	A-B	20.2	350	350	0.0236	342.566	single	175.175	4900	169.5294	175.175	0.871691	2Ø16
support	B	61.12	350	350	0.0713	326.3919	single	175.175	4900	538.3712	538.3712	2.678997	3Ø16
span	B-C	36.6	350	350	0.0427	336.2786	single	175.175	4900	312.9102	312.9102	1.557077	3Ø16
support	C	9.214	350	350	0.0107	346.649	single	175.175	4900	76.41808	175.175	0.871691	2Ø16

Axis 6

Roof

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	77.24	250	300	0.1716	244.1655	double	107.25	3000	909.4856	909.4856	4.525705	5Ø16
span	A-B	87.35	250	300	0.1941	234.1589	double	107.25	3000	1072.482	1072.482	5.336794	6Ø16
support	B	83.1	250	300	0.1847	238.5034	double	107.25	3000	1001.715	1001.715	4.98465	5Ø16
span	B-C	70.53	250	300	0.1567	250.2569	single	107.25	3000	810.2622	810.2622	4.031958	4Ø16
support	C	52.37	250	300	0.1164	265.1377	single	107.25	3000	567.87	567.87	2.825786	3Ø16

5th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	56.04	300	350	0.0762	324.6077	single	150.15	4200	496.3377	496.3377	2.469833	3Ø16
span	A-B	84.62	300	350	0.1151	309.8288	single	150.15	4200	785.2159	785.2159	3.907324	4Ø16
support	B	98.07	300	350	0.1334	302.2813	single	150.15	4200	932.7445	932.7445	4.641444	5Ø16
span	B-C	149.31	300	350	0.2031	268.0805	double	150.15	4200	1601.259	1601.259	7.968046	8Ø16
support	C	60.86	300	350	0.0828	322.2193	single	150.15	4200	543.0231	543.0231	2.702145	3Ø16

4th floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	71.27	300	350	0.0970	316.9238	single	150.15	4200	646.5316	646.5316	3.217215	4Ø16
span	A-B	86.59	300	350	0.1178		single	150.15	4200	806.3037	806.3037	4.01226	5Ø16
support	B	98.86	300	350	0.1345	301.8241	single	150.15	4200	941.6827	941.6827	4.685921	5Ø16
span	B-C	149.23	300	350	0.2030	268.1437	double	150.15	4200	1600.023	1600.023	7.9619	8Ø16
support	C	54.72	300	350	0.0744	308.75	single	150.15	4200	509.5385	509.5385	2.535522	3Ø16

3rd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	102.93	300	350	0.1400	299.4416	single	150.15	4200	988.252	988.252	4.917655	5Ø16
span	A-B	164.4	300	350	0.2237	255.2776	double	150.15	4200	1851.513	1851.513	9.213343	10Ø16
support	B	97.59	300	350	0.1328	302.5584	single	150.15	4200	927.3293	927.3293	4.614497	5Ø16
span	B-C	139.1	300	350	0.1893	275.8251	double	150.15	4200	1449.877	1449.877	7.214754	8Ø16
support	C	47.2	300	350	0.0642	328.8917	single	150.15	4200	412.5978	412.5978	2.053134	3Ø16

2nd floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	90.3	300	350	0.1229	306.6942	single	150.15	4200	846.4864	846.4864	4.212213	5Ø16
span	A-B	164.88	300	350	0.2243	254.8367	double	150.15	4200	1860.132	1860.132	9.256232	10Ø16
support	B	97.79	300	350	0.1330	302.443	single	150.15	4200	929.5842	929.5842	4.625718	5Ø16
span	B-C	138	300	350	0.1878	276.6243	double	150.15	4200	1434.256	1434.256	7.137021	8Ø16
support	C	47.9	300	350	0.0652	328.5569	single	150.15	4200	419.1436	419.1436	2.085707	3Ø16

1st floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	50.13	300	350	0.0682	327.4851	single	150.15	4200	440.0925	440.0925	2.189951	3Ø16
span	A-B	87.92	300	350	0.1196	308.0167	single	150.15	4200	820.6374	820.6374	4.083586	5Ø16
support	B	114.69	300	350	0.1560	292.286	single	150.15	4200	1128.12	1128.12	5.613655	6Ø16
span	B-C	152.9	300	350	0.2080	265.1995	double	150.15	4200	1657.573	1657.573	8.248271	9Ø16
support	C	35.11	300	350	0.0478	334.5647	single	150.15	4200	301.7092	301.7092	1.501339	2Ø16

Ground floor

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	0	350	350	0.0000	350	single	175.175	4900	0	175.175	0.871691	2Ø16
span	A-B	25.63	350	350	0.0299	340.5107	single	175.175	4900	216.3992	216.3992	1.076827	2Ø16
support	B	64.02	350	350	0.0747	325.1796	single	175.175	4900	566.018	566.018	2.816571	3Ø16
span	B-C	23.05	350	350	0.0269	341.4904	single	175.175	4900	194.0574	194.0574	0.965652	2Ø16
support	C	0	350	350	0.0000	350	single	175.175	4900	0	175.175	0.871691	2Ø16

Appendix G :Beam stirrup

AXIS2						
A-B	336.64	288.9567	93.75		224.7	0.0
B-C	276.54	247.0862	93.75	19.9	224.7	19.9
C-D	69.59	62.765	93.75	78.3	224.7	78.3
AXIS3						
A-B	351.12	270.1233	93.75	18.2	224.7	18.2
B-C	230	323.39	93.75	15.2	224.7	15.2
AXIS4						
A-B	326.34	230.23	93.75	21.3	224.7	21.3
B-C	263.9	193.51	93.75	25.4	224.7	25.4
AXIS 5						
A-B	267.4	230.34	93.75	21.3	224.7	21.3
B-C	212.6	36.64	93.75	134.1	224.7	93.8
AXIS6						
A-B	118.02	105.602	93.75	46.5	224.7	46.5
B-C	46.37	39.53	93.75	124.3	224.7	93.8
AXisA						
1-2	118.14	102.12	93.75	48.1	224.7	48.1
2-3	93.48	79.91	93.75	61.5	224.7	61.5
3-4	107.97	91.80	93.75	53.5	224.7	53.5
4-5	101.94	87.47	93.75	56.2	224.7	56.2
5-6	55.37	47.82	93.75	102.8	224.7	93.8
AXisB						
1-2	245.55	209.77	93.75	23.4	224.7	23.4
2-3	163.31	138.73	93.75	35.4	224.7	35.4
3-4	191.64	169.09	93.75	29.1	224.7	29.1
4-5	166.68	146.30	93.75	33.6	224.7	33.6

5-6	110.34	93.73	93.75	52.4	224.7	52.4
AXisC						
1-2	171.35	149.78	93.75	32.8	224.7	32.8
2-3	90.28	77.29	93.75	63.6	224.7	63.6
3-4	121.35	105.59	93.75	46.5	224.7	46.5
4-5	84.57	72.24	93.75	68.0	224.7	68.0
5-6	94.82	86.21	93.75	57.0	224.7	57.0
AxisD						
1-2	72.57	61.88	93.75	79.4	224.7	79.4
AXIS1		5th				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro
A-B	201.55	176.89	93.75	27.8	224.7	27.8
B-C	214.16	191.48	93.75	25.7	224.7	25.7
C-D	102.136	66.74	93.75	73.6	224.7	73.6
AXIS2						
A-B	250	225.76	93.75		224.7	0.0
B-C	200.16	178.31	93.75	27.6	224.7	27.6
C-D	121.1	76.73	93.75	64.1	224.7	64.1
AXIS3						
A-B	153.78	136.34	93.75	36.0	224.7	36.0
B-C	172.79	152.83	93.75	32.2	224.7	32.2
AXIS4						
A-B	110.23	97.2	93.75	50.6	224.7	50.6
B-C	283.79	253.93	93.75	19.4	224.7	19.4
AXIS 5						
A-B	127.83	113.97	93.75	43.1	224.7	43.1
B-C	149.91	129.29	93.75	38.0	224.7	38.0
AXIS6						
A-B	142.2	131.22	93.75	37.5	224.7	37.5

B-C	225.2	199.55	93.75	24.6	224.7	24.6
AXisA						
1-2	167.02	143.02	93.75	34.4	224.7	34.4
2-3	141.9	109.91	93.75	44.7	224.7	44.7
3-4	140.26	112.00	112 .06	43.9	224.7	43.9
4-5	101.94	108.71	93.75	45.2	224.7	45.2
5-6	118.8	83.5	93.75	58.9	224.7	58.9
AXisB						
1-2	115.79	276.35	93.75	17.8	224.7	17.8
2-3	115.82	88.37	93.75	55.6	224.7	55.6
3-4	148.02	121.33	93.75	40.5	224.7	40.5
4-5	146.23	114.65	93.75	42.9	224.7	42.9
5-6	27.93	19.51	93.75	251.9	224.7	93.8
AXisC						
1-2	225.4	204.06	93.75	24.1	224.7	24.1
2-3	108.13	88.00	93.75	55.8	224.7	55.8
3-4	121.35	105.59	93.75	46.5	224.7	46.5
4-5	72.48	58.63	93.75	83.8	224.7	83.8
5-6	67.51	46.22	93.75	106.3	224.7	93.8
AxisD						
1-2	93.06	77.16	93.75	63.7	224.7	63.7
AXIS1		4th				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro
A-B	201.55	176.89	93.75	27.8	224.7	27.8
B-C	214.16	191.48	93.75	25.7	224.7	25.7
C-D	102.136	66.74	93.75	73.6	224.7	73.6
AXIS2						
A-B	250	225.76	93.75		224.7	0.0
B-C	200.16	178.31	93.75	27.6	224.7	27.6

C-D	121.1	76.73	93.75	64.1	224.7	64.1
AXIS3						
A-B	153.78	136.34	93.75	36.0	224.7	36.0
B-C	172.79	152.83	93.75	32.2	224.7	32.2
AXIS4						
A-B	110.23	97.2	93.75	50.6	224.7	50.6
B-C	283.79	253.93	93.75	19.4	224.7	19.4
AXIS 5						
A-B	127.83	113.97	93.75	43.1	224.7	43.1
B-C	149.91	129.29	93.75	38.0	224.7	38.0
AXIS6						
A-B	142.2	131.22	93.75	37.5	224.7	37.5
B-C	225.2	199.55	93.75	24.6	224.7	24.6
AXisA						
1-2	167.02	143.02	93.75	34.4	224.7	34.4
2-3	141.9	109.91	93.75	44.7	224.7	44.7
3-4	140.26	112.00	112 .06	43.9	224.7	43.9
4-5	101.94	108.71	93.75	45.2	224.7	45.2
5-6	118.8	83.5	93.75	58.9	224.7	58.9
AXisB						
1-2	115.79	276.35	93.75	17.8	224.7	17.8
2-3	115.82	88.37	93.75	55.6	224.7	55.6
3-4	148.02	121.33	93.75	40.5	224.7	40.5
4-5	146.23	114.65	93.75	42.9	224.7	42.9
5-6	27.93	19.51	93.75	251.9	224.7	93.8
AXisC						
1-2	225.4	204.06	93.75	24.1	224.7	24.1
2-3	108.13	88.00	93.75	55.8	224.7	55.8
3-4	121.35	105.59	93.75	46.5	224.7	46.5

4-5	72.48	58.63	93.75	83.8	224.7	83.8
5-6	67.51	46.22	93.75	106.3	224.7	93.8
AxisD						
1-2	93.06	77.16	93.75	63.7	224.7	63.7
AXIS1		3rd				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro
A-B	288.5	253.142	93.75	19.4	224.7	19.4
B-C	214.16	191.48	93.75	25.7	224.7	25.7
C-D	59.3	42.07	93.75	116.8	224.7	93.8
AXIS2						
A-B	48.6	43,43	93.75		224.7	0.0
B-C	127.03	110.6	93.75	44.4	224.7	44.4
C-D	26.26	19.23	93.75	255.6	224.7	93.8
AXIS3						
A-B	118.94	104.8	93.75	46.9	224.7	46.9
B-C	121.5	104.72	93.75	46.9	224.7	46.9
AXIS4						
A-B	112.5	98.15	93.75	50.1	224.7	50.1
B-C	82.7	72.65	93.75	67.7	224.7	67.7
AXIS 5						
A-B	150.8	135.29	93.75	36.3	224.7	36.3
B-C	84.72	75.09	93.75	65.5	224.7	65.5
AXIS6						
A-B	113.98	100.97	93.75	48.7	224.7	48.7
B-C	67.15	60.12	93.75	81.7	224.7	81.7
AXisA						
1-2	167.02	143.02	93.75	34.4	224.7	34.4
2-3	141.9	109.91	93.75	44.7	224.7	44.7
3-4	140.26	112.00	112 .06	43.9	224.7	43.9

4-5	101.94	108.71	93.75	45.2	224.7	45.2
5-6	118.8	83.5	93.75	58.9	224.7	58.9
AXisB						
1-2	127.03	276.35	93.75	17.8	224.7	17.8
2-3	122.7	93.30	93.75	52.7	224.7	52.7
3-4	118.9	100.17	93.75	49.1	224.7	49.1
4-5	64.72	63.08	93.75	77.9	224.7	77.9
5-6	34.54	26.15	93.75	187.9	224.7	93.8
AXisC						
1-2	209.39	180.93	93.75	27.2	224.7	27.2
2-3	121.15	93.44	93.75	52.6	224.7	52.6
3-4	114.4	95.50	93.75	51.5	224.7	51.5
4-5	82.7	67.24	93.75	73.1	224.7	73.1
5-6	744.34	53.39	93.75	92.1	224.7	92.1
AxisD						
1-2	71.06	60.82	93.75	80.8	224.7	80.8
AXIS1		2nd				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro
A-B	201.55	176.89	93.75	27.8	224.7	27.8
B-C	214.16	191.48	93.75	25.7	224.7	25.7
C-D	102.136	66.74	93.75	73.6	224.7	73.6
AXIS2						
A-B	250	225.76	93.75		224.7	0.0
B-C	200.16	178.31	93.75	27.6	224.7	27.6
C-D	121.1	76.73	93.75	64.1	224.7	64.1
AXIS3						
A-B	153.78	136.34	93.75	36.0	224.7	36.0
B-C	172.79	152.83	93.75	32.2	224.7	32.2
AXIS4						

A-B	110.23	97.2	93.75	50.6	224.7	50.6
B-C	283.79	253.93	93.75	19.4	224.7	19.4
AXIS 5						
A-B	127.83	113.97	93.75	43.1	224.7	43.1
B-C	149.91	129.29	93.75	38.0	224.7	38.0
AXIS6						
A-B	142.2	131.22	93.75	37.5	224.7	37.5
B-C	225.2	199.55	93.75	24.6	224.7	24.6
AXisA						
1-2	167.02	143.02	93.75	34.4	224.7	34.4
2-3	141.9	109.91	93.75	44.7	224.7	44.7
3-4	140.26	112.00	112 .06	43.9	224.7	43.9
4-5	101.94	108.71	93.75	45.2	224.7	45.2
5-6	118.8	83.5	93.75	58.9	224.7	58.9
AXisB						
1-2	115.79	276.35	93.75	17.8	224.7	17.8
2-3	115.82	88.37	93.75	55.6	224.7	55.6
3-4	148.02	121.33	93.75	40.5	224.7	40.5
4-5	146.23	114.65	93.75	42.9	224.7	42.9
5-6	27.93	19.51	93.75	251.9	224.7	93.8
AXisC						
1-2	225.4	204.06	93.75	24.1	224.7	24.1
2-3	108.13	88.00	93.75	55.8	224.7	55.8
3-4	121.35	105.59	93.75	46.5	224.7	46.5
4-5	72.48	58.63	93.75	83.8	224.7	83.8
5-6	67.51	46.22	93.75	106.3	224.7	93.8
AxisD						
1-2	93.06	77.16	93.75	63.7	224.7	63.7
AXIS1		1st				

span	Vmax	Ved	Sbmax	Scal	Smin	Spro
A-B	288.5	253.142	93.75	19.4	224.7	19.4
B-C	214.16	191.48	93.75	25.7	224.7	25.7
C-D	59.3	42.07	93.75	116.8	224.7	93.8
AXIS2						
A-B	48.6	43,43	93.75		224.7	0.0
B-C	127.03	110.6	93.75	44.4	224.7	44.4
C-D	26.26	19.23	93.75	255.6	224.7	93.8
AXIS3						
A-B	118.94	104.8	93.75	46.9	224.7	46.9
B-C	121.5	104.72	93.75	46.9	224.7	46.9
AXIS4						
A-B	112.5	98.15	93.75	50.1	224.7	50.1
B-C	82.7	72.65	93.75	67.7	224.7	67.7
AXIS 5						
A-B	150.8	135.29	93.75	36.3	224.7	36.3
B-C	84.72	75.09	93.75	65.5	224.7	65.5
AXIS6						
A-B	113.98	100.97	93.75	48.7	224.7	48.7
B-C	67.15	60.12	93.75	81.7	224.7	81.7
AXisA						
1-2	167.02	143.02	93.75	34.4	224.7	34.4
2-3	141.9	109.91	93.75	44.7	224.7	44.7
3-4	140.26	112.00	112 .06	43.9	224.7	43.9
4-5	101.94	108.71	93.75	45.2	224.7	45.2
5-6	118.8	83.5	93.75	58.9	224.7	58.9
AXisB						
1-2	127.03	276.35	93.75	17.8	224.7	17.8
2-3	122.7	93.30	93.75	52.7	224.7	52.7

3-4	118.9	100.17	93.75	49.1	224.7	49.1
4-5	64.72	63.08	93.75	77.9	224.7	77.9
5-6	34.54	26.15	93.75	187.9	224.7	93.8
AXisC						
1-2	209.39	180.93	93.75	27.2	224.7	27.2
2-3	121.15	93.44	93.75	52.6	224.7	52.6
3-4	114.4	95.50	93.75	51.5	224.7	51.5
4-5	82.7	67.24	93.75	73.1	224.7	73.1
5-6	744.34	53.39	93.75	92.1	224.7	92.1
AxisD						
1-2	71.06	60.82	93.75	80.8	224.7	80.8
AXIS1		Ground				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro
A-B	31.93	28.02	93.75	175.4	224.7	93.8
B-C	34.4	31.86	93.75	154.3	224.7	93.8
C-D	56.3	52.41	93.75	93.8	224.7	93.8
AXIS2						
A-B	31.68	27.77	93.75		224.7	0.0
B-C	38.05	34.2	93.75	143.7	224.7	93.8
C-D	85.4	69.07	93.75	71.2	224.7	71.2
AXIS3						
A-B	30.95	27.04	93.75	181.8	224.7	93.8
B-C	46.04	42.15	93.75	116.6	224.7	93.8
AXIS4						
A-B	33.16	29.58	93.75	166.2	224.7	93.8
B-C	51.12	47.19	93.75	104.1	224.7	93.8
AXIS 5						
A-B	37.5	33.6	93.75	146.3	224.7	93.8
B-C	51.53	46.12	93.75	106.6	224.7	93.8

AXIS6						
A-B	39.92	36.01	93.75	136.5	224.7	93.8
B-C	44	40.11	93.75	122.5	224.7	93.8
AXisA						
1-2	33.87	30.81	93.75	159.5	224.7	93.8
2-3	45.07	39.64	93.75	124.0	224.7	93.8
3-4	21.87	19.61	112 .06	250.6	224.7	250.6
4-5	20.87	19.38	93.75	253.6	224.7	93.8
5-6	74.36	61.51	93.75	79.9	224.7	79.9
AXisB						
1-2	25.78	23.02	93.75	213.5	224.7	93.8
2-3	47.5	44.22	93.75	111.1	224.7	93.8
3-4	35.7	32.20	93.75	152.6	224.7	93.8
4-5	43.57	38.60	93.75	127.3	224.7	93.8
5-6	76.34	63.14	93.75	77.8	224.7	77.8
AXisC						
1-2	11.56	26.92	93.75	182.6	224.7	93.8
2-3	44.36	39.02	93.75	126.0	224.7	93.8
3-4	39.16	34.45	93.75	142.7	224.7	93.8
4-5	22.52	19.95	93.75	246.4	224.7	93.8
5-6	44.47	36.78	93.75	133.6	224.7	93.8
AxisD						
1-2	39.95	36.34	93.75	135.2	224.7	93.8

Appendix H :Column design

location	column	bc	hc	bb	hb	Lc	Lbx left	lbx right	lby left	lby right	k1x	k2x	k1y	k2y	Lox	Loy
ROOf	C1	300	300	250	300	2850	0	5000	0	3600	2.11	2.11	1.52	1.52	2599.05	2523.80
ROOf	C2	300	300	250	300	2850	5000	4700	0	3600	1.02	1.02	1.52	1.52	2413.80	2523.80
5th floor	C3	300	300	300	350	2850	0	5000	0	3600	1.10	1.10	0.80	0.80	2437.57	2335.13
5th floor	C4	300	300	300	350	2850	5000	4700	0	3600	0.54	0.54	0.80	0.80	2199.19	2335.13
4th floor	C5	300	300	300	350	2850	0	5000	0	3600	1.10	1.10	0.80	0.80	2437.57	2335.13
4th floor	C6	300	300	300	350	2850	5000	4700	0	3600	0.54	0.54	0.80	0.80	2199.19	2335.13
3rd floor	C8	300	300	300	350	2850	0	5000	0	3600	1.10	1.10	0.80	0.80	2437.57	2335.13
3rd floor	C9	300	300	300	350	2850	5000	4700	0	3600	0.54	0.54	0.80	0.80	2199.19	2335.13
2nd floor	C10	300	300	300	350	2850	0	5000	0	3600	1.10	1.10	0.80	0.80	2437.57	2335.13
2nd floor	C11	300	300	300	350	2850	5000	4700	0	3600	0.54	0.54	0.80	0.80	2199.19	2335.13
1st floor	C12	300	300	300	350	2850	0	5000	0	3600	1.10	1.10	0.80	0.80	2437.57	2335.13
1st floor	C13	300	300	300	350	2850	5000	4700	0	3600	0.54	0.54	0.80	0.80	2199.19	2335.13
Ground floor	C14	400	400	350	350	2850	0	5000	0	3600	2.99	2.99	2.15	2.15	2663.75	2603.83
Ground floor	C15	400	400	350	350	2850	5000	4700	0	3600	1.45	1.45	2.15	2.15	2512.53	2603.83

Location	column	Mx top	Mx botm	My top	My bot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slendernes(x)	slendernes(y)
ROOF	C1	95.00	77.30	83.25	91.00	0.50	77.31	83.26	95.01	91.01	30.01	29.14	616.38	546.04	short	short
ROOF	C2	8.50	6.40	8.40	7.35	5.42	6.51	7.46	8.61	8.51	27.87	29.14	199.39	173.93	short	short
5th floor	C3	110.45	94.40	96.45	109.35	101.23	96.42	98.47	112.47	111.37	28.15	26.96	41.19	39.87	short	short
5thfloor	C4	15.87	4.50	15.48	4.40	621.91	16.94	16.84	28.31	27.92	25.39	26.96	21.72	21.63	slender	slender
4th floor	C5	97.34	92.30	97.13	92.20	213.19	96.56	96.46	101.60	101.39	28.15	26.96	25.25	25.21	slender	slender
3rd floor	C6	20.04	5.90	20.15	5.36	142.60	8.75	8.21	22.89	23.00	25.39	26.96	54.26	55.31	short	short
3rd floor	C8	93.20	94.63	91.23	104.84	102.10	95.24	93.27	96.67	106.88	28.15	26.96	34.79	40.26	short	short
2nd floor	C9	30.82	7.23	29.81	6.73	125.13	9.73	9.23	33.32	32.31	25.39	26.96	61.90	62.17	short	short
2nd floor	C10	97.51	91.20	97.30	91.30	149.40	94.19	94.29	100.50	100.29	28.15	26.96	30.69	30.57	short	short
1st floor	C11	34.34	8.50	34.41	7.41	101.30	10.53	9.44	36.37	36.44	25.39	26.96	68.92	70.41	short	short
1st floor	C12	99.00	25.10	99.20	25.11	185.80	28.82	28.83	102.72	102.92	28.15	26.96	51.21	51.23	short	short
1st floor	C13	46.78	9.50	45.74	9.40	137.70	12.25	12.15	49.53	48.49	25.39	26.96	60.88	60.74	short	short
Ground floor	C14	7.61	6.35	7.71	7.32	102.90	8.41	9.38	9.67	9.77	23.07	22.55	53.66	47.83	short	short
Ground floor	C15	20.06	2.24	19.05	22.40	145.50	5.15	21.96	22.97	25.31	21.76	22.55	80.22	45.25	short	short

column	location	MEDx	MEDy	Vsd	μsdy	μsdx	ω	As	As, min	As, max	As, prov	ø	no of bar	Longitudinal reinforcment	shear reinforcment	
ROOF	C1	95.01	91.01	0.0	0.30	0.31	0.3	1034.74	180	3600	1034.74	16	5.15	4ø16	ø 8 c/c	300
ROOF	C2	8.61	8.51	0.0	0.03	0.03	1.3	4483.88	180	3600	4483.88	16	22.31	4ø16	ø 8 c/c	300
5^{th} floor	C3	22.56	9.25	0.1	0.36	0.37	0.25	862.284	180	3600	862.284	16	4.29	4ø16	ø 8 c/c	300
5^{th} floor	C4	28.31	27.92	0.6	0.09	0.09	0.1	344.914	180	3600	344.914	16	1.72	4ø16	ø 8 c/c	300
4th floor	C5	16.8	25.35	0.2	0.33	0.33	0.3	1034.74	180	3600	1034.74	16	5.15	4ø16	ø 8 c/c	300
4^{th} floor	C6	22.89	23.00	0.1	0.08	0.07	0.15	517.371	180	3600	517.371	16	2.57	4ø16	ø 8 c/c	300
3^{rd} floor	C8	96.67	29.6	0.1	0.35	0.32	6.3	21729.6	180	3600	21729.6	16	108.13	4ø16	ø 8 c/c	300
3^{rd} floor	C9	33.32	32.31	0.1	0.11	0.11	7.3	25178.7	180	3600	25178.7	16	125.29	4ø16	ø 8 c/c	300
2^{nd} floor	C10	34.67	31.33	0.1	0.33	0.33	8.3	28627.8	180	3600	28627.8	16	142.46	4ø16	ø 8 c/c	300
2^{nd} floor	C11	36.37	36.44	0.1	0.12	0.12	9.3	32077	180	3600	32077	24	70.94	6ø16	ø 8 c/c	300
1^{st} floor	C12	54.74		0.2	0.34	0.34	10.3	35526.1	180	3600	35526.1	16	176.78	4ø16	ø 8 c/c	300
1^{st} floor	C13	49.53	48.49	0.1	0.16	0.16	11.3	38975.3	180	3600	38975.3	16	193.95	4ø16	ø 8 c/c	300
Ground floor	C14	9.67	9.77	0.1	0.01	0.01	12.3	75421.1	320	6400	75421.1	16	375.30	4ø16	ø 8 c/c	320
Ground floor	C15	22.97	25.31	0.1	0.03	0.03	13.3	81552.9	320	6400	81552.9	16	405.82	4ø16	ø 8 c/c	320