Civil Engineering Department

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Structural Design of G+4 Mixed Used Building in Adama

A B.Sc. thesis/project (CENG5281) submitted as a partial fulfilment of the requirements for the Degree of Bachelor Science in Civil Engineering

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Summary

This project deals about the structural analysis and design of a G+4 mixed use building considering all the external and internal effects according to (ES EN Design of Concrete Structures 2015), the project is located in Adama town. The plan area covered by 1027m^2 and total building height is17m In the first section concern about the background of the project, overall objective project, literature review, and methodology includes basic principles of analysis and design of structures, limitations, study area and the second concern about structural analysis and design project includes superstructure and substructure. Structural analysis and design of superstructure it contains analysis and design roof, slab, stair case, beam, column, and lateral load analysis. The analysis of the frame is done by using ETABS, and finally, using the joint reaction output, the foundation for the structure was designed based on soil characteristics of Adama town. From the analysis, it observed that the structure is design safely and economically under the exposed load.

Acknowledgement

Our greatest thanks from the depth of our heart are to God for helping us with the courage, strength as well as health through-out school time and for the Successful accomplishment of this project. This project comes to realization with collaboration of different significant suggestions and ideas from various sources.

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Table of Contents

Summary i

Acknowledgement ii

List of Figure v

List of tables vi

Abbreviation and symbols ix

1 Introduction 1

1.1 Background 3

1.2 Literature review 6

2 Methodology 8

2.1 Study Area 9

2.2 Structural analysis and design 10

2.2.1 Roof analysis and design 10

2.2.2 Flat slab design 27

2.2.3 Design of Reinforced Concrete Slab according to (ES EN 1992) 48

2.2.4 First floor slab design 52

2.2.5 Second floor slab design 73

2.2.6 Third and fourth floor slab design 93

2.2.7 Stair case design 115

2.2.8 Earthquake load analysis 134

2.2.9 Frame Analysis Procedure using ETABS 173

2.2.10 Frame analysis 174

2.2.11 Beam design 180

2.2.12 Design of column 201

2.2.13 Foundation design 242
Reference 255
Appendix 256
Appendix A Detail of roof analysis 256
Appendix A1; Terrain categories and terrain parameter 256
Appendix A2 External pressure coefficients for flat roofs 257
Appendix B; detail slab analysis 258
Appendix B1; Minimum cover 258
Appendix B2;Basic span/effective depth ratio 261
Appendix C Lateral Load Analysis 328
Appendix D; Frame Analysis 362
Appendeix E beam design 369
Appendix F; design of Column 406

List of Figure

Figure 1 roof floor slab layout for roof 27
Figure 2 calculated moment for roof floor 36
Figure 3 support moment adjustment for roof 41
Figure 4 roof floor slab reinforcement 46
Figure 5 frist floor slab layout 52
Figure 6 calculated moment for frist floor 63
Figure 7 support moment adjustement for frist floor 67
Figure 8 frist floor slab reinforcement 71
Figure 9 second floor slab layout 73
Figure 10 calculated moment for second flloor 83
Figure 11 fypical 3rd and 4th floor slab layout 93
Figure 12 calculated moment for third and fouth floor 105
Figure 13 support moment adjustment for third and fourth floor 109
Figure 14 thrid and fourth floors slab reinforcement 113
Figure 15 stair case layout 117
Figure 16 detail reinforisment for stair case design 133
Figure 17 column detail 239
Figure 18moment @ axis B-B 362
Figure 19Axial Force @ axis B-B 363
Figure 20 Shear force @ axis B-B 364
Figure 21Shear force @ axis 1-1 365
Figure 22 Moment @ axis 1-1 366
Figure 23 Axial force @ 2-2 367
Figure 24 Shear @ axis 2-2 368

List of tables

Table 1design dead &live load for flat roof 33
Table 2 sectional elevation for first floor 56
Table 3 design dead &live for first floor solid slab 57
Table 4 support moment adjustments 64
Table 6 sectional elevation of second floor slabs 77
Table 7 design dead & live for second floor 78
Table 8 sectional elevation for third &fourth floor..... 97
Table 9 design dead & live for third &fourth floor 97
Table 11 Weight according to story level 137
Table 12 Distribution of base shear over the building height 142
Table 13 Grouping columns based on having equal axial load..... 241
Table 14 dead load computation..... 263
Table 15 Live Load for different functions based on EBCS-1..... 267
Table 16 depth determination for roof 268
Table 17 Moment Calculation for two way slab for roof..... 270
Table 18 moment cantilever slabs for roof..... 271
Table 19 support moment adjustment between panels for roof..... 272
Table 20 support moment adjustment for roof 275
Table 21 support moment adjustment..... 276
Table 22 Span Moment Adjustment 277
Table 23 Reinforcement design for roof 278
Table 24 depth determination for second floor..... 280
Table 25 Dead load of HCB for first floor 283
Table 26 Dead load of plaster for first floor..... 284
Table 27 imposed load for first floor 285
Table 28 Moment for two way slab for first floor 286
Table 29 Moment Calculation for cantilever first floors..... 287
Table 30Support Moment adjustment between panels for first floor 288
Table 31 Span Moment Adjustment for first floor 292

Table 32 design reinforisment for first floor 293
Table 33 Check Depth for Deflection: Serviceability Requirement for frist floor 295
Table 35 Dead load of HCB for second floor 300
Table 36 Dead load of plaster for second floor 301
Table 37 Moment Calculation for two way second floor 302
Table 38 Support Moment adjustment between panels for second floor 302
Table 39 Span Moment Adjustment for second floor 307
Table 40 Reinforcement design for second floor 308
Table 41 Check Depth for Deflection: Serviceability Requirement for second floor 310
Table 43 Dead load of HCB for third &fourth floor 315
Table 44 Dead load of plaster for third & fourth floor 317
Table 45 imposed load third &fourth floor 318
Table 46Moment Calculation for two way for third &fourth floor 319
Table 48 Support Moment adjustment between panels for third &fourth floor 321
Table 49 Moment adjustment between panels 322
Table 50 Moment adjustment between panels 323
Table 51 Span Moment Adjustment for third &fourth floor 324
Table 52 Reinforcement Design for Third Floor and 4th floor 325
Table 55 thrid and forth floor determine the center of mass 347
Table 56 D- values on each axise 353
Table 57 Correction factors in x -and y- axise 358
Table 58reinforcement weather single or double reinforcement 369
Table 60 Check second order effect for C1 407
Table 61 Longitudinal Reinforcement for C1 408
Table 62 Lateral Reinforcement calculation for C1 409
Table 64 Check seconde order effect for C4 411
Table 65 Longtudnal Reinforcement calculation C4 412
Table 66 Lateral Reinforcement calculation C4 413
Table 68 Check seconde order effect (C20) 415
Table 69 Longtudnal Reinforcement calculation (C20) 416

Table 70 Lateral Reinforcement calculation (C20) 417
Table 72 Check seconde order effect(C13) 419
Table 73 Longitudinal Reinforcement calculation(C13) 420
Table 74 Lateral Reinforcement calculation(C13) 421
Table 76Check seconde order effect(C14) 423
Table 77 Longtudnal Reinforcement calculation(C14) 424
Table 78 Lateral Reinforcement calculation(C14) 425

Abbreviation and symbols

A_c: Gross Area of concrete

A_s: Area of tensile reinforcement

A_s: Area of compressive reinforcement bar

b, h: Dimension of rectangular section (width and depth respectively)

CALT: Altitude factor

C_d: Dynamic coefficient

CDIR: Direction factor

C_e: Exposure coefficient

C_p: Pressure coefficient

C_r: Roughness coefficient

C_T: Topographic coefficient

t CTEM: Temperature coefficient

d: Effective depth

D: Gross Depth

E: Modulus of elasticity

E_c: Modulus of elasticity of concrete

f_{cy} Maximum compressive strain of concrete

EBCS: Ethiopian Building code of standard

E_s: Modulus of elasticity of steel

f_{cd}: Design Compressive strength of concrete in compression

f_{ck}: Characteristic cylindrical compressive strength of concrete

f_{ctd}: Tensile strength of concrete

f_{yd}: Design yield strength of Reinforcement Bar in Tension and Compression

f_{yk}: Characteristic strength of Reinforcement Bar in Tension

HCB: Hollow Concrete Block

I: Second Moment of Inertia

K: Equivalent roughness

KT: Terrain factor

Pd: Design value

qref: Reference mean velocity pressure

r: Radius of gyration

RC: Reinforced Concrete

t: Plate thickness

Vm: Mean wind velocity

V_{ref}: Relative wind velocity factor

Z: Height above the ground

Z_{min}: Minimum height

Z_o: Roughness length

W_e: external pressure

C_{pe}: external pressure coefficient

C_{pi}: internal pressure coefficient

Qk: live load

G_k: dead load

F_b: base shear

X_m: center of mass in x axis

Y_m: center of mass in y axis

L_y: long dimension

L_x: short direction

M_{ys}: support moment in the longer direction of panel

M_{xs}: support moment in the shorter direction of panel

M_{xf}: field moment in the shorter direction of the panel

M_{yf}: field moment in the longer direction of panel

1 Introduction

The primary aim of all structural design is to ensure that the structure will performs satisfactorily during its design life. Specifically, the design must check that the structure is capable of carrying the loads safely and that it will not deform excessively due to the applied loads. This requires the designer to make realistic estimates of the strengths of the materials composing the structure and the loading to which it may be subjected during its design life. Furthermore, the designer will need a basic understanding of structural behavior. The designer must make an assessment of the future likely level of loading, including self-weight, to which the structure may be subjected during its design life. Using computer methods or hand calculations the design loads acting on individual elements can be then be evaluated. The design loads are used to calculate the bending moments, shear forces and deflections at critical points along the elements. Finally, suitable dimensions for the element can be determined.

Building structure is a structure which is composed of both structural and non-structural elements. The primary purpose of structural members is to build up and maintains its shape under a considerable load action to have adequate stability strength and stiffness. Not only safety but it also fulfills the requirement of serviceability as well as it should be economical. Regarding safety it requires that the strength of the structure should be adequate for all the loads which are reasonably considered.

When we come to the serviceability, it requires that all kinds of loads likely to occur during use, everything should be satisfactory, for example excessive deflection should be adequately small; vibration should be within the tolerable limit, the maximum width of crack also should not be greater than the specified limit etc. The structure also should be economical keeping its safety against to collapse.

Generally; the building structure to serve its purpose, it must be safe against collapse, economical to user and gives its serviceability in use (functional)

- Material selection & design data

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.

a) 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 γ_c= 1.5
- Characteristic strength, $f_{ck} = \frac{f_{cu}}{1.25} = \frac{25}{1.25} = 20MPA \; where \; f_{cu} = 25MPA$
- Design strength, $f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} = \frac{0.85*20}{1.5} = 11.33MPA$

b) Reinforcement

- Steel S-400
- Partial safety factor for concrete γ_s= 1.15
- Characteristic strength, $f_{yk} = 400MPA$
- Design strength, $f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{400}{1.15} = 347.8MPA$
- Partial safety factors for load
- 1.35 for dead load
- 1.5 for live load

Design method & reference

- Method (Approach) = Limit state design method
- References (Code) = ES EN (Ethiopian Standard based on Euro Norms)

1.1 Background

In this project we will design a G+4, mixed use building located in Adama Town. The plan area covered by 1027m^2 and total building height is17m also the soil in the area is subsoil class C. use reinforced concrete frame structural system. The Story height of the building is 3 m in our design we included the lateral load analysis (wind and Earth quake) and detailed structural design of roof, slab, beam, column, and a typical foundation design. And all the structural members are designed as per the specification of Ethiopian Building code of Standards, (ES EN Design of Concrete Structures 2015) And we used Limit state design method. The frame is done using SAP /ETABS. The document incorporates the deign steps and design output but the layout of the slab, the reinforcement detailing of slab, beam, column, stair& foundation and ETABS analysis output s are included in the soft copy.

Objective of the project

a) General objective

The primary objective why we shall prefer such project is help us to know the definite and clear procedures of reinforced concrete building structural design. That's why we have never been got the chances and such kinds of tasks before, which shall need the knowledge of a combination of our major courses that we have taken in our department. Another objective of this task will be the need to prepare ourselves for the challenges that will face us mainly related with reinforced concrete structural building design in the real working environment. So that, we will able to solve problems easily and in well manner. Developing a potential of investigating possible alternative of design and structural alternatives and employ the best, safe, construct able, and economic type of design for reinforced concrete building is also the main objectives of constructing this project. The last but not the least of the objectives of constructing this project will be enhancing our knowledge and understanding on using civil engineering soft wares like SAP2000, ETABS, and some other developed software in a proper and relevant way after completion.

b) Specification Objective of project

- Getting basic structural design knowledge
- Strengthen and accomplishment of different capacity of structural Analysis and design
- Relating theoretical knowledge to the actual site condition
- Development of new skills such as development of software and applying existing software
- Using the courses that we took for required purpose
- Integrating different disciplines
- Creating an awareness for others to develop such kind of idea for either B. Sc thesis

1.2 Literature review

a) Coefficient method of slab analysis

Various design tables which has been in use over approximately the past decades are listed below. Many of these tables are still in use. They are all based on the limit of ultimate strength.

Taylor, S.R., Hayes, B. and Mohamedbhai, G.T.G states that the coefficients represented are derived from the yield line theory and apply to the full width of the of slab. It is recommended that the loading use should be the design load which is factored load. It is also suggest that although in theory the full width of the slab should be used. In practice only a middle strip of the slab might be reinforced in accordance with the moment produced from these coefficients and similarly for the length of the slab and edge strip is reinforced using minimum area of steel.

Thakkar, M.C and Rao, J.K.S; justify that the average moment distribution per unit width of the slab is derived from uniform orthotropic reinforcement throughout the whole width of the slab. That is, the slab analysis is by Hillerborg's strip method and then the average of moments for all strips along each edge is taken and this value is the moment per meter width quoted by tables.

b) Strip method of slab analysis

The yield line method of slab analysis was an upper bound approach to determine the flexural strength of slabs. An upper bound analysis, if in error, will be so on the unsafe side. The actual carrying capacity will be less than, or at best equal to, the capacity predicted, which is certainly a cause for concern in design. Also, when applying the yield line method, it is necessary to assume that the distribution of reinforcement is known over the whole slab; it is used for design only in an iterative sense, for calculating the capacities of trial design with varying reinforcement until a satisfactory arrangement is found. These motivate Hillerborg to develop the strip method for slab design, his first results being published in Swedish in 1956. In contrast to yield line analysis, it is a lower bound approach. If a distribution of moment can be found that satisfies both equilibrium and boundary conditions for a given external loading, and if the yield moment capacity of the slab is nowhere exceeded, then the given external loading will represent a lower bound of the carrying capacity.

The strip method gives results on the safe side, and the needed reinforcement can be calculated, leading the designer to economical arrangement of steel, as well as safe design. Generally, it is simple to use, even for slabs with whole or irregular boundaries.

Hillerborg set basic principles for edge supported slabs and introduced the expression “strip method”, in his first original work in1956. He later expanded the method to include the practical design of slabs on columns and L-shaped or in the simplest possible way to produce safe designs for most of the slabs that he will meet in practice, including slabs that are irregular in plan or that carry uneven distributed loads. The strip method encourages the use of strong bands of steel where needed, such as around openings or over columns, improving economy and reducing the likelihood of cracking or large deflections under service load. See from (Design of G+4 building using cofficient and strip method 2008)

2 Methodology

Basic principles of analysis and design of structures

The basic principles of analysis and design of structures will be employed in this project**.** Use Application of the theoretical approach of different courses we learnt as required. And During our task we will use different software including ETABS 16 & AutoCAD 2007, Spread sheet and others to facilitate our task. And also employing local building code standard (ES EN Design of Concrete Structures 2015). Using developed templates for different associated tasks.

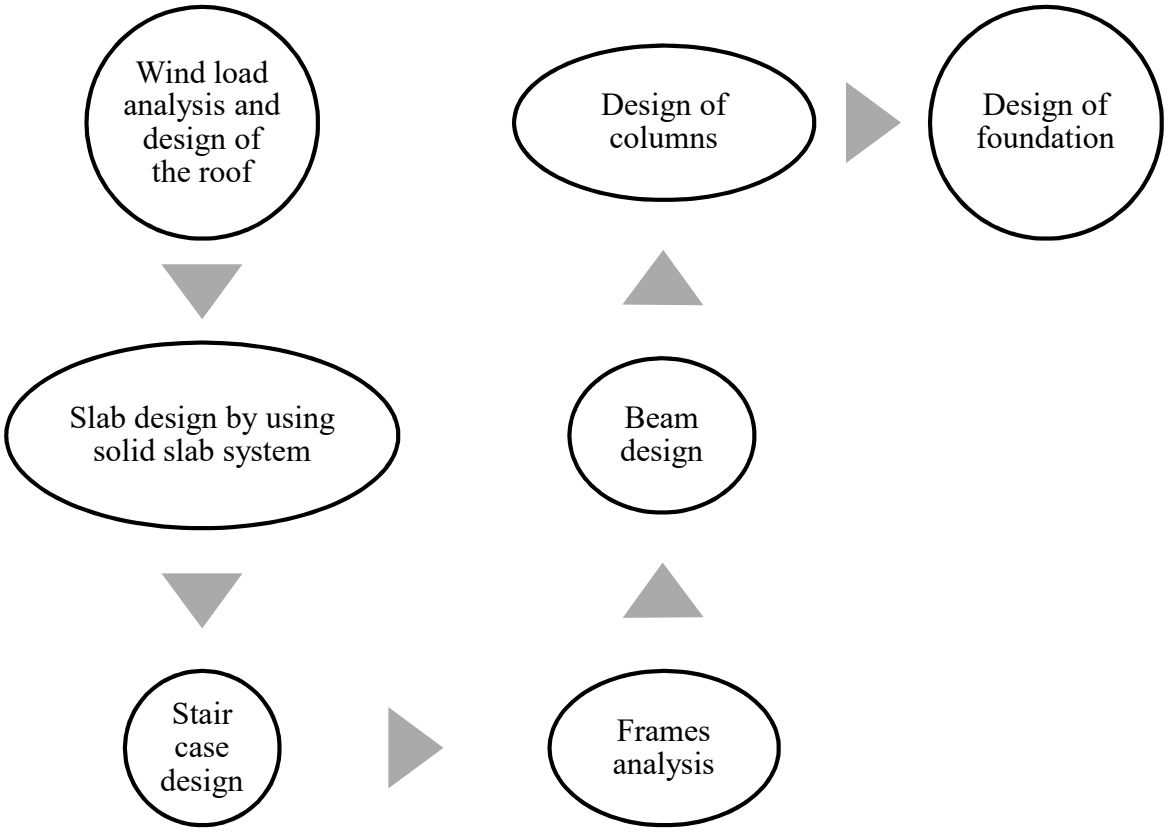


Figure 1: Framework of the study

2.1 Study Area

This project deals about the structural analysis and design of a G+4 mixed use building considering all the external and internal effects according to (ES EN Design of Concrete Structures 2015), The project is located in Adama

2.2 Structural analysis and design

The scope of our project to carry out a brief structural analysis and design of the building skeletal frame in concise manner. Also brief, simplified, and easy recognized structural elements detail which is ease and suitable to implement. As it is explained under introduction part this thesis has two main stages.

A. First stage

Structural analysis and design of superstructure

Which involves the following tasks

2.2.1 Roof analysis and design

The building is located in Adama city where there are same buildings around. Therefore according to (ES EN 1992) it is terrain category IV (urban areas in which at least 15% of the surface is covered with building and their average height exceeds 15m).

Roof design which involves**:**

Wind load analysis for the roof have two methods for wind load analysis

a) Quasi static method and
b) . Dynamic analysis we prefer Quasi static since our structure is assumed to be less susceptible to dynamic excitation and from (ES EN 1992) a building which satisfies the criterion:

(For c_d <1.2and building height less than 200m) can be analyzed using Quasi static

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 and the roughness of the surrounding

Action of the wind loads on structures is represented either as a wind pressure or 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. External wind pressure (Wex) is the wind pressure acting on

the external surfaces of a structure and internal wind pressure, is the wind pressure acting on the internal surfaces of a structure.

$w_{ex} = q_{ref} * C_e(Z_e) * C_{pe}$ Formula

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

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

Where: $qref$ = Reference mean wind pressure

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

cp e = External wind pressure coefficient

cp i = Internal wind pressure coefficient see from (ES- EN Actions on Structures 1991-1-4 2015)

First let us determine necessary information

Maximum height of building (height of roof) above ground level =17m

The roof of our building is flat roof

Location of the area –ADAMA

Procedures

a) Step:- Determination of basic wind velocity (vb)

$$Vb = Cdir * Csea * Vbo$$

Cdir=direction factor

Csea=seasonal factor

Vbo=fundamental basic wind velocity

Cdir=Cseason=1 see from (ES- EN Actions on Structures 1991-1-4 2015)

Vbo=22m/s for Ethiopia

b) Step: Determination of mean wind velocity, vm (z)

$$Vm(z) = Cr(z) * Co(z) * vb$$

Where; Co (Z) =orographic factor

Orographic factor is calculated by the following formula under different condition

$$Co(z) = 1 \ for \ \varphi < 0.05$$

$$Co(z) = 1 + 2 * S * \varphi \ for \ 0.05 < \varphi < 0.3$$

$$Co(z) = 1 + 0.6 * S * \varphi \ for \ \varphi > 0.3$$

Where, S – is the Orographic location factor

Φ- Is the upwind slope, Φ=H/LU in the wind direction

LU- is the actual length of the upwind slope

H-is the effective height

I. Wind pressure Coefficients

External pressure coefficients give the effect of the wind on the external surfaces of buildings.

Internal pressure coefficients give the effect of the wind on the internal surfaces of buildings.

a) External Pressure coefficients

The external pressure coefficients are divided into local coefficients and overall coefficients.

Local coefficients, Cpe1 give the pressure coefficients for loaded areas of 1m2.They may be used for the design of small elements and fixings.

Overall coefficients, Cpe 10 give the pressure coefficients for loaded areas of 10 m2.They may be used for loaded areas larger than 10 m2

The external pressure coefficients Cpe for buildings and parts of buildings depend on the size of the loaded area A, which is the area of the structure that produces the wind action in the section to be calculated.

Note: The loaded area is the area of the structure, which produces the wind action in the section to be calculated. For buildings with the loaded area between lm2and 10m2.the following procedure is recommended for determining the external pressure coefficient. See From (ES- EN Actions on Structures 1991-1-4 2015)Variation of external pressure coefficient for buildings with size of the loaded area A. The figure is based on the following:

A ≤ 1 m2, Cpe = Cpe1

1 $m2 < A <$ 10$m2$, $Cpe = Cpe1 + (Cpe10 - Cpe1) \log 10A$

$A \geq 10m2$, = $Cpe10$

The values of $Cpel$ and $Cpel0$ are given on Tables see from(ES- EN Actions on Structures 1991-1-4 2015) . Refer from appendix A

b) Internal Wind Pressure

$Wi = qref * Ce\ (Ze) * Cpi$

The internal wind pressure is the pressure found inside the house due to opening of windows and door.

Internal pressure coefficient (Cpi)

For closed buildings with internal partitions and opening windows the external values

Cpi=0.8 or Cpi=-0.5

Location of the area Adama

- Design information

The building is located in adama city. Therefore, according to (ES- EN Actions on Structures 1991-1-4 2015) TABLE 4.1 the area is categorized according to the following:

Terrain Category: According to (ES- EN Actions on Structures 1991-1-4 2015) TABLE 4.1 the location of the building in adama fall under Terrain Category- Iv

Category IV is characterized with

- roughness length, z0 = 1m
- minimum height, zmin = 10m (refer from appendix A)

Topography coefficient: no escarpment or hills are located around the building site which means the site is topography unaffected zone and therefore,

Ct (z) = 1 according to terrain condition

Height of the building is 17m

Width of the roof 27.41m and length of building is 37.44m

Roughness coefficient: 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:

$Cr\ (z) = krLn\ (z/zo)$, for $zmin \leq Z \leq 200m$

$Cr\ (z) = Cr(zmin)$, $for\ z < zmin$

Where z0 is the roughness length and zmin is the minimum height. Both z0 and zmin are dependent on the ground roughness and are given in Table 4.1.

In our case the building height is between z min and 200m .hence we can use the first formula.

$Cr\ (z) = ktLn\ (z/zo)$, but

$Kr=0.19*(Zo/Z0,II)^{0.07}$

- Calculations of mean wind velocity (Vm(zo))

Vm(z) = Cr(z)*Co(z)*Vb

Z= 17m Terrain Category- Iv

Zmin= 10m

Zo= 1m

zo,III=0.3m

Roughness coefficient Cr(z)

Cr(z) = Krln(z/zo) for Zmin < Z < Zmax

Kr = 0.19*(zo/zo,III)^0.07= 0.19*(1/0.3)^0.07= 0.21

Cr(z)= 0.21ln(17/0.3) =0.83

Orographic factor

(Co(z))Co(z)= 1 , Ø<0.05

basic wind velocity Vb=22m/recommended value

Vm $(z) = (z) * (z) * vb$

Vm(z)= 0.83*1*22=18.26m/s

Altitude

Temperature and Barometer

But ES EN1991-1-4-2015 recommended as 1.25kg/m3

- calculation of peak velocity pressure (qp (z))

qp(z) = [1 + 7Iv(z)] ∗1/2∗ ρair ∗ $vm^2(z)$

ρair=density of air, it is depend on :- Altitude Temperature and Barometer But ES EN1991-1-4-2015 recommended as 1.25kg/m^3

Wind terbulance (lv(zo)

lv= k1/[Co(z)ln(z/zo)] for Zmin < Z < Zmax=200m

Where, K1 is turbulences factor, recommended value is 1.0

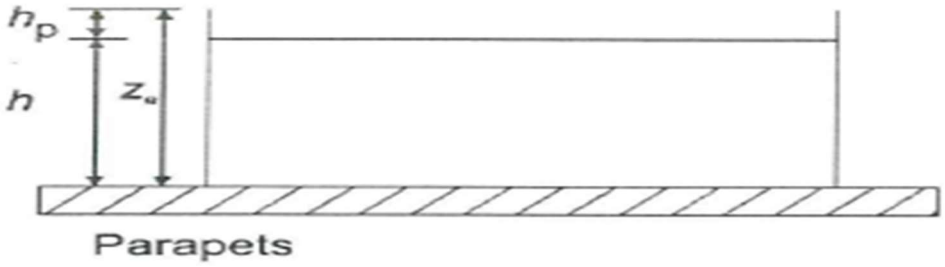
Lv = 1/[1*ln(17/1)]= 0.35

q(17) = [1+7*0.35]*1/2*1.25*18.26^2= 0.72kpa

- Wind load analysis for Flat Roof

Determination of external pressure coefficients

External wind pressure coefficient cpe are derived from Appendix A. of ES-EN-2015



Case 1

b=37.44m

ES EN 1991-1-4:2015

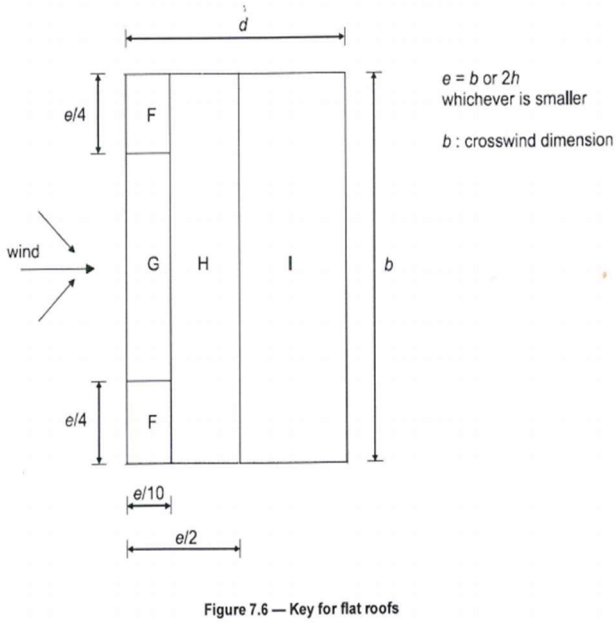


Figure 7.6 — Key for flat roofs

d=27.41m

e=37.44

e/4=9.36

e/2=18.72

e/10=3.74

hp=1

h=16

hp/H=1/16=0.0625

Determination of External and Internal Pressure Coefficient

Region	F		G		H		I	
hp/h	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10
0.05	-1.4	-2.0	-0.9	-1.6	-0.7	-1.2	-0.2	-0.2
0.10	-1.2	-1.8	-0.8	-1.4	-0.7	-1.2	-0.2	-0.2
0.0625	**-1.33**	**-1.95**	**-0.987**	**-1.55**	**-0.7**	**-1.2**	**-0.2**	**-0.2**

WIND PRESSURE COMPUTATION

Determination of External and Internal Pressure Coefficient

Areas calculation

Zones	F	G	H	I
Areas	=e/4*e/10 =9.36*3.74 =35m^2	=(b-2*e/4)*e/10 =(18.72)*(3.74) =70m^2	=(e/2-e/10)*b =14.98*37.44 =560.85 m^2	=(d-e/2)*b =8.69*37.44 =325.36m^2
Ext. Press. Coeff.				
Int. Press. Coeff.				

Zones	Areas	Ext. Press. Coeff	Int. Press. Coeff
F	35	-1.95	0.8
			-0.5
G	70	-1.55	0.8
			-0.5
H	560.85	-1.2	0.8
			-0.5
I	325.36	-0.2	0.8
		0.2	-0.5

Computations of External Wind Pressure

We = Qref * Cpe

Zone	peak velocity Pressure	Ext Press. Coeff	Ext. Wind Press
F	0.72	-1.95	-1.404
	0.72	0.00	0.00
G	0.72	-1.55	-1.12
	0.72	0.00	0.00
H	0.72	-1.2	-0.864
	0.72	0.00	0.00
I	0.72	-0.2	-0.144
	0.72	0.2	0.144
		Critical Pressures are	-1.404
			0.144

Computations of Net Wind Pressure

The net wind pressure is given by the sum of the internal and external pressure.

$$Wnet = We + Wi$$

From the computation of external pressure above, the critical loads are

Zone	External Press.
F	-1.404
I	0.144

peak velocity Pressure Inter. Press. Coeff. Inter. Wind Press.(KN/m2) q(z) Cpi

peak velocity Pressure q(z)	Inter. Press. Coeff. q(z) Cpi	Inter. Wind Press.(KN/m2
0.72	0.8	0.576
0.72	-0.5	-0.36

Zone	External Press. We (KN/m2)	Internal Press. Wi (KN/m2)	Net Press. $W_{net} = W_e - W_i$ (KN/m2)
F	-1.404	0.576	-1.98
I	0.144	-0.36	0.504

Case -2

ES EN 1991-1-4:2015

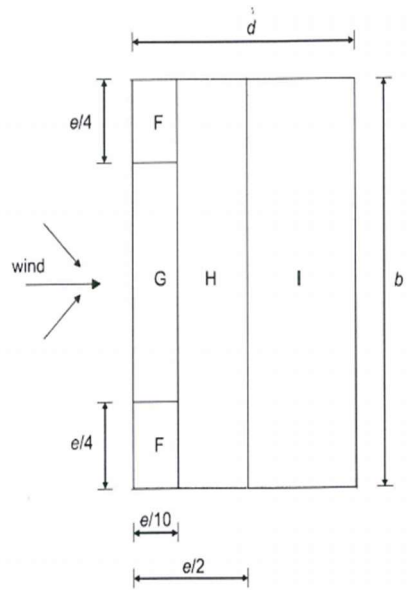


Figure 7.6 — Key for flat roofs

b=27.41m

d=37.44m

e=27.41

e/4=6.85

e/2=13.71

e/10=2.74

hp=1

h=16

hp/H=1/16=0.0625

Determination of External and Internal Pressure Coefficient

Region	F		G		H		I	
hp/h	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10
0.05	-1.4	-2.0	-0.9	-1.6	-0.7	-1.2	-0.2	-0.2
0.10	-1.2	-1.8	-0.8	-1.4	-0.7	-1.2	-0.2	-0.2
0.0625	**-1.33**	**-1.95**	**-0.987**	**-1.55**	**-0.7**	**-1.2**	**-0.2**	**-0.2**

WIND PRESSURE COMPUTATION

Determination of External and Internal Pressure Coefficient

Areas calculation

Zones	F	G	H	I
Areas	=e/4*e/10 =6.85*2.74 =18.77m^2	=(b-2*e/4)*e/10 =(13.71)*(2.74) =37.57 m^2	=(e/2-e/10)*b =10.97*27.41 =300m^2	=(d-e/2)*b =23.73*27.41 =650.44m^2
Ext. Press. Coeff.				
Int. Press. Coeff.				

Zones	Areas	Ext. Press. Coeff	Int. Press. Coeff
F	18.77	-1.95	0.8
			-0.5
G	37.57	-1.55	0.8
			-0.5
H	300	-1.2	0.8
			-0.5
I	650.44	-0.2	0.8
		0.2	-0.5

Computations of External Wind Pressure

We = Qref * Cpe

Zone	peak velocity Pressure	Ext Press. Coeff	Ext. Wind Press
F	0.72	-1.95	-1.404
	0.72	0.00	0.00
G	0.72	-1.55	-1.12
	0.72	0.00	0.00
H	0.72	-1.2	-0.864
	0.72	0.00	0.00
I	0.72	-0.2	-0.144
	0.72	0.2	0.144

Critical Pressures are -1.404

0.144

Computations of Net Wind Pressure

The net wind pressure is given by the sum of the internal and external pressure.

Wnet = We + Wi

From the computation of external pressure above, the critical loads are

Zone	External Press.
F	-1.404
I	0.144

peak velocity Pressure Inter. Press. Coeff. Inter. Wind Press.(KN/m2) q(z) Cpi

peak velocity Pressure q(z)	Inter. Press. Coeff. q(z) Cpi	Inter. Wind Press.(KN/m2
0.72	0.8	0.576
0.72	-0.5	-0.36

Zone	External Press. We (KN/m2)	Internal Press. Wi (KN/m2)	Net Press. Wnet = We - Wi (KN/m2)
F	-1.404	0.576	-1.98
I	0.144	-0.36	0.504
F	-1.404	0.576	-1.98
I	0.144	-0.36	0.504

Hence, the critical net wind loads on the roof are

Critical Press.are (KN/m2) -1.98

0.504

THEREFORE THE CRITICAL PRESSURE IS

Wnet min = 0.504

Wnet max = -1.98

TOTAL DESIGN LOAD DEAD LOAD [GK]

FOR ROOF SLAB

RC Slab =0.2*25=5KN/m2

Cement screed[30mm]=0.03*23=0.69KN/m2

marble[20mm]=0.02*27=0.54KN/m2

Plastering and Painting /mortar[20mm]=0.02*23=0.46KN/m^2

Total dead load Gk= 6.49 KN/m^2

LIVE LOAD [Qk]

Live load = Q_k= 0.4 KN/m_2

LOAD COMBINATION

Combination -1 Pd =1.35DL+1.5LL = (1.35*6.49) + (1.5*0.4) =9.36KN/m2

Combination-2 Pd=1.35DL+1.5WL = (1.35*6.49) + (1.5*1.98)=10.94KN/m2

Combination-3 Pd=1.35DL+1.5WL = (1.35*6.49) - (1.5*1.98)=6.58KN/m2

Combination – 4 Pd = 1.35DL+1.5LL+0.9WL =(1.35*6.49)+(1.5*0.4)+(0.9*1.98)= 10.67KN/m2

Combination – 5 pd =1.35DL+1.5LL - 0.9WL =(1.35*6.49)+(1.5*0.4)-(0.9*1.98)=8.055KN/m2

Therefore, the governing combination is combination-2

2.2.2 Flat slab design

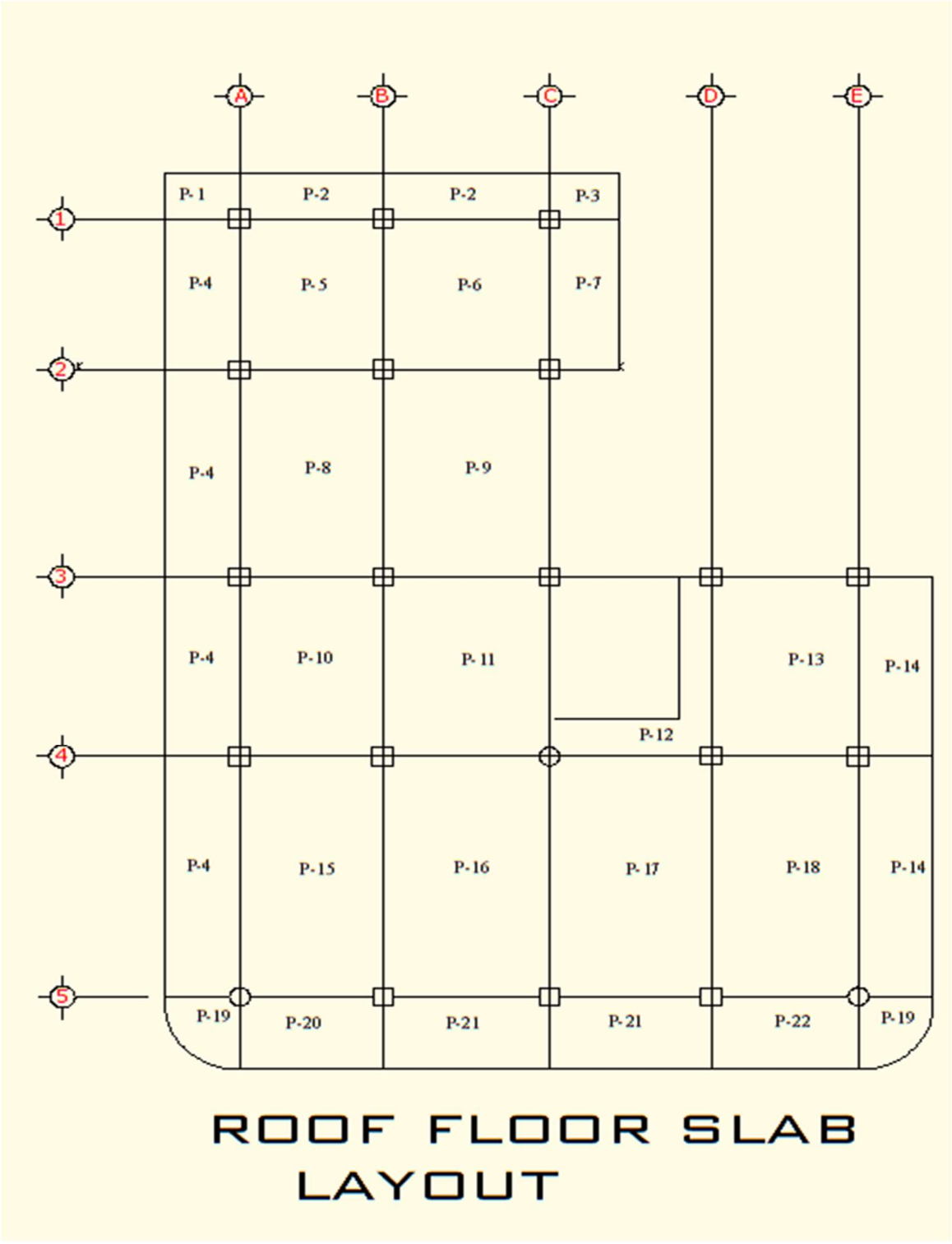


Figure 1 roof floor slab layout for roof

- Concrete cover design (ES EN Art 4.4.1.2 2015)

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.

a) 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 Φ12 longitudinal bar and Φ25 nominal maximum aggregate size;
Therefore; Cmin, b=12mm.
If nominal maximum aggregate size is greater than 32mm Cmin, b should be increased by 5mm

b) 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) tables 4.4N.
Using structural class, S3 and XC1
First, the concrete cover

With:

$\mathbf{c_{min,b}}$ = 12 mm

$\mathbf{c_{min,dur}}$ = 10 mm

^{C}min = max (cmin,b; cmin,dur ; 10 mm) =max (12; 10; 10 mm) = 12 mm

$\mathbf{\Delta c_{dev}}$ = 10 mm.

ΔCdev (allowance in Design for Variation)

Note:The value of Δc_{dev} for use in a country may be found in its National Annex .The recommended values is 10 mmWe obtain from relation

$\mathbf{c_{nom} = c_{min} + \Delta c_{dev}}$ = 12 + 10 = 22 mm .

c) 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 (ES EN 1992)

Cfire = 20mm

Hs = 120mm

Governing Cover for Design

Governing cover accounting for

- Corrosion
- Bond/Durability
- Fire

Cover = 22mm

Concrete C 20/25

Rebar S400

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

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

d=l/N*K*F1*F2*F3

WHERE

Derivation of these Expressions.

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o \ \ldots\ldots\ldots \quad (7.16a)(\text{ES EN } 1992;2015 \text{ art .7.42 } 1992)$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \text{if } \rho > \rho_o \ \ldots\ldots\ldots \quad (7.16b)(\text{ES EN } 1992;2015 \text{ art .7.42 } 1992)$$

- $N = 11 + 1.5\sqrt{f_{ck}} = 17.71$

- $\rho_o = \frac{\sqrt{f_{ck}}}{\frac{1000}{\sqrt{20}}} = 0.447$

a) Depth Determination

d=l/N*K*F1*F2*F3 $F1 = \frac{500}{\text{fyk}} = \frac{500}{400} = 1.25$

F2=1, F3=1

Sample example panel 7

L_y/L_x = 6640/1500 = 4.42>2.0 (one Way Cantilever)

d=l/N*K*F1*F2*F3= l/17.71*0.4*1.25*1*1=169.4mm is the govering depth

$D_{overall}$=d+c+ϕ/2=169.4+22 +6=197.4mm take D=200mm.

d=D-Cover -ϕ/2=200-22-12/2=174mm table 1 Computation of Depth for deflection (refer from in a tabular form appendix B)

b) Load Calculation

Design Dead Load and Live Loads for each Panel

FOR ROOF SLAB

RC Slab =0.2*25=5KN/m2

Cement screed[30mm]=0.03*23=0.69KN/m2

marble[20mm]=0.02*27=0.54KN/m2

Plastering and Painting /mortar[20mm]=0.02*23=0.46KN/m^2

Total dead load Gk= 6.49 KN/m^2

LIVE LOAD [Qk]

Live load = Qk= 0.4 KN/m2

PD'=1.35D.L+1. 5L.L…….(2015)

1.35*6.49+1.5*0.4=9.3615 KN/m^2

Table 1design dead &live load for flat roof

Panel	TYPE FF	Se.wt+FF+Mo+Cei	QK	DesignLoad
p1	Marble	6.49	0.4	9.3615
p2	Marble	6.49	0.4	9.3615
p3	Marble	6.49	0.4	9.3615
p4	Marble	6.49	0.4	9.3615
p5	Marble	6.49	0.4	9.3615
p6	Marble	6.49	0.4	9.3615
p7	Marble	6.49	0.4	9.3615
p8	Marble	6.49	0.4	9.3615
p9	Marble	6.49	0.4	9.3615
p10	Marble	6.49	0.4	9.3615
p11	Marble	6.49	0.4	9.3615
p12	Marble	6.49	0.4	9.3615
p13	Marble	6.49	0.4	9.3615
p14	Marble	6.49	0.4	9.3615
p15	Marble	6.49	0.4	9.3615
p16	Marble	6.49	0.4	9.3615
p17	Marble	6.49	0.4	9.3615
p18	Marble	6.49	0.4	9.3615
p19	Marble	6.49	0.4	9.3615
p20	Marble	6.49	0.4	9.3615
p21	Marble	6.49	0.4	9.3615
p22	Marble	6.49	0.4	9.3615

c) 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

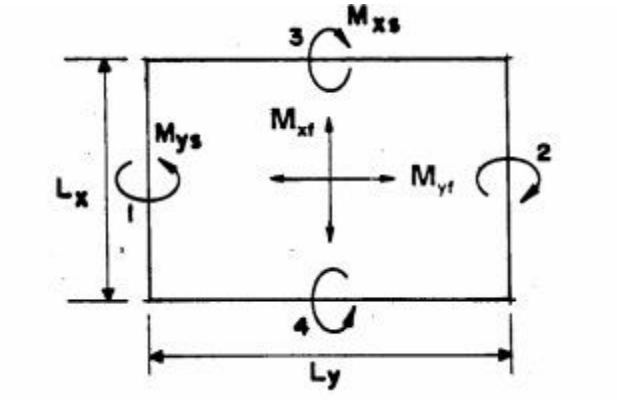
$$M_i=\beta_i n L_x^2 \quad \text{.....(2015)}$$

Where M_i=Design moment per unite width of reference

P_d=or n Uniformly Distributed Design Load

β_i=Coefficient given in table A-1

=f(Ly/Lx), Support condition.



$M_{XS}=\beta_{XS} n L_x^2$

$M_{XF}=\beta_{XF} n L_x^2$

$M_{YS}=\beta_{YS} n L_x^2$

$M_{YF}=\beta_{YF} P_d L_x^2$

Sample example panel 9

Design action, $n_{d\ or\ pd}$= 1.35*6.49+1.5*0.4=9.3615 KN/m^2

L_y/L_x = 9.17/6.52= 1.4 < 2.0 (Two way slab)

Four edges continuous

M_{xs}= $\beta_{sx}nlx^2$ = 0.039x 9.3615 x42.5 = 15.52kNm/m

M_{ys}= $\beta_{sy}nlx^2$ = 0.037x 9.3615 x42.5 = 14.72kNm/m

M_{yf}= $\beta_{syf}nlx^2$ = 0.024 x 9.3615 x 42.5 = 11.027kNm/m

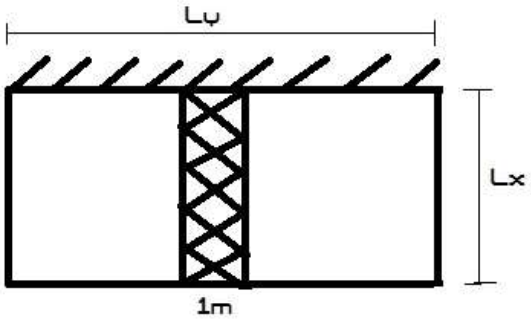
M_{xf} = $\beta_{sxf}nlx^2$ = 0.03x 9.3615 x 42.5=11.936 kNm/m Table 5 Moment Calculation for two way slab using coefficient method In a tabular form(refer from in a tabular form appendix B)

d) Moment Calculation for cantilever slabs.

Material Data

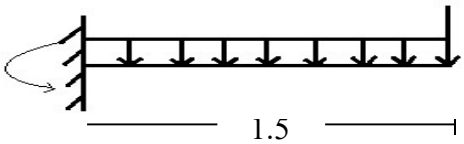
Sample example panel 7

- Taking 1m strip of the slab in the shorter direction



Pd= 9.3615 KN/m

9.3615 KN/m



Mxs

Mxs= 9.3615KN *$(\frac{1.5m}{2})^2$ =5.266KN.m Table 6 Moment Calculation for cantilever slabs. (Refer from in a tabular form appendix B)

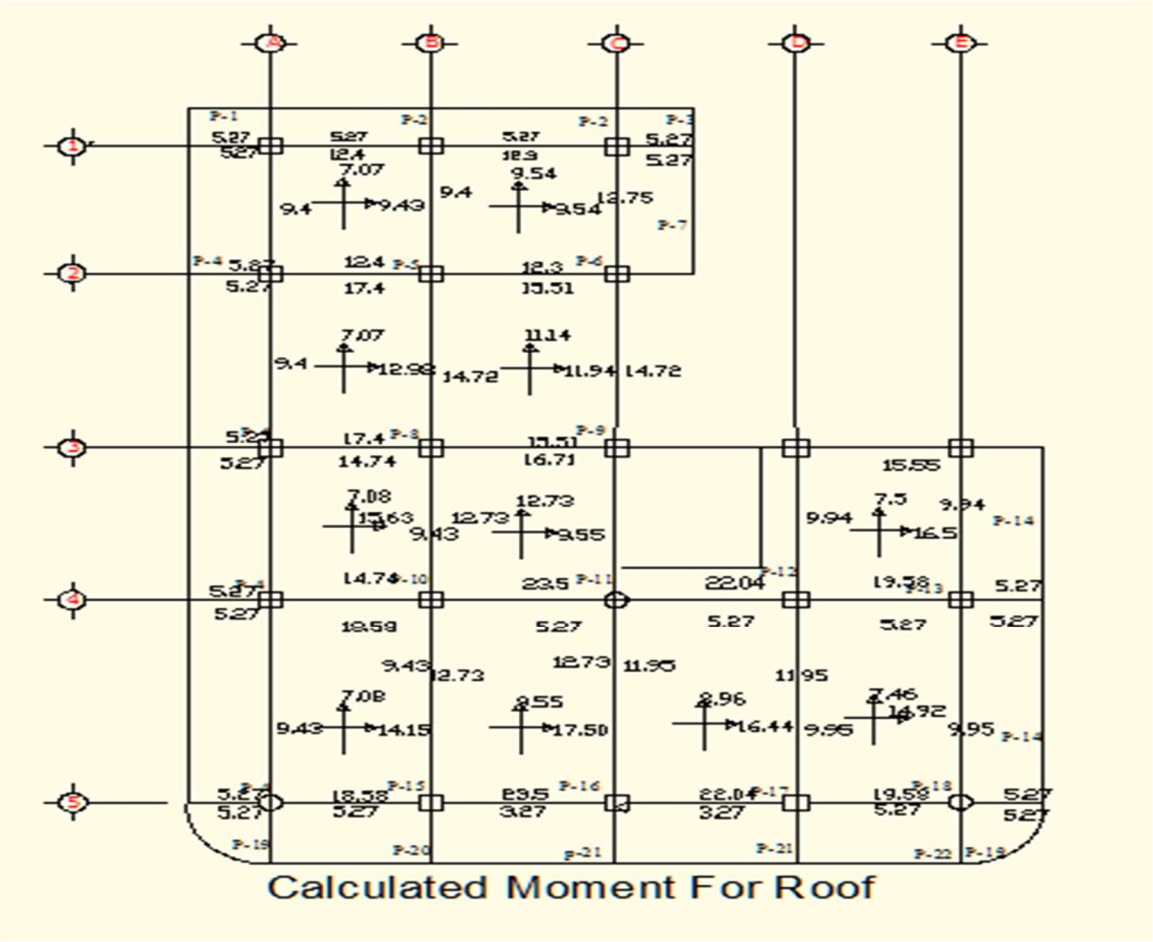


Figure 2 calculated moment for roof floor

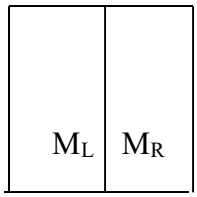
e) Moment adjustment between panels

Support moment adjustment

There are two cases.

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

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….$M_R>M_L$

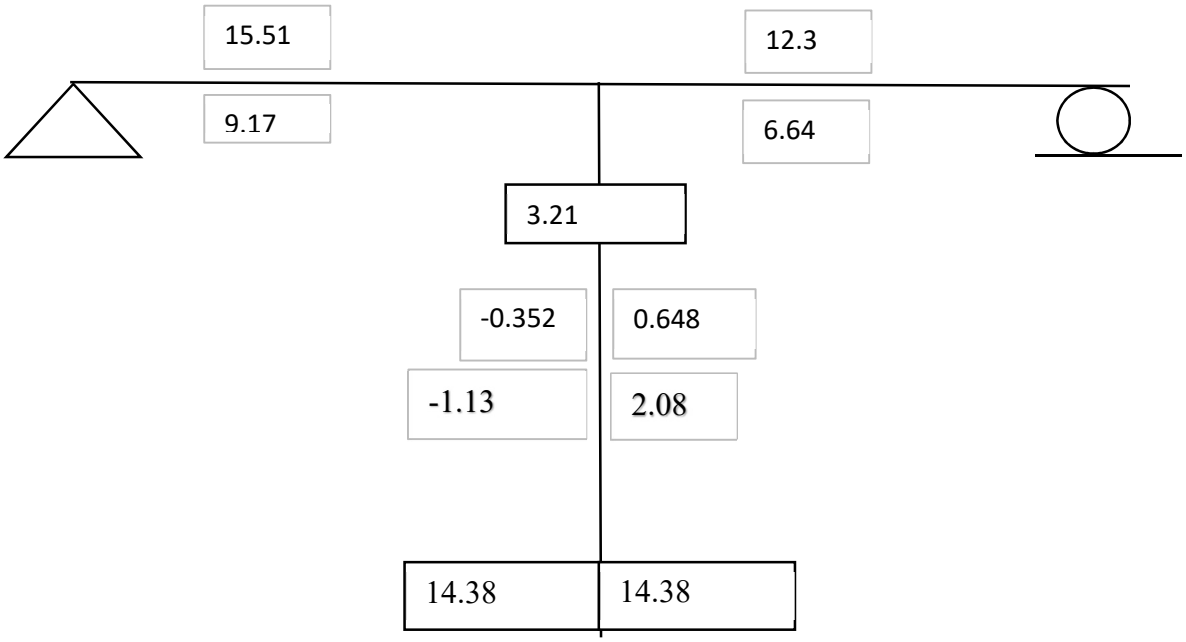


$M_R>M$

$M_d=M_R-\frac{K_R}{K_R+K_L}*\Delta M$……Considering right(ES EN 1992)

$M_d=M_R+\frac{K_R}{K_R+K_L}*\Delta M$……Considering Left(ES EN 1992)

Sample example panel Between T5&T6



M1= 15.51KNm

M2 =12.3KNm

ΔMS= M1- M2= 15.51KNm -12.3KNm = 3.21KNm

20%of larger moment=0.2*15.51KN.m=3.102KN.m

Implies that, 20%of larger moment=3.102KN.m< ΔMS=3.21KN.m

Therefore, we have to use moment distribution method to adjust support moment.

For continuous slab, coefficients could be as the following;

$K_{AB}=\frac{3I}{4L}=\frac{3I}{4*9.17}=0.082I$ $KBC=\frac{I}{L}=\frac{I}{6.64}=0.1506I$

$Df_{AB}=\frac{KAB}{KBC+KAB}=\frac{0.082I}{0.082I+0.1506I}=-0.352$ $DfBC=\frac{0.01506I}{0.082I+0.01506I}=0.648$

ΔMdf1 = 3.21* -0.352= -1.13KNm

ΔMdf2 = 3.21 * 0.648= 2.08KNm

Madj = 14.38KNm Table 7 Support Moment adjustment between panels (refer from in a tabular form appendix B)

f) Span Moment Adjustment

If the support moment is decreased the span moments M_{xf} and M_{yf} are then increased allow for the changes of support moment.

$M_{xdf} = M_{xf} + C_x \Delta M'$

$M_{ydf} = M_{yf} + C_y \Delta M'$

Where, $\Delta M' = M_R - M_d$ (on all the supports)

C_x, C_y = Coefficients for adjusting span moments given in table.

If the support moment is increased, no adjustment shall be made to the span moment

Thus span moment adjustments is required for **P8, P_{13}, P_{15}, P16, P_{17}, P_{18},**
Consider panel (P_{15}) as sample example. It requires adjustment both in shorter and longer span.

$$\frac{Ly}{Lx} = \frac{10.65}{5.61} = 1.9$$

Cx1	Cy1	Cx2	Cy2
0.481	0.251	0.258	0.06

Mxf(adj) = ΔMxf+Mxf, ΔMxf= Cx1*ΔMxs+ Cx2*ΔMys

Myf(adj)= ΔMyf+Myf, ΔMyf= Cy1*ΔMxs+ Cy2*ΔMys

ΔMxs=Mxs (unadj)-Mxs (adj) =18.58-17.07=1.51

ΔMys= Mys(unadj)-Mys(adj)=9.43-11.4=--2.01KN.m

ΔMyf=0.251*1.51+0.06*-2.01=0.2584KN.m

ΔMxf=0.0481*1.51+0.258*-2.01=0.2077KN.m

Mxf(adj)= 14.15+0.2077=14.3577KN.m

Myf(adj)= 7.08 +0.258=7.338KN.m Table 10 Span Moment Adjustment (refer from in a tabular form appendix B)

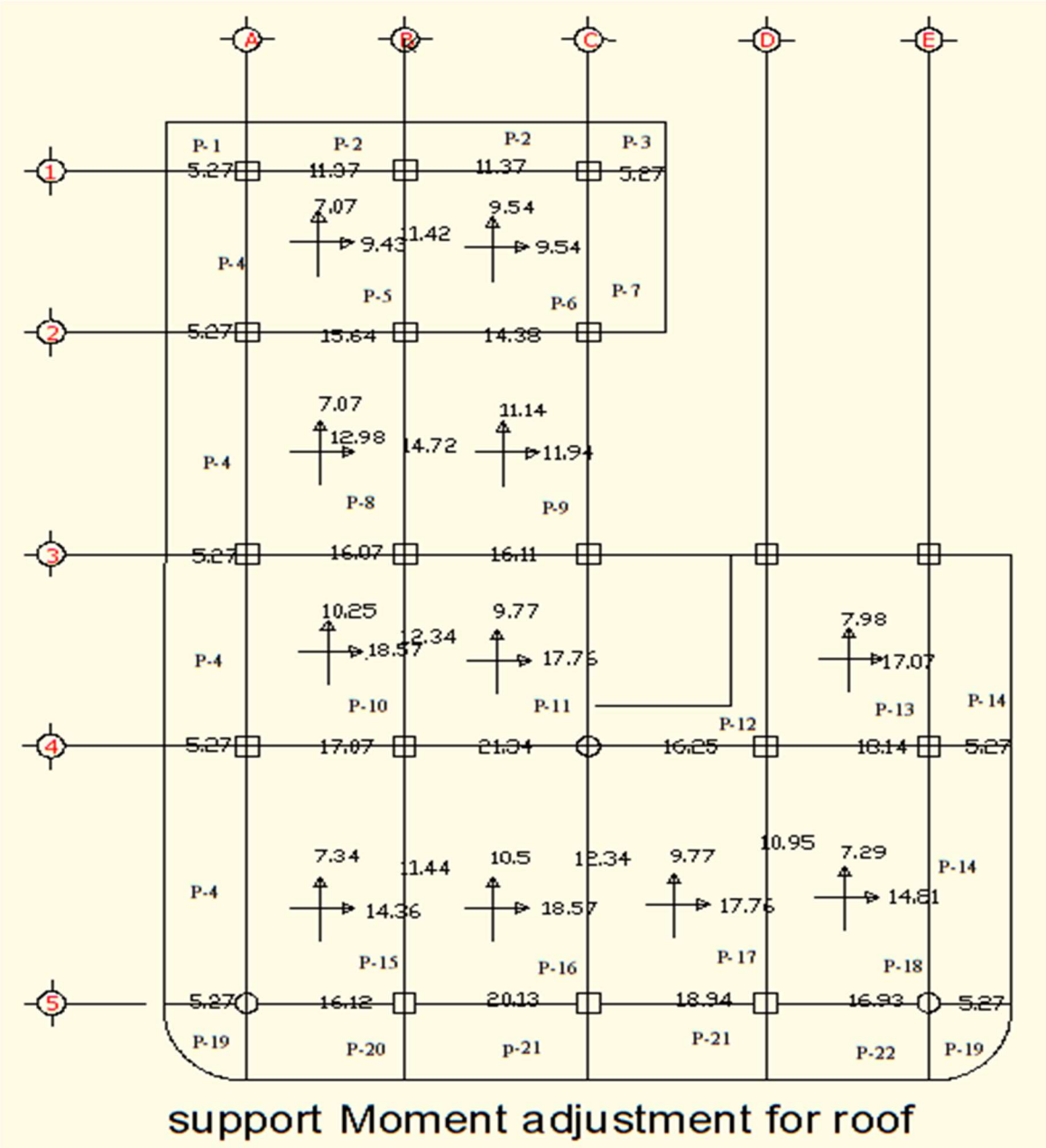


Figure 3 support moment adjustment for roof

g) DESIGN OF REINFORCEMENT (refer from in a tabular form appendix B)

Sample example panel 6

panel	Moment	D	B	K	Z	ASmin	Asrequred	ASmax	As	Scalculated	Sprovided	Remark
6mxs	14.38	174	1000	0.019	0.994	226.2	238.96	8000	113.04	473.0535931	400	F12c/c400
mxf	9.54	174	1000	0.013	0.981	226.2	160.62	8000	113.04	499.734748	400	F12c/c400
mys	11.42	174	1000	0.015	0.985	226.2	191.47	8000	113.04	499.734748	400	F12c/c400
myf	9.54	174	1000	0.013	0.981	226.2	160.62	8000	113.04	499.734748	400	F12c/c400

$fctm = 0.3(Fck^{2/3})$

$$A_{s,min}=Max\begin{cases}\frac{0.26fctmbtd}{fyk}\\0.00013btd\end{cases}$$ .....(2015) $Art.9.2.1.1$ (1)

Ast, max=0.04*Ac……...(2015) Art.9.2.1.1 (3)

Effective depth:

d= 200 – 20 -0.5(**F**) = 174mm

Minimum and maximum reinforcement area

fctm = $0.3(fck)^{(2/3)}$ = 0.3(25)^(2/3) = 2.6

$A_{s,min} = 0.26\ (f_{ctm}/f_{yk})bd = 0.26(2.6/400)*1000*174 = 294mm^2/m$

$= 0.0013bd = 0.0013(1000))(174) = 226.2mm2/m$

$A_{s,max} = 0.04A_c = 0.04(1000)(200) = 8000\ mm^2/m$

Short span:

Support:

M_{xs}= 14.38kNm/m

$K = \mu_{sd} = M/f_{ck}bd^2 = (14.38x10^6) / (25 \times 1000 \times 174^2) = 0.018999 < K_{bal} = \mu_{sd,lim} = 0.205$ by using general design chart

IF $\mu_{sd} \leq \mu_{sd,lim}$,Then Compression reinforcement not required

Kz or z read from the table

Kz=z=0.994d

A_s = M/0.87f_{yk}z = 14.38x 10^6/0.87 x 400 x 0.994 x 174= 266.38mm^2

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/266.38=473.05

Smax= $\begin{Bmatrix} 3D \\ 400 \end{Bmatrix}$ take Smax=3*200=600mm take Smax=400mmtake Spacing =400mm

Provide, Ø12 C/C 400mm

Mid Span

M_{xf}= 9.54 kNm/m

K = μ_{sd}= M/$f_{ck}bd^2$ = (9.54x10^6) / (25 x 1000 x 174^2) = 0.0126 < K_{bal}= $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=0.981d

A_s = M/0.87f_{yk}z = (9.54 x 10^6/0.87 x 400 x 0.981x 174mm = 160.616 mm^2/m <A_{Smin}=226.2mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/226.2=499.73mm

Smax= $\begin{Bmatrix} 3D \\ 400 \end{Bmatrix}$ take Smax=3*200=600mm take Smax=400mmtake Spacing =400mm

Provide, Ø12 C/C 400mm

Long span:

Support:

M_{ys}= 11.42kNm/m

K = μ_{sd}= $M/f_{ck}bd^2$ = $(11.42x10^6) / (25 \times 1000 \times 174^2)$ = 0.01509 < K_{bal}= $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=0.9850d

$A_s = M/0.87f_{yk}z$ = 11.42 x 10^6/0.87 x 400 x 0.9850 x 174 = 191.4699 mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/226.2=499.73 mm

Smax= $\begin{Bmatrix} 3D \\ 400 \end{Bmatrix}$ take Smax=3*200=600mm take Smax=400mmtake Spacing =400mm

Provide, Ø12 C/C 400mm

Long span:

Mid Span

M_{yf}= 9.54 kNm/m

K = μ_{sd}= $M/f_{ck}bd^2$ = $(9.54x10^6) / (25 \times 1000 \times 174^2)$ = 0.0126 < $K_{bal=}$ $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd<}$ $\mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=0.981d

A_s = $M/0.87f_{yk}z$ = $(9.54 \times 10^6/0.87 \times 400 \times 0.981x 174mm$ = 160.616 mm^2/m <A_{Smin}=226.2mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi*12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/226.2=499.73mm

Smax= $\begin{Bmatrix} 3D \\ 400 \end{Bmatrix}$ take Smax=3*200=600mm take Smax=400mmtake Spacing =400mm

Provide, Ø12 C/C 400mm

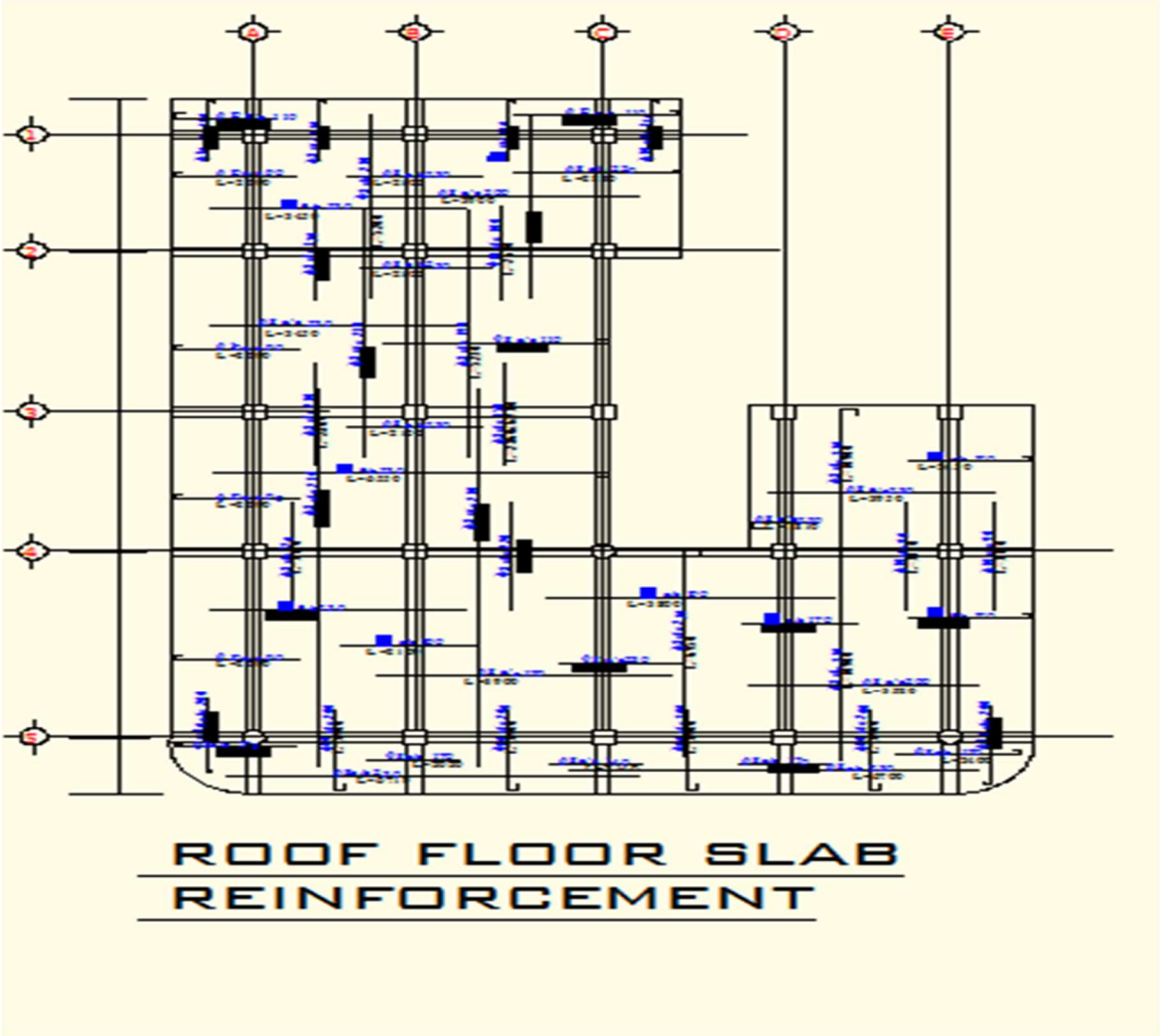


Figure 4 roof floor slab reinforcement

Check Depth for Deflection: Serviceability Requirement

To check the depth for deflection weathers it satisfies the serviceability requirement we use the new codes ES EN 1991:2015.

As an example panel 6

Initially we assumed $\rho_o = \rho$.but we can calculate them as follows

$\rho = \frac{A_{st,req}}{bd} = \frac{239}{1000*174} = 0.014\%$ ……..ES E*N*1992: 2017. 7.4.2(7.16*a*)

$\rho o = \frac{\sqrt{fck}}{1000} = \sqrt{20}/1000 = 0.4472\%$

Now, $\rho = 0.014\% < \rho o = 0.44721\%$

$l/d = N * K$*F1*F2*F3

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o \text{ ……..} \quad (7.16a)(ES\ EN\ 1992;2015\ art\ .7.42\ 1992)$$

$$N = \left[11 + 1.5\sqrt{20}\frac{0.4472}{0.014} + 3.2\sqrt{20}\left(\frac{0.4472}{0.014} - 1\right)^{3/2}\right] = 49.33$$

Where

K……. from appendix F

F1=500 /($f_{yk}A_{s,req}$ / $A_{s,prov}$)……[1]…. (7.17)

F2&F3=1, (because span of slab ≤ 7m)……. … section 5.3.2.2 (1).

Then we work similarly other slabs by the excel appendix

2.2.3 Design of Reinforced Concrete Slab according to (ES EN 1992)

Based on their load transfer mechanism solid slabs are classified as one-way slabs they are those either supported on the two out of four opposite sides or the longer span to short span ratio is greater than two. The second slab is two-way slabs they are those supported on all four sides and the longer span to short span ratio is less than or equal to two. The third slab Cantilever slabs they are those with a fixed support on only one out of four sides there design system of the slab system by using Coefficient method, Yield line method and Strip method

The slabs are designed by following the procedures stated bellow.

a) Step:- Depth determination:

The minimum depth required for the slab can be calculated from the minimum depth required for deflection.

Concrete cover

$$c_{min} = \max \{c_{min,b}; c_{min,dur}; 10\ mm\}$$

$$c_{nom} = c_{min} + \Delta c_{dev}$$

Where:

$c_{min,b}$ minimum cover due to bond requirement,

$c_{min,dur}$ minimum cover due to environmental conditions,

are also given. (Refer from appendix B)

$$D_{overall}=d+c+\phi/2$$

b) Step:- analysis of the design load.

Design load (Pd) = $1.35DL + 1.5LL$..... (2015)

dead load(DL) analysis design load

The dead load is composed of the self weight of the slab itself , weights of the partition walls,weight of the finishing and other considerable permanent loads.

Self weight of the slab is equal to the over all depth times unit weight of concrete.

Live load (LL) analysis

c) Step:- analysis of the design moment:

Analysis of the design moment will be done as for two-way solid slabs and for one way solid slabs the calculation will be performed as 1m wide beam.

d) Step-: Moment adjustment:

The support and the span moments will be adjusted to avoid over reinforcements and to achieve economical design.

e) Step:-reinforcement details:

After calculation of the design moment reinforcement detailing follows the appropriate rebar spacing.

Note: For the purpose of construction simplicity and monolithic construction the governing overall depth has been taken.

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b * Ab}{As}$

Smax= $\begin{Bmatrix} 3D \\ 400 \end{Bmatrix}$ take

Smax=3*200=600mm take Smax=400mm

Smax $at\ edge = \begin{Bmatrix} 3.5D \\ 450 \end{Bmatrix}$

Smax at edge=3.5*200=700mm take Smax=450mm

Provide, Ø12 C/C 400mm

The slabs are designed by following the procedures stated bellow.

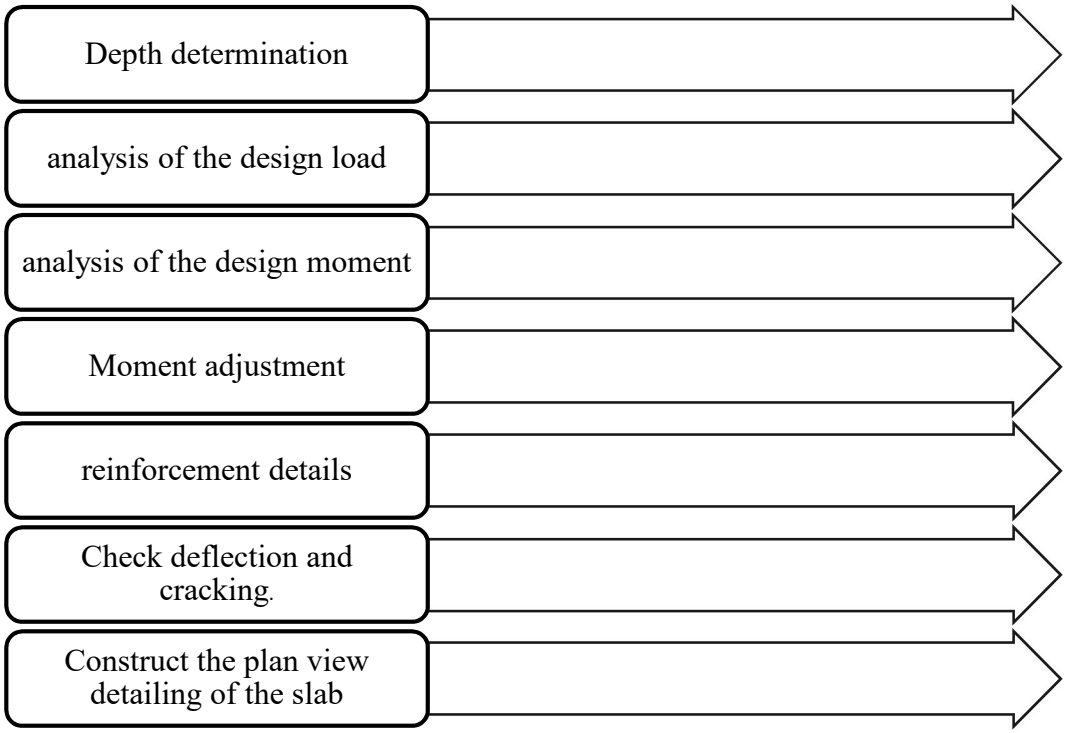


Fig 3; designed procedures for slabs.

2.2.4 First floor slab design

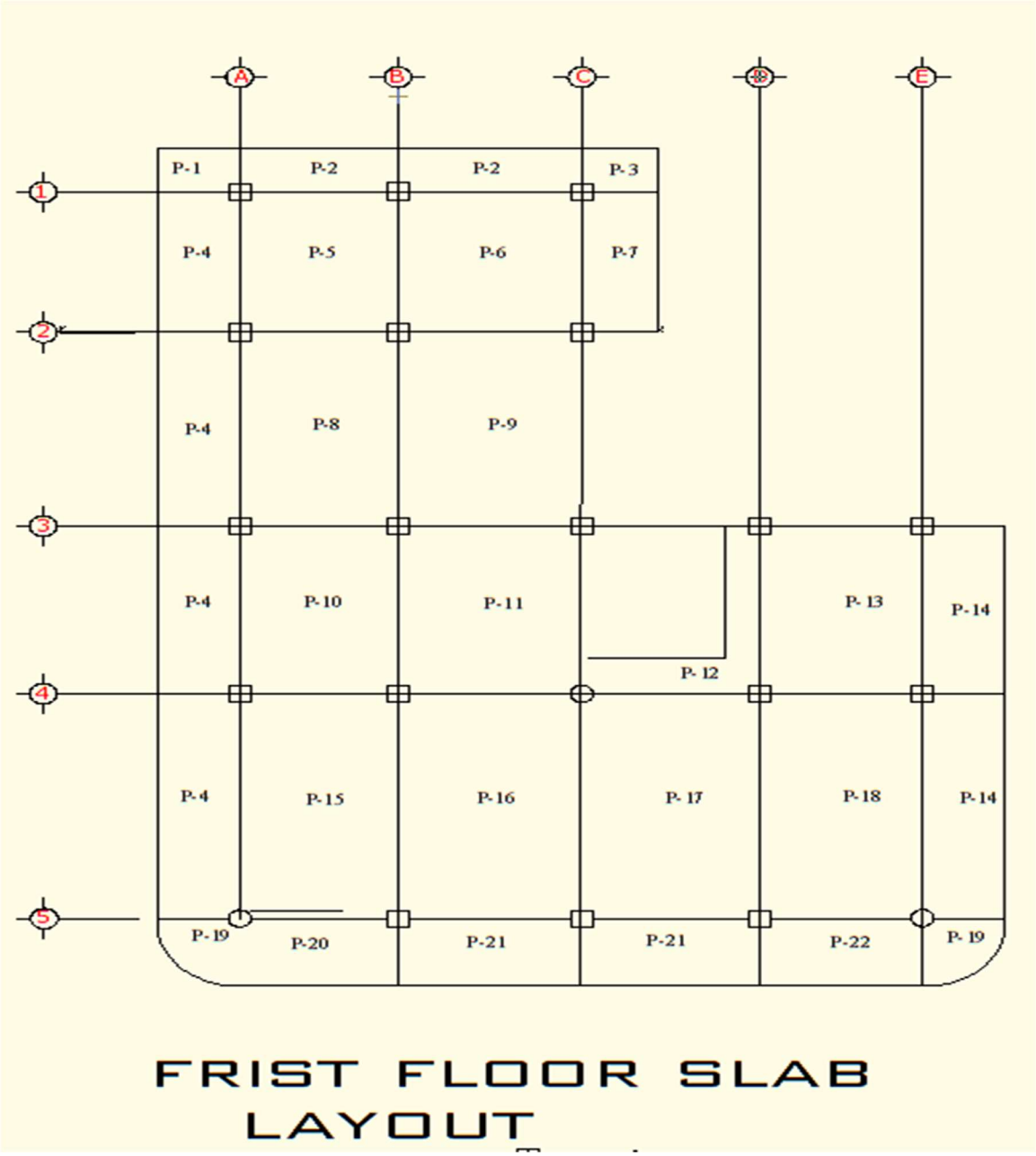


Figure 5 frist floor slab layout

A. Solid Slab Design

a) Concrete cover design (ES EN Art 4.4.1.2 2015)

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.

b) 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 Φ12 longitudinal bar and Φ25 nominal maximum aggregate size;

Therefore; Cmin, b=12mm.

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

c) 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 usingES EN 1992:2015 tables 4.4N.

Using structural class, S3 and XC1

First, the concrete cover

With:

$c_{min,b}$ = 12 mm

$c_{min,dur}$ = 10 mm

^{C}min = max (cmin,b; cmin,dur ; 10 mm) =max (12; 10; 10 mm) = 12 mm

Δc_{dev} = 10 mm.

ΔCdev (allowance in Design for Variation)

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

We obtain from relation

$c_{nom} = c_{min} + \Delta c_{dev}$ = 12 + 10 = 22 mm .

d) 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

Cfire = 20mm

Hs = 120mm

Governing Cover for Design

Governing cover accounting for

- Corrosion
- Bond/Durability
- Fire

Cover = 22mm

Concrete C 20/25

Rebar S400

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

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

d=l/N*K*F1*F2*F3

WHERE

Derivation of these Expressions.

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o \ldots\ldots\ldots \quad (7.16a)(ES\ EN\ 1992;2015\ art\ .7.42\ 1992)$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \text{if } \rho > \rho_o \ldots\ldots\ldots\ldots \quad (7.16b)(ES\ EN\ 1992;2015\ art\ .7.42\ 1992)$$

- Fck=20*0.85 =17
- $N = 11 + 1.5\sqrt{f_{ck}} = 17.71$
- $\rho_o = \frac{\sqrt{f_{ck}}}{\frac{1000}{\sqrt{20}}} = 0.447$

a) Depth Determination

d=l/N*K*F1*F2*F3 $F1 = \frac{500}{\text{fyk}} = \frac{500}{400} = 1.25$

F2=1, F3=1

Sample example panel 7

L_y/L_x = 6640/1500 = 4.43 >2.0 (One Way Cantilever)

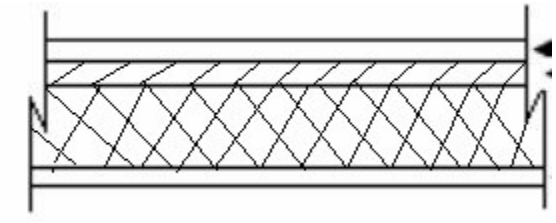
d=l/N*K*F1*F2*F3= l/17.71*0.4*1.25*1*1=169.4mm is the govering depth

$D_{overall}$=d+c+ϕ/2=169.4+22 +6=197.4mm take D=200mm.

d=D-Cover -ϕ/2=200-22-12/2=174mm table 1 Computation of Depth for deflection (refer from in a tabular form appendix B)

b) Load Calculation

Sectional (detail) elevation of floor slabs



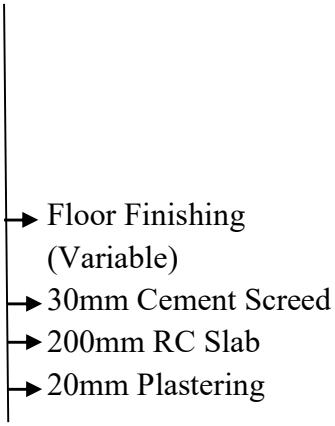


Table 2 sectional elevation for first floor

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……..(2015)

Table 3 design dead &live for first floor solid slab

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)
F1	Kitchen	6.59	6.59	2	2	11.8965
F2	Kitchen	6.59		2		
	Shower	6.59	6.59	2	2	11.8965
	Toilet	6.59		2		
F3	Toilet	6.59	6.59	2	2	11.8965
F4	Kitchen	6.59		2		
	Cafeteria	6.63	6.63	3	3	13.4505
F5	Kitchen	6.59	6.59	2	2	11.8965
F6	Toilet	6.59		2		
	Shower	6.59	6.59	2	2	11.8965
F7	Toilet	6.59	6.59	2	2	11.8965
F8	Cafeteria	6.63		3		
	Kitchen	6.59	6.63	2	3	13.4505
F9	Corridor	6.98		5		
	Kitchen	6.59	6.98	2	5	16.923
	Cafeteria	6.63		3		

F10	Cafeteria	6.63	6.63	3	3	13.4505
F11	Cafeteria	6.63		3		
	Corridor	6.98	6.98	5	5	16.923
F12	Shop	6.63	6.63	5	5	16.4505
F13,14	Shop	6.63	6.63	5	5	16.4505
F15,16	Cafeteria	6.63	6.63	3	3	13.4505
F17	Retail	6.63		5		
	Corridor	6.98	6.98	5	5	16.923
F18	Shop	6.63	6.63	5	5	16.4505
F19,20 ,21,22	Balcony	6.98	6.98	4	4	15.423

c) Partition wall dead load computation

Material data

- Thickness of HCB=40cm
- Thickness of plastering on two side=20mm
- Unit weight of HCB=14 KN/m^3
- Unit weight of plastering =23 KN/m^3
- Height of wall=2.8m

Dead load of partition wall=Dead load of plaster+ Dead load of HCB

$D.L_{pw}=D.L_{pl}+D.L_{HCB}$

$D.L_{pw}(kN)=(H_{pl}*L_{pl}*t_{pl}*\gamma_{pl})+(H_{HCB}*L_{HCB}*t_{HCB}*\gamma_{HCB})$

Therfore $D.L_{pw}(kN/m^2)=\frac{D.L_{pw}(kN)}{\text{Area of Panel}}$

Where, $D.L_{pw}$=Dead load of Partition Wall

H_{pl} , L_{pl} , t_{pl} & γ_{pl} =Height, Length, Thickness, Unit Weight of Plastering Respectively.

H_{HCB} , L_{HCB} , t_{HCB} & γ_{HCB}= Height, Length, Thickness, Unit Weight of HCB Respectively.

Dead Load of Partition Wall on First Flour Panels

Sample example panel 3

$D.L_{HCB}=H_{HCB}*L_{HCB}*t_{HCB}*\gamma_{HCB}=2.8*4.3*0.4*14=68.14$

$D.L_{pw}(kN/m^2)=\frac{D.L_{pw}(kN)}{\text{Area of Panel}}=68.14/4.2=16.22kN/m^2$

$D.L_{pl})=H_{pl}*L_{pl}*t_{pl}*\gamma_{pl}=2.8*4.3*0.03*23=8.395$

$D.L_{pw}(kN/m^2)=\frac{D.L_{pw}(kN)}{\text{Area of Panel}}=8.395/4.2=1.999kN/m^2$

$D.L_{pw}=D.L_{pl}+D.L_{HCB}=1.999+16.22=18.222kN/m^2$. Table 4 Dead load of Partition wall (refer from in a tabular form appendix B)

Summary of Imposed load

$$Pd=Pd'+1.35\ D.L_{pw}$$

Where, pd'=1.35 D.L +1. 5 L.L

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

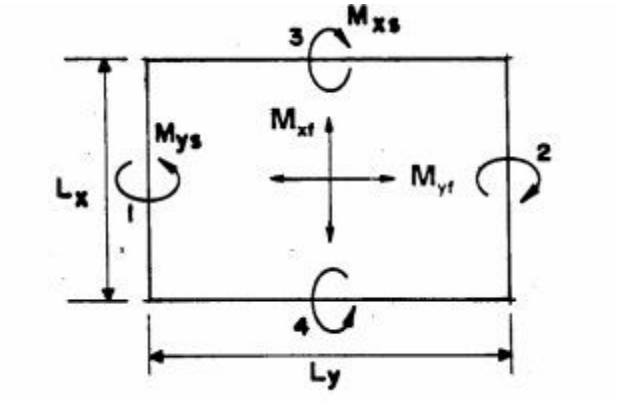
$$M_i=\beta_i n L_x^2 \quad \text{.....(2015)}$$

Where M_i=Design moment per unite width of reference

P_d=or n Uniformly Distributed Design Load

β_i=Coefficient given in table A-1

=f(Ly/Lx), Support condition.



$M_{XS}=\beta_{XS} n L_x^2$

$M_{XF}=\beta_{XF} n L_x^2$

$M_{YS}=\beta_{YS} n L_x^2$

$M_{YF}=\beta_{YF} P_d L_x^2$

Sample example panel 5

Design action, $n_{d\ or\ pd}$= 1.35(6.59) + 1.5(2) = 12.74 kN/m^2

L_y/L_x = 6640/5610 = 1.2 < 2.0 (Two way slab)

Four edges continuous

Long span: M_{xs}= $\beta_{sx1}nlx^2$ = 0.042 x 12.74 x31.5 = 16.85 kNm/m

M_{ys}= $\beta_{sx1}nlx^2$ = 0.032x 12.74 x31.5 = 12.84 kNm/m

Short span: M_{yf}= $\beta_{syf}nlx^2$ = 0.024 x x 31.5 = 9.6kNm/m

M_{xf} = $\beta_{syf}nlx^2$ = 0.024 x 12.74 x 31.5 = 12.84 kNm/m Table 5Moment Calculation for two way slab using coefficient method In a tabular form286 (refer from in a tabular form appendix B)

d) Moment Calculation for cantilever slabs.

Material Data

- Thickness of HCB=400mm
- Thickness plastering on two side=20mm
- Unit weight of HCB=14KN/m^3
- Unit weight of plastering=23 KN/m^3
- Height of wall=2.8m

Load from external wall=(H_{pl} * t_{pl} * γ_{pl}) + (H_{HCB} * t_{HCB} * γ_{HCB})

Where: H_{pl} ,t_{pl} & γ_{pl} =Height, Thickness, Unit Weight of Plastering Respectively.

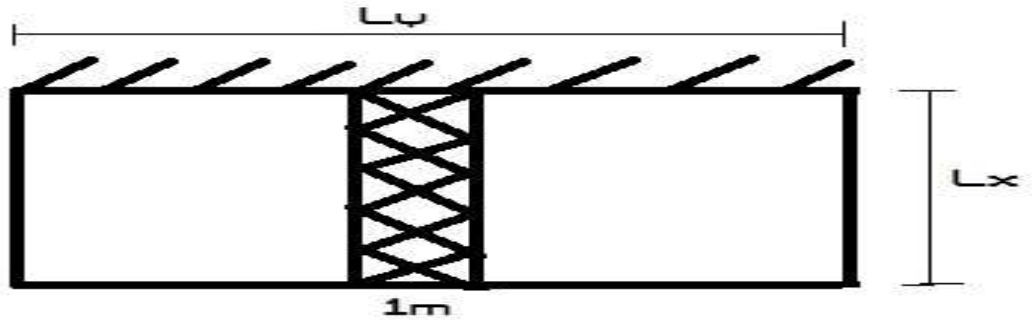
H_{HCB} , t_{HCB} & γ_{HCB}= Height, Thickness, Unit Weight of HCB Respectively.

D.L $_{ex}$=(0.4m x 2.81m x 14 KN/m^3)+(0.02m x 2.8m x 23 KN/m^3)

=17.03 KN/m

Sample example panel 3

- Taking 1m strip of the slab in the shorter direction

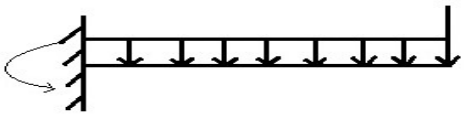


D.L_{ex}=17.03 kN

Pd= 36.47KN/m

17.03 KN

36.47KN/m



Mxs

1.5

Mxs=17.03KN*1.5m + 36.47KN $*(\frac{1.5m}{2})^2$ =46.06KN.m Table 6 Moment Calculation for cantilever slabs. (refer from in a tabular form appendix B)

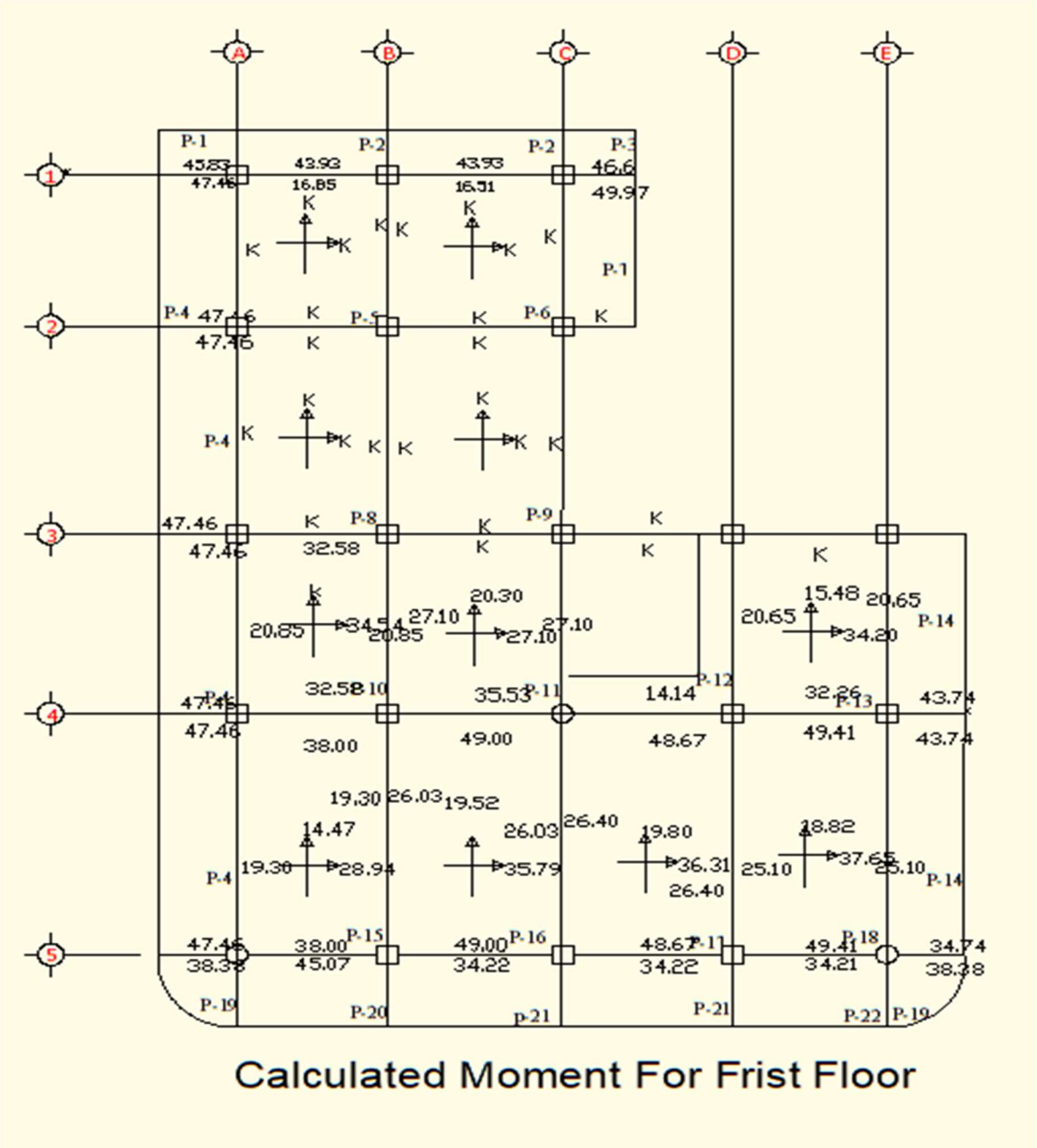


Figure 6 calculated moment for frist floor

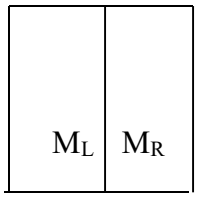
e) Moment adjustment between panels

Support moment adjustment

There are two cases.

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

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….$M_R>M_L$



$M_R>M$

$M_d=M_R-\frac{K_R}{K_R+K_L}*\Delta M$……Considering right(ES EN 1992)

$M_d=M_R+\frac{K_R}{K_R+K_L}*\Delta M$……Considering Left(ES EN 1992)

Sample example panel Between F2&F5

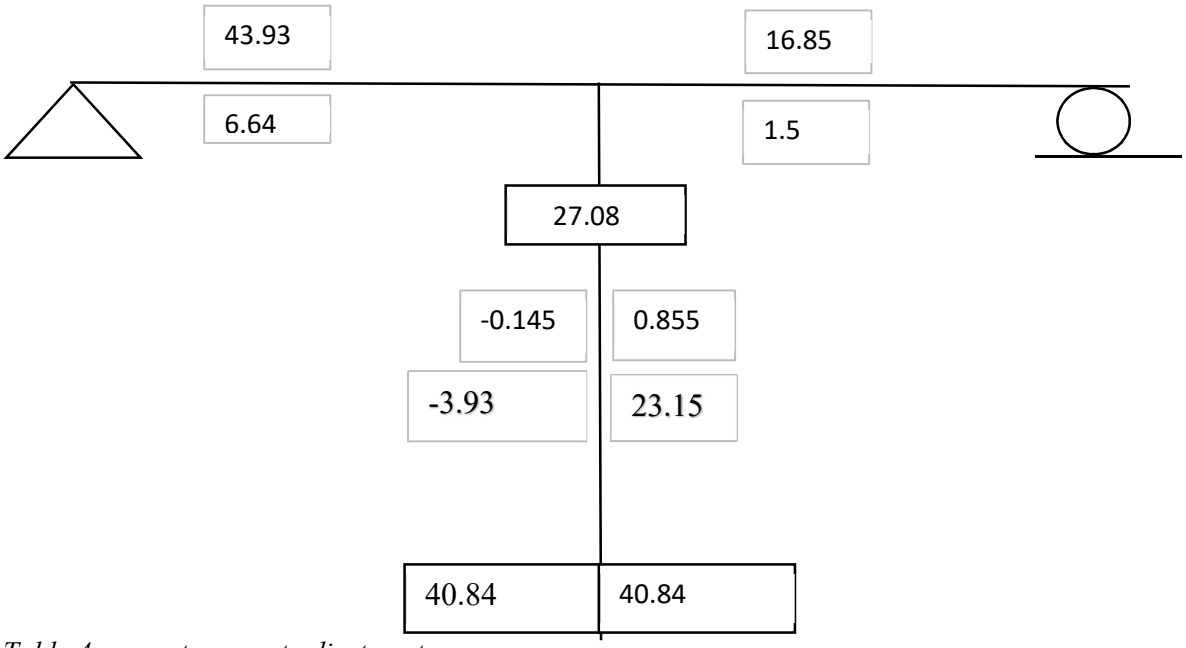


Table 4 support moment adjustments

M1= 43.93KNm

M2 =16.85KNm

ΔMS= M1- M2= 43.93KNm -16.85KNm = 27.08KNm

20%of larger moment=0.2*43.93KN.m=8.786KN.m

Implies that, 20%of larger moment=8.786KN.m< ΔMS=27.08KN.m

Therefore, we have to use moment distribution method to adjust support moment.

For continuous slab, coefficients could be as the following;

$K_{AB}=\frac{3I}{4L}=\frac{3I}{4*6.64}=0.113I$ $KBC=\frac{I}{L}=\frac{I}{1.5}=0.67I$

$Df_{AB}=\frac{KAB}{KBC+KAB}=\frac{0.113I}{0.113I+0.67I}=0.145$ $DfBC=\frac{0.0.67I}{0.113I+0.67I}=0.855$

ΔMdf1 = 27.08 * -0.145 = -3.93KNm

ΔMdf2 = 27.08 * 0.855 = 23.15KNm

Madj = 40.84KNm Table 7 Support Moment adjustment between panels (refer from in a tabular form appendix B)

f) Span Moment Adjustment

If the support moment is decreased the span moments M_{xf} and M_{yf} are then increased allow for the changes of support moment.

$M_{xdf}= M_{xf}+ C_x\Delta M'$

$M_{ydf}= M_{yf}+ C_y\Delta M'$

Where, $\Delta M'= M_R-M_d$ (on all the supports)

C_x, C_y = Coefficients for adjusting span moments given in table.

If the support moment is increased, no adjustment shall be made to the span moment

Thus span moment adjustments is required for **P_{16}, P_{17}, P_{18},**

Consider panel (P_{18}) as sample example. It requires adjustment both in shorter and longer span.

$$\frac{Ly}{Lx} = \frac{10.65}{5.76} = 2$$

Shorter span Cx1=0.461, Cy1=0.272

Longer span Cx2=0.285, Cy2=0.074

Mxf(adj) = ΔMxf+Mxf, ΔMxf= Cx1*ΔMxs+ Cx2*ΔMys

Myf(adj)= ΔMyf+Myf, ΔMyf= Cy1*ΔMxs+ Cy2*ΔMys

ΔMxs=Mxs (unadj)-Mxs (adj) =49.14-46.6=2.81

ΔMys= Mys(unadj)-Mys(adj)=25.9-25.75=0.15KN.m

ΔMxf=0.461*2.81+0.285*0.15=0.79713KN.m

ΔMyf=0.272*2.81+0.074*0.15=0.20625KN.m

Mxf(adj)= 0.79713KN +37.65=38.44713KN.m

Myf(adj)= 0.20625KN.m +18.82=19.02625KN.m Table 10Span Moment Adjustment (refer from in a tabular form appendix B)

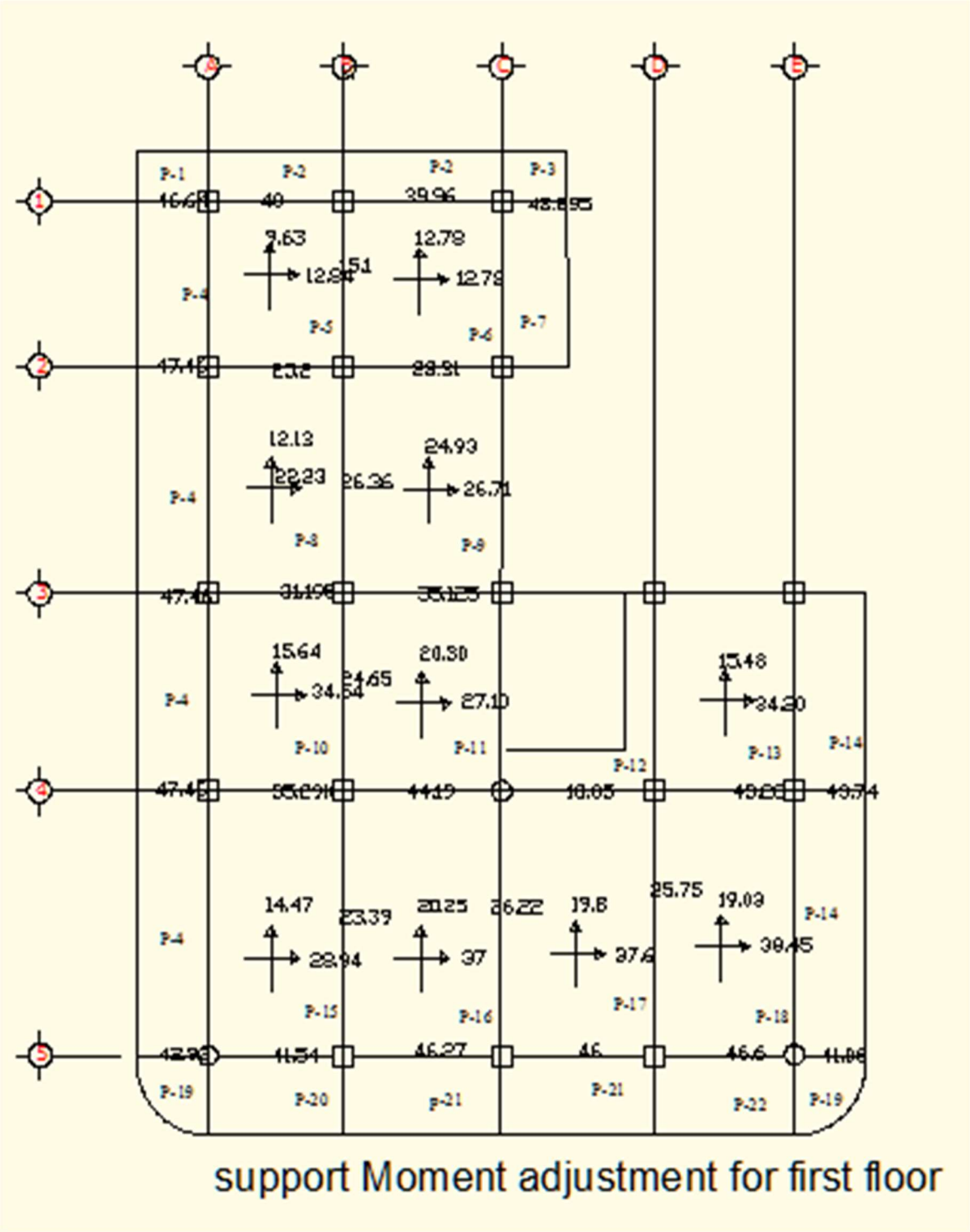


Figure 7 support moment adjustement for frist floor

g) DESIGN OF REINFORCEMENT (refer from in a tabular form appendix B)

Sample example panel 5

panel	Moment	D	B	K	Z	ASmin	Asrequred	ASmax	As	Scalculated	Sprovided
5mxs	40.84	174	1000	0.053	0.93	226.2	718.46	8000	113.04	157.33	150
mxf	11.5	174	1000	0.01	0.91	226.2	207.41	8000	113.04	544.98	400
mys	25.55	174	1000	0.03	0.93	226.2	452.24	8000	113.04	249.95	240
myf	8.61	174	1000	0.01	1	226.2	142.19	8000	113.04	499.73	400

$fctm = 0.3(Fck^{2/3})$

$$A_{s,min} = Max\begin{cases}\frac{0.26 fctm b t d}{fyk}\\ 0.00013 b t d\end{cases}$$ .....(2015) $Art.9.2.1.1$ (1)

Ast, max=0.04*Ac……...(2015) Art.9.2.1.1 (3)

- Effective depth:

d= 200 – 20 -0.5(F) = 174mm

Minimum and maximum reinforcement area

fctm = 0.3(fck)$^{(2/3)}$ = 0.3(25)^(2/3) = 2.6

$A_{s,min}$ = 0.26 (f_{ctm}/f_{yk})bd =0.26(2.6/400)*1000*174=294mm^2/m

= 0.0013bd = 0.0013(1000))(174)= 226.2mm2/m

$A_{s,max}$ = 0.04A_c = 0.04(1000)(200) = 8000 mm^2/m

Short span:

Support:

M_{xs}= 40.84 kNm/m

K = μ_{sd}= M/$f_{ck}bd^2$ = (40.84x10^6) / (25 x 1000 x 174^2) = 0.05395< K_{bal}= $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=0.9388d

A_s = M/0.87f_{yk}z = 40.84 x 10^6/0.87 x 400 x 0.9388 x 174= 718.454 mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b * Ab}{As}$ =1000*113.04/718.454=157.33take Spacing =150mm

Provide, Ø12 C/C 165mm

Mid Span

M_{xf}= 11.5 kNm/m

K = μ_{sd}= M/$f_{ck}$$bd^2$ = (11.5x10^6) / (25 x 1000 x 174^2) = 0.0152 < $K_{bal=}$ $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=0.9156d

A_s = M/0.87f_{yk}z = 11.5 x 10^6/0.87 x 400 x 0.9156 x 174mm = 207.417 mm^2/m <A_{Smin}=226.2 mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b * Ab}{As}$ =1000*113.04/226.2=499.1 mm

Smax= $\begin{Bmatrix} 3D \\ 400 \end{Bmatrix}$ take Smax=3*200=600mm take Smax=400mmtake Spacing =400mm

Provide, Ø12 C/C 400mm

Long span:

Support:

M_{ys}= 25.55 kNm/m

K = μ_{sd}= $M/f_{ck}bd^2$ = $(25.55x10^6) / (25 \times 1000 \times 174^2)$ = 0.03375< K_{bal}= $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=z=0.933d

A_s = $M/0.87f_{yk}z$ = 25.55 x 10^6/0.87 x 400 x 0.933 x 174 = 452.249 mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/452.249=249.95 mm take Spacing =240mm

Provide, Ø12 C/C 240mm

Long span:

Mid-span:

M_{yf}= 8.61 kNm/m

K = μ_{sd}= $M/f_{ck}bd^2$ = $(8.16x10^6) / (25 \times 1000 \times 174^2)$ = 0.01138 < K_{bal}= $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=1d

A_s = $M/0.87f_{yk}z$ = 8.16 x 10^6/0.87 x 400 x 1 x 174 = 142.2 mm^2/m <A_{Smin}=226.2 mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/226.2=499.1 mm

Smax= $\begin{Bmatrix} 3D \\ 400 \end{Bmatrix}$ take Smax=3*200=600mm take Smax=400mmtake Spacing=400mm

Provide, Ø12 C/C 400mm Table 5 Reinforcement design for first floor slab(refer from in a tabular form appendix B)

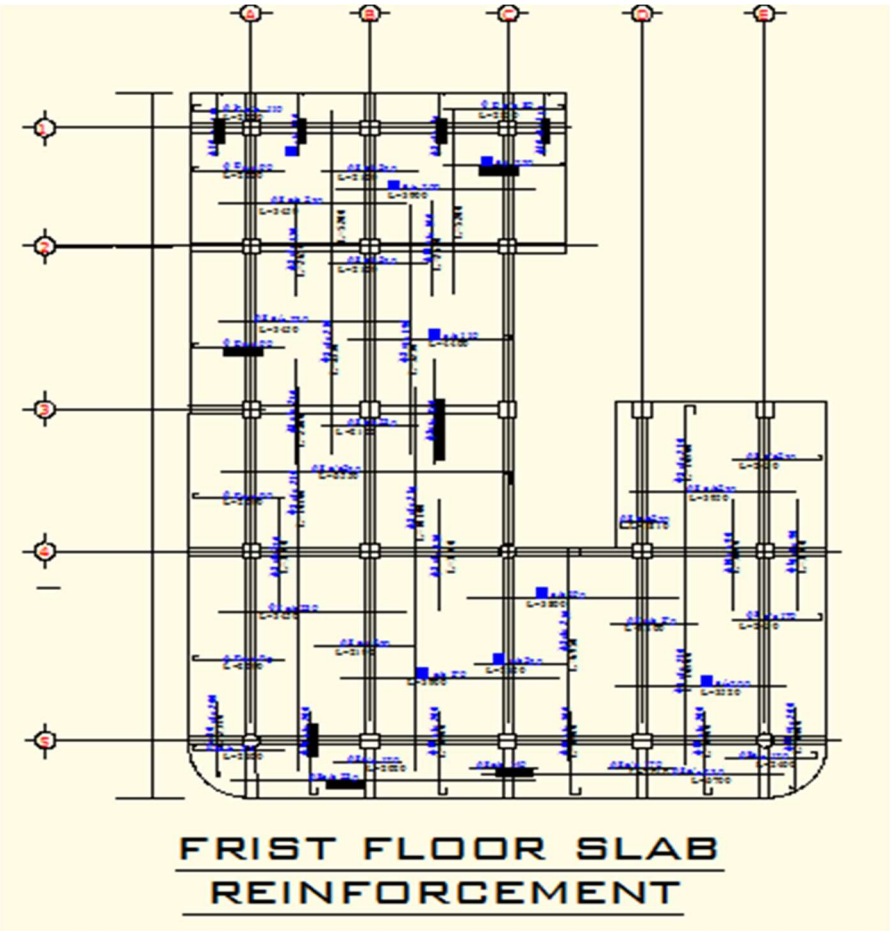


Figure 8 frist floor slab reinforcement

Check Depth for Deflection: Serviceability Requirement

To check the depth for deflection weathers it satisfies the serviceability requirement we use the new codes ES EN 1991:2015.

As an example panel 5

Initially we assumed $\rho_o = \rho$.but we can calculate them as follows

$\rho = \frac{A_{st,req}}{bd} = \frac{718.46}{1000*174} = 0.0413\%$ …….ES E*N*1992: 2017 . 7.4.2(7.16*a*)

$\rho o = \frac{\sqrt{fck}}{1000} = \sqrt{20}/1000 = 0.4472\%$

Now, $\rho = 0.19\% < \rho o = 0.44721\%$

$l/d = N * K$*F1*F2*F3

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o$$ (7.16a)(ES EN 1992;2015 art .7.42 1992)

$$N = \left[11 + 1.5\sqrt{20}\frac{0.4472}{0.0413} + 3.2\sqrt{20}\left(\frac{0.4472}{0.0413} - 1\right)^{3/2}\right] = 39.33$$

Where

K……. from appendix F

F1=500 /($f_{yk}A_{s,req} / A_{s,prov}$)……(2015)…. (7.17)

F2&F3==1, (because span of slab ≤ 7m)…….(2015) … section 5.3.2.2 (1).

Then we work similarly other slabs by the excel appendix

2.2.5 Second floor slab design

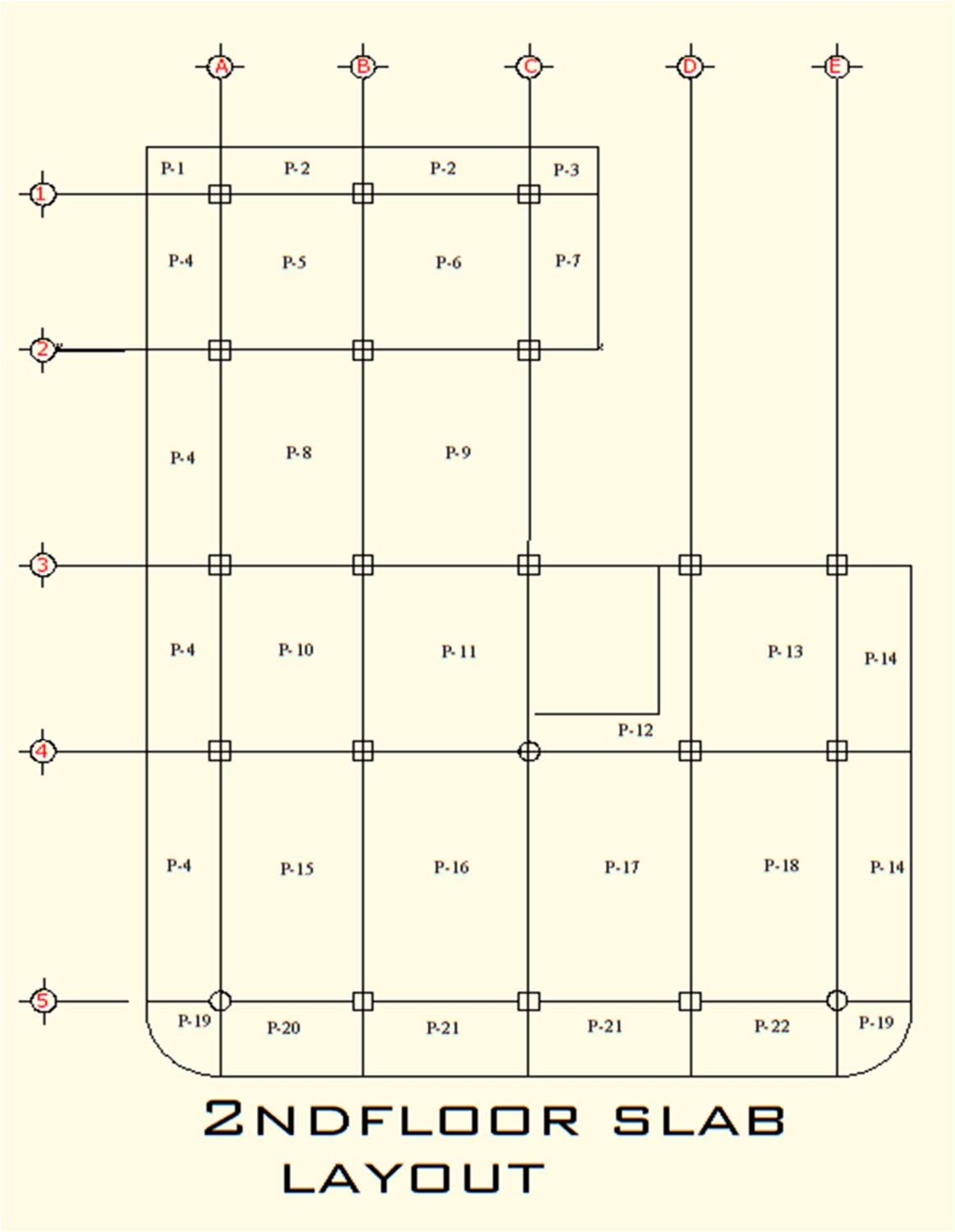


Figure 9 second floor slab layout

Solid Slab Design

a) Concrete cover design (ES EN Art 4.4.1.2 2015)

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.

b) 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 Φ12 longitudinal bar and Φ25 nominal maximum aggregate size;
Therefore; Cmin, b=12mm.
If nominal maximum aggregate size is greater than 32mm Cmin, b should be increased by 5mm

c) 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:

$c_{min,b}$ = 12 mm

$c_{min,dur}$ = 10 mm

^{C}min = max (cmin,b; cmin,dur ; 10 mm) =max (12; 10; 10 mm) = 12 mm

Δc_{dev} = 10 mm.

ΔCdev (allowance in Design for Variation)

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

We obtain from relation

$c_{nom} = c_{min} + \Delta c_{dev}$ = 12 + 10 = 22 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

Cfire = 20mm

Hs = 120mm

Governing Cover for Design

Governing cover accounting for

- Corrosion
- Bond/Durability
- Fire

Cover = 22mm

Concrete C 20/25

Rebar S400

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

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

d=l/N*K*F1*F2*F3

WHERE

Derivation of these Expressions.

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o \quad \ldots\ldots\ldots \quad (7.16a)(\text{ES EN } 1992;2015 \text{ art .7.42 } 1992)$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \text{if } \rho > \rho_o \quad \ldots\ldots\ldots \quad (7.16b)(\text{ES EN } 1992;2015 \text{ art .7.42 } 1992)$$

- Fck=20*0.85 =17
- $N = 11 + 1.5\sqrt{f_{ck}} = 17.71$
- $\rho_o = \frac{\sqrt{f_{ck}}}{\frac{1000}{\sqrt{20}}} = 0.447$

a) Depth Determination

d=l/N*K*F1*F2*F3 $F1 = \frac{500}{\text{fyk}} = \frac{500}{400} = 1.25$

F2=1, F3=1

Sample example panel 7

L_y/L_x = 6640/1500 = 4.43 >2.0 (One Way Cantilever)

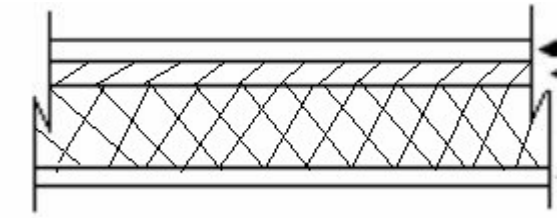
d=l/N*K*F1*F2*F3= l/17.71*0.4*1.25*1*1=169.4mm is the govering depth

$D_{overall}$=d+c+ϕ/2=169.4+22 +6=197.4mm take D=200mm.

d=D-Cover -ϕ/2=200-22-12/2=174mm table 1 Computation of Depth for deflection (refer from in a tabular form appendix B)

b) Load Calculation

Sectional (detail) elevation of floor slabs



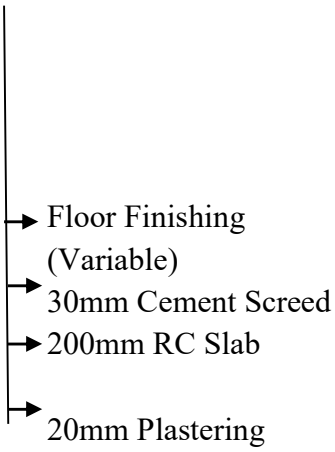


Table 6 sectional elevation of second floor slabs

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……(2015)

Second Floor Panels

Table 7 design dead & live for second floor

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)
S1	Internet Café	6.63	6.63	3	3	13.4505
S2	Internet Café	6.63		3		
	Toilet	6.63	6.63	2	3	13.4505
S3	Toilet	6.63	6.63	2	2	11.9505
S4	Internet Café	6.63		3		
	Shop	6.63	6.98	5	5	16.923
	Corridor	6.98		5		
S5	Internet Café	6.63	6.63	3	3	13.4505
S6	Toilet	6.59	6.59	2	2	11.8965
S7	Toilet	6.59	6.59	2	2	11.8965
S8	Internet Café	6.63	6.63	3	3	13.4505

S9	Internet Café	6.63	6.63	3	3	13.4505
	Corridor	6.98	6.98	5	5	16.923
S10	Shop	6.63		5		
S11	Shop	6.63	6.63	5		
	Corridor	6.98	6.98	5	5	16.923
S12,13,14	Shop	6.63	6.63	5	5	16.4505
S15,16,17,18	Shop	6.63	6.63	5	5	16.4505
	Corridor	6.98	6.98	5	5	16.923
S19,20,21,22	Shop	6.63	6.63	5		

c) Partition wall dead load computation

Material data

- Thickness of HCB=40cm
- Thickness of plastering on two side=20mm
- Unit weight of HCB=14 KN/m^3
- Unit weight of plastering =23 KN/m^3
- Height of wall=2.8m
  - Unit weight of HCB for light weight aggregate range from 10-14 KN/m^3 & lower values are used for smaller size thickness (100mm to 200mm) ,so we use γ_{HCB}=14 KN/m^3

- Dead load of partition wall=Dead load of plaster+ Dead load of HCB

$D.L_{pw} = D.L_{pl} + D.L_{HCB}$

$D.L_{pw}(kN) = (H_{pl} * L_{pl} * t_{pl} * \gamma_{pl}) + (H_{HCB} * L_{HCB} * t_{HCB} * \gamma_{HCB})$

Therfore $D.L_{pw}(kN/m^2) = \frac{D.L_{pw}(kN)}{\text{Area of Panel}}$

Where, $D.L_{pw}$=Dead load of Partition Wall

H_{pl} , L_{pl} , t_{pl} & γ_{pl} =Height, Length, Thickness, Unit Weight of Plastering Respectively.

H_{HCB} , L_{HCB} , t_{HCB} & γ_{HCB}= Height, Length, Thickness, Unit Weight of HCB Respectively.

Dead Load of Partition Wall on First Flour Panels

Sample example panel 6

D.L $_{HCB}$=H_{HCB} * L_{HCB} * t_{HCB} * γ_{HCB}=2.8*18.35*0.4*14=287.73

D.L $_{pw}$(kN/m^2)= $\frac{\text{D.L pw(kN)}}{\text{Area of Panel}}$=287.73/43.29=6.65kN/m^2

D.L pl)=H_{pl} * L_{pl} * t_{pl} * γ_{pl} =2.8*18.35*0.03*23=35.57

D.L $_{pw}$(kN/m^2)= $\frac{\text{D.L pw(kN)}}{\text{Area of Panel}}$=35.45/43.29=0.82kN/m^2

D.L $_{pw}$=D.L $_{pl}$ + D.L $_{HCB}$=6.65+0.82=7.49 kN/m^2 . Table 4 Dead load of Partition wall (refer from in a tabular form appendix B)

Summary of Imposed load

Pd=Pd'+1.35 D.L $_{pw}$……..(2015)

Where, pd'=1.35 D.L +1. 5 L.L

d) 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

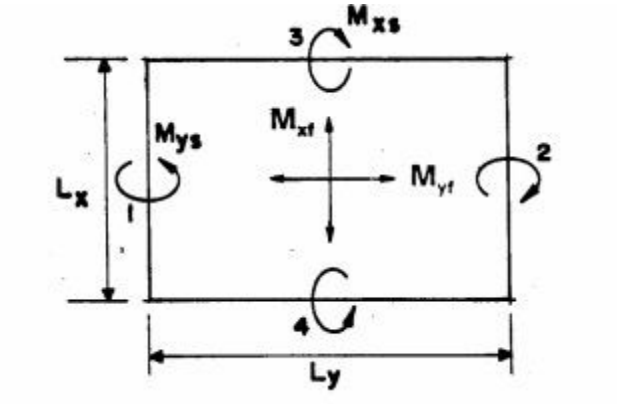
$$M_i=\beta_i n L_x^2 \quad \text{.....(2015)} \quad \text{...}$$

Where M_i=Design moment per unite width of reference

P_d=or n Uniformly Distributed Design Load

β_i=Coefficient given in table A-1

=f(Ly/Lx), Support condition.



$M_{XS}=\beta_{XS}nL_x^2$

$M_{XF}=\beta_{XF}nL_x^2$

$M_{YS}=\beta_{YS}nL_x^2$

$M_{YF}=\beta_{YF}P_dL_x^2$

Sample example panel 6

Design action, $n_{d\ or\ pd}$= 1.35(6.59) + 1.5(2) = 22.025 kN/m^2

L_y/L_x = 6640/6520 = 1 < 2.0 (Two way slab)

Four edges continuous

Long span: $M_{xs}= \beta_{sx1}nlx^2$ = 0.031 x 22.025 x42.5 = 29.017kNm/m

$M_{ys}= \beta_{sy1}nlx^2$ = 0.032x 22.025 x42.5 = 29.954kNm/m

Short span: $M_{yf}= \beta_{syf}nlx^2$ = 0.024 x22.025 x 42.5= 22.4655kNm/m

$M_{xf}= \beta_{syf}nlx^2$ = 0.024 x 22.025 x 42.5 = 22.4655 kNm/m Table 5Moment Calculation for two way slab using coefficient method In a tabular form286 (refer from in a tabular form appendix B)

e) Moment Calculation for cantilever slabs.

Material Data

- Thickness of HCB=400mm
- Thickness plastering on two side=20mm
- Unit weight of HCB=14KN/m^3

- Unit weight of plastering=23 KN/m^3
- Height of wall=2.8m

  Load from external wall=(H_{pl} * t_{pl} * γ_{pl}) + (H_{HCB} * t_{HCB} * γ_{HCB})

  Where: H_{pl} ,t_{pl} & γ_{pl} =Height, Thickness, Unit Weight of Plastering Respectively.

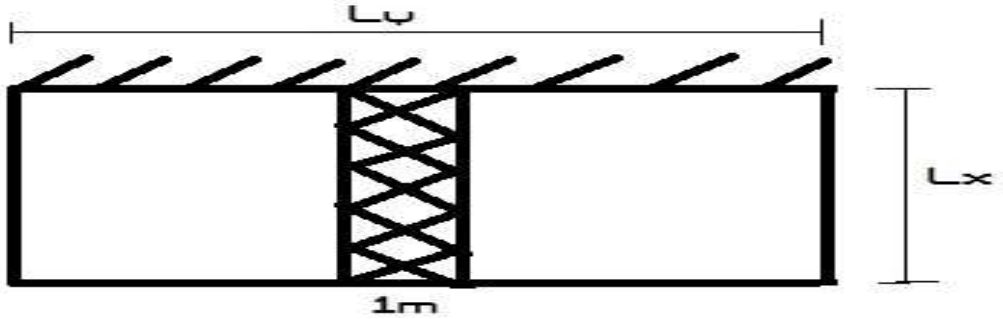
  H_{HCB} , t_{HCB} & γ_{HCB}= Height, Thickness, Unit Weight of HCB Respectively.

  D.L $_{ex}$=(0.4m x 2.81m x 14 KN/m^3)+(0.02m x 2.8m x 23 KN/m^3)

  =17.03 KN/m

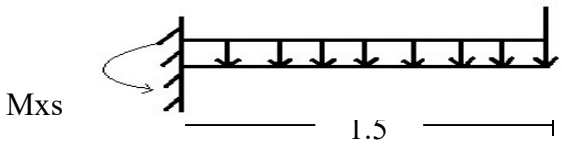
Sample example panel 3

- Taking 1m strip of the slab in the shorter direction

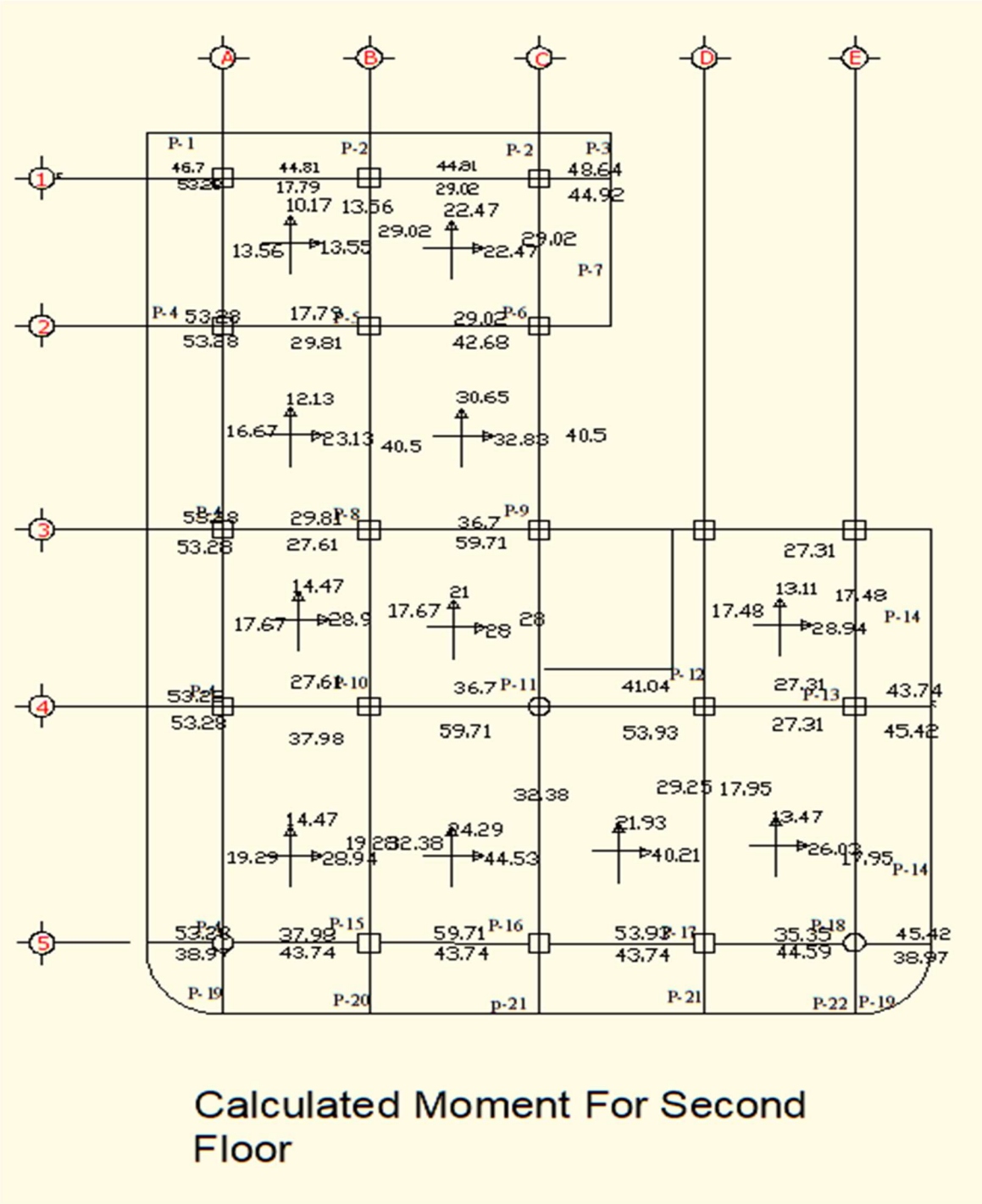


D.L$_{ex}$=17.03 kN

Pd= 41.047KN/m



Mxs=17.03KN*1.5m + 41.047KN *$(\frac{1.5m}{2})^2$ =48.64KN.m Table 6 Moment Calculation for cantilever slabs. (refer from in a tabular form appendix B)



Calculated Moment For Second Floor

Figure 10 calculated moment for second flloor

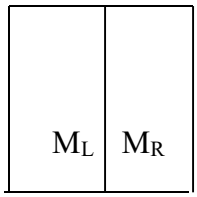
f) Moment adjustment between panels

Support moment adjustment

There are two cases.

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

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….$M_R>M_L$

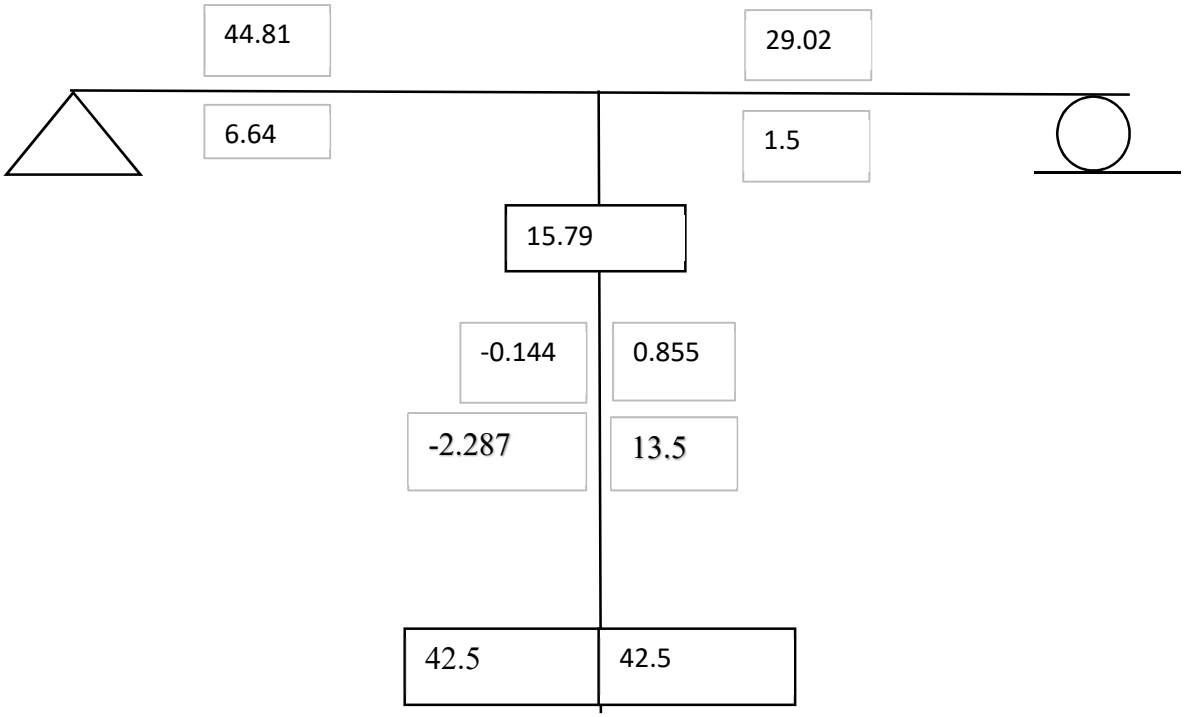


$M_R>M$

$M_d=M_R-\frac{K_R}{K_R+K_L}*\Delta M$……Considering right(ES EN 1992)

$M_d=M_R+\frac{K_R}{K_R+K_L}*\Delta M$……Considering Left(ES EN 1992)

Sample example panel Between S2&S6



M1= 44.81KNm

M2 =17.79KNm

ΔMS= M1- M2= 44.81KNm -17.79KNm = 15.79KNm

20%of larger moment=0.2*44.81KN.m=8.962KN.m

Implies that, 20%of larger moment=8.962KN.m< ΔMS=15.79KN.m

Therefore, we have to use moment distribution method to adjust support moment.

For continuous slab, coefficients could be as the following;

$K_{AB}=\frac{3I}{4L}=\frac{3I}{4*6.64}=0.113I$ $K_{BC}=\frac{I}{L}=\frac{I}{1.5}=0.67I$

$Df_{AB}=\frac{KAB}{KBC+KAB}=\frac{0.113I}{0.113I+0.67I}=0.144$ $Df_{BC}=\frac{0.0.67I}{0.113I+0.67I}=0.855$

ΔMdf1 = 15.79* -0.144= --2.287KNm

ΔMdf2 = 15.79 * 0.855 = 13.5KNm

Madj = 42.5KNm Table 7 Support Moment adjustment between panels (refer from in a tabular form appendix B)

g) Span Moment Adjustment

If the support moment is decreased the span moments M_{xf} and M_{yf} are then increased allow for the changes of support moment.

$M_{xdf}= M_{xf}+ C_x\Delta M'$

$M_{ydf}= M_{yf}+ C_y\Delta M'$

Where, $\Delta M'= M_R-M_d$ (on all the supports)

C_x, C_y = Coefficients for adjusting span moments given in table.

If the support moment is increased, no adjustment shall be made to the span moment

Thus span moment adjustments is required for **P_9, P_{16}, P_{17},**

Consider panel (P_9) as sample example. It requires adjustment both in shorter and longer span.

$$\frac{Ly}{Lx}=\frac{9.17}{6.52}=1.4$$

Cx1	Cy1	Cx2	Cy2
0.398	0.331	0.315	0.11

Mxf(adj) = ΔMxf+Mxf, ΔMxf= Cx1*ΔMxs+ Cx2*ΔMys

Myf(adj)= ΔMyf+Myf, ΔMyf= Cy1*ΔMxs+ Cy2*ΔMys

ΔMxs=Mxs (unadj)-Mxs (adj) =42.68-39.7=2.98

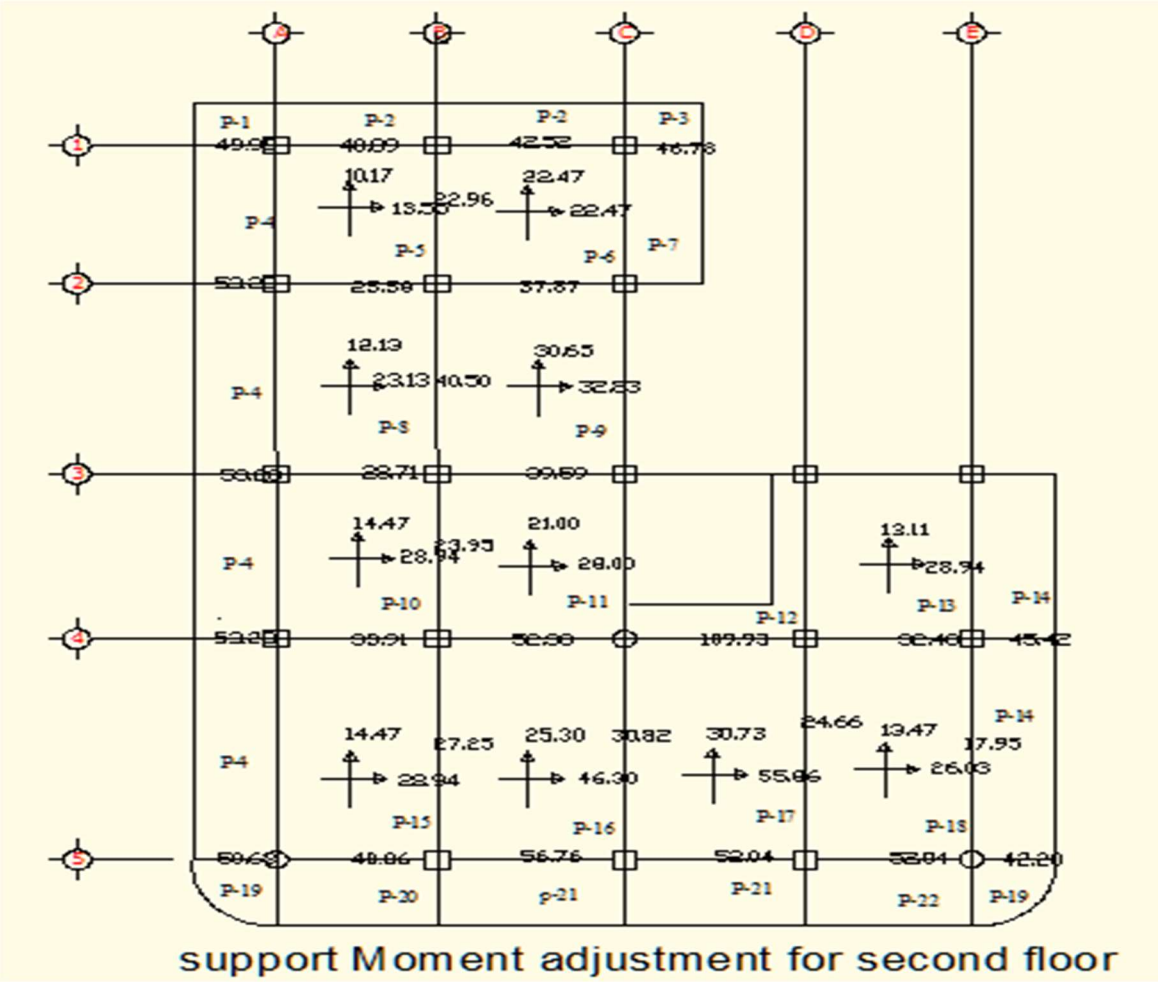
ΔMys= Mys(unadj)-Mys(adj)=40.5-30.96=9.54KN.m

ΔMyf=0.331*2.98+0.0.11*9.54=2.0358KN.m

ΔMxf=0.0.398*2.98+0.315*9.54=4.19KN.m

Mxf(adj)= 32.83KN +4.19=37.02KN.m

Myf(adj)= 30.65KN.m +2.0358=32.686KN.m Table 10Span Moment Adjustment (refer from in a tabular form appendix B)



support Moment adjustment for second floor

h) DESIGN OF REINFORCEMENT (refer from in a tabular form appendix B)

Sample example panel 8

panel	Moment	D	B	K	Z	ASmin	Asrequred	ASmax	As	Scalculated	Sprovided
8mxs	28.71	174	1000	0.04	0.97	226.2	491.16	6960	113.04	230.2	230
Mxf	22.13	174	1000	0.03	0.97	226.2	375.41	6960	113.04	301.1	300
Mys	30.96	174	1000	0.04	0.96	226.2	531.20	6960	113.04	212.8	210
Myf	12.13	174	1000	0.02	0.99	226.2	203.24	6960	113.04	499.7	400

$fctm = 0.3(Fck^{2/3})$

$A_{s,min} = Max\begin{cases}\frac{0.26fctmbtd}{fyk}\\ 0.00013btd\end{cases}$ .....(2015) $Art.9.2.1.1$ (1)

Ast, max=0.04*Ac……...(2015) Art.9.2.1.1 (3)

Effective depth:

d= 200 – 20 -0.5(F) = 174mm

Minimum and maximum reinforcement area

fctm = 0.3(fck)$^{(2/3)}$ = 0.3(25)^(2/3) = 2.6

$A_{s,min}$ = 0.26 (f_{ctm}/f_{yk})bd =0.26(2.6/400)*1000*174=294mm^2/m

= 0.0013bd = 0.0013*(1000)* (174) = 226.2mm2/m

$A_{s,max}$ = 0.04A_c = 0.04(1000)(200) = 8000 mm^2/m

Short span:

Support:

M_{xs}= 28.71kNm/m

K = μ_{sd}= M/$f_{ck}bd^2$ = (28.71x10^6) / (25 x 1000 x 174^2) = 0.0379< $K_{bal=}$ $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd<}$ $\mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=0.9654d

A_s = M/0.87f_{yk}z = 28.71x 10^6/0.87 x 400 x 0.9654 x 174= 491.156mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/475.13=230.15 take Spacing =230mm

Provide, Ø12 C/C 230mm

Mid Span

M_{xf}= 22.13 kNm/m

K = μ_{sd}= M/f_{ck}bd^2 = (22.13x10^6) / (25 x 1000 x 174^2) = 0.0292 < $K_{bal=}$ $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd<}$ $\mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=0.9735d

A_s = M/0.87f_{yk}z = (22.13 x 10^6/0.87 x 400 x 0.9735 x 174mm = 375.414mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/375.414=301mm take Spacing =300mm

Provide, Ø12 C/C 300mm

Long span:

Support:

M_{ys}= 30.96kNm/m

K = μ_{sd}= M/f_{ck}bd^2 = (30.96x10^6) / (25 x 1000 x 174^2) = 0.0409 < $K_{bal=}$ $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd<}$ $\mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=0.9625d

A_s = M/0.87f_{yk}z = 30.96x 10^6/0.87 x 400 x 0.9625x 174 = 531.2mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/531.2=212.8 mm take Spacing =210mm

Provide, Ø12 C/C 210mm

Long span:

Mid Span

M_{yf}= 12.13kNm/m

K = μ_{sd}= M/$f_{ck}bd^2$ = (12.13x10^6) / (25 x 1000 x 174^2) = 0.01603 < $K_{bal=}$ $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd<}$ $\mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=0.9857d

A_s = M/0.87f_{yk}z = 11.5 x 10^6/0.87 x 400 x 0.9857 x 174mm = 203.24mm^2/m <A_{Smin}=226.2 mm^2/m

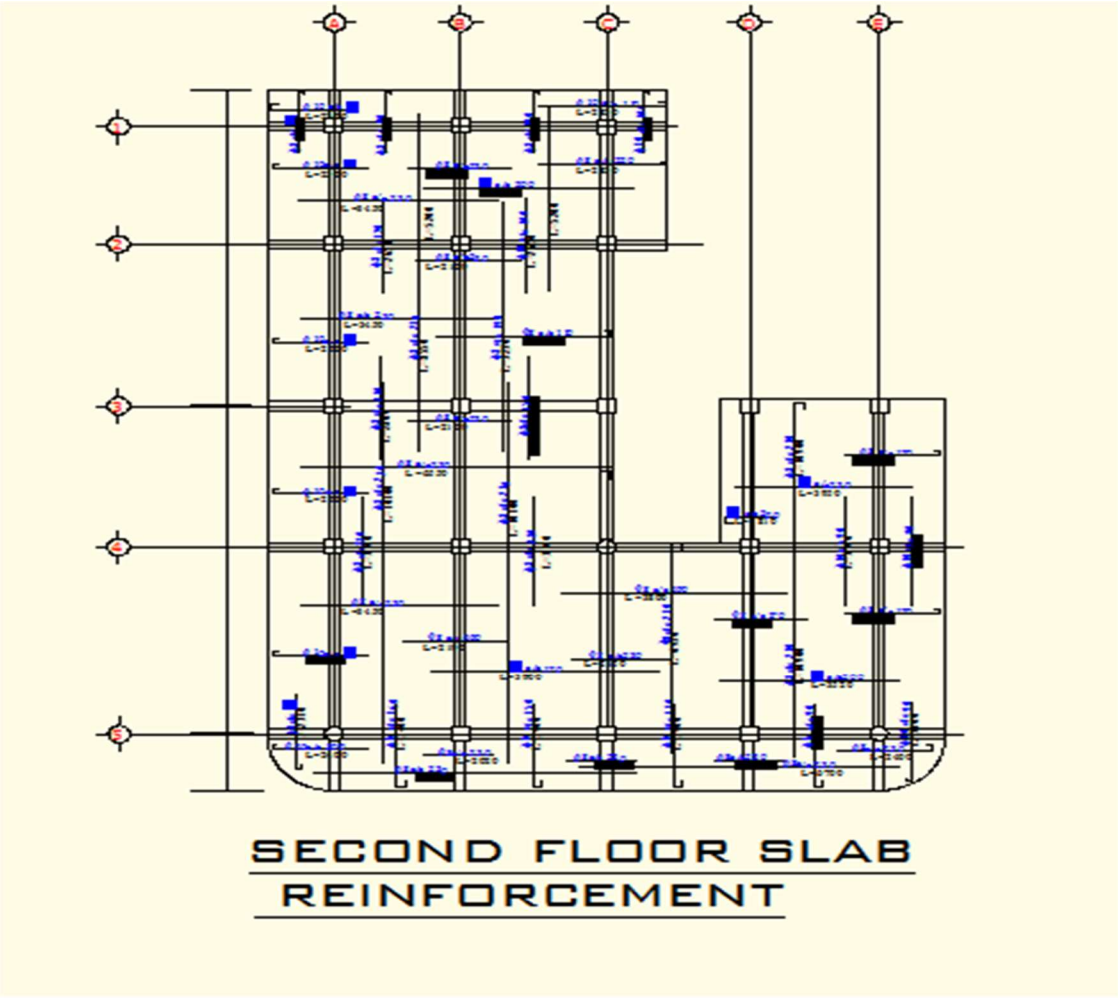
Ø = 12mm

Ab= $\frac{\pi d^2}{4}$ = $\frac{\pi*12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/226.2=499.1 mm

Smax= $\begin{Bmatrix} 3D \\ 400 \end{Bmatrix}$ take Smax=3*200=600mm take Smax=400mmtake Spacing =400mm

Provide, Ø12 C/C 400mm



SECOND FLOOR SLAB REINFORCEMENT

Check Depth for Deflection: Serviceability Requirement

To check the depth for deflection weathers it satisfies the serviceability requirement we use the new codes ES EN 1991:2015.

As an example panel 8

Initially we assumed $\rho_o = \rho$.but we can calculate them as follows

$\rho = \frac{A_{st,req}}{bd} = \frac{491.16}{1000*174} = 0.282\%$ ……..*N*1992: 2017 . 7.4.2(7.16*a*)

$\rho o = \frac{\sqrt{fck}}{1000} = \sqrt{20}/1000 = 0.4472\%$

Now, $\rho = 0.19\% < \rho o = 0.44721\%$

$l/d = N * K * F1 * F2 * F3$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o \quad (7.16a)$$

$$N = \left[11 + 1.5\sqrt{20}\frac{0.4472}{0.282} + 3.2\sqrt{20}\left(\frac{0.4472}{0.282} - 1\right)^{3/2}\right] = 33$$

Where

K……. from appendix B

F1=500 /($f_{yk}A_{s,req}$ / $A_{s,prov}$)……(2015)…. (7.17)

F2&F3==1, (because span of slab ≤ 7m)…….(2015) … section 5.3.2.2 (1).

2.2.6 Third and fourth floor slab design

Solid Slab Design

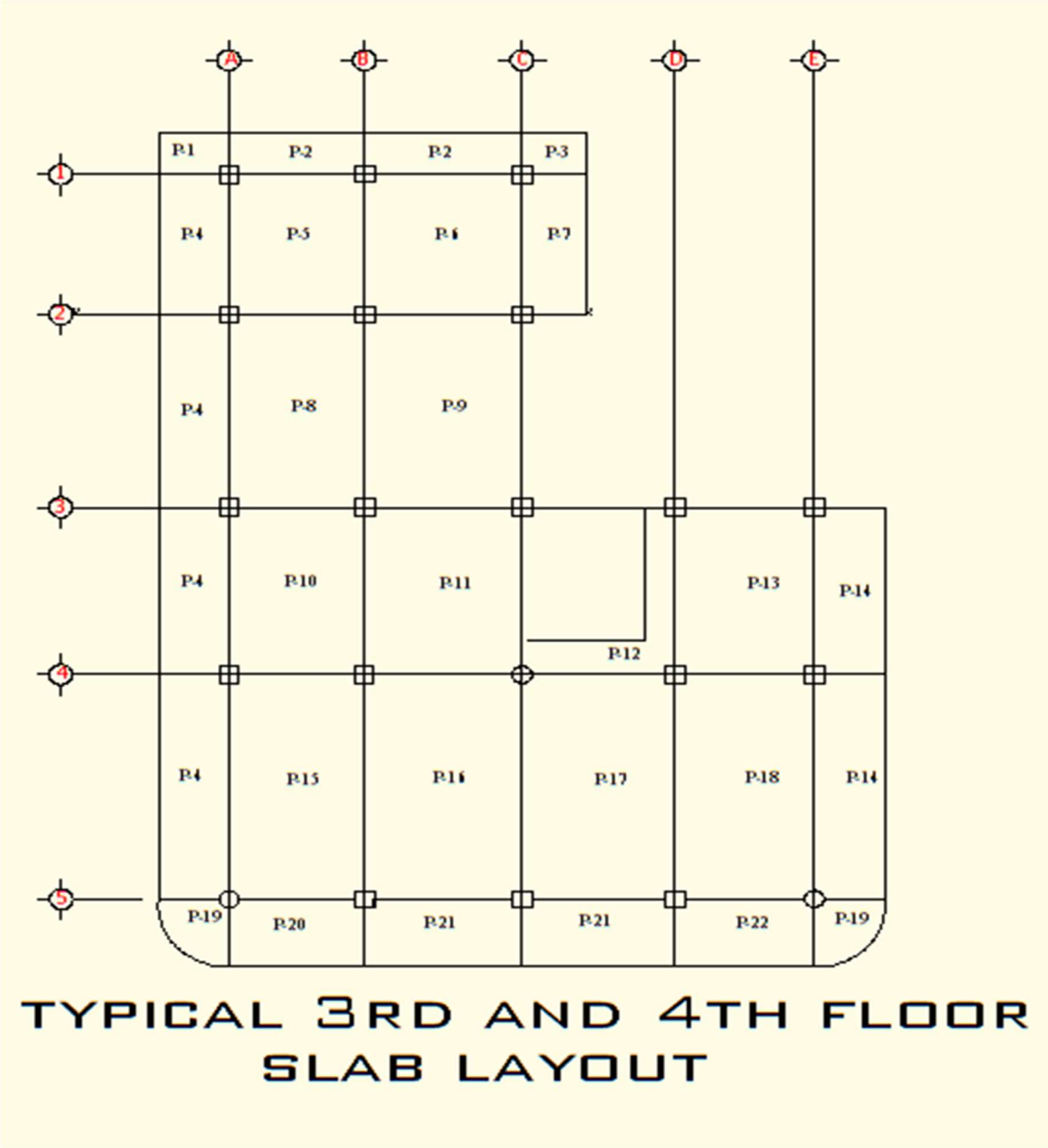


Figure 11 fypical 3rd and 4th floor slab layout

a) Concrete cover design (ES EN Art 4.4.1.2 2015)

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.

b) 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 Φ12 longitudinal bar and Φ25 nominal maximum aggregate size;

Therefore; Cmin, b=12mm.

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

c) 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:

$c_{min,b}$ = 12 mm

$c_{min,dur}$ = 10 mm

^{C}min = max (cmin,b; cmin,dur ; 10 mm) =max (12; 10; 10 mm) = 12 mm

Δc_{dev} = 10 mm.

ΔCdev (allowance in Design for Variation)

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

We obtain from relation

$c_{nom} = c_{min} + \Delta c_{dev}$ = 12 + 10 = 22 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

Cfire = 20mm

Hs = 120mm

Governing Cover for Design

Governing cover accounting for

- Corrosion
- Bond/Durability
- Fire

Cover = 22mm

Concrete C 20/25 Rebar S400

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

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

d=l/N*K*F1*F2*F3

WHERE

Derivation of these Expressions.

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o \quad \ldots\ldots\ldots$$ (7.16a)(ES EN 1992;2015 art .7.42 1992)

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \text{if } \rho > \rho_o \ldots..$$ (7.16b)(ES EN 1992;2015 art .7.42 1992)

Fck=20*0.85 =17

- Fck=20*0.85 =17
- $N = 11 + 1.5\sqrt{f_{ck}} = 17.71$
- $\rho_o = \frac{\sqrt{f_{ck}}}{\frac{1000}{\sqrt{20}}} = 0.447$

- Depth Determination

d=l/N*K*F1*F2*F3 $F1 = \frac{500}{\text{fyk}} = \frac{500}{400} = 1.25$

F2=1, F3=1

Sample example panel 7

L_y/L_x = 6640/1500 = 4.42>2.0 (one Way Cantilever)

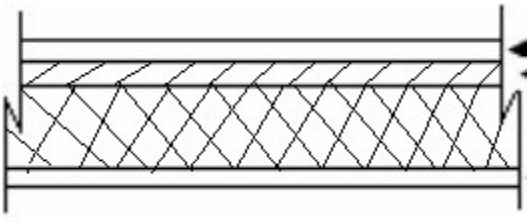
d=l/N*K*F1*F2*F3= l/17.71*0.4*1.25*1*1=169.4mm is the govering depth

$D_{overall}$=d+c+ϕ/2=169.4+22 +6=195.4mm take D=200mm.

d=D-Cover -ϕ/2=200-22-12/2=174mm table 1 Computation of Depth for deflection (refer from in a tabular form appendix B)

- Load Calculation

Sectional (detail) elevation of floor slabs



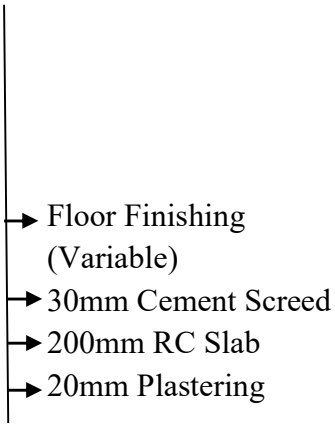


Table 8 sectional elevation for third &fourth floor

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.

Summary of Imposed load

Pd=Pd'+1.35 D.L $_{pw}$

Where, pd'=1.35 D.L +1. 5 L.L

PD'=1.35D.L+1. 5L.L

Table 9 design dead & live for third &fourth floor

- Partition wall dead load computation

Material data

- Thickness of HCB=40cm
- Thickness of plastering on two side=20mm
- Unit weight of HCB=14 KN/m^3
- Unit weight of plastering =23 KN/m^3
- Height of wall=2.8m

Dead load of partition wall=Dead load of plaster+ Dead load of HCB

Panel	Function	Calculated Dead Load (KN/m2)	Governing Dead Load (KN/m2)	Live Load (KN/m2)	Governing Live Load (KN/m2)	Pd'=1.35D.L+1.5L.L (KN/m2)
T1	Bed Room	6.21	6.21	2	2	11.3835
T2	Bed Room	6.21		2		
	Shower	6.59	6.63	2	5	16.4505
	Janitors Room	6.63		5		
T3	Janitors Room	6.63	6.63	5	5	16.4505
T4	Bed Room	6.21		2		
	Shower	6.59	6.59	2	2	11.8965
	Toilet	6.59		2		
T5	Bed Room	6.21	6.21	2	2	11.3835
T6	Toilet	6.59		2		
	Janitors Room	6.63	6.63	5	5	16.4505
T7	Janitors Room	6.63	6.63	5	5	16.4505
T8	Bed Room	6.21	6.21	2	2	11.3835
T9	Bed Room	6.21		2		

							D.L
	Corridor	6.98	6.98	5	5	16.923	
T10	Bed Room	6.21	6.21	2	2	11.3835	
T11	Bed Room	6.21		2			
	Corridor	6.98	6.98	5	5	16.923	
T12	Bed room	6.21		2			
			6.98	2	5	16.923	
	Corridor	6.98		5			
T13	Bed room	6.21		2			
	Toilet	6.59	6.21	2	2	11.3835	
T14	Bed Room	6.21		2			
	Corridor	6.98	6.98	5	5	16.923	
	Shower	6.59		2			
T15,16,17,18	Bed Room	6.21		2			
	Corridor	6.98	6.98	5	5	16.923	
	Toilet	6.59		2			
T19	Balcony	6.98	6.98	4	4	15.423	
T20,21,22	Balcony	6.98		4			
	Toilet	6.59	6.98	2	4	15.423	
	Shower	6.59		2			

$_{pw}$=D.L $_{pl}$ + D.L $_{HCB}$

D.L $_{pw}$(kN)=(H_{pl} * L_{pl} * t_{pl} * γ_{pl}) + (H_{HCB} * L_{HCB} * t_{HCB} * γ_{HCB})

Therfore D.L $_{pw}(kN/m^2)=\frac{D.L_{pw}(kN)}{\text{Area of Panel}}$

Where, D.L $_{pw}$=Dead load of Partition Wall

H_{pl} , L_{pl} , t_{pl} & γ_{pl} =Height, Length, Thickness, Unit Weight of Plastering Respectively.

H_{HCB} , L_{HCB} , t_{HCB} & γ_{HCB}= Height, Length, Thickness, Unit Weight of HCB RespectivelyDead Load of Partition Wall on First Flour Panels

Sample example panel 8

D.L $_{HCB}$=H_{HCB} * L_{HCB} * t_{HCB} * γ_{HCB}=2.8*7.72*0.4*14=121.48

D.L $_{pw}(kN/m^2)=\frac{D.L_{pw}(kN)}{\text{Area of Panel}}$=121.48/51.44=2.36kN/$m^2$

D.L pl)=H_{pl} * L_{pl} * t_{pl} * γ_{pl} =2.8*7.72*0.03*23=14.968

D.L $_{pw}(kN/m^2)=\frac{D.L_{pw}(kN)}{\text{Area of Panel}}$=14.968/51.4=0.291kN/$m^2$

D.L $_{pw}$=D.L $_{pl}$ + D.L $_{HCB}$=2.36+0.291=2.651 kN/m^2 . Table 4 Dead load of Partition wall (refer from in a tabular form appendix B)

- 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

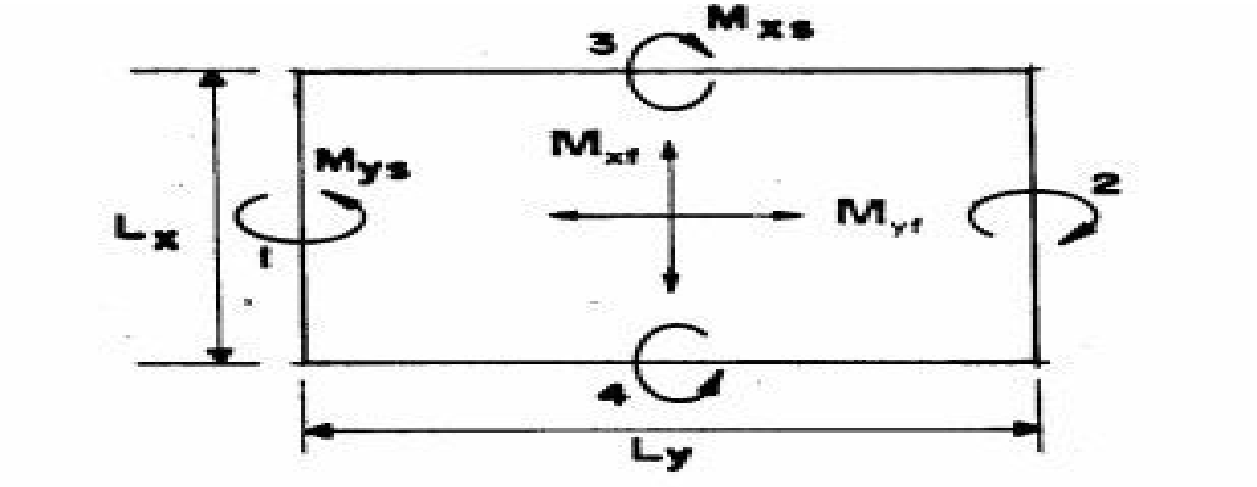
$$M_i=\beta_i n L_x^2 \quad \text{.....(2015)}$$

Where M_i=Design moment per unite width of reference

P_d=or n Uniformly Distributed Design Load

β_i=Coefficient given in table A-1

=f(Ly/Lx), Support condition.



$M_{XS}=\beta_{XS} n L_x^2$

$M_{XF}=\beta_{XF} n L_x^2$

$M_{YS}=\beta_{YS} n L_x^2$

$M_{YF}=\beta_{YF} P_d L_x^2$

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

$M_i = \beta_s n l_x^2$

Where M_i=Design moment per unite width of reference

P_d=or n Uniformly Distributed Design Load

β_i=Coefficient given in table A-1

=f(Ly/Lx), Support condition.

$M_{xs} = \beta_{sx} n l_x^2$

$M_{ys} = \beta_{sy} n l_x^2$

$M_{yf} = \beta_{syf} n l_x^2$

$M_{xf} = \beta_{sxf} n l_x^2$

Sample example panel 8

Design action, $n_{d\ or\ pd} = 1.35(6.59) + 1.5(2) = 22.025\ kN/m^2$

$L_y/L_x = 6640/6520 = 1 < 2.0$ (Two way slab)

Four edges continuous

$M_{xs} = \beta_{sx} n l_x^2 = 0.059 \times 14.586 \times 31.5 = 27.11 kNm/m$

$M_{ys} = \beta_{sy} n l_x^2 = 0.032 \times 14.586 \times 31.5 = 14.702 kNm/m$

$M_{yf} = \beta_{syf} n l_x^2 = 0.024 \times 14.586 \times 31.5 = 11.027 kNm/m$

$M_{xf} = \beta_{sxf} n l_x^2 = 0.044 \times 14.586 \times 31.5 = 20.216$ kNm/m Table 5 Moment Calculation for two way slab using coefficient method In a tabular form286 (refer from in a tabular form appendix B)

- Moment Calculation for cantilever slabs.

Material Data

- Thickness of HCB=400mm
- Thickness plastering on two side=20mm
- Unit weight of HCB=14KN/m^3
- Unit weight of plastering=23 KN/m^3
- Height of wall=2.8m

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

  Where: H_{pl} ,t_{pl} & γ_{pl} =Height, Thickness, Unit Weight of Plastering Respectively.

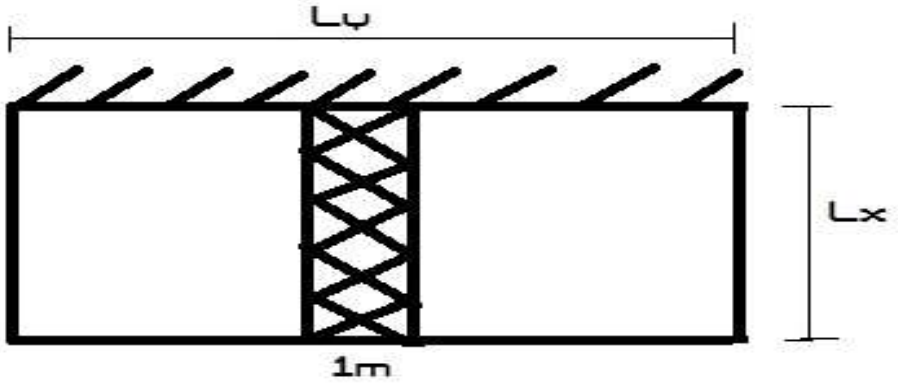
  H_{HCB} , t_{HCB} & γ_{HCB}= Height, Thickness, Unit Weight of HCB Respectively.

  D.L $_{ex}$=(0.4m x 2.81m x 14 KN/m^3)+(0.02m x 2.8m x 23 KN/m^3)

  =17.03 KN/m

Sample example panel 7

- Taking 1m strip of the slab in the shorter direction

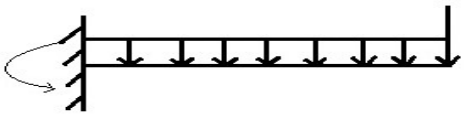


D.L$_{ex}$=17.03 kN

Pd= 36.538 KN/m

17.03 KN

36.538KN/m



Mxs

Mxs=17.03KN*1.5m + 36.538KN $*(\frac{1.5m}{2})^2$ =46.097KN.m Table 6 Moment Calculation for cantilever slabs. (Refer from in a tabular form appendix B)

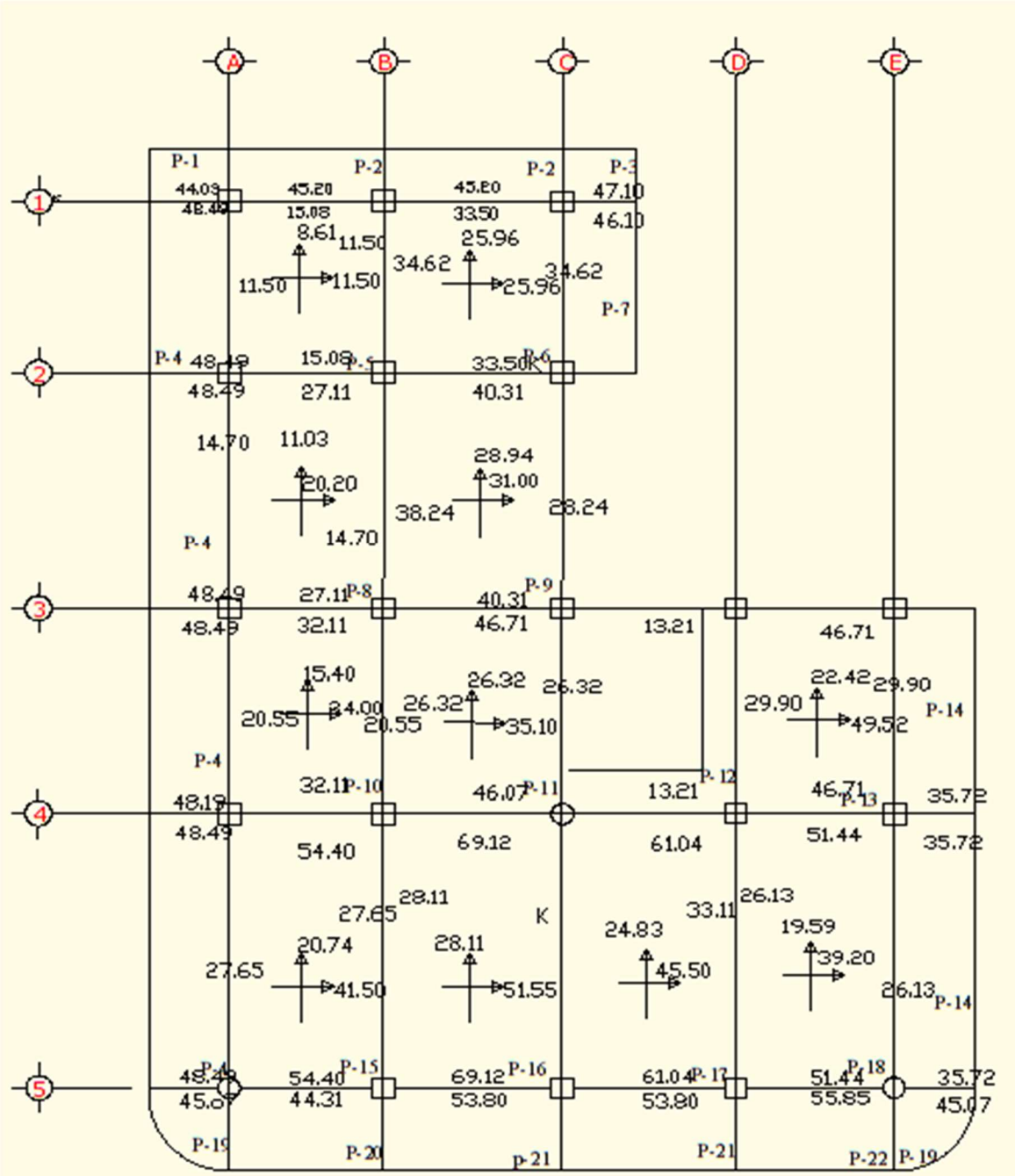


Figure 12 calculated moment for third and fouth floor

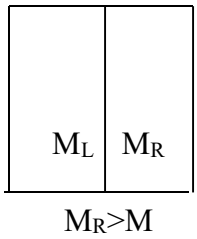
Moment adjustment between panels

Support moment adjustment

There are two cases.

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

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….$M_R>M_L$



$M_R>M$

$M_d=M_R-\frac{K_R}{K_{R+}K_L}*\Delta M$…….Considering right(ES EN 1992)

$M_d=M_R+\frac{K_R}{K_{R+}K_L}*\Delta M$……Considering Left(ES EN 1992)

Sample example panel Between T5&T6

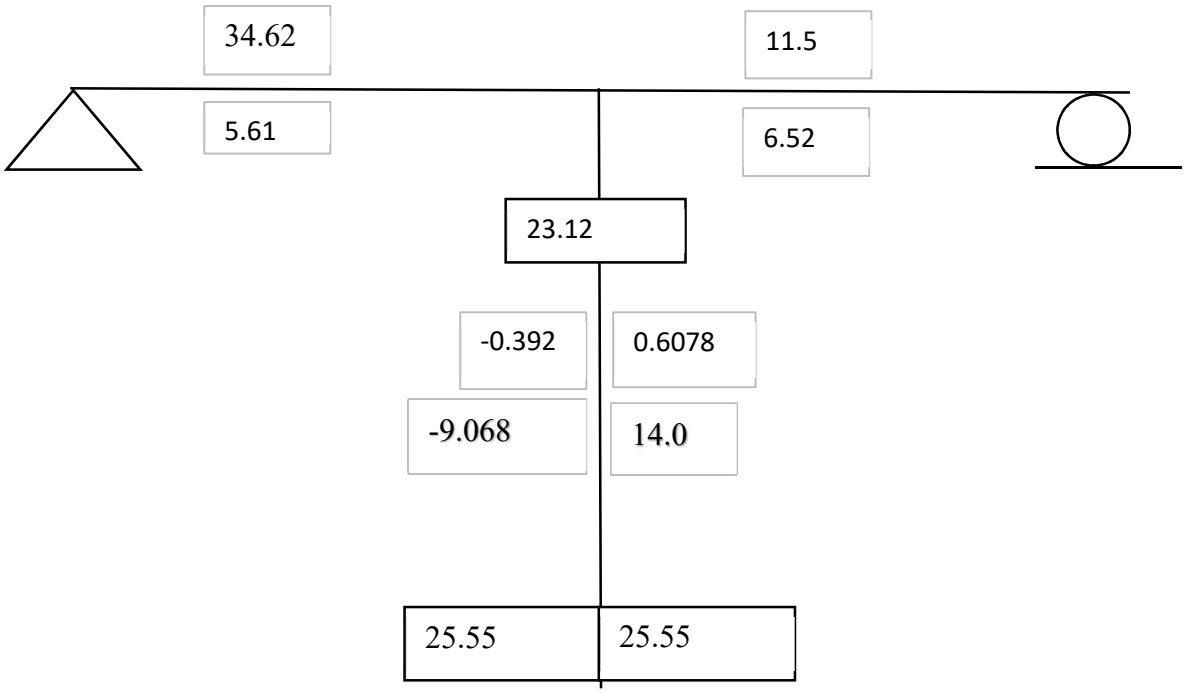


Table 10 support

M1 = 34.62KNm

M2 =11.5KNm

ΔMS= M1- M2= 34.62KNm -11.5KNm = 23.12KNm

20%of larger moment=0.2*34.62KN.m=6.924KN.m

Implies that, 20%of larger moment=6.924KN.m< ΔMS=23.12KN.m

Therefore, we have to use moment distribution method to adjust support moment.

For continuous slab, coefficients could be as the following;

$K_{AB}=\frac{3I}{4L}=\frac{3I}{4*5.61}=0.115I$ $K_{BC}=\frac{I}{L}=\frac{I}{6.52}=0.178I$

$Df_{AB}=\frac{KAB}{KBC+KAB}=\frac{0.115I}{0.115I+0.178I}=-0.3922$ $Df_{BC}=\frac{0.0.178I}{0.115I+0.178I}=0.60778$

ΔMdf1 = 23.12* -0.3922= -9.068KNm

ΔMdf2 = 23.12 * 0.60778= 14.0KNm

Madj = 25.55KNm Table 7 Support Moment adjustment between panels (refer from in a tabular form appendix B)

- Span Moment Adjustment

If the support moment is decreased the span moments M_{xf} and M_{yf} are then increased allow for the changes of support moment.

$M_{xdf} = M_{xf} + C_x\Delta M'$

$M_{ydf} = M_{yf} + C_y\Delta M'$

Where, $\Delta M' = M_R - M_d$ (on all the supports)

C_x, C_y = Coefficients for adjusting span moments given in table.

If the support moment is increased, no adjustment shall be made to the span moment

Thus span moment adjustments is required for **P_{15}, P_{16}, P_{17},**

Consider panel (P_{15}) as sample example. It requires adjustment both in shorter and longer span.

$$\frac{Ly}{Lx} = \frac{10.65}{5.61} = 1.9$$

Cx1	Cy1	Cx2	Cy2
0.481	0.251	0.258	0.06

Mxf(adj) = ΔMxf+Mxf, ΔMxf= Cx1*ΔMxs+ Cx2*ΔMys

Myf(adj)= ΔMyf+Myf, ΔMyf= Cy1*ΔMxs+ Cy2*ΔMys

ΔMxs=Mxs (unadj)-Mxs (adj) =54.4-49.35=5.05

ΔMys= Mys(unadj)-Mys(adj)=27.65-27.88=-0.23KN.m

ΔMyf=0.251*5.05+0.0.06*0.23=1.2537KN.m

ΔMxf=0.0481*5.05+0.258*0.23=2.37KN.m

Mxf(adj)= 41.5+2.37=43.869KN.m

Myf(adj)= 20.74 +1.2537=21.99KN.m Table 10 Span Moment Adjustment (refer from in a tabular form appendix B)

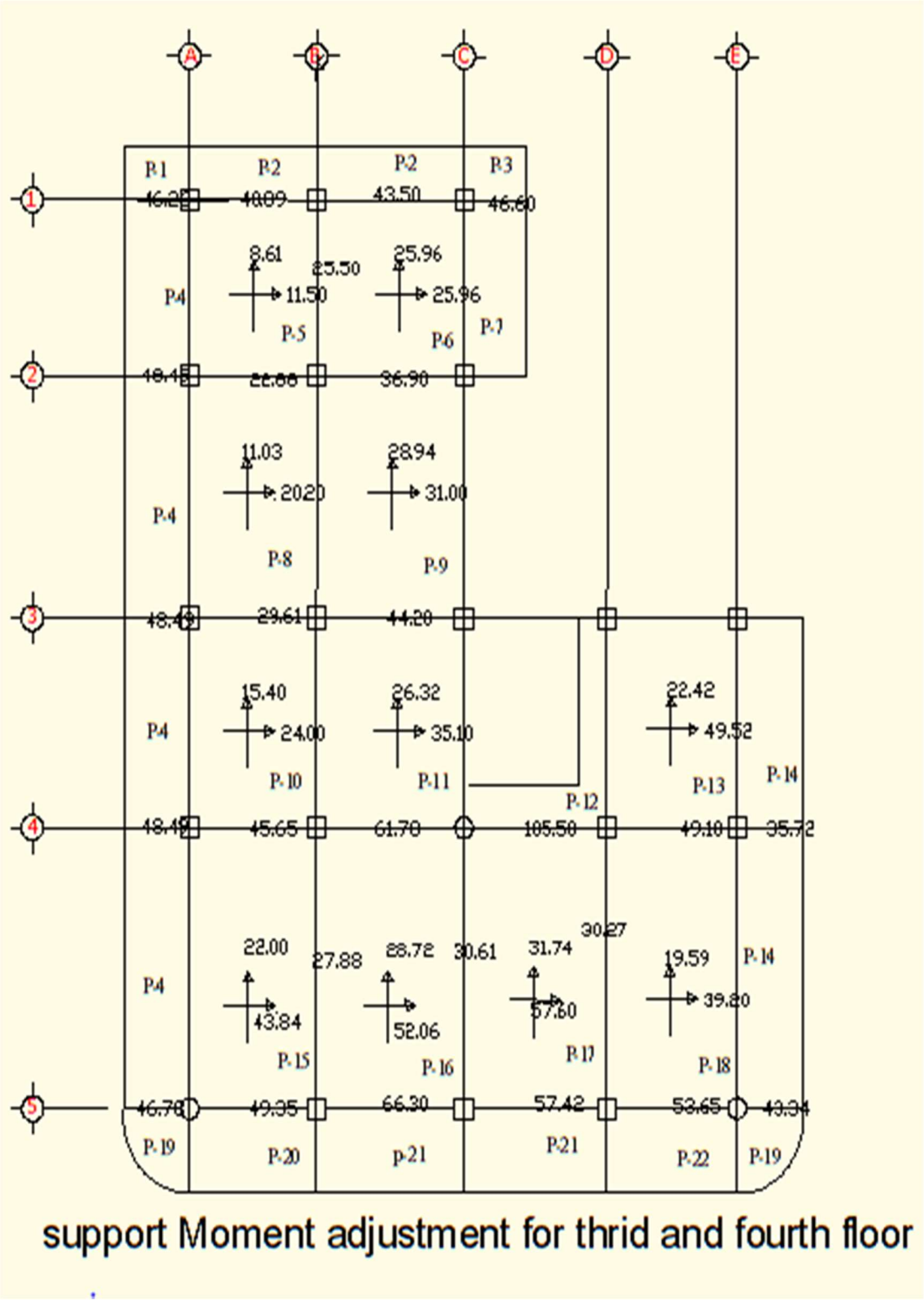


Figure 13 support moment adjustment for third and fourth floor

- DESIGN OF REINFORCEMENT (refer from in a tabular form appendix B)

Sample example panel 6

Panel	Moment	D	B	K	Z	ASmin	Asrequred	ASmax	as	Scalculated	Sprovided	Remark
6mxs	43.5	174	1000	0.057	0.94	226.2	762.31	6960	113.04	148.29	140	F12c/c140
Mxf	25.96	174	1000	0.034	0.96	226.2	444.63	6960	113.04	254.23	250	F12c/c250
Mys	25.55	174	1000	0.034	0.98	226.2	432.38	6960	113.04	261.44	260	F12c/c260
Myf	25.96	174	1000	0.034	0.97	226.2	444.28	6960	113.04	254.44	250	F12c/c250

$fctm = 0.3(Fck^{2/3})$

$A_{s,min} = Max\begin{cases}\frac{0.26fctmbtd}{fyk}\\ 0.00013btd\end{cases}$ .....(2015) $Art.9.2.1.1$ (1)

Ast, max=0.04*Ac……...(2015) Art.9.2.1.1 (3)

- Effective depth:

d= 200 – 20 -0.5(F) = 174mm

Minimum and maximum reinforcement area

fctm = 0.3(fck)$^{(2/3)}$ = 0.3(25)^(2/3) = 2.6

$A_{s,min}$ = 0.26 (f_{ctm}/f_{yk})bd =0.26(2.6/400)*1000*174=294mm^2/m

= 0.0013bd = 0.0013(1000))(174)= 226.2mm2/m

$A_{s,max}$ = 0.04A_c = 0.04(1000)(200) = 8000 mm^2/m

Short span:

Support:

M_{xs}= 43.5kNm/m

K = μ_{sd}= M/f_{ck}bd^2 = (43.5x10^6) / (25 x 1000 x 174^2) = 0.05747 < K_{bal}= $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=z=0.9424d

A_s = M/0.87f_{yk}z = 43.5x 10^6/0.87 x 400 x 0.9424 x 174= 762.3mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/762.3=148.288take Spacing =140mm

Provide, Ø12 C/C 140mm

Mid Span

M_{xf}= 25.96 kNm/m

K = μ_{sd}= M/$f_{ck}$$bd^2$ = (25.96x10^6) / (25 x 1000 x 174^2) = 0.0343

< K_{bal}= $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=z=0.964d

A_s = M/0.87f_{yk}z = (25.96 x 10^6/0.87 x 400 x 0.964 x 174mm = 444.63 mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/444.63=254.23 mm take Spacing =250mm

Provide, Ø12 C/C 250mm

Long span:

Support:

M_{ys}= 25.55 kNm/m

K = μ_{sd}= M/$f_{ck}$$bd^2$ = (25.55x10^6) / (25 x 1000 x 174^2) = 0.03375< K_{bal}= $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=z=0.976d

A_s = M/0.87f_{yk}z = 25.55 x 10^6/0.87 x 400 x 0.976 x 174 = 432.38 mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/432.38 =261.4 mm take Spacing =260mm

Provide, Ø12 C/C 260mm

Long span:

Mid Span

M_{xf}= 25.96 kNm/m

K = μ_{sd}= M/$f_{ck}bd^2$ = (25.96x10^6) / (25 x 1000 x 174^2) = 0.0343< $K_{bal=}$ $\mu_{sd,lim}$ = 0.205 by using general design chart

IF $\mu_{sd} < \mu_{sd,lim}$,Then Compression reinforcement not required

Kz read from the table

Kz=z=0.964d

A_s = M/0.87f_{yk}z = (25.96 x 10^6/0.87 x 400 x 0.964 x 174mm = 444.63 mm^2/m

Ø = 12mm

Ab= $\frac{\pi d^2}{4} = \frac{\pi * 12^2}{4}$ =113.04mm^2

Spacing, S = $\frac{b*Ab}{As}$ =1000*113.04/444.63=254.23 mm take Spacing =250mm

Provide, Ø12 C/C 250mm

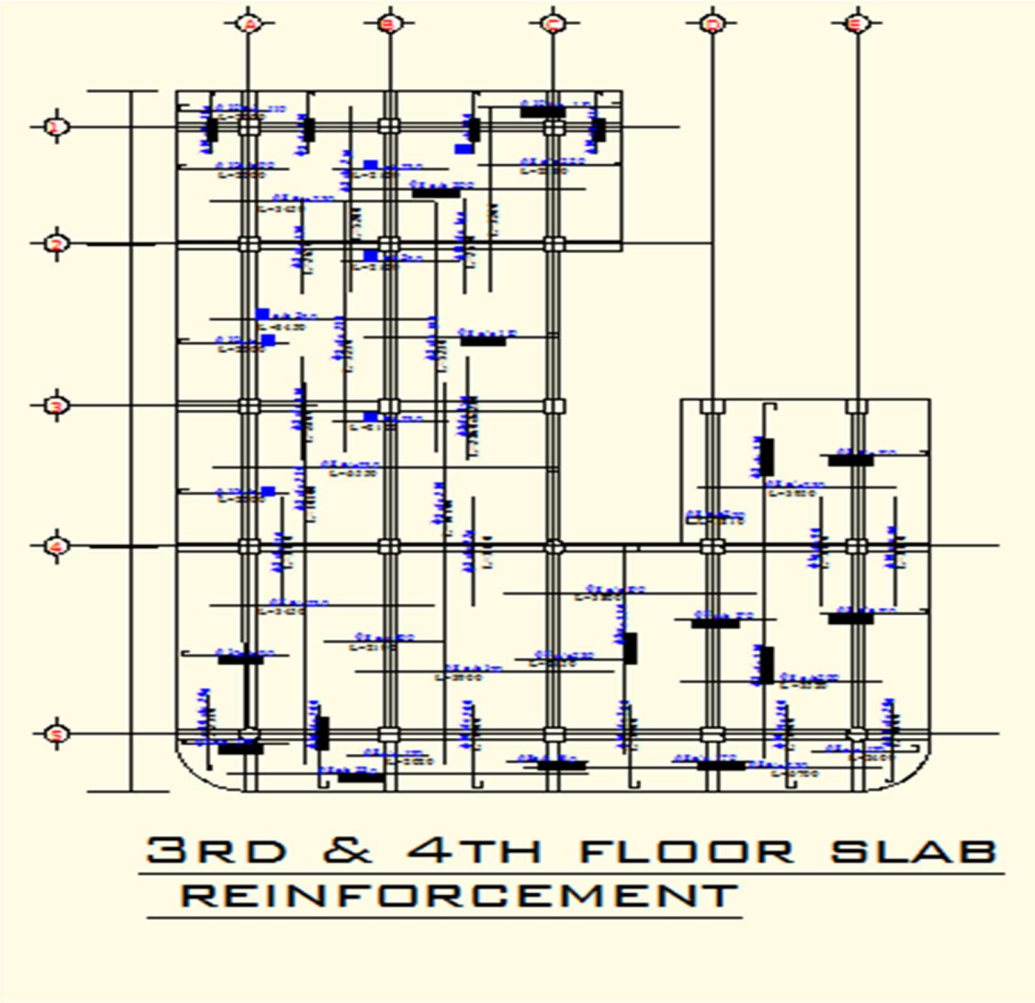


Figure 14 thrid and fourth floors slab reinforcement

Check Depth for Deflection: Serviceability Requirement

To check the depth for deflection weathers it satisfies the serviceability requirement we use the new codes ES EN 1991:2015.

As an example panel 6

Initially we assumed $\rho_o = \rho$.but we can calculate them as follows

$\rho = \frac{A_{st,req}}{bd} = \frac{762.3}{1000*174} = 0.438\%$ ……..N1992: 2017 . 7.4.2(7.16a)

$\rho o = \frac{\sqrt{fck}}{1000} = \sqrt{20}/1000 = 0.4472\%$

Now, $\rho = 0.19\ \% < \rho o = 0.44721\ \%$

$l/d = N * K$*F1*F2*F3

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o$$ (7.16a)(ES EN 1992;2015 art .7.42 1992)

$$N = \left[11 + 1.5\sqrt{20}\frac{0.4472}{0.0438} + 3.2\sqrt{20}\left(\frac{0.4472}{0.0438} - 1\right)^{3/2}\right] = 19.33$$

Where

K……. from appendix F

F1=500 /($f_{yk}A_{s,req}$ / $A_{s,prov}$)……(2015)…. (7.17)

F2&F3==1, (because span of slab ≤ 7m)…….(2015) … section 5.3.2.2 (1).

Then we work similarly other slabs by the excel below

2.2.7 Stair case design

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).

GUID LINES AND TYPES OF STIAR IN OUR BUIDING

The type of stair and its layout is governed essentially by the available size of staircase room and positions beams and columns along the boundary of stair case.

Following are some useful guide lines in deciding the layout and type of stair:

- The stair slab, in general are heavy compared to floor slab because of

i. Heavy dead load due to inclined length of slab acting over horizontal span and due to additional weight of steps,

ii. Greater live load in stair than on floors. Therefore, longer span for flight should be avoided as far as possible.

Stair flight shall be preferably being supported on beam or wall. Supporting on landing slab should be avoided as possible essentially when the span of the landing exceed twice the width of stair, because this Cause stress concentration in the supporting landing slab at their junction. We have two kinds of stair based on their support condition

Pinned –pinned support

Fixed -pinned support

We have two basic ways in which stairs are planned

A straight flight stair, which rises from floor to floor in one direction with or without landing .

Open well stairs, where a space or well exists between flights

The stairs mentioned above are generally free standing ones (simple straight flights of stairs).

Modeling the staircase

In modeling the staircase the part that's supported on the ground and the landing part was assumed to be hinged support and the one which is monolithically casted with the slab was assumed to be fixed

- DESIGN PROCEDURE

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

Loading: which determines the total load in the stair and landing

Analysis: determines moment and shear forces based on the analyzed moment

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.

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

Stair case 1-1

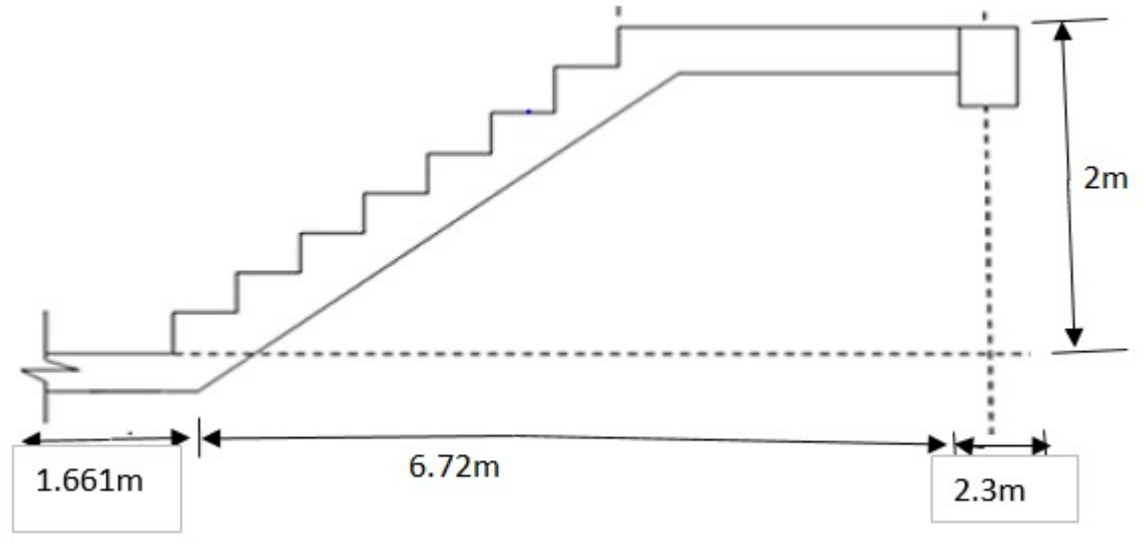


Figure 15 stair case layout

Concrete class C-25

fck = 20MPa

fcd = 11.33MPa

Steel grade S-400

fyk=400MPa

a) 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. The nominal cover shall be specified on the drawings. t is defined as Cmin

Cnom=Cmin+ΔCdev

Where, Cmin – minimum cover and ΔCdev is allowance in design for deviation.

Minimum cover, shall provide in order to ensure

Safe transmission of bond force

Corrosion resistance/ Durability

Fire resistance

$$C_{min} = \max \begin{cases} C_{min,b} \\ C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur,add} \\ 10mm \end{cases}$$

Where;

Cmin, b -minimum cover due to bond requirement, see ES EN1992:2015 Art. 4.4.1.2 (3).

Cmin,dur - minimum cover due to environmental conditions, see ES EN 1992:2015 Art 4.4.1.2 (5)

ΔCdur,γ- additive safety element, see ES EN1992:2015 Art 4.4.1.2 (6)

ΔCdur,st -reduction of minimum cover for use of stainless steel, see ES EN 1992:2015 Art 4.4.1.2 (7)

ΔCdur,add -reduction of minimum cover for use of additional protection, see ES EN1992:2015 Art 4.4.1.2 (8)

But; the recommended value of ΔCdur,γ , ΔCdur,st , and ΔCdur,add is zero see Art. 4.4.1.2 (6, 7, and 8).

b) 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, arangment of bars for separated minimum cover

Cmin, b= diameter of bar

Assume Φ10 longitudinal bar and Φ25 nominal maximum aggregate size; Therefore; Cmin, b=10mm.

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

c) 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 streangth 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:

Cmin, dur=20mm then

$$C_{min} = \max \begin{cases} C_{min,b}=10mm \\ C_{min,dur}=10mm \\ 10mm \end{cases} \qquad C_{min}=10mm$$

ΔCdev (allowance in Design for Variation

Note: The value of ΔCdev for use in a Country may be found in its National Annex. The recommended value is 10 mm.

Then Cnom=Cmin+ΔCdev=10+10=20mm

d) Cover Design for Fire

For the slab to sustain fire incident for 60 minutes the required cover and minimum height of the Section can be determined form Table 5.8 0f EN 1992-1-2

Cfire = 15mm

Hs = 80mm

Governing Cover for Design

Governing cover accounting for

- Corrosion
- Bond/Durability
- Fire cover = 20mm

a) Step 1-Geometrical data height of stair in one flight(clear height between landing)=2

Number of riser(R) = 13

Number of thread (T) = number of riser -1=13-1 =12

Height of Riser= $\frac{\text{Height of Stair}}{\text{No of Stair}} = \frac{2\text{m-0m}}{13}$ = 15.38m

Therefore we use riser height 16 cm for the design.

Width of thread = 56cm 560mm

breadth of step along slope(B) G

B= √ (G^2+R^2)

= $\sqrt{(0.56^2+0.16^2)}$

= 0.582m

take noising =25mm

Width of going=T-N

= 560-25 = 535mm

a) Step 2:

Determination of depth for deflection

According to ES EN 1992:2015; the limit state of deformation may be checked by either:

By limiting the span/depth ratio, according to 7.4.2 or

By comparing a calculated deflection, according to 7.4.3, with a limit value

l/d=K(11+1.5$\sqrt{fck}$ ∗ $\rho o/\rho$+3.2*$\sqrt{fck}(\rho o/\rho$-1)^(3/2))*F1*F2*F3 ……ifρ <ρo art.7.4.2.(7.16a) l/d=K(11+1.5$\sqrt{fck}$ ∗ $\rho o/(\rho$ − ρ‘)+1/12*$\sqrt{fck}\sqrt{(\rho o/\rho)}$)*F1*F2*F3 ……if$\rho > \rho o$ (ES EN 1992;2015 art .7.42 1992)

Where;

l/d - is the limit span/depth

K -is the factor to take into account the different structural systems

ρo - is the reference reinforcement ratio = $10^{-3}\sqrt{fck}$

ρ - is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)

ρ’ - is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers) fck is in MPa units.

F1=500/(fyk*As,req/As,pro)

F2=0.8, for flanged sections where the ratio of the flange breadth to the rib breadth exceeds 3. Otherwise; F2=1 for other cases.

F3=7/leff , For beams and slabs, other than flat slabs, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflections (leff in meters, see Art. 5.3.2.2 (1)). Or F3=8.5/leff, For flat slabs where the greater span exceeds 8.5 m, and which support partitions liable to be damaged by excessive deflections (leff in meters). Otherwise; F3=1 for both cases.

Assumption

Initially we can't know ρ and ρ'. So let's assume ρ= ρo and use equation 7.16a.

As,req = As,provided.

l/d=K*N*F1*F2*F3

N= 11+1.5√fck∗$\rho o/\rho$+3.2*√($\rho o/\rho$-1)^(3/2)

$$N = 11 + 1.5\sqrt{f_{ck}}$$
$$= 17.71$$

ρo=10^{-3}√fck=10^{-3}√20=0.447%

F1=500/(fyk*As,req/As,pro)=500/400=1.25

F2=1 and F3=7/leff=7/10.68=0.655

Depth Determination

d=l/N*K*F1*F2*F3 $F1 = \frac{500}{\text{fyk}} = \frac{500}{400} = 1.25$

F2=1, F3=0.655, l=10680mm

l/d=K*N*F1*F2*F3

(10680/d)*(7/10.68)=17.71*11.3*1.25*1*0.655

d=42.72mm

$D_{overall}$=d+c+ϕ/2=42.72+20 +6=68.72mm take D=150mm.

d=D-Cover -ϕ/2=150-20-12/2=124mm

Note:- The stair case slab is designed as one way reinforced slab. The depth of the section for design purpose of longitudinally spanning slab is taken as the minimum thickness perpendicular to soffit of stair case. I.e. the waist (w). For transverse stair case, there is no universal method to determine effective depth.

Step 3-Loading condition on stair

Material data

Material	thickness (m)	unit weight (KN/m^3)
Marblr	0.03	27
cement screed	0.02	23
Concrete	0.15	25
Plastering	0.02	23

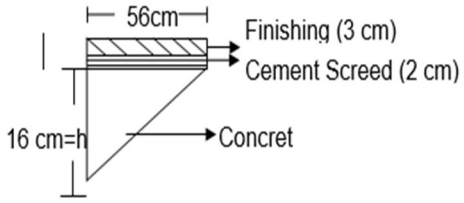
Load computation

1- DL for flight (inclined slab)

The live load for stair case given in codes is the loading on horizontal projection area. Hence the dead load due to waist is calculated along the slope of the stair case and it is then converted to along the horizontal plane.

Take 1m width strip

Step dead load



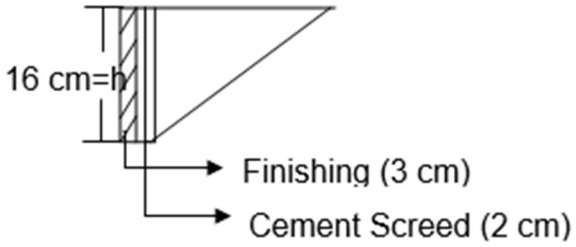
D.L of cement screed=$t_{sc}*\gamma_{sc}$=0.02*23 KN/m^3=0.46 KN/m

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*0.15m*25 KN/m^3 =1.88KN/m

Therefore DL of step=0.46 KN/m + 0.81 KN/m + 1.88 KN/m=3.15 KN/m

Riser Dead Load

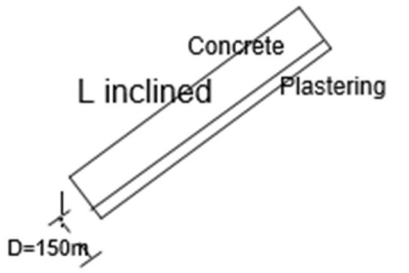


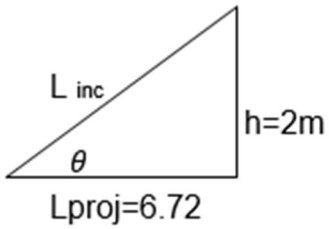
D.L of cement screed=$\frac{\text{No of riser (hcs*tcs*}\gamma\text{sc)}}{\text{Projected length (12*28 cm)}}=\frac{13(0.16m*0.02m*23KN/m^3}{6.72}$=0.14 KN/m

D.L of Finishing =$\frac{\text{No of riser (hcs*tcs*}\gamma\text{sc)}}{\text{Projected length (12*28 cm)}}=\frac{13(0.16m*0.03m*27KN/m^3}{6.72}$=0.25 KN/m

Therefore D.L of riser (16cm) = 0.14 KN/m + 0.25 KN/m=0.39 KN/m

- Waist Dead Load





Tan θ=2/6.72

θ =$\tan^{-1}(\frac{2}{6.72})$=16.57°

Sin θ=2/L_{inc}

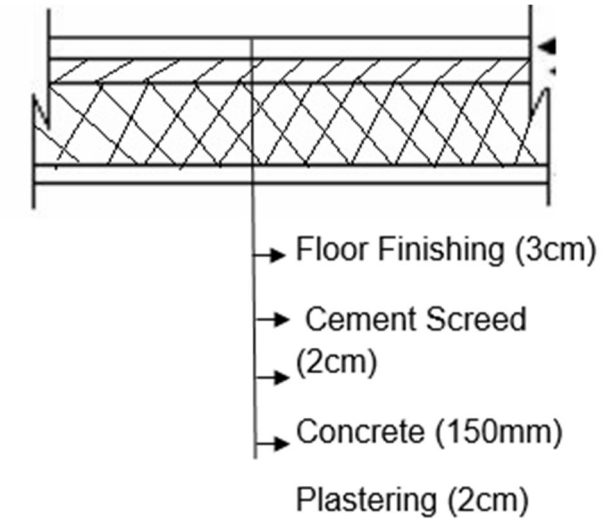
L_{inc}\=2/sin θ =2m/sin 16.57°=7 m

D.L of concrete = $\frac{\text{D*L inc*}\gamma\text{conc}}{\text{Lprojected}}=\frac{0.15m*7m*25\ KN/m^3}{6.72\ m}$=3.91 KN/m

D.L of Plastering = $\frac{\text{tpl*Linc*}\gamma\text{pl}}{\text{Lprojected}}=\frac{0.02m*7m*23\ KN/m^3}{6.72\ m}$=0.48 KN/m

Therefore D.L of waist = 3.91.KN/m + 0.48 KN.m=4.39 KN/m

b) 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*25 KN/m^3 +0.02m*23 KN/m^3=5.48 KN/m

Therefore D.L of Landing=5.48 KN/m

Total Dead Load and design load

c) For the inclined slab

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

=3.15 KN/m+0.39 KN/m+4.39 KN/m=7.92 KN/m

Live load=5 KN/m^2*1m=5 KN/m

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

=1.3*7.92 KN/m + 1.5*5 KN/M =18.2KN/m

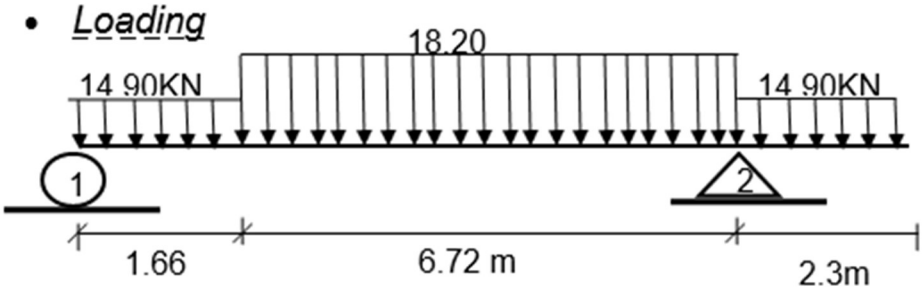
For the landing

- D.L=5.48 KN/m
- L.L=5 KN/m^2 * 1m=5KN/m
- Deisgn load, pd=1.35 D.L + 1.5LL

  =1.35*5.48 KN/m + 1.5*5 KN/m

  =14.99 KN/m

- Loading



Moment Analysis

↻+ΣM at 1=0

14.90*(1.66)*1.66/2+18.20*6.72*(1.66+6.72/2)+14.90*2.30*(8.38+2.30/2)-R_2*8.38m=0

R_2*8.38m=20.53+613.97+326.59 KN.m

R_2=961.09/8.38 KN

R_2=114.69KN

ΣFy=0 (+) ↓

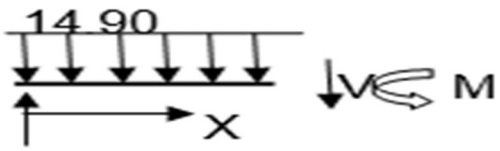
R_1+R_2=14.90 KN/m*1.66 m+18.20 KN/m*6.72 m+14.90 KN/m*2.3m

R_1=181.31+114.69

R_1=66.62 KN

Analyzing using the method of section,

- For Z=0, to Z=1.66m (Z in measured from support 1)



M(x)+14.90 KN/m + x^2/2=66.62x

M(x)=66.62x-7.45 x^2

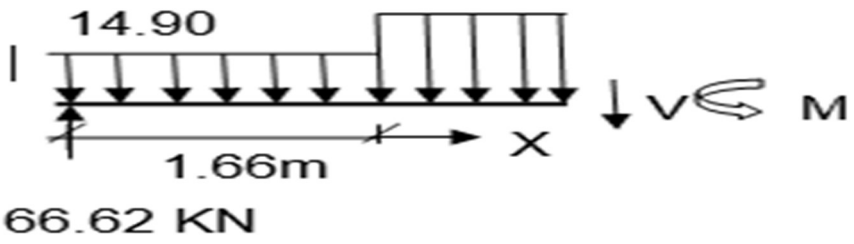
- At Z=0, X=0

M(0)=0

- At Z=1.66, x=1.66

M(1.66)= 66.62(1.66) -7.45$(1.66)^2$ KN.m=90.02KNM

- For Z=1.66 to Z=8.38m



M(x)+18.20 KN/m*x^2/2+14.90 KN/m*1.66 m*(x+1.66/2)=66.62 KN (x+1.66)

M(x)=-9.1x^2+-24.73x+-20.53+66.62x + 110.59

= -9.1x^2 + 41.89x + 90.06

At Z=1.66m, x=0

M(0)=90.06 KN.m

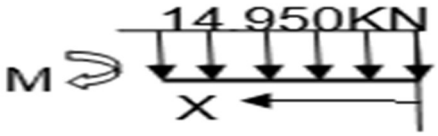
At Z=8.38m, X=6.72m

M(6.72)= -9.1(6.72)^2+41.89(6.72)+90.06 KN.m

=-410.94+281.47+90.06

M(6.72)=-39.41

- For Z=8.38 to Z=10.68 m



M(x) =-14.9 x^2/2=

= - 7.45x^2

At Z= 10. 68m, x=0

M(0)=0

At Z=4.93m, x=2.3m

M(2.3)=-14.9(2.3)^2/2 =-39.41

- Mmax (+ve) is b/n Z=1.66 to Z=8.38m

M(x)=-9.1x^2+41.87x+90.06

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

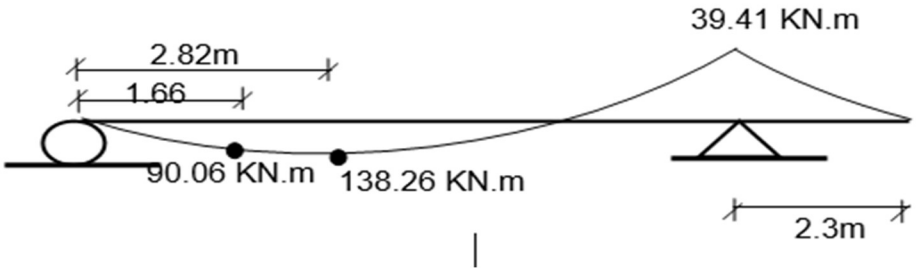
dM(x)/dx=d/dx(9.1 x^2+41.89x+90.06=0

=-18.2x+41.89=0, x=41.89/18.2m=2.3 , Z=1.66+2.30=, Z=2.82m

M(2.3m)=-9.1x^2+41.89x+90.06 KN.m

= -9.1$(2.3)^2$ + 41.88x^2 + 90.06=138.26

Bending Moment Diagram



Shear Force computation

V(x)=dM(x)/dx

- For Z=0 to Z=0.83m

M(x)=66.62x-7.45x^2

dM(x)/dx=V(x)=-14.9x+66.62

at Z=0, x= 0)

V(0)=66.62 KN

At Z=166m, x=1.66 m

V(1.66)=-14.9(1.66)+66.62 KN

- For Z=1.66m to Z=8.38m

M(x)=-9.1x^2+41.89x+90.06

dM(x)/dx=V(x)=-18.x+41.89

at Z=1.66m, x=0

V(0)=-18.2(0)+41.89=41.89

At Z=8.38m to 10.68, x=6.72m

V(6.72)=-18.2x+41.89

V(6.72)=-80.42

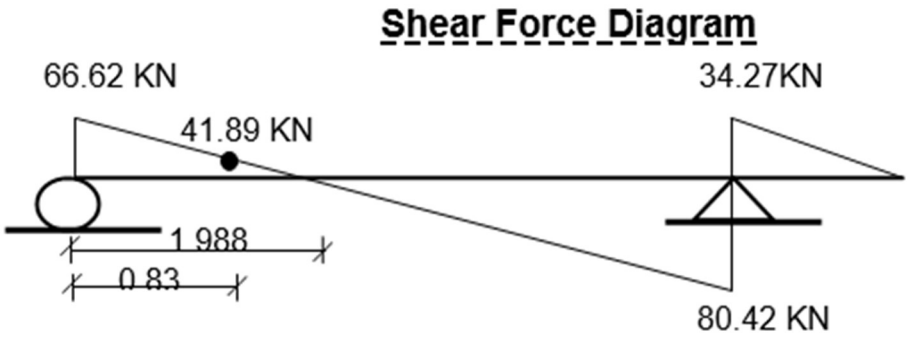
For Z=8.38m to Z=10.68m

$M(x)= -7.45x^2$

dM(x)/dx= -14.9x

at Z=8.38m, x=2.3m

V(2.3)= -14.9x2.3=-34.27



Reinforcement calculation

According to Eurocode 2 Part 1,1 - EN 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. $As, \geq 20\%\ As$,

Max design moment at support= 39.41KNm

Max design moment in the span= 138KNm

Considering maximum moment: M =138 KN.m

Material Data, C-25, S-400 $using\ \emptyset 12$

Effective depth=74.12mm=80mm

d = D –cover –Φbar/2 = 136-2-12/2 =124 mm

Width, b=1000mm

Moments calculated for each panels

Using design charts

For span moment

Using Mu = 138KN/m, calculate reinforcement,

Using general design table no.1 from ES EN1992-1-1:2015

$$K_m = \sqrt{\frac{M_{sd,s}}{bd^2}}$$

$$A_{S=} = \frac{K_s M_{sd,s}}{d}$$

Using corresponding value of Km take Ks value from the table 1a

Design moment, M=138KNm

$$K_m = \sqrt{\frac{M_{sd,s}}{bd^2}} = \sqrt{\frac{138}{1*0.124^2}} = 94.74$$

Compression reinforcement is not required.

$$K_s = 3.53$$

$$A_{Scalc} = \frac{K_s M_{sd,s}}{d} = \frac{3.53 * 138kNm}{0.124} = 3928.55mm^2$$

The minimum and maximum value of area of reinforcement is computed in accordance with the provision of the code.

$fctm = 0.3(Fck^{2/3}) = 0.3 * 20^{2/3} = 2.2$

$$A_{s,min}=Max\begin{cases}\frac{0.26fctmbtd}{fyk}=\frac{0.26*2.2*1000*124}{400}=177.32mm^2\\0.00013btd=0.00013*1000*124=16.12\end{cases}$$ .....(2015) $Art.9.2.1.1\ (1)$

$$Asreq=Max\begin{cases}A_{s,calc}=3928.55mm^2\\A_{s,min}=177.32mm^2\end{cases}$$

Ast, max=0.04*Ac……...(2015) Art.9.2.1.1 (3)

$$a_{s=}\frac{\pi d^2}{4}=\frac{3.14*10^2}{4}=78.5mm^2$$

$$Spacing=\frac{\text{b}*\text{as}}{\text{Asreq}}=\frac{1000*78.5}{3928.55mm^2}=20\text{ mm}$$

For slab maximum spacing is defined in section 9.3.1.1(3) of ES EN 1992-1-1

$$Sprovide=min\begin{cases}3D=3*150=450mm\\400mm\\Spacing=96mm\end{cases}$$ ……..(2015)

Provide, Ø12 C/C 90mm

Transverse reinforcement

According to Euro code 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.

- $As, \geq 20\%\ As,$

$A_{s,} \geq 0.2*$ 3928.55 =785.71mm2

Therefore take the minimum reinforcement

d=150-20-8-4=118mm, for Ø8 main transverse rebar

A_s=785.71mm^2

$$A_{s,min} = \frac{0.26fctmbtd}{fyk} = \frac{0.26 * 2.2 * 1000 * 124}{400} = 177.32mm2 < A_{s,} = 785.71mm2$$

$$a_{s=} \frac{\pi d^2}{4} = \frac{3.14 * 8^2}{4} = 50.24mm^2$$

$$Spacing = \frac{b * as}{Asreq} = \frac{1000*50.24}{785.71} = 160mm$$

Provide, Ø8 C/C 160mm

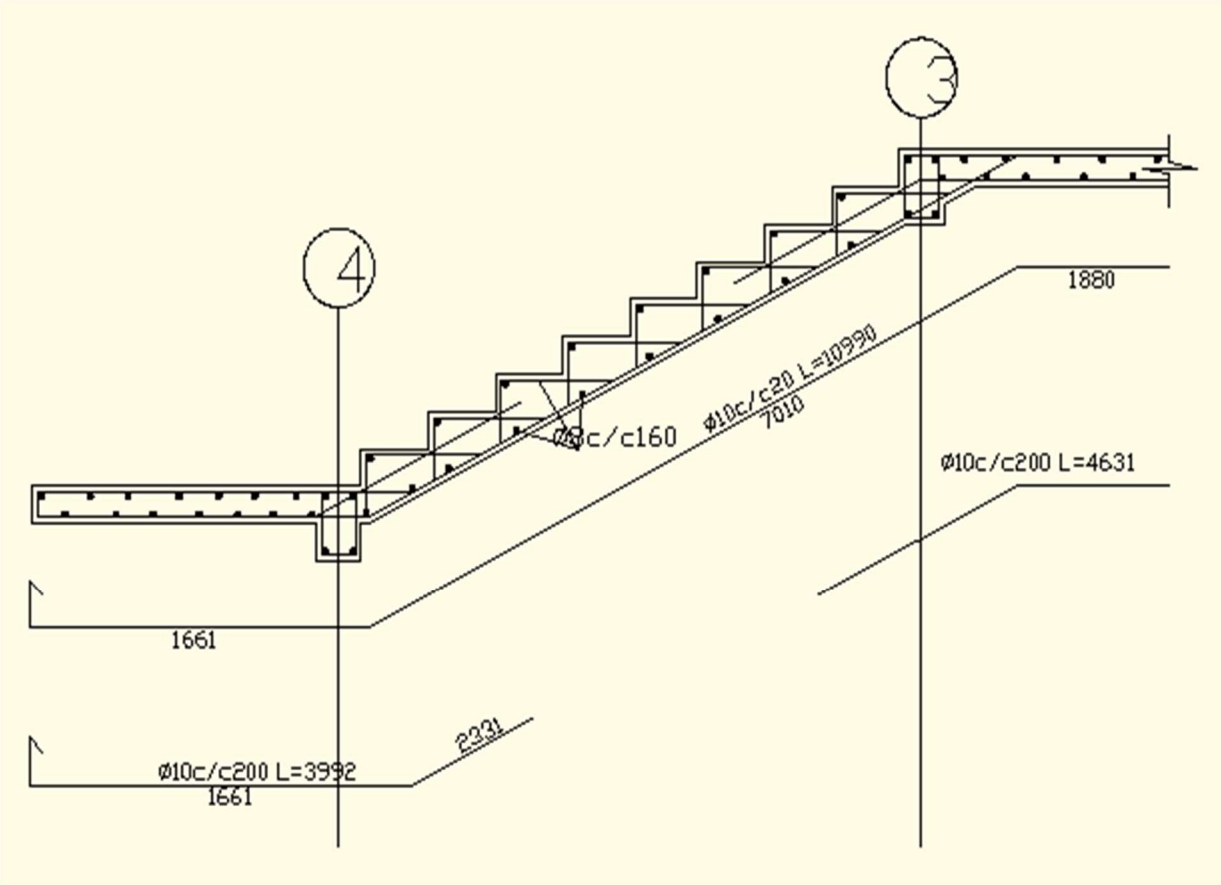


Figure 16 detail reinforisment for stair case design

2.2.8 Earthquake load analysis

An earthquake is the vibration of earth produced by the rapid release of accumulated energy in elastically strained rock. The building is to be constructed in Adama, which is located in Longitude 39.5565-E and Latitude 8.5386 -N. it is categorized seismic hazard zone four.

- Analysis of earthquake loads on building on (2015)ES EN 2015

The objective of seismic design in accordance with ES EN 1998-1:2015 is explicitly stated. Its purpose is to ensure that in the event of earthquakes. For human lives are protected, Damage is limited and Structures important for civil protection remain operational. Ø Distribution of the base shear over the height of the building

The horizontal story shear force determined above shall be distributed to the lateral load resisting system based on the assumption that the slab floor at every level are rigid enough. The storey shear force will be distributed to each frame system according to their stiffness, the procedures in the story shear force distribution include:

- Determination of center of mass for each floors
- Determination of center of stiffness for each floors
- Determination of eccentricity for each floors
- Calculation of the direct shear force for each frame system
- Weight and Center of mass calculation

Summary of weight (refer from in a tabular form appendix C)

a) Foundation

- Weight of footing column=336 KN
- Weight of footing pad=10% weight of footing column=33.6 KN
- Total weight =369.6 KN

b) Ground Floor

- Weight of grade beam=1373.86 KN
- ½(weight of ground floor column=336 KN
- Weight of wall=3489.1969KN
- Weight of staircase=170.87 KN
- Total weight=5369.93KN

c) First Floor

- Weight of slab=5129.27 KN
- Weight of beam=1194.528 KN
- ½ (weight of ground floor column)=336 KN
- ½(weight of first floor column)=259.2 KN
- Weight of wall=3856.61 KN
- Weight of staircase=170.87 KN
- Total weight=10946.48 KN

d) Second Floor

- Weight of slab=4889.37 KN
- Weight of beam=1381.35 KN
- ½(weight of 1^{st} floor column)=259.2 KN
- ½(weight of 2^{nd} floor column)=300 KN
- Weight of wall=2775.13 KN
- Weight of Stair Case=170.87 KN
- Total Weight =9775.95 KN

e) Third Floor

- Weight of slab=5200.2 KN
- Weight of beam=1381.35KN
- ½(weight of 2nd floor column)=300KN
- ½(weight of 3rd floor column)=300 KN
- Weight of wall=3092.387 KN
- Weight of staircase=170.87 KN
- Total weight=10444.807 KN

f) Fourth Floor

- -Weight of slab=5200.2 KN
- -Weight of beam=1381.35KN
- -½(weight of 2nd floor column) =300 KN
- -½(weight of 3rd floor column) =300 KN
- -Weight of wall=3092.387 KN
- -Weight of staircase=170.87 KN
- -Total weight=10444.807KN

g) Roof Level

- Weight of top tie beam=200.62 KN
- 1/2(weight of 4th floor column)=300 KN
- Total weight=500.62KN

General in every building, the earth quake and wind load effect should be considered as a part of load in order to build a structure that can safely transfer the loads to the foundation and finally to the ground and absorb some of the energy present rather than suffering damage.

The objective of seismic design in accordance with ES EN 1998-1:2015 is explicitly stated. Its purpose is to ensure that in the event of earthquakes. For Human lives are protected, Damage is limited and Structures important for civil protection remain operational. Since our structure is "regular structure" means, we assume that our building has uniform distribution of mass and stiffness; therefore, we select static analysis (coefficient method)

Design Basis:

- Ground condition: (Assume sub soil class C)
- Seismic zone: (Adama, Zone 4)

Table 11 Weight according to story level

Story	Floor	Wi (KN)	mass(kg)
W1	F+G	5739.53	585.07
W2	1st	10946.48	1115.85
W3	2nd	9775.9	996.52
W4	3rd	10444.8	1064.71
W5	4th	10444.8	1064.71
W6	Roof	500.6	51.03
Total Weight of the Building		478522	4785.2

Determination of design earthquake force

a) Determination of base shear
b) Distribution of base shear over the height of the building

Base Shear is an estimate of the maximum expected lateral force that will occur due to seismic ground motion at the base of a structure. Calculation of base shear (V) depends on:

- Soil condition at the site is C Proximity to potential sources of seismic activity
- The level of ductility of the building.

Fb = Sd(T1) * m * λ……(ES EN, 2003:4.3.3.2.2 Eq(4.5))

Where, Sd(T1) design spectrum at period T1

T1 = Fundamental period of vibration

λ= correction factor

$$\lambda = \begin{cases} 0.85. \text{ If } T1 <= 2Tc, \text{ building more than twostory} \\ 1, \text{ other wise} \end{cases}$$

λ= correction factor=0.85 the case satisfied

ES EN 1998-1:2015

Table 3.3: Values of the parameters describing the recommended Type 2 elastic response spectra

Ground type	S	T_B(s)	T_C(s)	T_D(s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

Identification of Ground type

The ground type for in our case is type B, hence the

The values of the Parameters from type 1 elastic response spectra are

Ground type	S	$T_{B(S)}$	$T_{C(S)}$	$T_{D(S)}$
C	1.15	0.20	0.6	2.0

For the determination of the fundamental period of vibration $T1$ of the building, expressions based on methods of structural dynamics.

T1 = Ct ·H ¾ =0.63sec

- Ct is 0.075 for moment resistant space concrete frames
- H is17m the height of the building,

Calculation of the design spectrum depend on the following criteria-

- The fundamental vibration period(T1)

- The design ground acceleration on type Aground (ag = γI.agR);
- The lower limit of the period of the constant spectral acceleration branch ;(TB)
- The upper limit of the period of the constant spectral acceleration branch ;(TC)
- The value defining the beginning of the constant displacement response range of the spectrum ;(TD)
- The soil factor;(S)
- The damping correction factor with a reference value of η = 1

The site and the building properties are

- The ground type of the site is C and
- The important class our building is II from ESEN 1998-1:2015 page (53) Table4.3
- The building is frame system
- The building is Ordinary buildings αu/α1=1;

Table D1: Bedrock Acceleration Ratio α_o

Zone	5	4	3	2	1	0
$\alpha_o = a_g/g$	0.20	0.15	0.10	0.07	0.04	0

- Seismic zone of the site is zone four which is Adama Where αo = ag/g = 0.15

Therefore, Ag = α_0 * g = 0.15 * 9.81 = 1.472

For structural type frame system, dual system, Coupled Wall system; 3.0

αu/α1 q= 3.0 αu/α1 =3 * 1 = 3

Sd (T) = { = ag * S * 2.5/q * (Tc/T) = 1.34 ; ≥ β * ag = 0.2 * 1.472= 0.29

Therefore, Sd (T1) = 1.34

- Determination of Base Shear

Fb= Sd (T1) * m * λ, λ= 0.85, since T1≤2Tc = 0.63 ≤ 2 * 0.5 = 1

m=47852.2 KN

Fb = 1.34 * 4785.2 * 0.85 = 4498.1KN

Distribution of base Shear over the Height of the Building

$Fi = ((Zi \times Mi)/(\sum Mj * Zj)$ From ESEN1998-1:2015 page (58)

Where,

- Fi is the horizontal force acting on story i;
- Fb is the seismic base shear i
- zi, zj are the heights of the masses mi,mj above the level of application of the seismic action (foundation or top of a rigid basement).

Table 12 Distribution of base shear over the building height

story	Remark	floor	Weight	Mi	hi	Mihi	mihi/∑mihi	fb	fb * mihi/∑mihi
1st	W1	F+G	5739.53	585.07	2	1170.14	0.025	4498.1	360.71
2nd	W2	1st	10946.48	1115.85	6	6695.09	0.144	4498.1	292.42
3rd	W3	2nd	9775.9	996.52	9	8968.72	0.193	4498.1	1088.46
4th	W4	3rd	10444.8	1064.71	12	12776.51	0.275	4498.1	1145.72
5th	W5	4th	10444.8	1064.71	15	15970.64	0.343	4498.1	1432.15
6th	W6	Roof	500.6	51.03	18	918.53	0.020	4498.1	178.63
			∑ mihi			46499.64			

Determination of center of stiffness (Xs, Ys) of the floor.

It is a point where the stiffness or strength of the floor is concentrated.

$Xs=\Sigma Diy*Xi/\Sigma Diy$ $Ys=\Sigma DiX*Yi/\Sigma Dix$

Where,

Xs=center of stiffness of the floor.

Ys=center of stiffness of i^{th} element (beam, column).

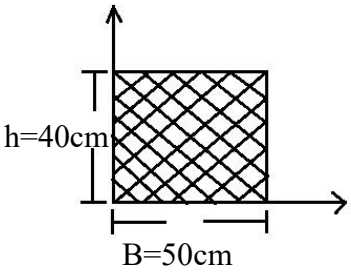
- Dix=aKiXc
- $Kxc=iyc/L=X^3Y/12L$
- Diy=aKiyc
- $Kyc=ixc/L=XY^3/12L$

Where, a=factor depending on boundary condition

Kc=column stiffness

Column Stiffness (Kxc & Kyc)

a) Rectangular columns



- Foundation column
- L=2m
- $Kxc=iyc/L=b^3h/12L=(50\ cm)^3*40\ cm)^3/(12*200\ cm)=2083.3\ cm^3$

- Kyc=ixc/L=$bh^3/12L$=Kxc=2083.3 cm^3

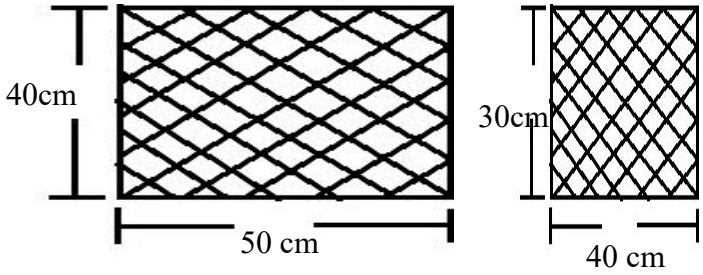
- 1st Floor Column
- L=4m
- Kxc=iyc/L=$b^3h/12L$=$(50\ cm)^3*(40\ cm)^3/(12*400\ cm)$=1041.7$cm^3$
- Kyc=ixc/L=$bh^3/12L$=Kxc=1041.7 cm^3
- 2nd, 3rd & 4th Floor Columns
- L=3m
- Kxc=iyc/L=$b^3h/12L$=$(50\ cm)^3*(40\ cm)^3/(12*300\ cm)$=1388.9 cm^3
- Kyc=ixc/L=$bh^3/12L$=Kxc=1388.9 cm^3

b) Circular columns

- D=40 cm
- Kxc=Kyc=$\pi D^4/64L$
- Foundation Columns
- L=2m
- Kxc= $\pi D^4/64L$= $\pi*(40\ cm)^4/64*200\ cm$=628.32 cm^3
- Kyc=Kxc= 628.32 cm^3
- 1st Floor Column
- L=4m
- Kxc= $\pi D^4/64L$= $\pi*(40\ cm)^4/64*400\ cm$=314.16 cm^3
- 2nd, 3rd , & 4th Floor columns
- L=3m
- Kxc= $\pi D^4/64L$= $\pi*(40\ cm)^4/64*300cm$=418.87 cm^3
- Kyc=Kxc=418.87 cm^3

Beam Stiffness

Type I Type II

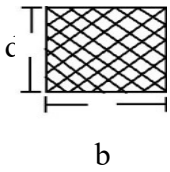


Grade Beam and Intermediate beam Top tie beam

In the fig below beams are named using letters & numbers the letter represents their type & the number represents beams with identical length.

- G-Grade Beam
- I-Intermediate beam
- T-top the beam

- Beam stiffness is;



Kbi=i/L ,

Kbi=$bd^3/12L$

- Grade Beams
  - b=50cm
  - d=40 cm
- KGi=50cm * $(40cm)^3/12$Li=266,666.6/Li $(cm)^3$

In tabular form

Beam	G1	G2	G3	G4	G5	G6	G7	G8	G9	G10	G11	G12
Li (cm)	150	561	652	652	280	576	150	664	917	798	1065	150
KGi=266,666.6 (cm3)/Li	1778	475	409	409	952	463	1778	402	291	334	250	1778

- intermediate Beam
  - b=50 cm
  - d=40 cm
  - Ki_i=50 cm*(40 cm)3/12L_i=266,666.6 (cm^3)/Li

In tabular form

Beam	i1	i2	i3	i4	i5	i6	i7	i8	i9	G10	G11	G12
Li (cm)	150	561	652	652	280	576	150	664	917	798	1065	150
Ki_i=266,666.6 (cm3)/Li	1778	475	409	409	952	463	1778	402	291	334	250	1778

- Top tie beam
  - b=40 cm
  - d=30 cm
  - KTi=(40cm)*(30 cm)^3/12*Li=90000 (cm^3)/Li

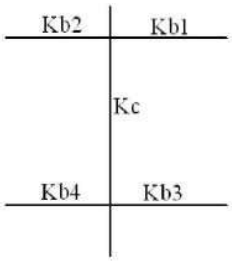
In tabular form

Beam	T1	T2	T3	T4	T5	T6	T7	T8	T9	G10	G11	G12
Li (cm)	150	561	652	652	280	576	150	664	917	798	1065	150
KTi=90,000 (cm3)/Li	600	160	138	138	321	156	600	136	309	113	85	600

For frame other than fixed column base (refer from in a tabular form appendix C)

- $K_i=\frac{\Sigma Kbj \text{ (upper \& lower beam)}}{2Kci}$
- $a_i=\frac{K'}{2+k'}$
- $D_i=a_iKc_i$
- Where: - a = constant determined by
- KC = Stiffness ratio of column

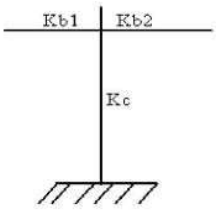
Case-1 General



$a=\frac{\bar{K}}{2+\bar{K}}$

$\bar{K}=\frac{\Sigma K_b}{2K_c}$

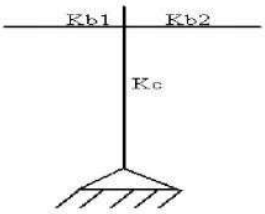
case-2 One end fixed



$a=\frac{0.5+\bar{K}}{2+\bar{K}}$

$\bar{K}=\frac{\Sigma K_b}{K_c}$

case-3 One end pin supported



$a=\frac{0.5\bar{K}}{2+\bar{K}}$

$\bar{K}=\frac{\Sigma K_b}{1+2K_c}$

Kt_1	600	Kt_2	160	Kt_3	138	Kt_5	321

K_c	1388.9	K_c	1388.9	K_c	1388.9		
K'	1.084671323	K'	0.425516596	K'	0.393836849		
a	0.351632705	a	0.175433389	a	0.164521174		
D	488.3826646	D	243.6594336	D	228.5034589	∑D	960.5455571
KI_1	1778	KI_2	475	KI_3	409	KI_5	952

K_c	1388.9	K_c	1388.9	K_c	1388.9		
K'	1.622147023	K'	0.636474908	K'	0.588955288		
a	0.447841298	a	0.241411328	a	0.227487624		
D	622.0067782	D	335.2961931	D	315.9575616	∑D	1273.260533
KI_1	1778	KI_2	475	KI_3	409	KI_5	952

K_c	1388.9	K_c	1388.9	K_c	1388.9		
K'	1.622147023	K'	0.636474908	K'	0.588955288		
a	0.447841298	a	0.241411328	a	0.227487624		
D	622.0067782	D	335.2961931	D	315.9575616	∑D	1273.260533
KI_1	1778	KI_2	475	KI_3	409	KI_5	952

K_c	1388.9	K_c	1388.9	K_c	1388.9		
K'	1.622147023	K'	0.636474908	K'	0.588955288		
a	0.447841298	a	0.241411328	a	0.227487624		
D	622.0067782	D	335.2961931	D	315.9575616	∑D	1273.260533
KI_1	1778	KI_2	475	KI_3	409	KI_5	952

K_c	1041.7	K_c	1041.7	K_c	1041.7		
K'	2.16281079	K'	0.848612844	K'	0.785254872		
a	0.519555392	a	0.297903889	a	0.28193286		

	D	541.2208514	D	310.3264811	D	293.6894603	∑D 1145.236793
KG_1 1778		KG_2 475		KG_3 409		KG_5 952	
	K_c	2083.3	K_c	2083.3	K_c	2083.3	
	K'	1.081457303	K'	0.424326789	K'	0.392646282	
	a	0.350956446	a	0.175028709	a	0.164105445	
	D	731.1475637	D	364.6373104	D	341.8808731	∑D 1437.665747

Diagram of Determination of K',a and D Along X-axis on Axis 1-1 and 2-2

Kt_{12} 600 Kt_{11} 85 Kt_{10} 113

	K_c	418.87	K_c	1388.9	K_c	1388.9
	K'	3.238474944	K'	0.281517748	K'	0.160918713
	a	0.618209494	a	0.123390558	a	0.074467731
	D	258.9494107	D	171.3771459	D	103.4282311 ∑D 533.7547878
KI_{12}	1778		KI_{11}	250	KI_{10}	334
	K_c	418.87	K_c	1388.9	K_c	1388.9
	K'	4.841597632	K'	0.420476636	K'	0.240478076
	a	0.707670619	a	0.173716461	a	0.107333376
	D	296.4219922	D	241.2747933	D	149.0753262 ∑D 686.7721117
KI_{12}	1778		KI_{11}	250	KI_{10}	334
	K_c	418.87	K_c	1388.9	K_c	1388.9
	K'	4.841597632	K'	0.420476636	K'	0.240478076
	a	0.707670619	a	0.173716461	a	0.107333376
	D	296.4219922	D	241.2747933	D	149.0753262 ∑D 686.7721117
KI_{12}	1778		KI_{11}	250	KI_{10}	334
	K_c	418.87	K_c	1388.9	K_c	1388.9
	K'	4.841597632	K'	0.420476636	K'	0.240478076
	a	0.707670619	a	0.173716461	a	0.107333376
	D	296.4219922	D	241.2747933	D	149.0753262 ∑D 686.7721117
KI_{12}	1778		KI_{11}	250	KI_{10}	334
	K_c	314.16	K_c	1041.7	K_c	1041.7
	K'	6.455309396	K'	0.56062206	K'	0.32062974
	a	0.763462233	a	0.218939792	a	0.138164971
	D	239.8492953	D	228.0695809	D	143.9264499 ∑D 611.845326

KG_{12} 1778		KG_{11} 250		KG_{10} 334	
K_c	628.32	K_c	2083.3	K_c	2083.3
K'	3.227654698	K'	0.280324485	K'	0.160322565
a	0.617419261	a	0.12293184	a	0.074212327
D	387.9368698	D	256.1039027	D	154.6065414 ∑D 798.6473138

Diagram of Determination of K',a and D Along Y-axis on Axis E-E

Kt_{12}	600	Kt_{11}	85	Kt_{10}	113	
K_c	1388.9	K_c	1388.9	K_c	1388.9	
K'	0.976672187	K'	0.281517748	K'	0.160918713	
a	0.328108749	a	0.123390558	a	0.074467731	
D	455.7102412	D	171.3771459	D	103.4282311	∑D 730.5156182
KI_{12}	1778	KI_{11}	250	KI_{10}	334	
K_c	1388.9	K_c	1388.9	K_c	1388.9	
K'	1.460148319	K'	0.420476636	K'	0.240478076	
a	0.421990095	a	0.173716461	a	0.107333376	
D	586.1020434	D	241.2747933	D	149.0753262	∑D 976.4521628
KI_{12}	1778	KI_{11}	250	KI_{10}	334	
K_c	1388.9	K_c	1388.9	K_c	1388.9	
K'	1.460148319	K'	0.420476636	K'	0.240478076	
a	0.421990095	a	0.173716461	a	0.107333376	
D	586.1020434	D	241.2747933	D	149.0753262	∑D 976.4521628
KI_{12}	1778	KI_{11}	250	KI_{10}	334	
K_c	1388.9	K_c	1388.9	K_c	1388.9	
K'	1.460148319	K'	0.420476636	K'	0.240478076	
a	0.421990095	a	0.173716461	a	0.107333376	
D	586.1020434	D	241.2747933	D	149.0753262	∑D 976.4521628
KI_{12}	1778	KI_{11}	250	KI_{10}	334	
K_c	1041.7	K_c	1041.7	K_c	1041.7	

K'	1.946817702	K'	0.56062206	K'	0.32062974
a	0.493262636	A	0.218939792	a	0.138164971
D	513.8316875	D	228.0695809	D	143.9264499 ∑D 885.8277183
KG_{12} 1778		KG_{11} 250		KG_{10} 334	
K_c	2083.3	K_c	2083.3	K_c	2083.3
K'	0.973455575	K'	0.280324485	K'	0.160322565
a	0.327381913	A	0.12293184	a	0.074212327
D	682.0347399	D	256.1039027	D	154.6065414 ∑D 1092.745184

Diagram of Determination of K',a and D Along Y-axis on Axis D-D

Then $Xs=\frac{\Sigma Diy\ Xi}{\Sigma Diy}$, $Ys=\frac{\Sigma Dix\ Yi}{\Sigma Dix}$

Calculation of center of stiffness for 1st story

Axis	Dx	y	Dx*y
5	1872.99	0	0.00
4	2432.68	10.65	25908.06
3	2526.83	18.63	47074.75
2	1437.67	27.8	39967.11
1	1437.66	34.44	49513.01
SUM =	**9707.82**		**162462.94**

Axis	DY	X	Dy*X
A	1928.46	0	0.00
B	2222.56	5.61	12468.56
C	2165.81	12.13	26271.28
D	1092.75	18.65	20379.69
E	798.65	24.41	19494.97
SUM =	**8208.22**		**78614.50**

Ys =Σ(Dx*y)/ΣDx= **16.73526652**

Xs = Σ(Dy*x)/ΣDy = **9.577529269**

Calculation of center of stiffness for 2nd story

Axis	Dx	y	Dx*y
5	1402.39	0.00	0.00
4	1876.42	10.65	19983.84
3	1992.43	18.63	37118.90
2	1145.24	27.80	31837.67
1	1145.24	34.44	39442.07
SUM =	**7561.71**		**128382.48**

Axis	DY	X	Dy*X
A	1500.97	0	0.00
B	1774.95	5.61	9957.47
C	1698.22	12.13	20599.41
D	885.83	18.65	16520.69
E	611.85	24.41	14935.14
SUM =	**6471.81**		**62012.71**

Ys = Σ(Dx*y)/ΣDx = **16.98**

Xs = Σ(Dy*x)/ΣDy = **9.58**

Calculation of center of stiffness for 3rd,4th and 5th story

Axis	Dx	y	Dx*y
5	1593.31	0.00	0.00
4	2116.25	10.65	22538.02
3	2225.27	18.63	41456.72
2	1273.26	27.80	35396.63
1	1273.26	34.44	43851.07
SUM =	**8481.34**		**143242.44**

Axis	DY	X	Dy*X
A	1680.83	0	0.00
B	1970.51	5.61	11054.56
C	1901.29	12.13	23062.67
D	976.45	18.65	18210.83
E	686.77	24.41	16764.10
SUM =	**7215.86**		**69092.17**

Ys =Σ(Dx*y)/ΣDx= 16.88912308

Xs = Σ(Dy*x)/ΣDy = 9.58

Calculation of center of stiffness for 6th story

Axis	Dx	y	Dx*y
5	1250.95	0	0
4	1625.14	10.65	17307.69
3	1688.18	18.63	31450.79
2	960.55	27.8	26703.18
1	960.55	34.44	33081.20
SUM =	6485.36		108542.86

Axis	DY	X	Dy*X
A	1364.77	0	0.00
B	1561.53	5.61	8760.18
C	1523.45	12.13	18479.45
D	730.52	18.65	13624.10
E	533.76	24.41	13028.96
SUM =	5714.02		53892.70

Ys = Σ(Dx*y)/ΣDx = 16.74

Xs = Σ(Dy*x)/ΣDy = 9.43

Computation of frame shear, Qi

$$Qi=\frac{Di\ frame}{\Sigma Di\ frame}*V$$,where v is a storey shear

Computation of frame shear in Y-direction

Axis	Story	Diy	Σ(Diy)	V	Qi
A		1928.46	8208.22	360.71	84.75
B		2222.56	8208.22	360.71	97.67
C	1st	2165.81	8208.22	360.71	95.18
D		1092.75	8208.22	360.71	48.02
E		798.65	8208.22	360.71	35.10
A		1500.97	6471.81	292.42	67.82
B		1774.95	6471.81	292.42	80.20
C	2nd	1698.22	6471.81	292.42	76.73
D		885.83	6471.81	292.42	40.02
E		611.85	6471.81	292.42	27.65
A		1680.83	7215.86	1088.46	253.54
B		1970.51	7215.86	1088.46	297.24
C	3rd	1901.29	7215.86	1088.46	286.80
D		976.45	7215.86	1088.46	147.29
E		686.77	7215.86	1088.46	103.59
A		1680.83	7215.86	1145.72	266.88
B		1970.51	7215.86	1145.72	312.87
C	4th	1901.29	7215.86	1145.72	301.88
D		976.45	7215.86	1145.72	155.04
E		686.77	7215.86	1145.72	109.04
A		1680.83	7215.86	1432.15	333.60
B		1970.51	7215.86	1432.15	391.09

C	5th	1901.29	7215.86	1432.15	377.35
D		976.45	7215.86	1432.15	193.80
E		686.77	7215.86	1432.15	136.31
A		1364.77	5714.02	178.63	42.67
B		1561.53	5714.02	178.63	48.82
C	6th	1523.45	5714.02	178.63	47.63
D		730.52	5714.02	178.63	22.84
E		533.76	5714.02	178.63	16.69

Computation of frame shear in X-direction

Axis	Story	DiX	Σ(DiX)	V	Qi
5		1872.99	9707.82	360.71	69.59
4		2432.68	9707.82	360.71	90.39
3	1st	2526.83	9707.82	360.71	93.89
2		1437.67	9707.82	360.71	53.42
1		1437.66	9707.82	360.71	53.42
5		1402.39	7561.71	292.42	54.23
4		1876.42	7561.71	292.42	72.56
3	2nd	1992.43	7561.71	292.42	77.05
2		1145.24	7561.71	292.42	44.29
1		1145.24	7561.71	292.42	44.29
5		1593.31	8481.34	1088.46	204.48
4		2116.25	8481.34	1088.46	271.59
3	3rd	2225.27	8481.34	1088.46	285.58
2		1273.26	8481.34	1088.46	163.40
1		1273.26	8481.34	1088.46	163.40

5		1593.31	8481.34	1145.72	215.24
4		2116.25	8481.34	1145.72	285.88
3	4th	2225.27	8481.34	1145.72	300.60
2		1273.26	8481.34	1145.72	172.00
1		1273.26	8481.34	1145.72	172.00
5		1593.31	8481.34	1432.15	269.04
4		2116.25	8481.34	1432.15	357.35
3	5th	2225.27	8481.34	1432.15	375.76
2		1273.26	8481.34	1432.15	215.00
1		1273.26	8481.34	1432.15	215.00
5		1250.95	6485.36	178.63	34.46
4		1625.14	6485.36	178.63	44.76
3	6th	1688.18	6485.36	178.63	46.50
2		960.55	6485.36	178.63	26.46
1		960.55	6485.36	178.63	26.46

Determination of eccentricities, ei

a) Actual eccentricities, e_i.....(2020a)

- $e_x=x_s-x_m$, where x_s & y_s-center of stiffness
- $e_y=y_s-y_m$, x_m & y_m- center of mass.

Determination of eccentricities

ex =Xs-Xm ey = Ys-Ym

Story	Xs	Ys	Xm	Ym	ex	ey
1st	9.58	16.74	13.71	18.72	-4.13	1.99
2nd	9.58	16.98	13.71	18.72	-4.12	1.74
3rd	9.58	16.89	13.71	18.72	-4.13	1.83
4th	9.58	16.89	13.71	18.72	-4.13	1.83
5th	9.58	16.89	13.71	18.72	-4.13	1.83
6th	9.43	16.74	13.71	18.72	-4.28	1.98

b) Accidental eccentricities e_{li} and design eccentricity……..(2020a)

- Is an additional eccentricity to the actual eccentricity for uncertainties in locating masses and spatial variation
- $e_{li}=\pm 0.05\ L_i$

  where Li=floor dimension perbndicular to the direction of seismic action.

  $e_{lx}=\pm 0.05\ L_y$

  $e_{ly}=\pm 0.05\ L_x$

- then the design eccentricity, $ed_{tot}=e\pm e_{li}$

Accidental eccentricities

eli = -+.05L_i.....

.(2020a)

along x-axis ; e_{lx}= -+.05*27.4=-+1.37

along y-axis ; e_{ly}= -+.05*37.4= -+1.87

Design eccentricities

ed_{tot} = e-+e_{li}

Story	e_x	e_y	ed_{x1}	ed_{x2}	ed_{y1}	ed_{y2}
1st	-4.13	1.99	-2.76	-5.50	3.86	0.12
2nd	-4.12	1.74	-2.75	-5.49	3.61	-0.13
3rd	-4.13	1.83	-2.76	-5.50	3.70	-0.04
4th	-4.13	1.83	-2.76	-5.50	3.70	-0.04
5th	-4.13	1.83	-2.76	-5.50	3.70	-0.04

6^{th}	-4.28	1.98	-2.91	-5.65	3.85	0.11

Determination of shear correction factor for torsion refer from appendix(ES EN 1992)

- $\alpha ix=\frac{(1+(Dix)ey_{tot}*Yi')}{Jr}$……(2020a)
- $\alpha iy=\frac{(1+(Diy)ex_{tot}*xi')}{Jr}$……..(2020a)
- $Jr=Jx+Jy=\sum DixYi'^{2}+\sum DiyXi'^{2}$……
- Xi'=Xs-Xi

  Yi'=Ys-Yi

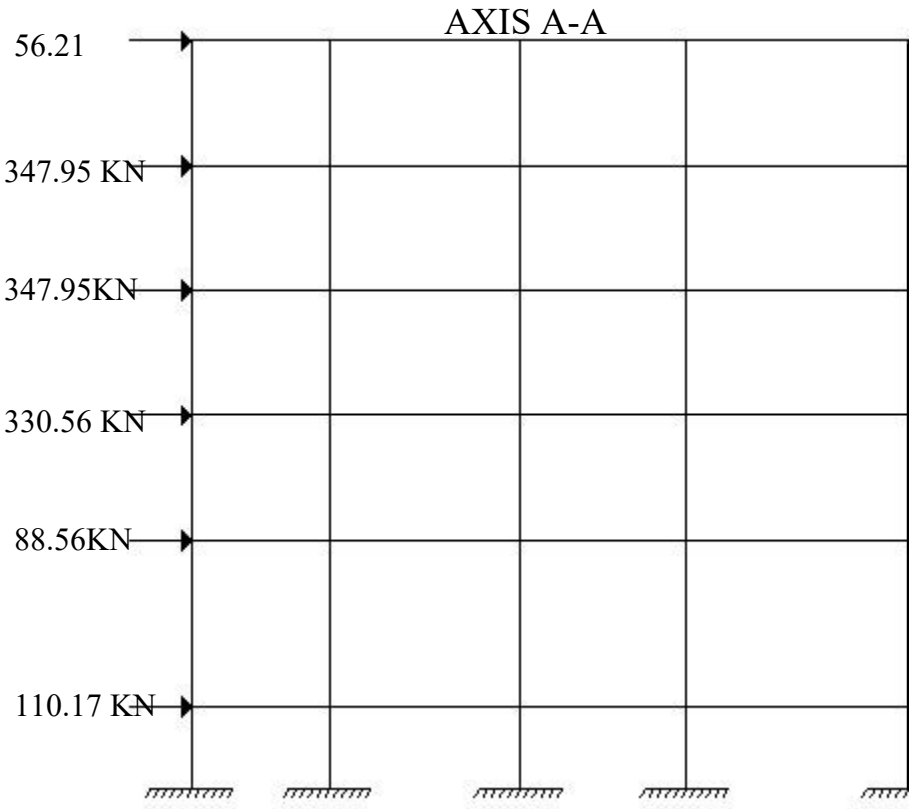
Calculation of total shear

(Lateral force)

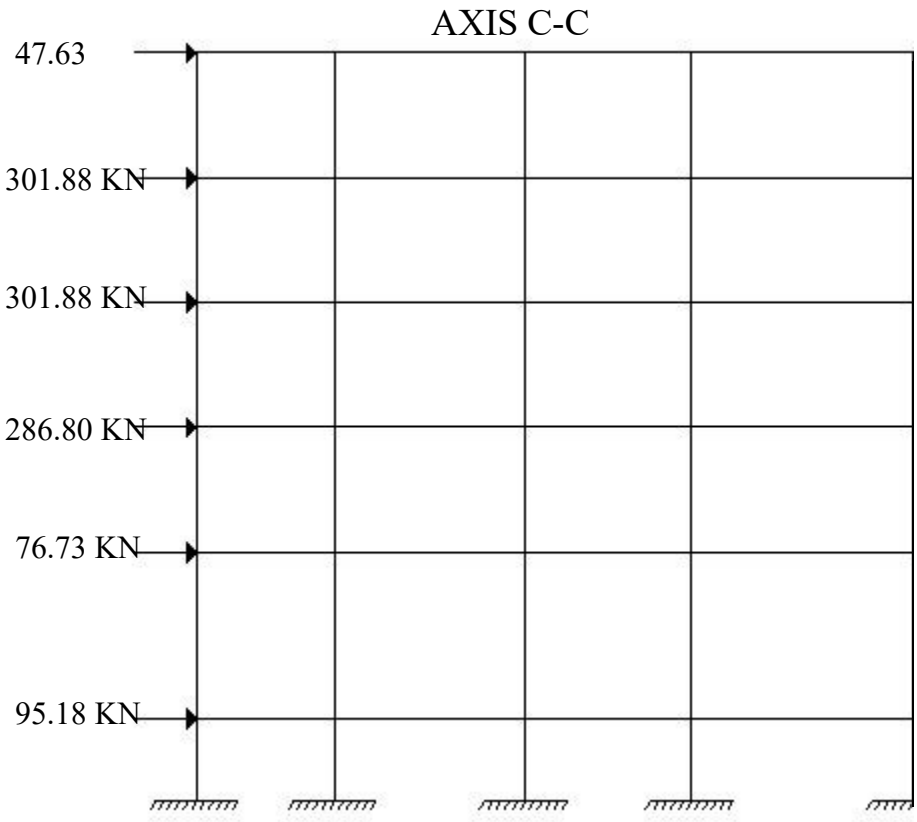
story	Frame on axis	Qi	αx	Corrected force(Fi=αx* Qi)
	5	69.59	0.99	68.80
	4	90.39	1.00	89.95
	3	93.89	1.00	93.89
	2	53.42	1.19	63.47
1st	1	53.42	1.32	70.75
	A	84.75	1.30	110.17
	B	97.67	1.16	113.42
	C	95.18	1.00	95.18
	D	48.02	0.92	44.26
	E	35.10	0.85	29.85
	5	54.23	1.01	54.94
	4	72.56	1.01	72.97
	3	77.05	1.00	77.05
	2	44.29	1.18	52.15
	1	44.29	1.31	57.85
	A	67.82	1.31	88.56
	B	80.20	1.16	93.38
	C	76.73	1.00	76.73
2nd	D	40.02	0.92	36.83
	E	27.65	0.85	23.42
	5	204.48	1.00	205.29

	4	271.59	1.00	272.05
	3	285.58	1.00	285.58
	2	163.40	1.18	193.10
3rd	1	163.40	1.31	214.61
	A	253.54	1.30	330.56
	B	297.24	1.16	345.77
	C	286.80	1.00	286.80
	D	147.29	0.92	135.62
	E	103.59	0.85	87.91
	5	215.24	1.00	216.09
	4	285.88	1.00	286.37
	3	300.60	1.00	300.60
	2	172.00	1.18	203.26
4th	1	172.00	1.31	225.90
	A	266.88	1.30	347.95
	B	312.87	1.16	363.96
	C	301.88	1.00	301.88
	D	155.04	0.92	142.76
	E	109.04	0.85	92.54
	5	269.04	1.00	270.12
	4	357.35	1.00	357.96
	3	375.76	1.00	375.76
	2	215.00	1.18	254.08
5th	1	215.00	1.31	282.37
	A	266.88	1.30	347.95
	B	312.87	1.16	363.96
	C	301.88	1.00	301.88

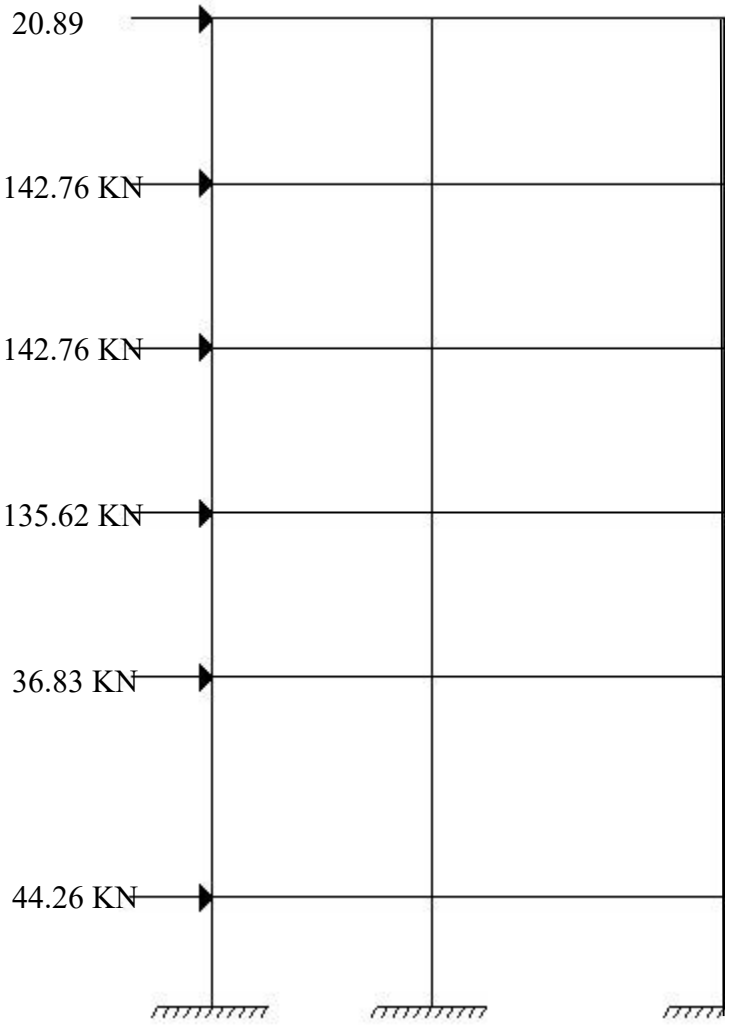
	D	155.04	0.92	142.76
	E	109.04	0.85	92.54
	5	34.46	0.99	34.07
	4	44.76	1.00	44.55
	3	46.50	1.00	46.50
	2	26.46	1.19	31.38
6th	1	26.46	1.32	34.94
	A	42.67	1.32	56.21
	B	48.82	1.17	57.14
	C	47.63	1.00	47.63
	D	22.84	0.91	20.89
	E	16.69	0.84	13.97



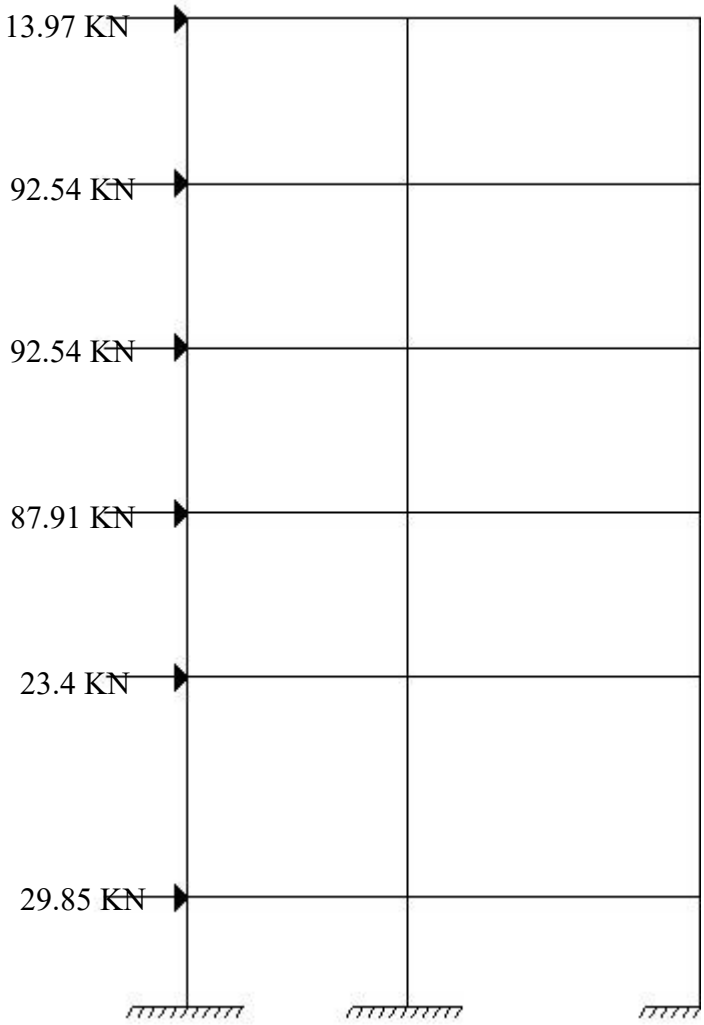
Earthquake Load on Frames

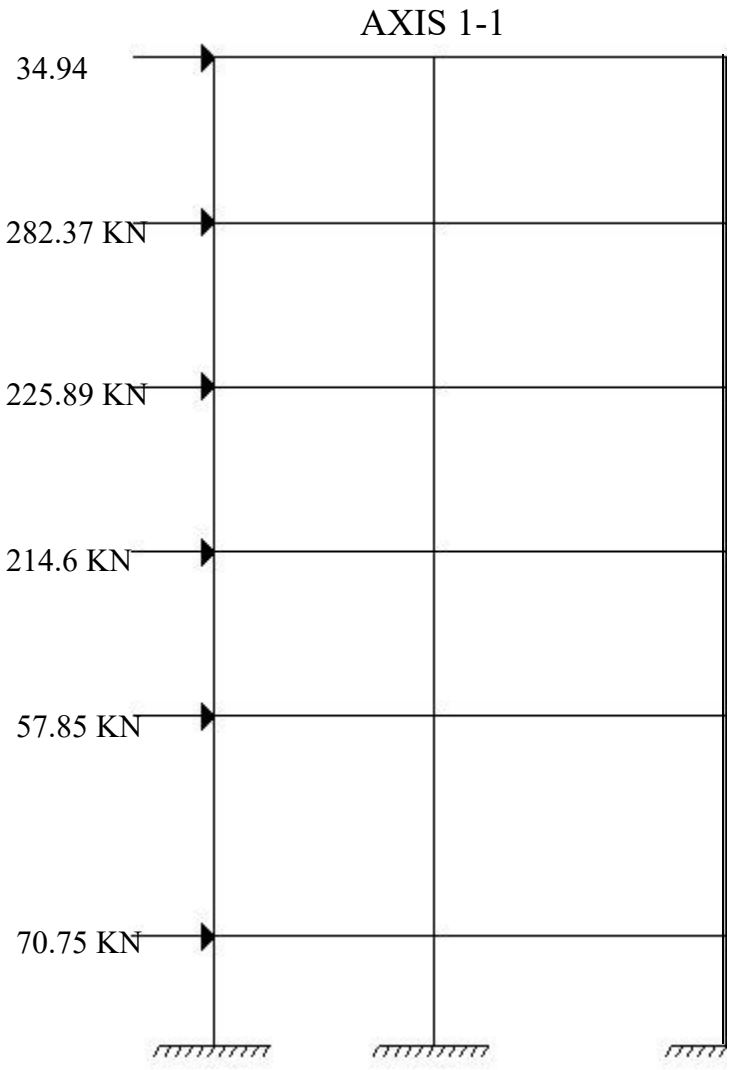

AXIS C-C
47.63
301.88 KN
301.88 KN
286.80 KN
76.73 KN
95.18 KN

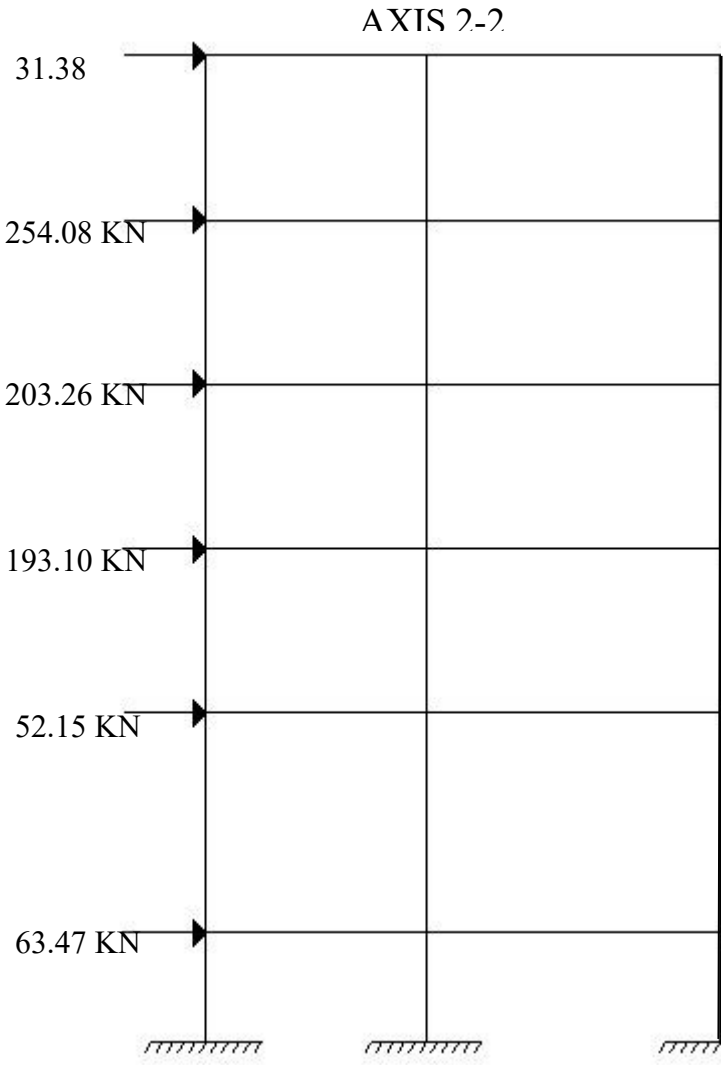
AXIS D-D



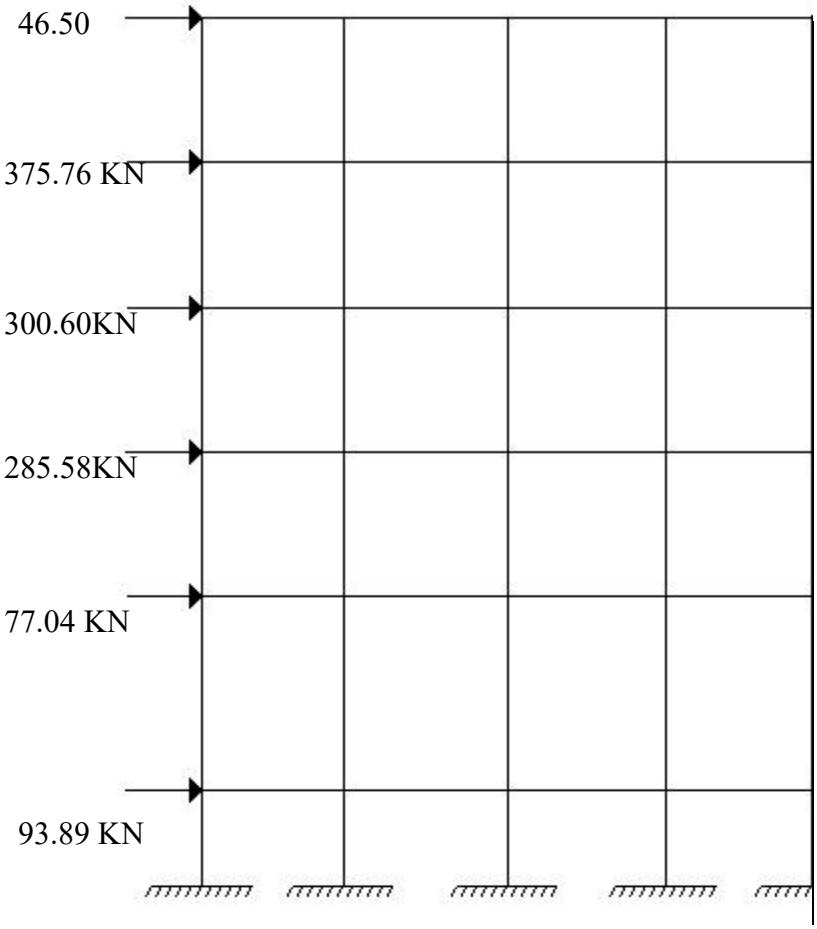
AXIS E-E



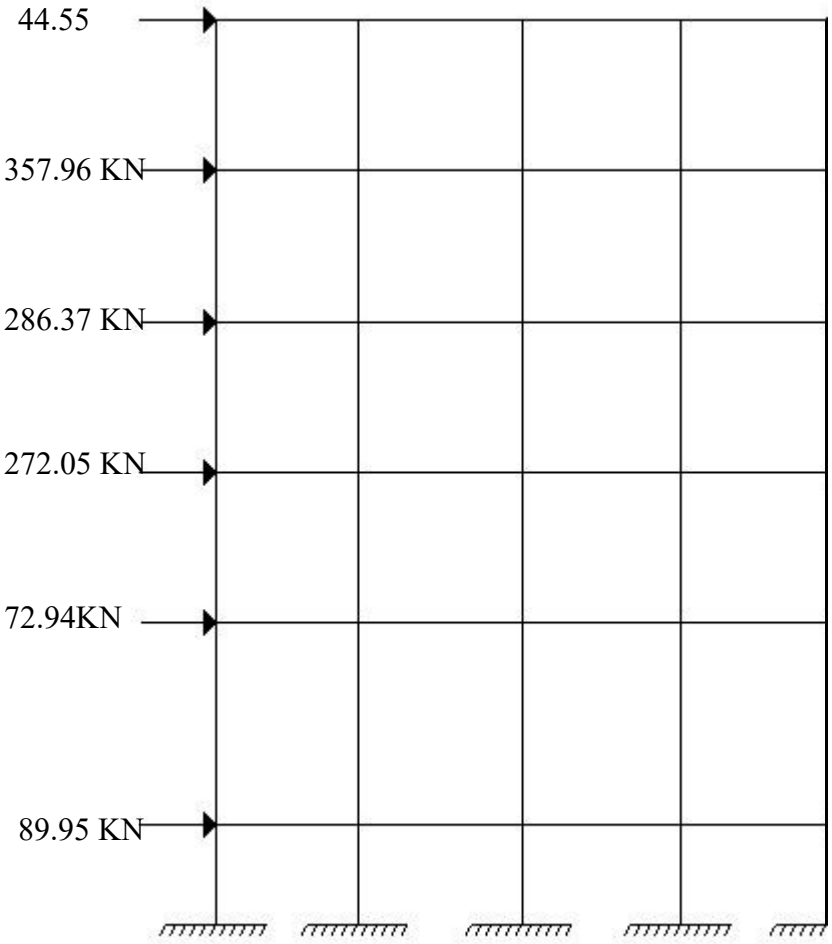

AXIS 1-1
34.94
282.37 KN
225.89 KN
214.6 KN
57.85 KN
70.75 KN


AXIS 2-2
31.38
254.08 KN
203.26 KN
193.10 KN
52.15 KN
63.47 KN

AXIS 3-3



AXIS 4-4



2.2.9 Frame Analysis Procedure using ETABS

Our frame was analyzed using ETABS 2016. The procedure for modelling the frame will be as follows:

2.2.10 Frame analysis

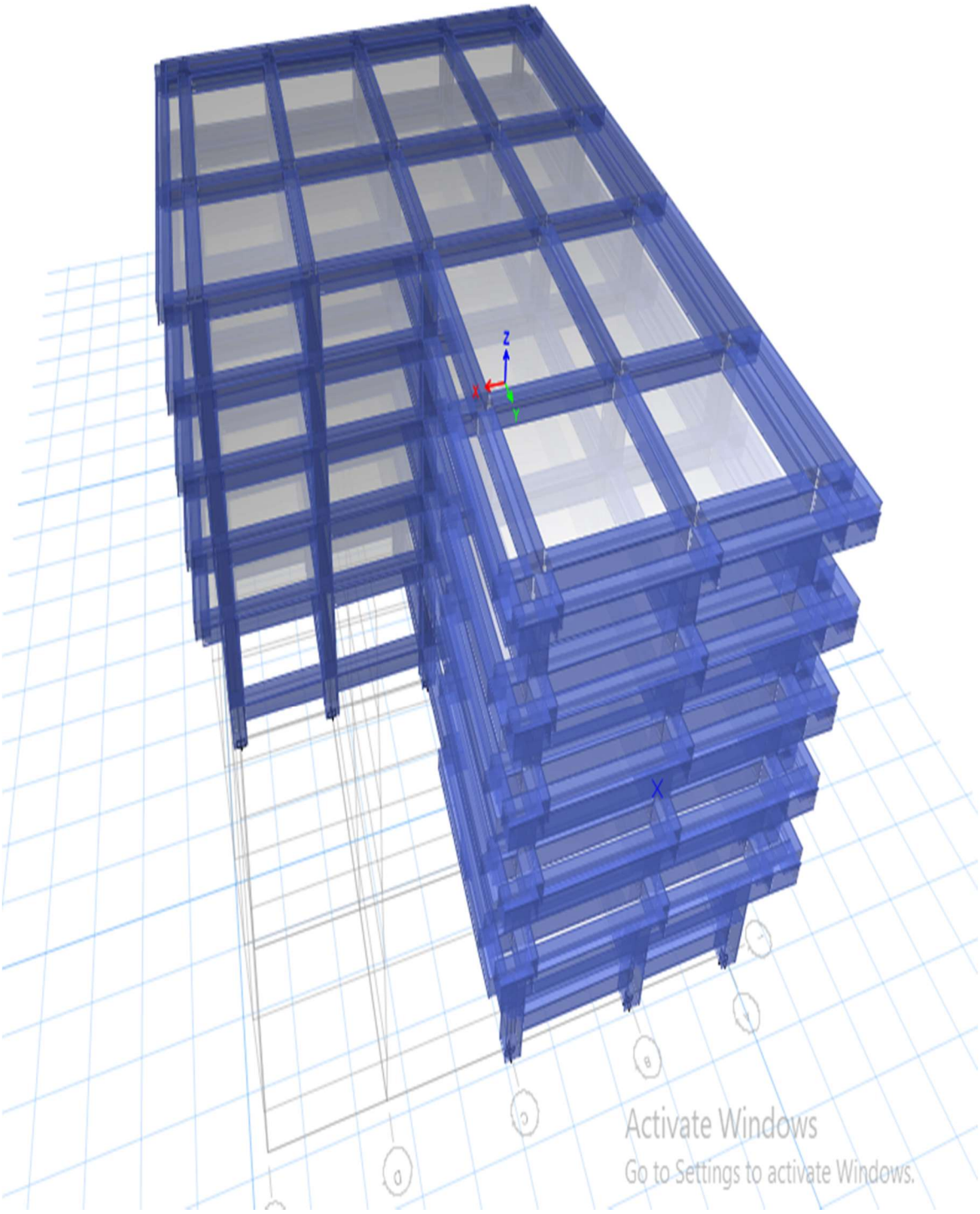


Figure 3D model

a) STEP: Plot Grid Coordinates

Plot grid coordinates that represent the given structural layout.

b) STEP : Define Material

Our building is made of reinforced concrete structures. Therefore, the materials which we define are concrete & steel rebar.

A. Concrete

- Grade: C-25
- Symmetry type: Isotropic
- Modulus of elasticity: 29GPA
- Poisson's ratio: 0.2
- Shear modulus (G): 12083.33MPa
- Coefficient of thermal expansion: 10 E-6
- Unit weight: 25KN/m^3

B. Rebar

- Grade: S-400
- Unit weight: 77KN/m3
- Modulus of elasticity: 200000MPa
- Poisson's ratio: 0.2
- Minimum yield stress, fy: 347.83MPa
- Minimum tensile stress, fu: 521.75MPa

c) STEP : Define Load and Frame Section

Define the following loads in the load pattern.

- Dead load
- Live load
- Wind load (X+)
- Wind load (X-)
- Wind load (Y+)
- Wind load (Y-)
- Eqx(X+)
- Eqx (X-)
- Eqx (Y+)
- Eqx (Y-)

Then, we have used the default load combinations in the ETABS itself.

- Beam

Top tie beam 300x400 mm

Inter mediate beam 500x400 mm

Grade beam 500x400 mm

- Column

Rectangular column 500x400 mm

Circular column 400mm

Those materials are defined in which their composition is C-25 and S-500 defined previously.

d) STEP : Draw the structure

Use the previously defined frame sections to draw the structure and it also includes assigning of restraints.

The restraints are assumed to be fixed joints.

e) STEP : Assigning loads

f)STEP : Analysis

Finally run the model and check for any errors.

Sample example output ETABS

Moment 3-3 @ axis A-A

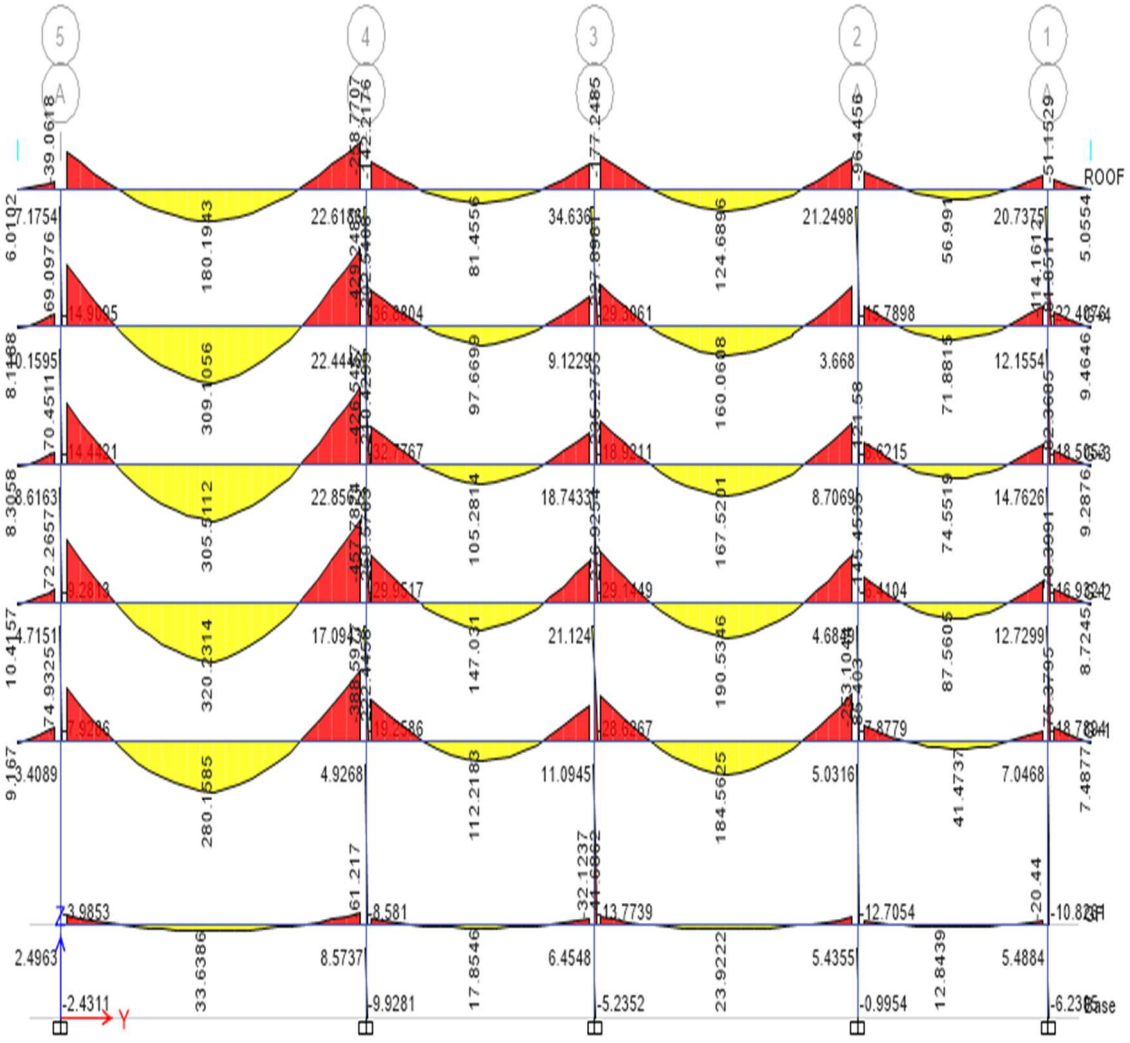


Figure moment @ axis A-A

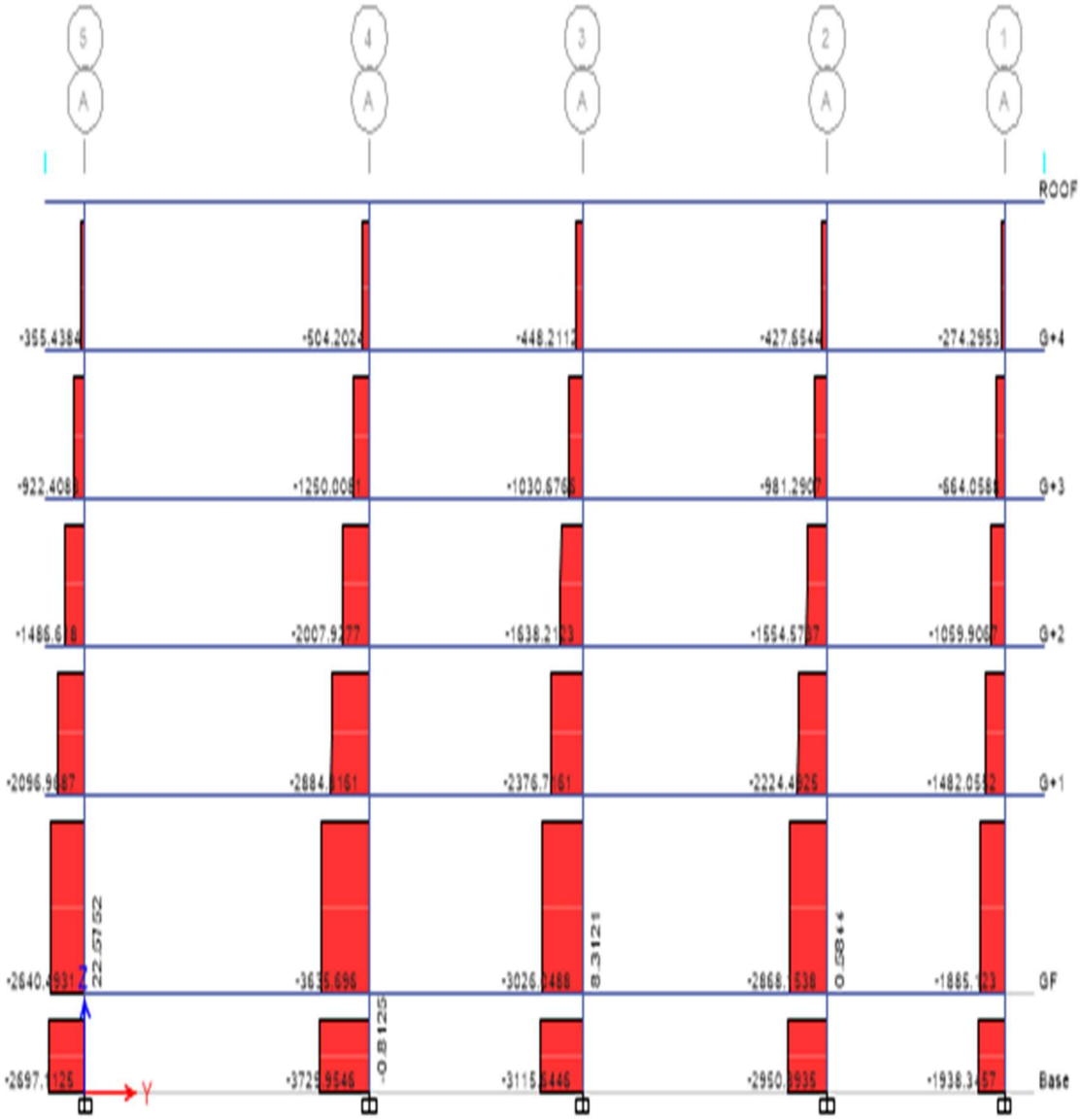


Figure Axi al force @A-A

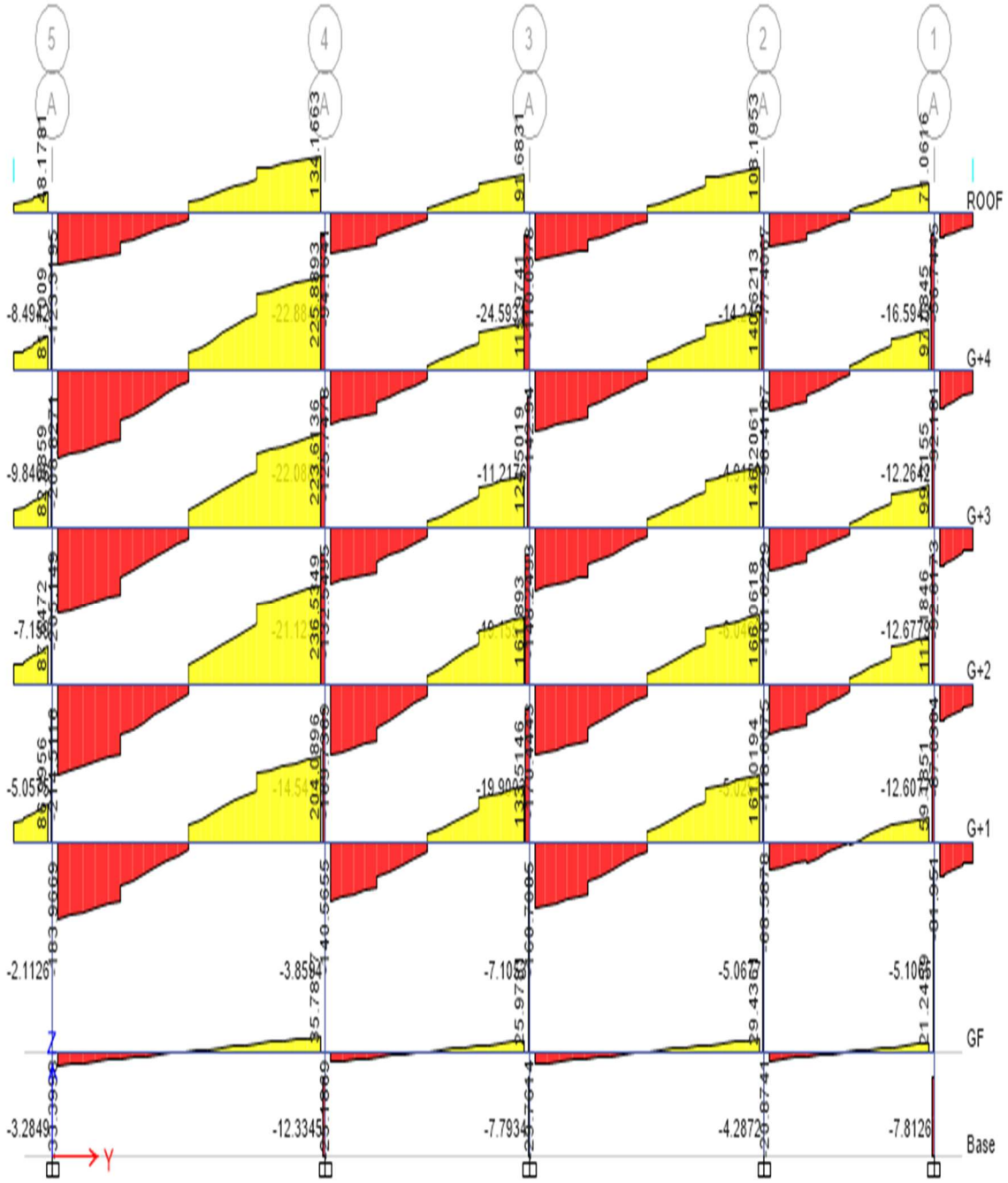


Figure Shear 2-2 @A-A

2.2.11 Beam design

Introduction

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
- Ø 14 bar and Ø 16 for flexure
- bar for shear
- Use concrete cover C = 25mm

Cross sections of beams are

- Top tie beam =400mm*300mm
- 1stfloor beams ,2nd floor beams,3rd floor ,4th floor beams = 500*400mm
- Grade beam, =500*400mm

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

From design chart

For μsd =$\boldsymbol{Msd/fcd*b*d2}$= μsd=0.295

No moment redistribution is allowed

Concrete Cover Determination

The nominal concrete cover is the distance b/n the surface of reinforcement closest to the nearest concrete surface including links and stirrups.

$Cnom=Cmin+\Delta Cdev$ *ES EN* 1992:2015 *Art* 4.4.12(1)

$Cmin=\{C_{min,b}Cmin,dur+\Delta Cdur,\gamma-\Delta Cdur,st-\Delta Cdur,\ add10mm$

Where;

Cmin, b -minimum cover due to bond requirement, see ES EN Art. 4.4.1.2 (3).

Cmin,dur - minimum cover due to environmental conditions, see ES EN Art 4.4.1.2 (5)

ΔCdur,γ- additive safety element, see ES EN Art 4.4.1.2 (6)

ΔCdur, st -reduction of minimum cover for use of stainless steel, see ES EN Art 4.4.1.2 (7)

ΔCdur, add -reduction of minimum cover for use of additional protection, see ES EN Art 4.4.1.2 (8)

But; the recommended value of ΔCdur,γ , ΔCdur,st , and ΔCdur,add is zero see Art. 4.4.1.2 (6, 7, and 8).

Effective Depth Determination: Serviceability requirement

According to ES EN 1992:2015; the limit state of deformation may be checked by either:

By limiting the span/depth ratio, according to 7.4.2 or by comparing a calculated deflection, according to 7.4.3, with a limit value

$l/d = [11+1.5\sqrt{fck}\rho o/\rho+3.2\sqrt{fck}(\rho o/\rho-1)32]*F1*F2*F3$……$if\ \rho\leq\rho o$ $Art.7.4.2$ $(7.16a)$

$l/d=K[11+1.5\sqrt{fck}\rho o/\rho-\rho'+112\sqrt{fck}\sqrt{\rho'/\rho}]*F1*F2*F3$……$if\ \rho\geq\rho o$ $Art.7.4.2$ $(7.16b)$

Where;

l/d - is the limit span/depth

K -is the factor to take into account the different structural systems

ρo - is the reference reinforcement ratio

ρ - is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)

ρ' - is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers) fck is in MPa units.

$$F1 = \frac{310}{\sigma s} = \frac{500}{fyk*As,req/As,prov}$$

F2=0.8, for flanged sections where the ratio of the flange breadth to the rib breadth exceeds 3. Otherwise; F2=1 for other cases.

F3=7/leff, for beams and slabs, other than flat slabs, with spans exceeding 7 m, which support

Partitions liable to be damaged by excessive deflections (leff in meters, see Art. 5.3.2.2 (1)). Or

F3=8.5/leff, for flat slabs where the greater span exceeds 8.5 m, and which support partitions

Liable to be damaged by excessive deflections (leff in meters). Otherwise; F3=1 for both cases.

Assumption

Initially we can't know ρ and ρ'. So let's assume ρ= ρo and use equation 7.16a.

As, req = As, provided.

$l\,d = K * N * F1 * F2 * F3\ Where;$

$N = 11 + 1.5\sqrt{fck}\ \rho o/\ \rho + 3.2\sqrt{(\rho o\ /\rho - 1)32}\ But\ \rho o = \rho\ (assumption)$

$\rho o = \sqrt{fck} * 10{-}3 = \sqrt{20} * 10{-}3 = 0.447\%$

$N = 11 + 1.5\sqrt{fck} = 11 + 1.5\sqrt{20} = 17.71$

$F1 = \frac{310}{\sigma s} = \frac{500}{fyk * As,req/As,prov}$

= 500 (400) = 1.75

F2 = 1 and F3 = 1 (because span of beam < 7m) $Now: determine\ K\ using\ table\ 7.4N\ of\ ES\ EN1992$: 2015

F2 = 1 and F3 = 1 (because span of beam < 7m) $Now: determine\ K\ using\ table\ 7.4N\ of\ ES\ EN1992$: 2015

So K = 1.3, F3 =1, and F2 = 1

$l\,d = K * N * F1 * F2 * F3 = 1 * 17.71 * 1.75 * 1 * 1 = 31$

L/d =28.78 Where; l = 5900 mm, this is the maximum span length.

d = 5900/31 = 190.32mm

So Effective depth d, = 190mm

Depth, D=d+d' Where: d' = cover + Φst + Φlon /2 $use\ \Phi st = 8mm\ and\ \Phi long = 20mm$

d' = cover + Φst + Φlon /2 = $30mm + 10mm + 20mm$ /2= $50mm$

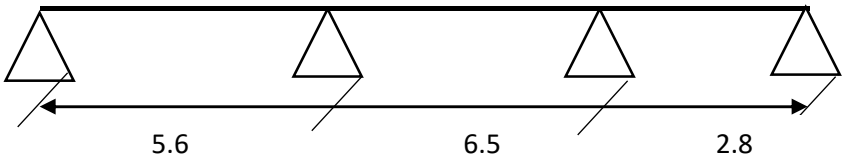
d=h-d'=500-50=450mm

Effective width of flanges (all limit states)

(1) T beams the effective flange width, over which uniform conditions of stress can be assumed, depends on the web and flange dimensions, the type of loading, the span, the support conditions and the transverse reinforcement.

(2) The effective width of flange *l*0 should be based on the distance between points of zero moment.

Figure6.2: Effective flange width parameters



L_O=0.15(L1+L2)=0.15(6.5+2.8)=1.395m

B1=(2.8-0.3)/2=1.25m ; beff1=0.2b1+0.1 L_O=0.2*1.25+0.1*1.39=0.389m=389mm

B2=(6.5-0.3)/2=3.1m ; beff2=0.2b2+0.1 L_O=0.2*3.1+0.1*1.39=0.759m=759mm

0.2 L_O=0.2*1.395=0.279m=279mm

From this beff1&beef2<b1&b2 but greater than 0.2 LO=296mm

Know, beff = Σbeffi +bw≤ b

Beff=0.389+0.759+0.3=1.45m=1450mm

Assume neutral axis depth is within the flange.

ρ =0.5[c1-$\sqrt{c * 12 - \frac{4*Med}{c2bd2}}$] or ρ =0.5[0.087-$\sqrt{(1 - \frac{2Med}{fcdbeffd2}}$]

Where, m=fyd/0.8fcd=347.83/(0.8*11.330=38.37

c1=2.5/m=2.5/38.37=0.065

c2=0.32*m^2fcd =0.32*38.37^2*11.33=5339.17

ρ=0.00095

From the relation Kx= ρ*m and kx=X/d we can determine the neutral axis depth

Then; X= ρ*m*d = 0.00095*38.37*400 = 14.58mm < hf = 240mm…..ok

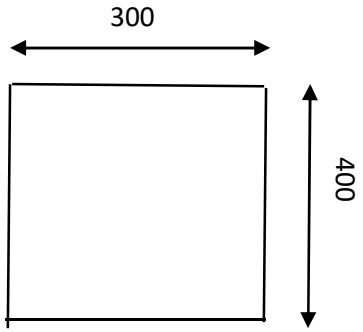
Therefore our assumption is correct.

Members not requiring design shear reinforcement

- Etabs output for moments

For Top tie beam axis 1 section A-B and B-C

Section A-B



Span moment=MEd=42.36kNm

Support moment=MEd=60.82kNm

VED=50.15KN

Section B-C

Span moment= MEd=62.83kNm

Support moment=MEd=80.46kNm

VED=61.68KN

Section A-B

$\mu sds = \frac{Msd,s}{fcd*b*d2}$

$\mu sds=60.82*10\text{^}6/11.33*300*400^2=0.112$

$\mu sd,s=0.112 < \mu us,s=0.295$(for 0%moment redistribution) design as singly reinforced section using $\mu sd,s=0.11984$read kz from the general design chart No 1A

kz= 0.934

Z=kz*d= 0.934*400=376.223mm

Thus Ast, calc=MED/fyd*z

Ast, calc= $60.82*10^6/(347.28*326.9)= 464.71mm^2$.

Before calculating the area of steel, required let us calculate Ast, min and Ast, max

$$\text{Ast, min} = \text{Max}\begin{cases} 0.26(fctm*btd)/\text{fyk} \\ 0.0013btd \end{cases}$$

Fctm =2.2Mpa by using fck=20Mpa from EBCS- EN1992-1-1-2014 table3.1

Fctm=2.2Mpa by using fck=20Mpa from EBCS- EN1992-1-1-2014 table3.1

$$\text{Ast, min} = \text{Max}\begin{cases} 0.26(2.2*300*400)/400=171.6mm^2 \\ 0.0013*300*400= 156mm^2 \end{cases}$$

The minimum and the maximum value of area of reinforcement is computed in accordance with the provision of the code.

Astt cal> Ast, min=464.71mm2 >156mm^2; use Ast, calc 464.71mm^2

Know no bar=Ast. Cal/as=464.71mm2/314mm2=1.4 so we proved 2Φ16

Ast, provide=2*3.14*16^2/4=402mm2

6.3 Check failure type

$\rho prov = \frac{Astprovid}{bd} = \frac{402}{300*400}$=0.00335=0.335%

ϵyd = fyd/Es=347.83/2*105 =1.74%

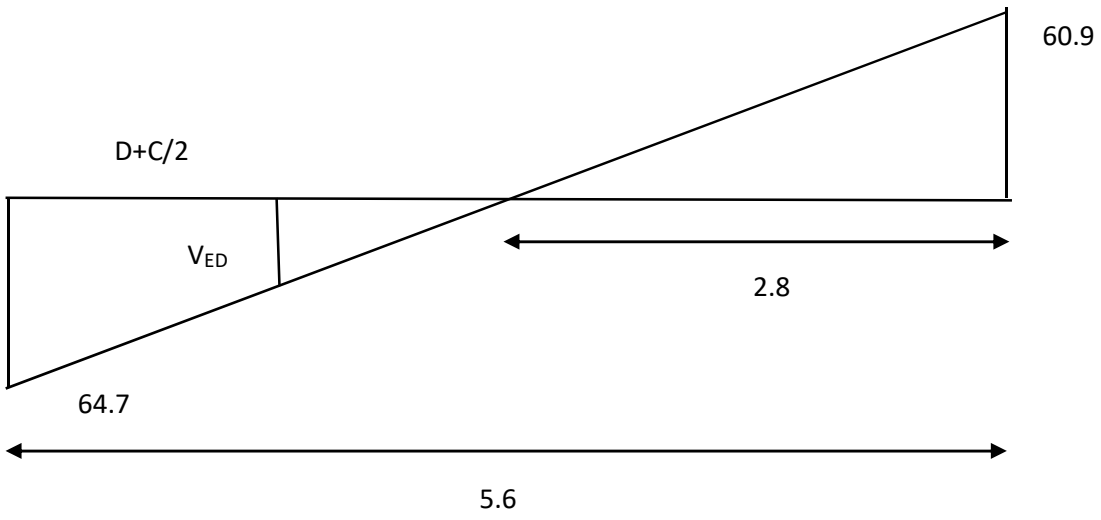
$\rho b = 0.8(\epsilon \frac{\rho cu}{\epsilon cu+\epsilon yd}) \frac{fcd}{fyd}$=$0.8\left(\epsilon \frac{3.5}{3.5+3.5}\right)\frac{11.33}{347.83} = 0.074$

Since ρ=0.598%> ρo=0.447%..... tension failure…..ok

a) Check the section for shear resistance

The design shear is located d distance from the face of the column.

By similarity of triangle, VEd can obtain as follows:



Considerθring 200mm frame beam width, D + C/2=400mm+200/2mm=500mm.

Hence, from similarity of triangles;

$\frac{60.9}{5.6-X} = \frac{64.7}{X}$

X=2.885m

$\frac{60.9}{2.8} = \frac{VED}{2.885-0.5}$

VED = 50.15KN

b) Check for diagonal compression failure

$Vrd,= V1*fcd*bw*\frac{Z}{\cot\theta+\tan\theta}$

Let us tray, θ<=21.8° =>cotθ=2.5,tanθ=0.4

d= D-cover-ϕ/2/-stirrup

d=400-34-16/2-10=348mm

Z =0.9*d =0.9*348mm = 313.2mm

V1 = 0.6(1- fck/250)

V1 = 0.6(1-25/250) = 0.552

Fcd=11.33

Bw =200

$VRD,MAX=\frac{(0.552*11.33*200*313.2/1+1)}{1000}$

VRD, MAX=195.88

Since it resistance (VRD, MAX=195.88)>>acting shear (VED=50.15), So the section is doesn't require shear reinforcement for diagonal.

Check concrete capacity of section for shear

Concrete capacity check

$$v_{Rd.c} = \left[0.12K(100\rho_1 f_{CK})^{\frac{1}{3}}\right] \text{bd} \geq V_{min}$$

$$K = 1 + \left(\frac{200}{d}\right)^{0.5} \leq 2$$

d= 400

b= 300

Asl= 200.96

k= 1.71

$$\rho_1 = \frac{A_{sl}}{bd} \leq 0.02$$

taking minimum reinforcment ø 16

$\rho_1 = 0.0017$

$v_{Rd,c} = 36.780$

$v_{Rd,c}$ = VRdc <= VEd, shear reinforcment design required

longtudinal bar diam.= 16

minimum spacing = 240

c) Reinforcement calculation

For member with vertical shear reinforcement the shear resistance VRD is the smaller value of]

Vrd =Max { Vrds= $\frac{\text{As}}{\text{ws}} * zfydcot\theta$; Vrds= $\propto c * bw * z * v * \frac{\text{fcd}}{\text{cot}\theta + \text{ta}\ \theta}$

Where,

α= is the angle between shear reinforcement and the main tension chord (Recommended value of 1 for non pre stressed structures)

θ= is the angle between concrete compression struts and the main tension chord

Ftd = is the design value of the tensile force in the longitudinal reinforcement

Fcd = is the design value of the concrete compression force in the direction of the longitudinal member axis.

bw =is the minimum width between tension and compression chords

z =is the inner lever arm, for a member with constant depth, corresponding to the maximum bending moment in the element under consideration. In the shear analysis, the approximate value z = 0.9d may normally be used.

Asw =is the cross-sectional area of the shear reinforcement

s =is the spacing of the stirrups

fywd =is the design yield strength of the shear reinforcement

V = 0.6 for fck ≤ 60 MPa Now let's determine the angle θ between the concrete compression strut and the main tension chord by considering Vrd,s = VEd

$VEd = \propto c * bw * z * v * \frac{fcd}{\cot\theta + \tan\theta} = \propto c * bw * z * v * \frac{fcd}{\sin 2\theta/2}$

$\theta = 0.5 * SIN - 1(2 * VED/(\alpha * bw * z * v * fcd))$

$\theta = 0.5 * SIN - 1(2 * 50.15/(1 * 200 * 315 * 0.6 * 11.33))$

Θ=0.1116

$\frac{VED}{Z * Fyd * \cot\theta} = \frac{AS}{WS}$

$ASW = 2 * 3.14 * 8^2/4 = \frac{100.53\ 35.51 * 10\hat{}3}{361.8 * 11.33 * 1} = 100.53/S$

S=1116.05mm

Determination of maximum spacing

The ratio of shear reinforcement is given by the expression

$\rho w = \frac{Asw}{S * bW * \sin\infty}$:Where, ρw = is the shear reinforcement ratio

bw = is the breadth of the web of the member

$\rho w \leq \rho w,min = \frac{0.08\sqrt{fck}}{fyk} = \frac{0.08\sqrt{20}}{400} = 0.000894$

→take ρw=0.000894

Asw= is area of shear reinforcement with length s

α = is the angle between shear reinforcement and the longitudinal axis

s= is the spacing of the shear reinforcement measured along the longitudinal axis of the member S1, max = 0.75d (1+cotα), considering α=900

S1max = 0.75d=0.75*400=300mm

Thus Smax = $\frac{Asw}{\rho w * bW * sin\alpha}$ $\frac{100.53}{0.000894*200*\sin 90}$=562.25mm

S calculated=116.05mm< s max=116.05mm <562.25

So we provide S=116.05mm

Check depth for flexure

Mrd = 0.8kx (1-0.4kx) $fcdbd^2$

Mrd = 0 .8*0.448(1-0.4*0.448)*11.33*300*350*350=122.48KNm

Mrd=122.48KNm>MED=60.82KNm…...ok!

Check depth for Deflection/Serviceability requirement

$\rho o = \sqrt{fck} * 10^{-3} = \sqrt{20} * 10^{-3} = 0.447\%$

$\rho = \frac{Astprovid}{bd} = \frac{402}{300*350} = 0.00383 = 0.383\%$

Since ρ=0.383%

$\frac{l}{d} = K*N*F1*F2*F3$ Where $N=(11+1.5\sqrt{fck}*\frac{\rho}{\rho-\rho o}+(\frac{1}{12})*\sqrt{fck}*\sqrt{\frac{\rho'}{\rho o}})$

ρ'=0

$N=11+1.5*\sqrt{20}*\frac{0.447}{0.383}=18$

K=1

$F1=\frac{500}{400*\frac{402}{439.54}}=1.5$

F2=1 since beff/bw is<3

F3=1 since the span is less than 7

$\frac{l}{d} = 1*18*1.5*1*1=27$

d=4720/27=174.8mm< 350mm ……..ok

Axis B

Section 5-4

Span moment= MEd=534.23kNm

Support moment=MEd=106.98 kNm

VED=203.53KN

μsds=Msd,s/fcd*b*d^2

μsds=534.23*10^^{6}/11.33*400*450^2=0.581

μsd,s=0.581>μus,s=0.295(for 0%moment redistribution) design as double reinforced section using

Designing the beam as double reinforced beam using Design Chart

Support moment MED=534.23KNm

Span moment MED=106.98KNm

For MEd=534.23KNm

μsds=Msd,s/fcd*b*d2

μsds=534.23*10^6/11.33*400*450^2=0.581

μsds=0.581> μsds=0.295

(for 0% moment redistribution) => design the beam as doubly reinforced section (provide compression reinforcement)

μsds=0.295, Msd'= μsds*fcd*bw*d^2=0.581*11.33*400*450^2= 533.2KNm

kz=0.8 from chart , Z=kz*d=0.8*450=360mm

AS1=$\frac{Msd'}{fydZ}+\frac{Msd-Msd'}{(d-d')*fyd}=\frac{533.23}{347.83*360}+\frac{534.23-5\quad.23}{40\ *347.83}$ =551.88mm^2

The minimum and maximum value of area of reinforcement is computed in accordance withthe provision of the code.

Asmax=0.04 AC=0.04*400*500=8000mm^2

ASmin=286mm^2

As,min=286mm^2 <As,calcu=4958.86mm^2 <As,max=8000mm^2

#no bar=$\frac{AScalcu}{ASsingle}=\frac{4958.86}{314}$ =15.79 Therefore we provided 16Φ16

AS2=$\frac{Msd-Msd'}{(d-d')*fyd}=\frac{146.1-122.83}{300*347.83}$=223.001mm2

#no bar=223.001/200.96=0.493, therefore we provided 1Φ16.

Strain distribution at ULS

For μus∗ = 0.295 => Kx = 0.448 => X = Kx *d = 0.448*350 = 156.8mm

Using triangular similarity

Єcm=0.35%

Using triangular similarity

єs1= Єcm(d/x-1)=0.0035(350/156.8-1)=0.0431

єyd=fyd/E =347.83/200000=0.00174

$\frac{(\epsilon s2)}{x-d2} = \frac{\epsilon cm}{x}$ from the similarity $\epsilon s2 = \frac{\epsilon cm}{x(x-d2)}$ єs2= 0.0035/156.8(156.8−50) =0.00238

Since єyd =0.00174 <єs2 =0.00238 the bottom reinforcement yielded

Determine Maximum number of bar Placed in a single Row Spacing of bars according to EN 1992-1-1-2015 The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should be not less than

S= Max { 20; k1*diameter of longitudinal bar; Dg*k2 }

Where dg is the maximum size of aggregate the recommended values of k1 and k2 are 1 and 5 respectively.

Assume maximum size of aggregate size is 20mm and diameter of stirrup is 8mm

S= Max { 20; 1*24; 20+5 }

Therefore; clear spacing between bars is 25mm.

Width, bw =2*cover*nΦlong+(n-1)s+Φst

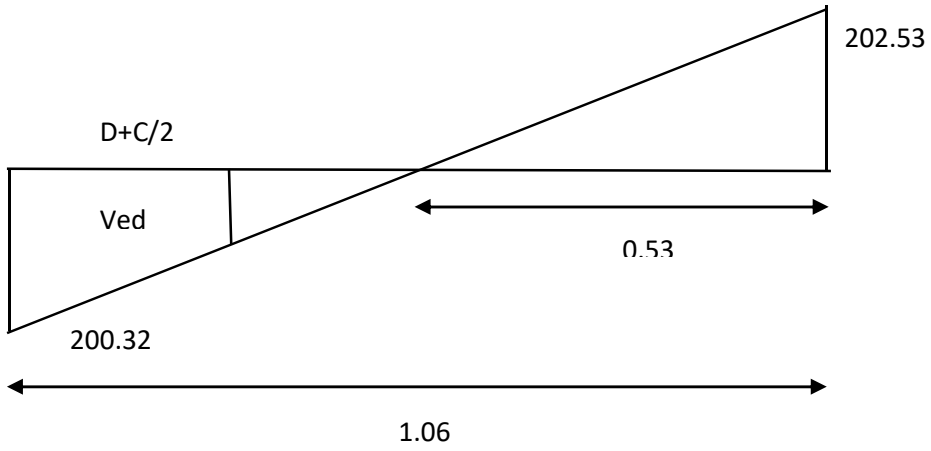
400=2*30*n*24+(n-1)*25+2*10= 70+24n+25n-25=55*49n

, n = (400-55)/49=7bar

Therefore maximum number of bar in a single row should be 7. But we have 5 bar.

Check the section for shear resistance

The design shear is located d distance from the face of the column. By similarity of triangle, VEd can obtain as follows:



Considering 200mm frame beam width, D + C/2=400mm+200/2mm=500mm.

Hence, from similarity of triangles;

$\frac{202.53}{1.06-X}=\frac{200.32}{X}$

X=0.53m

$\frac{202.53}{0.53}=\frac{VEd}{0.53-0.50}$ VED =11.46KN

Check for diagonal compression failure

$Vrd,x=V1*fcd*bw*\frac{z}{\cot\theta+\tan\theta}$

Let us tray, θ<=21.8o =>cotθ=2.5,tanθ=0.4

d= D-cover-ϕ/2/-stirrup

d=400-34-16/2-10=348mm

Z =0.9*d =0.9*348mm = 313.2mm

V1 = 0.6(1- fck/250)

V1 = 0.6(1-25/250) = 0.552

Fcd=11.33

bw=200

$VRD,MAX=(\frac{0.552*11.33*200*\frac{313.2}{1+1}}{1000})$

VRD, MAX=195.88

Since it resistance (VRD, MAX=195.88)>acting shear (VED=60.23)

So the section is doesn't require shear reinforcement for diagonal.

Check concrete capacity of section for shear

Vrd, c = max of. $\{CR,K(100P1fck))^{1/3}bwd,Vminbwd\}$

C Rdc = $\frac{0.18}{\gamma c} = \frac{0.18}{1.5} = 0.12$

k = 1+√200/d = 1+√200/348 = 1.57 ≤2 ……...ok!!

V min = 0.035 * K3/2 * fck1/2 = 0.035*1.573/2 * 251/2 = 0.344

ρ1=AS provided/bw*d=1484.13/ (200*348) =0.0213

Vrd, c =0.12*1.57*(100*0.0213*20) ^ (1/3) *200*348=45.795

Vrd, c 0.3896*200*348=27.12

take ,Vrd, c=45.795

VED =60.23KN>Vrd, c=45.795 ,the concrete section doesn't resist the coming shear alone, thus the section require design for shear reinforcement ES EN 1992-1-1 Section For members with vertical shear reinforcement, the shear resistance,

Reinforcement calculation

For member with vertical shear reinforcement the shear resistance, VRD is the smaller value of]

For member with vertical shear reinforcement the shear resistance, VRD is the smaller value of]

Vrd =Max

Vrds= $\frac{\text{As}}{\text{ws}} * zfydcot\theta$

Vrds= $\propto c * bw * z * v * \frac{\text{fcd}}{cot\theta + tan}$

Where,

α= is the angle between shear reinforcement and the main tension chord (Recommended value of 1 for non pre stressed structures)

θ= is the angle between concrete compression struts and the main tension chord

Ftd = is the design value of the tensile force in the longitudinal reinforcement

Fcd = is the design value of the concrete compression force in the direction of the longitudinal member axis.

bw =is the minimum width between tension and compression chords

z =is the inner lever arm, for a member with constant depth, corresponding to the maximum bending moment in the element under consideration. In the shear analysis, the approximate value z = 0.9d may normally be used.

Asw =is the cross-sectional area of the shear reinforcement

s =is the spacing of the stirrups

fywd =is the design yield strength of the shear reinforcement

V = 0.6 for fck ≤ 60 MPa Now let's determine the angle θ between the concrete compression strut and the main tension chord by considering Vrd,s = VEd

VEd= $\propto c*bw*z*v*\frac{\text{fcd}}{cot\theta+tan\theta}= \propto c*bw*z*v*\frac{\text{fcd}}{\frac{\text{si}}{2}}$

VEd= $\propto c*bw*z*v*\frac{\text{fcd}}{cot\theta+tan\theta}$

VED= $\propto c*bw*z*v*\frac{\text{fcd}}{\frac{\sin 2\theta}{2}}$

θ=0.5$*SIN-1(2*VED/(\alpha*bw*z*v*fcd))$

θ=0.5$*SIN-1(2*95.834/(1*200*361.8*0.6*11.33))$

Θ=0.1116

$\frac{\text{VED}}{\text{Z*Fyd*cotθ}}=\frac{\text{AS}}{\text{WS}}$

ASW=2*3.14*82/4=100.53

$\frac{60.123*10\text{^}3}{361.8*11.33*1}$=100.53/$S$

S=68.54MM

Determination of maximum spacing

The ratio of shear reinforcement is given by the expression

ρw = $\frac{\text{Asw}}{S*bW*Sin\propto}$:Where, ρw = is the shear reinforcement ratio

bw = is the breadth of the web of the member

ρw≤ ρw,min=$\frac{0.08\sqrt{\text{fck}}}{\text{fyk}}=\frac{0.08\sqrt{20}}{400}$ = 0.000894 →take ρw=0.000894

Asw= is area of shear reinforcement with length s α = is the angle between shear reinforcement and the longitudinal axis

s= is the spacing of the shear reinforcement measured along the longitudinal axis of the member S1,max = 0.75d(1+cotα), considering α=90^0

S1max = 0.75d=0.75*400=300mm

Thus Smax =$\frac{Asw}{\rho w*bw*\sin\alpha}$ =$\frac{100.53}{0.000894*200*\sin 90}$ =562.25mm

S calculated=68.54mm< s max=68.123mm <562.25

So we provide S=68.123mm

Check depth for flexure

Mrd=0.8kx (1-0.4kx) fcdbd2

Mrd=0.8*0.448(1-0.4*0.448)*11.33*300*400*400=159.98KNm

Mrd=159.98KNm>MED=146.1KNm….. ok!

Increase the cross section

Mrd=0.8kx (1-0.4kx)fcdbd^2

Mrd=0.8*0.448(1-0.4*0.448)*11.33*300*400*400=159.9KNm

Mrd=159.9KNm>MED=146.1KNm….. ok!

Check depth for Deflection/Serviceability requirement

ρo=$\sqrt{fck}*10^{-3}$=$\sqrt{20}*10^{-3}$=0.447%

ρ =($\frac{\text{Astprovid}}{\text{bd}}$)= $\frac{402}{300*400}$=0.00335 =0.335%

Since ρ=0.523% > ρo =0.447% using ES-EN 1992 Art.7.4.2 (7.16b)

$\frac{l}{d}$ =$K*N*F1*F2*F3$

Where N = (11+1.5$\sqrt{fck}*\frac{\rho}{\rho-\rho o}$ +($\frac{1}{12}$)*$\sqrt{fck}*\sqrt{\frac{\rho'}{\rho o}}$)

ρ'=0

N=11+1.5*$\sqrt{20}*\frac{0.447}{0.523}$=16.7

K=1

F1=$\frac{500}{400*\frac{628}{1250.2}}$=2.48

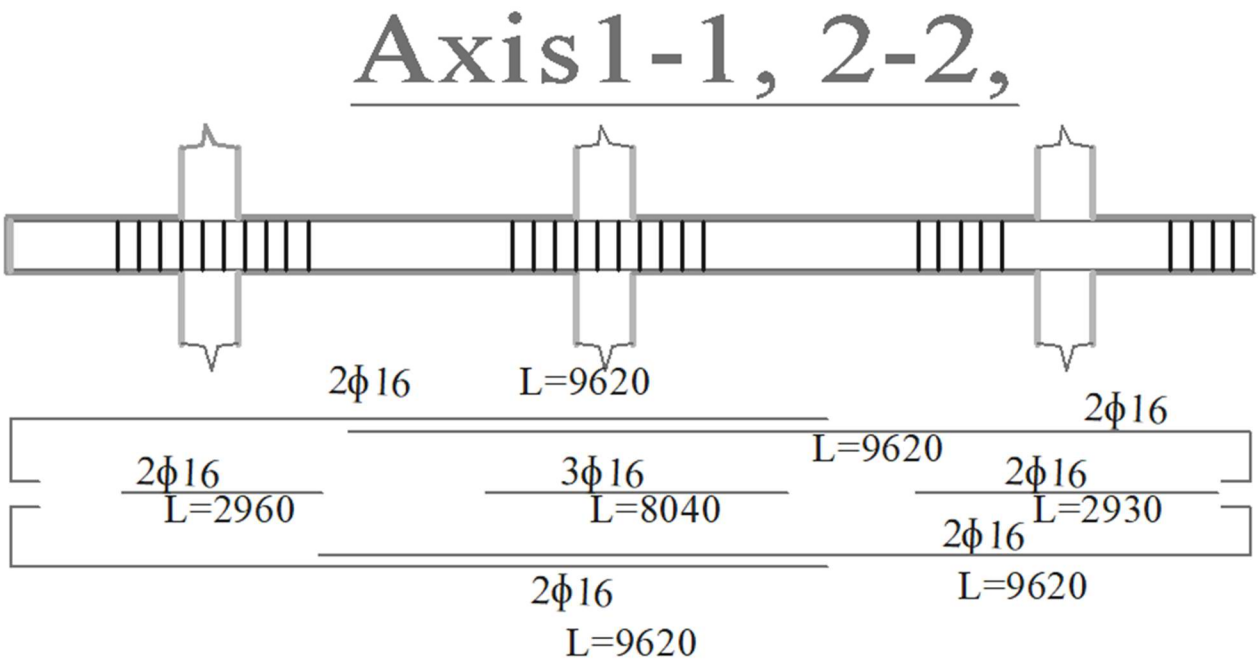
F2=1 since beff/bw is<3

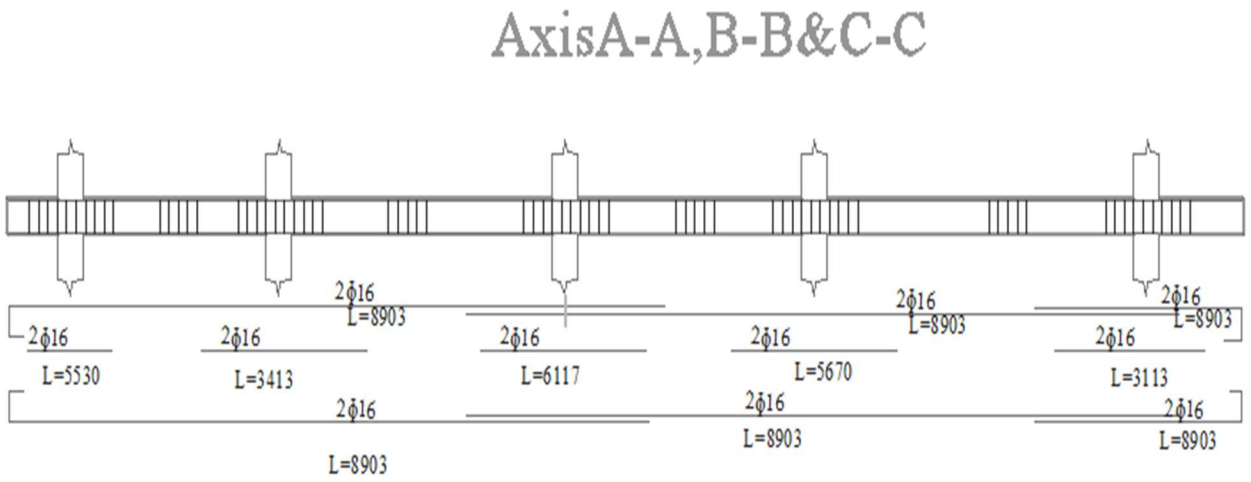
F3=1 Since the span is less than 7

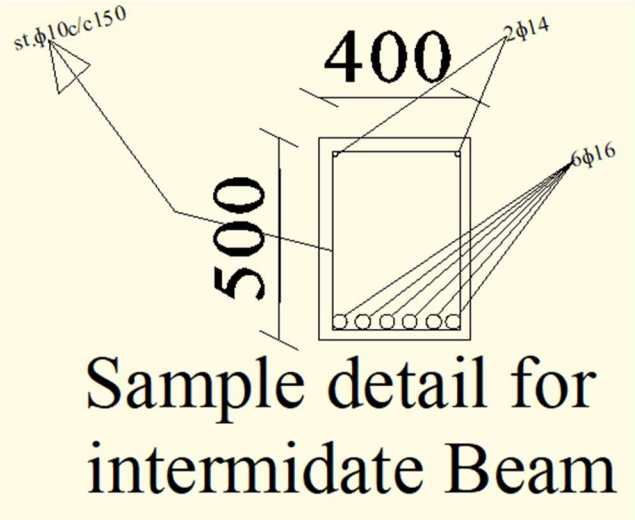
$\frac{l}{d}$=1∗16.7∗2.48∗1∗1=41.42

d=5900/41.42=142.45mm< 400mm ……..ok

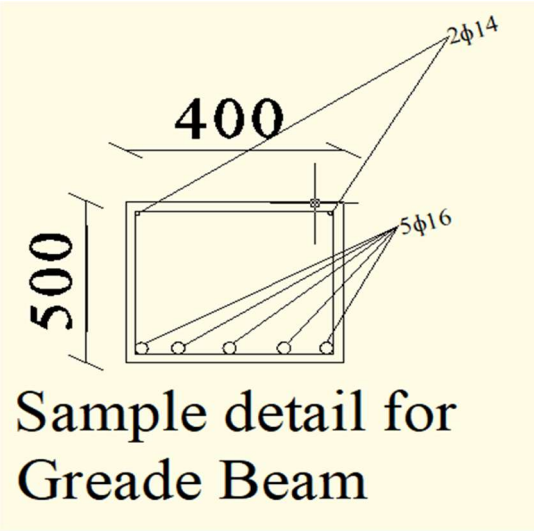
Rebar detail for for Top Tie beam



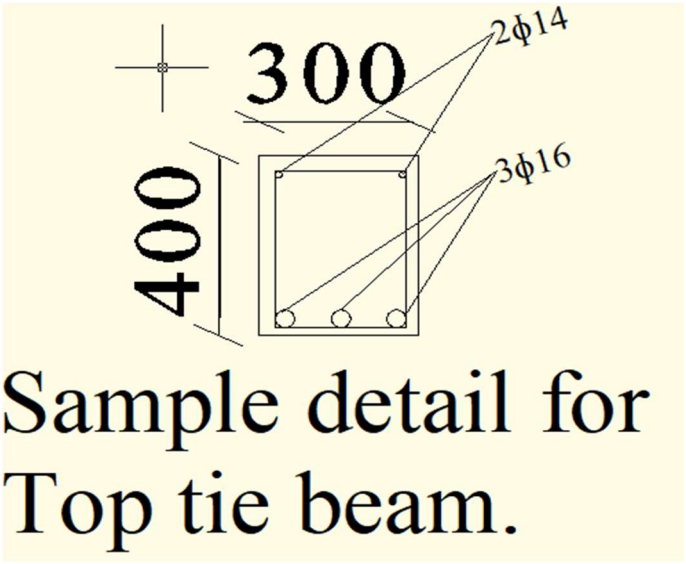




Sample detail for intermidate Beam



Sample detail for Greade Beam



Sample detail for Top tie beam.

2.2.12 Design of column

A 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:

- The 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

A. Rectangular column sample calculation for Column two on the 4th floor column

Given data

Top beam	depth(m)	width(m)	Length(m)
b1(x,x)	0.4	0.3	5.61
b2(x,x)	0.4	0.3	6.52
b3(y,y)	0.4	0.3	6.64
b4(y,y)	0.4	0.3	1.5

bottom beam	depth(m)	width(m)	length(m)
b1(x,x)	0.5	0.4	5.61
b2(x,x)	0.5	0.4	6.52
b3(y,y)	0.5	0.4	6.64
b4(y,y)	0.5	0.4	1.5
Column	0.5	0.4	3

M_{topx-x}=24.08KNm

M_{topy-y}=74.26KNm

M_{botx-x}=39.04KNm

M_{boty-y}=106.98KNm

NED=357.45KN

a) Concrete cover design (ES EN Art 4.4.1.2 2015)

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.

b) 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 Φ16 longitudinal bar and Φ25 nominal maximum aggregate size;

Therefore; Cmin, b= 16mm.

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

c) 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 Δc_{dev} for use in a country may be found in its National Annex .The recommended values is 10 mm

We obtain from relation

$c_{nom} = c_{min} + \Delta c_{dev}$ = 16 + 10 = 26 mm .

d) 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

Cfire = 20mm

Hs = 120mm

Governing Cover for Design

Governing cover accounting for

- Corrosion
- Bond/Durability
- Fire

Cover = 26mm

Concrete C 20/25

Rebar S400

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

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

B. Step 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)

lo=0.5 *l√((1+ $\boldsymbol{K1\ 0.45+k1}$) (1+ $\boldsymbol{K2\ 0.45+k2}$))

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

$\mathbf{K1}\ x-x=\frac{\left(\frac{EI}{l}\right)c}{\left(2*\frac{EI}{l}\right)b}>0.1$……(Asmare 2020)

K1x-x=(($\boldsymbol{EI/l}$)$\boldsymbol{c}$/($\boldsymbol{2*EI/l}$)$\boldsymbol{b}$)= 4166666667/1061207.528=3926.344806

K2x-x=(($\boldsymbol{EI/l}$)$\boldsymbol{c}$/($\boldsymbol{2*EI/l}$)$\boldsymbol{b}$)= 4166666667/2763561.271=1507.716406

K1y-y=4166666667/2615261.044=1593.212531

K2y-y=4166666667/2763561.271=611.7936118

Lox-x=0.5 *l√((1+$\boldsymbol{K1x-x/0.45+k1x-x}$) (1+$\boldsymbol{K2x-x0.45+k2x-x}$))

Lox-x=0.5*((1+3926.35/3926.8) (1+1507.722/1508.172))

Lox-x=2.997m

Loy-y=0.5 *l√((1+$K1y-y/0.45+k1y-y$) (1+$K2y-y/0.45+k2y-y$))

Loy-y=0.5*4((1+1593.21/1593.66) (1+611.79/0.45+612.24))

Loy-y=2.997m

C. Step maximum and minimum reinforcement calculation

As,min=max $\begin{cases} (0.1*NED/fyd)=0.1*357.48/347.8=102.7mm2 \\ 0.002*Ac=0.002*500*400=400mm2 \end{cases}$

As,min=400mm2

As,max=$0.08*Ac$=0.04*500*400=16000mm2

Step 4 design action

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

eix-x= max $\begin{cases} lo/400=2997/400=7.49mm \\ h/30=300/30=10mm \\ 20mm \end{cases}$

eix-x = 20mm

eiy-y= max $\begin{cases} lo/400=2997/400=7.49mm \\ h/30=300/30=10mm \\ 20mm \end{cases}$

eiy-y = 20mm

Note:

The first order effect includes the geometric imperfection, but not consideration of slenderness.

The second order effect is considered the slenderness of structural members or deformation of the structure.

First order iteration

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-dirn

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

M01x-x=min {|24.08|,|39.04|+0.02*357.45}

M01x-x=35.23KNm

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

M02x-x=max {|24.08,|39.04|+0.02*357.45}

M02x-x=46.19KNm

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

Moex-x=0.4*35.23+0.6*46.19≥0.4*46.19

Moex-x=41.81KNm

MEDx-x=18.48KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|74.26|,|106.98|+0.02*357.45}

M01y-y=81.41KNm

M02y-y=max {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M02y-y=max {|74.26|,|1036.98|+0.02*357.45}

M02y-y=114.13KNm

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

Moey-y=0.4*81.41+0.6*114.13≥0.4*114.13

Moey-y=101.04KNm

MEDy-y=45.56KNm

Check slenderness

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

i-is radius of gyration of the uncracked concrete

The minimum limiting value of slenderness is

$\lambda lim = 20*A*B*C/\sqrt{n}$

$\lambda limx\text{-}x$

$=20*A*B*C/\sqrt{n}$

Where, A – $1/(1 + 0.2\varphi ef)$, φef-effective creep ratio

B - $\sqrt{1 + 2\omega}$,-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 =mo1/mo2=24.08/39.04=0.62

y- dxn

r_m =mo1/mo2=74.26/106.98=0.69

$Ac = b * h = 500 * 400 = 20000mm^2$

for rectangular column

n=NEd/Acfcd=(357.48*1000/(20000*11.33)=1.58

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

$=20*0.7*1.1*0.62/\sqrt{1.58}=7.58$

A=0.7

B=1.1

C=1.7-(0.69)=1.01

n=NEd/Acfcd=(357.48*1000/(200000*11.33)=1.58

$\lambda limy\text{-}y = 20*A*B*C/\sqrt{n} = 20*0.7*1.1*0.69/\sqrt{1.58} = 7.59$

$\lambda x = \frac{lox}{ix}$ where i= √ Ic-x/Ac=√4166666667*10^{-12}/0.4*0.5=2.08 $\lambda y = \frac{loy}{iy} =$

where iy =√ Ic-y/Ac=√32000000000*10^{-12}/0.4*0.5=1.6

Check slenderness

λx-x >λ limx-x, slender column otherwise short column

λy- y>λ limy-y, slender column otherwise short column

λx-x < λ limx-x= 2.0888< 7.58, short column

λy- y< λ limy-y= 1.6 < 7.59, short column

vsd=Nsd/(Ac*fcd,) =357.48*1000/ (500*400*11.33) =0.157

μsdx-x=MEDx-x/(fcd*Ac*h)=18.48*1000000/ (11.33*200000*500) =0.016

μsdy-y= MEDy-y/(fcd*Ac*h) =45.56*1000000/ (11.33*200000*500) =0.04

Using biaxial rectangular chart number

ω=0.2

Step 5 reinforcement calculation

 Longitudinal reinforcement

As,tot = ω*Ac*fcd/fyd

As,tot=0.2*200000*11.33/347.8

As,tot=1303.5m²

As min=400mm² and As max=1600mm²

As min <As<As max……OK

Take Ø16mm, as=3.14*(16)²/4=200.96mm²

Number of bar=As/as=1303.5mm²/200.96mm²=5.48≈8bar

Provide 6 Ø16mm

(kassimal n.d.)

- For shear reinforcement(2015)

Diameter of bar = max
- ¼*longitudinal bar diameter=16/**4** =4
- 6mm
- Diameter of stirrup = 8mm

From above take Ø8mm

Spacing of shear reinforcement= min
- 20*$\emptyset long$, =20*16mm=320mm
- lesser dimension of column =300mm
- 400mm

Finally, provideØ8 $/C$ 300mm

A. Step Rectangular column sample calculation for Column two on the 3rd floor column

Given data

intermediate beam	depth(m)	width(m)	Length(m)
b1(x,x)	0.5	0.4	5.61
b2(x,x)	0.5	0.4	6.52
b3(y,y)	0.5	0.4	6.64
b4(y,y)	0.5	0.4	1.5

bottom beam	depth(m)	width(m)	length(m)
b1(x,x)	0.5	0.4	5.61
b2(x,x)	0.5	0.4	6.52
b3(y,y)	0.5	0.4	6.64
b4(y,y)	0.5	0.4	1.5
Column	0.5	0.4	3

$M_{topx\text{-}x}$=27.3KNm

$M_{topy\text{-}y}$=68.9KNm

$M_{botx\text{-}x}$=40KNm

$M_{boty\text{-}y}$=98.6KNm

NED=915.3KN

B. Step; 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)

lo=0.5 *l√((1+ $\boldsymbol{K1\ 0.45+k1}$) (1+ $\boldsymbol{K2\ 0.45+k2}$))

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

$\mathbf{K1}\ x-x=\frac{\left(\frac{EI}{l}\right)c}{\left(2*\frac{EI}{l}\right)b}>0.1$……..(Asmare 2020)

K1x-x=($\boldsymbol{(EI/l)c/(2*EI/l)b}$)= 4166666667/2763561.271=3926.344806

K2x-x=($\boldsymbol{(EI/l)c/(2*EI/l)b}$)= 4166666667/2763561.271=1507.716406

K1y-y=4166666667/6810575.636=1593.212531

K2y-y=4166666667/2763561.271=611.7936118

Lox-x=0.5 *l√((1+$\boldsymbol{K1x-x/0.45+k1x-x}$) (1+$\boldsymbol{K2x-x/0.45+k2x-x}$))

Lox-x=0.5*((1+1507.72/1508.17) (1+1507.722/1508.172))

Lox-x=3m

Loy-y=0.5 *l√((1+$K1y-y/0.45+k1y-y$) (1+$K2y-y/0.45+k2y-y$))

Loy-y=0.5*4((1+611.79/612.24) (1+611.79/0.45+612.24))

Loy-y=3m

C. Step maximum and minimum reinforcement calculation

As,min=max $\begin{cases}(0.1*NED/fyd)=0.1*915.3/347.8=26m2\\0.002*Ac=0.002*500*400=400mm2\end{cases}$

As,min=400mm2

As,max=$0.08*Ac$=0.04*500*400=16000mm2

D. Step design action (ES EN 1992;Art 5.2.7 ., 2)

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

eix-x= max { lo/400=3000/400=7.5mm ; h/30=300/30=10mm ; 20mm }

eix-x = 20mm

eiy-y= max { lo/400=3000/400=7.5mm ; h/30=300/30=10mm ; 20mm }

eiy-y = 20mm

Note:

The first order effect includes the geometric imperfection, but not consideration of slenderness.

The second order effect is considered the slenderness of structural members or deformation of the structure.

First order iteration

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-dirn

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

M01x-x=min {|27.3|,|40|+0.02*915.3}

M01x-x=45.61KNm

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

M02x-x=max {|27.3,|40|+0.02*915.3}

M02x-x=58.31KNm

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

Moex-x=0.4*45.61+0.6*58.31≥0.4*58.31

Moex-x=53.23KNm

MEDx-x=23.33KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|66.9|,|98.6|+0.02*915.3}

M01y-y=85.23KNm

M02y-y=max {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M02y-y=max {|66.9|,|98.6|+0.02*915.3}

M02y-y=117KNm

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

Moey-y=0.4*85.23+0.6*117≥0.4*117

Moey-y=104.3KNm

MEDy-y=46.8KNm

Check slenderness

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

i-is radius of gyration of the uncracked concrete

The minimum limiting value of slenderness is

$\lambda lim = 20*A*B*C/\sqrt{n}$

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=27.3/40=0.7

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

y- dxn

r_m =mo1/mo2=68.9/98.6=0.7

r_m=mo1/mo2=27.3/40=0.68

Ac=$b * h$ = 500 * 400 = 20000mm^2

for rectangular column

n=NEd/Acfcd=(915.3*1000/(200,000*11.33)=0.4

Therefore, slenderness limit (ƛlimx-x) =20*A*B*C/√n

=20*0.7*1.1*1.02/√0.4=25

A=0.7

B=1.1

C=1.7-(0.7)=1

n=NEd/Acfcd=(915.3*1000/(200000*11.33)=0.4

λlimy-y =20*A*B*C/√n=20*0.7*1.1*1/√0.4=24.44

λ x $= \frac{lox}{ix}$ where i= √ Ic-x/Ac=√4166666667*10^{-12}/0.4*0.5=2.08

$$\lambda x = \frac{3}{2.08} = 1.44$$

λ y $= \frac{loy}{iy}$where iy =√ Ic-y/Ac=√32000000000*10^{-12}/0.4*0.5=1.6

$$\lambda y = \frac{3}{1.6} = 1.88$$

Check slenderness

λx-x >λ limx-x, slender column otherwise short column

λy- y>λ limy-y, slender column otherwise short column

λx-x < λ limx-x=1.44< 25, short column

λy- y< λ limy-y= 1.88<24.44, short column

vsd=Nsd/(Ac*fcd,) =915.3*1000/ (500*400*11.33) =0.4

μsdx-x=MEDx-x/(fcd*Ac*h)=23.33*1000000/ (11.33*200000*500) =0.021

μsdy-y= MEDy-y/(fcd*Ac*h) =46.8*1000000/ (11.33*200000*500) =0.04

Using biaxial rectangular chart number

ω=0.2

Step 5 reinforcement calculation

 Longitudinal reinforcement

As,tot = ω*Ac*fcd/fyd

As,tot=0.2*200000*11.33/347.8

As,tot=1303.5m²

As min=400mm² and As max=1600mm²

As min <As<As max……OK

Take $\emptyset 16mm$, as=3.14*(16)²/4=200.96mm²

Number of bar=As/as=1303.5mm²/200.96mm²=5.48≈8bar

Provide 5 Ø16mm

 For shear reinforcement

Diameter of bar = max
- ¼*longitudinal bar diameter=16/**4** =4
- 6mm
- Diameter of stirrup = 8mm

From above take Ø8mm

Spacing of shear reinforcement= min
- 20*$\emptyset \boldsymbol{long}$, =20*16mm=320mm
- lesser dimension of column =300mm
- 400mm

Finally, provide$\emptyset 8 \ /C \ 300mm$

A. Step Rectangular column sample calculation for Column two on the 2nd floor column

Given data

intermediate beam	depth(m)	width(m)	Length(m)
b1(x,x)	0.5	0.4	5.61
b2(x,x)	0.5	0.4	6.52
b3(y,y)	0.5	0.4	6.64
b4(y,y)	0.5	0.4	1.5

bottom beam	depth(m)	width(m)	length(m)
b1(x,x)	0.5	0.4	5.61
b2(x,x)	0.5	0.4	6.52
b3(y,y)	0.5	0.4	6.64
b4(y,y)	0.5	0.4	1.5
Column	0.5	0.4	3

$M_{topx\text{-}x}$=26.31.KNm

$M_{topy\text{-}y}$=62.91KNm

$M_{botx\text{-}x}$=26.75KNm

$M_{boty\text{-}y}$=84.38KNm

NED=2678.3KN

B. Step 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)

$lo=0.5 * l\sqrt{((1+ \boldsymbol{K1}\ 0.45+\boldsymbol{k1})\ (1+ \boldsymbol{K2}\ 0.45+\boldsymbol{k2}))}$

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

$$\mathbf{K1}\ x-x=\frac{\left(\frac{EI}{l}\right)c}{\left(2*\frac{EI}{l}\right)b}>0.1\text{……..(Asmare 2020)}$$

K1x-x=(($\boldsymbol{EI/l}$)$\boldsymbol{c}$/(2∗$\boldsymbol{EI/l}$)$\boldsymbol{b}$)= 4166666667/2763561.271=3926.344806

K2x-x=(($\boldsymbol{EI/l}$)$\boldsymbol{c}$/(2∗$\boldsymbol{EI/l}$)$\boldsymbol{b}$)= 4166666667/2763561.271=1507.716406

K1y-y=4166666667/6810575.636=1593.212531

K2y-y=4166666667/2763561.271=611.7936118

Lox-x=0.5 *l√((1+$\boldsymbol{K1x-x/0.45}$+$\boldsymbol{k1x-x}$) (1+$\boldsymbol{K2x-x/0.45}$+$\boldsymbol{k2x-x}$))

Lox-x=0.5*((1+1507.72/1508.17) (1+1507.722/1508.172))

Lox-x=3m

Loy-y=0.5 *l√((1+$K1y-y/0.45+k1y-y$) (1+$K2y-y/0.45+k2y-y$))

Loy-y=0.5*4((1+611.79/612.24) (1+611.79/0.45+612.24))

Loy-y=3m

Step 3 maximum and minimum reinforcement calculation

As,min=max $\begin{cases}(0.1*NED/fyd)=0.1*915.3/347.8=26\text{m2}\\0.002*Ac=0.002*500*400=400\text{mm2}\end{cases}$

As,min=400mm2

As,max=0.08∗Ac=0.04∗500∗400=16000mm2

Step 4 design action

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

eix-x= max $\begin{cases}lo/400=3000/400=7.5mm\\h/30=300/30=10mm\\20\text{mm}\end{cases}$

eix-x = 20mm

eiy-y= max $\begin{cases}lo/400=3000/400=7.5mm\\h/30=300/30=10mm\\20\text{mm}\end{cases}$

eiy-y = 20mm

Note:

The first order effect includes the geometric imperfection, but not consideration of slenderness.

The second order effect is considered the slenderness of structural members or deformation of the structure.

First order iteration

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-dirn

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

M01x-x=min {|26.31|,|26.75|+0.02*2678.3}

M01x-x=79.88KNm

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

M02x-x=max {|26.31,|26.75|+0.02*2678.3}

M02x-x=80.33KNm

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

Moex-x=0.4*79.88+0.6*80.33≥0.4*80.33

Moex-x=80.15KNm

MEDx-x=32.13KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|62.91|,|84.38|+0.02*2678.3}

M01y-y=116.5KNm

M02y-y=max {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M02y-y=max {|62.91|,|84.38|+0.02*2678.3}

M02y-y=138KNm

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

Moey-y=0.4*116.5+0.6*138≥0.4*138

Moey-y=129.4KNm

MEDy-y=55.2KNm

Check slenderness

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

i-is radius of gyration of the uncracked concrete

The minimum limiting value of slenderness is

λlim=20*A*B*C/√n

λlimx-x

=20*A*B*C/√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=26.31/26.75=0.98

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

y- dxn

r_m =mo1/mo2=62.91/84.38=0.75

r_m=mo1/mo2=27.3/40=0.98

Ac=$b * h$ = 500 * 400 = 20000mm^2

for rectangular column

n=NEd/Acfcd=(2678.3*1000/(200,000*11.33)=1.2

Therefore, slenderness limit (ƛlimx-x) =20*A*B*C/√n

=20*0.7*1.1*0.72/√1.2=10.08

A=0.7

B=1.1

C=1.7-(0.98)=0.72

n=NEd/Acfcd=(2678.3*1000/(200000*11.33)=1.2

λlimy-y =20*A*B*C/√n=20*0.7*1.1*0.95/√1.2=13.33

λ x $= \frac{lox}{ix}$ where i= √ Ic-x/Ac=√4166666667*10^{-12}/0.4*0.5=2.08

$$\lambda x = \frac{3}{2.08} = 1.44$$

λ y $= \frac{loy}{iy}$where iy =√ Ic-y/Ac=√32000000000*10^{-12}/0.4*0.5=1.6

$$\lambda y = \frac{3}{1.6} = 1.88$$

Check slenderness

λx-x >λ limx-x, slender column otherwise short column

λy- y>λ limy-y, slender column otherwise short column

λx-x < λ limx-x=1.44< 10.08, short column

λy- y< λ limy-y= 1.88<13.33, short column

vsd=Nsd/(Ac*fcd,) =2678.3*1000/ (500*400*11.33) =1.2

μsdx-x=MEDx-x/(fcd*Ac*h)=32.13*1000000/ (11.33*200000*500) =0.03

μsdy-y= MEDy-y/(fcd*Ac*h) =55.2*1000000/ (11.33*200000*500) =0.05

Using biaxial rectangular chart number

ω=0.2

C. Step reinforcement calculation

 Longitudinal reinforcement

As,tot = ω*Ac*fcd/fyd

As,tot=0.2*200000*11.33/347.8

As,tot=1303.5m²

As min=400mm² and As max=1600mm²

As min <As<As max……OK

Take Ø16mm, as=3.14*(16)²/4=200.96mm²

Number of bar=As/as=1303.5mm²/200.96mm²=5.48≈8bar

Provide 6 Ø16mm

 For shear reinforcement

Diameter of bar = max
- ¼*longitudinal bar diameter=16/**4** =4
- 6mm
- Diameter of stirrup = 8mm

From above take Ø8mm

Spacing of shear reinforcement= min { 20*$\emptyset long$, =20*16mm=320mm ; lesser dimension of column =300mm ; 400mm }

Finally, provide$\emptyset 8 \ /C \ 300mm$

A. Step Rectangular column sample calculation for Column two on the 1st floor column

Given data

intermediate beam	depth(m)	width(m)	Length(m)
b1(x,x)	0.5	0.4	5.61
b2(x,x)	0.5	0.4	6.52
b3(y,y)	0.5	0.4	6.64
b4(y,y)	0.5	0.4	1.5

bottom beam	depth(m)	width(m)	length(m)
b1(x,x)	0.5	0.4	5.61
b2(x,x)	0.5	0.4	6.52
b3(y,y)	0.5	0.4	6.64
b4(y,y)	0.5	0.4	1.5
Column	0.5	0.4	3

M_{topx-x}=17.04.KNm

M_{topy-y}=55.06KNm

M_{botx-x}=31.46KNm

M_{boty-y}=90.58KNm

NED=2678.3KN

B. Step 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)

lo=0.5 *l√((1+ $\boldsymbol{K1\ 0.45+k1}$) (1+ $\boldsymbol{K2\ 0.45+k2}$))

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

$$\mathbf{K1}\ \mathrm{x}-\mathrm{x}=\frac{\left(\frac{EI}{l}\right)c}{\left(2*\frac{EI}{l}\right)b}>0.1$$

K1x-x=($\boldsymbol{(EI/l)c/(2*EI/l)b}$)= 4166666667/2763561.271=3926.344806

K2x-x=($\boldsymbol{(EI/l)c/(2*EI/l)b}$)= 4166666667/2763561.271=1507.716406

K1y-y=4166666667/6810575.636=1593.212531

K2y-y=4166666667/2763561.271=611.7936118

Lox-x=0.5 *l√((1+$\boldsymbol{K1x-x/0.45+k1x-x}$) (1+$\boldsymbol{K2x-x/0.45+k2x-x}$))

Lox-x=0.5*((1+1507.72/1508.17) (1+1507.722/1508.172))

Lox-x=3m

Loy-y=0.5 *l√((1+$K1y-y/0.45+k1y-y$) (1+$K2y-y/0.45+k2y-y$))

Loy-y=0.5*4((1+611.79/612.24) (1+611.79/0.45+612.24))

Loy-y=3m

C. Step maximum and minimum reinforcement calculation

As,min=max $\begin{cases} (0.1*NED/fyd)=0.1*3751/347.8=107.8m2 \\ 0.002*Ac=0.002*500*400=400mm2 \end{cases}$

As,min=400mm2

As,max=$0.08*Ac$=$0.04*500*400$=16000mm2

Step 4 design action

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

eix-x= max $\begin{cases} lo/400=3000/400=7.5mm \\ h/30=300/30=10mm \\ 20mm \end{cases}$

eix-x = 20mm

eiy-y= max $\begin{cases} lo/400=3000/400=7.5mm \\ h/30=300/30=10mm \\ 20mm \end{cases}$

eiy-y = 20mm

Note:

The first order effect includes the geometric imperfection, but not consideration of slenderness.

The second order effect is considered the slenderness of structural members or deformation of the structure.

First order iteration

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-dirn

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

M01x-x=min {|17.04|,|31.46|+0.02*3751}

M01x-x=92.06KNm

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

M02x-x=max {|17.04,|31.46|+0.02*3751}

M02x-x=106.48KNm

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

Moex-x=0.4*92.06+0.6*106.48≥0.4*106.48

Moex-x=100.7KNm

MEDx-x=42.6KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|55.06|,|90.58|+0.02*3751}

M01y-y=130.08KNm

M02y-y=max {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M02y-y=max {|55.06|,|90.58|+0.02*3751}

M02y-y=165.6KNm

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

Moey-y=0.4*130.08+0.6*165.6≥0.4*138

Moey-y=151.4KNm

MEDy-y=55.2KNm

Check slenderness

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

i-is radius of gyration of the uncracked concrete

The minimum limiting value of slenderness is

$\lambda lim=20*A*B*C/\sqrt{n}$

$\lambda limx-x$

$=20*A*B*C/\sqrt{n}$

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=17.04/31.46=0.54

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

y- dxn

r_m =mo1/mo2=55.06/90.58=0.61

r_m=mo1/mo2=17.04/31.46=0.54

Ac=$b * h$ = 500 * 400 = 20000mm^2

for rectangular column

n=NEd/Acfcd=(2678.3*1000/(200,000*11.33)=1.2

Therefore, slenderness limit (λlimx-x) =20*A*B*C/√n

=20*0.7*1.1*1.16/√1.2=17.86

A=0.7

B=1.1

C=1.7-(0.54)=1.16

n=NEd/Acfcd=(2678.3*1000/(200000*11.33)=1.2

λlimy-y =20*A*B*C/√n=20*0.7*1.1*1.09/√1.2=16.79

λ x $= \frac{lox}{ix}$ where i= √ Ic-x/Ac=√4166666667*10^{-12}/0.4*0.5=2.08

$$\lambda x = \frac{3}{2.08} = 1.44$$

λ y $= \frac{loy}{iy}$where iy =√ Ic-y/Ac=√32000000000*10^{-12}/0.4*0.5=1.6

$$\lambda y = \frac{3}{1.6} = 1.88$$

Check slenderness

λx-x >λ limx-x, slender column otherwise short column

λy- y>λ limy-y, slender column otherwise short column

λx-x < λ limx-x=1.44< 17.86, short column

λy- y< λ limy-y= 1.88<16.79, short column

vsd=Nsd/(Ac*fcd,) =3751*1000/ (500*400*11.33) =1.65

μsdx-x=MEDx-x/(fcd*Ac*h)=42.6*1000000/ (11.33*200000*500) =0.04

μsdy-y= MEDy-y/(fcd*Ac*h) =55.2*1000000/ (11.33*200000*500) =0.05

Using biaxial rectangular chart number

ω=0.2

D. Step reinforcement calculation

 Longitudinal reinforcement

As,tot = ω*Ac*fcd/fyd

As,tot=0.2*200000*11.33/347.8

As,tot=1303.5m²

As min=400mm² and As max=1600mm²

As min <As<As max……OK

Take Ø16mm, as=3.14*(16)²/4=200.96mm²

Number of bar=As/as=1303.5mm²/200.96mm²=5.48≈8bar

Provide 6 Ø16mm

 For shear reinforcement

Diameter of bar = max
- ¼*longitudinal bar diameter=16/**4** =4
- 6mm
- Diameter of stirrup = 8mm

From above take Ø8mm

Spacing of shear reinforcement= min
- 20*Ø$\boldsymbol{long}$, =20*16mm=320mm
- lesser dimension of column =300mm
- 400mm

Finally, provideØ8 /C 300mm

A. Steps rectangular column sample calculation for Column two on the ground floor column

Given data

intermediate beam	depth(m)	width(m)	Length(m)
b1(x,x)	0.5	0.4	5.61
b2(x,x)	0.5	0.4	6.52
b3(y,y)	0.5	0.4	6.64
b4(y,y)	0.5	0.4	1.5

bottom beam	depth(m)	width(m)	length(m)
b1(x,x)	0.5	0.4	5.61
b2(x,x)	0.5	0.4	6.52
b3(y,y)	0.5	0.4	6.64
b4(y,y)	0.5	0.4	1.5
Column	0.5	0.4	3

$M_{topx\text{-}x}$=12.34.KNm

$M_{topy\text{-}y}$=37.79KNm

$M_{botx\text{-}x}$=9.32KNm

$M_{boty\text{-}y}$=25.71KNm

NED=4703.46KN

B. Step 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)

lo=0.5 *l√((1+ $\boldsymbol{K1\ 0.45+k1}$) (1+ $\boldsymbol{K2\ 0.45+k2}$))

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

$$\mathbf{K1}\ \mathrm{x}-\mathrm{x}=\frac{\left(\frac{EI}{l}\right)c}{\left(2*\frac{EI}{l}\right)b}>0.1$$

K1x-x=(($\boldsymbol{EI/l}$)$\boldsymbol{c}$/(**2**∗$\boldsymbol{EI/l}$)$\boldsymbol{b}$)= 4166666667/2763561.271=3926.344806

K2x-x=(($\boldsymbol{EI/l}$)$\boldsymbol{c}$/(**2**∗$\boldsymbol{EI/l}$)$\boldsymbol{b}$)= 4166666667/2763561.271=1507.716406

K1y-y=4166666667/6810575.636=1593.212531

K2y-y=4166666667/2763561.271=611.7936118

Lox-x=0.5 *l√((1+$\boldsymbol{K1x-x/0.45+k1x-x}$) (1+$\boldsymbol{K2x-x/0.45+k2x-x}$))

Lox-x=0.5*((1+1507.72/1508.17) (1+1507.722/1508.172))

Lox-x=3m

Loy-y=0.5 *l√((1+$K1y-y/0.45+k1y-y$) (1+$K2y-y/0.45+k2y-y$))

Loy-y=0.5*4((1+611.79/612.24) (1+611.79/0.45+612.24))

Loy-y=3m

C. Step maximum and minimum reinforcement calculation

As,min=max $\begin{cases} (0.1*NED/fyd)=0.1*4703.46/347.8=135m2 \\ 0.002*Ac=0.002*500*400=400mm2 \end{cases}$

As,min=400mm2

As,max=$0.08*Ac$=0.08∗500∗400=16000mm2

Step 4 design action

T`he effect of imperfection taken from According to ES EN 1992: Art 5.2.7

eix-x= max $\begin{cases} lo/400=3000/400=7.5mm \\ h/30=400/30=13.33mm \\ 20mm \end{cases}$

eix-x = 20mm

eiy-y= max ` $\begin{cases} lo/400=3000/400=7.5mm \\ h/30=400/30=13.33mm \\ 20mm \end{cases}$

eiy-y = 20mm

`Note:

The first order effect includes the geometric imperfection, but not consideration of slenderness.

The second order effect is considered the slenderness of structural members or deformation of the structure.

First order iteration

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-dirn

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

M01x-x=min {|12.34|,|9.32|+0.02*4703.46}

M01x-x=103.4KNm

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

M02x-x=max {|12.34,|9.32|+0.02*4703.46}

M02x-x=106.41KNm

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

Moex-x=0.4*103.4+0.6*106.41≥0.4*106.41

Moex-x=105.2KNm

MEDx-x=42.6KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|37.79|,|25.71|+0.02*4703.46}

M01y-y=119.78KNm

M02y-y=max {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M02y-y=max {|37.79|,|25.71|+0.02*4703.46}

M02y-y=131.86KNm

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

Moey-y=0.4*119.78+0.6*131.86≥0.4*131.86

Moey-y=127.03KNm

MEDy-y=55.74KNm

Check slenderness

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

i-is radius of gyration of the uncracked concrete

The minimum limiting value of slenderness is

λlim=20*A*B*C/√n

λlimx-x

=20*A*B*C/√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=12.34/9.32=1.32

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

y- dxn

r_m =mo1/mo2=37.79/25.71=1.47

r_m=mo1/mo2=12.4/9.32=1.32

Ac=$b * h$ = 500 * 400 = 20000mm^2

for rectangular column

n=NEd/Acfcd=(4703.46*1000/(200,000*11.33)=2.07

Therefore, slenderness limit (ƛlimx-x) =20*A*B*C/√n

=20*0.7*1.1*0.38/√2.07=8.4

A=0.7

B=1.1

C=1.7-(1.32)=0.38

n=NEd/Acfcd=(4703.46*1000/(200000*11.33)=2.07

λlimy-y =20*A*B*C/√n=20*0.7*1.1*0.23/√2.07=2.53

λ x $= \frac{lox}{ix}$ where i= √ Ic-x/Ac=√4166666667*10^{-12}/0.4*0.5=2.08

$$\lambda x = \frac{3}{2.08} = 1.44$$

λ y $= \frac{loy}{iy}$where iy =√ Ic-y/Ac=√32000000000*10^{-12}/0.4*0.5=1.6

$$\lambda y = \frac{3}{1.6} = 1.88$$

Check slenderness

λx-x >λ limx-x, slender column otherwise short column

λy- y>λ limy-y, slender column otherwise short column

λx-x < λ limx-x=1.44< 8.4, short column

λy- y< λ limy-y= 1.88<2.53, short column

vsd=Nsd/(Ac*fcd,) =4703.46*1000/ (500*400*11.33) =2.07

μsdx-x=MEDx-x/(fcd*Ac*h)=42.6*1000000/ (11.33*200000*500) =0.04

μsdy-y= MEDy-y/(fcd*Ac*h) =55.74*1000000/ (11.33*200000*500) =0.05

Using biaxial rectangular chart number

ω=0.2

D. Step reinforcement calculation

 Longitudinal reinforcement

As,tot = ω*Ac*fcd/fyd

As,tot=0.2*200000*11.33/347.8

As,tot=1303.5m²

As min=400mm² and As max=1600mm²

As min <As<As max……OK

Take Ø16mm, as=3.14*(16)²/4=200.96mm²

Number of bar=As/as=1303.5mm²/200.96mm²=7.48≈8bar

Provide 8 Ø16mm

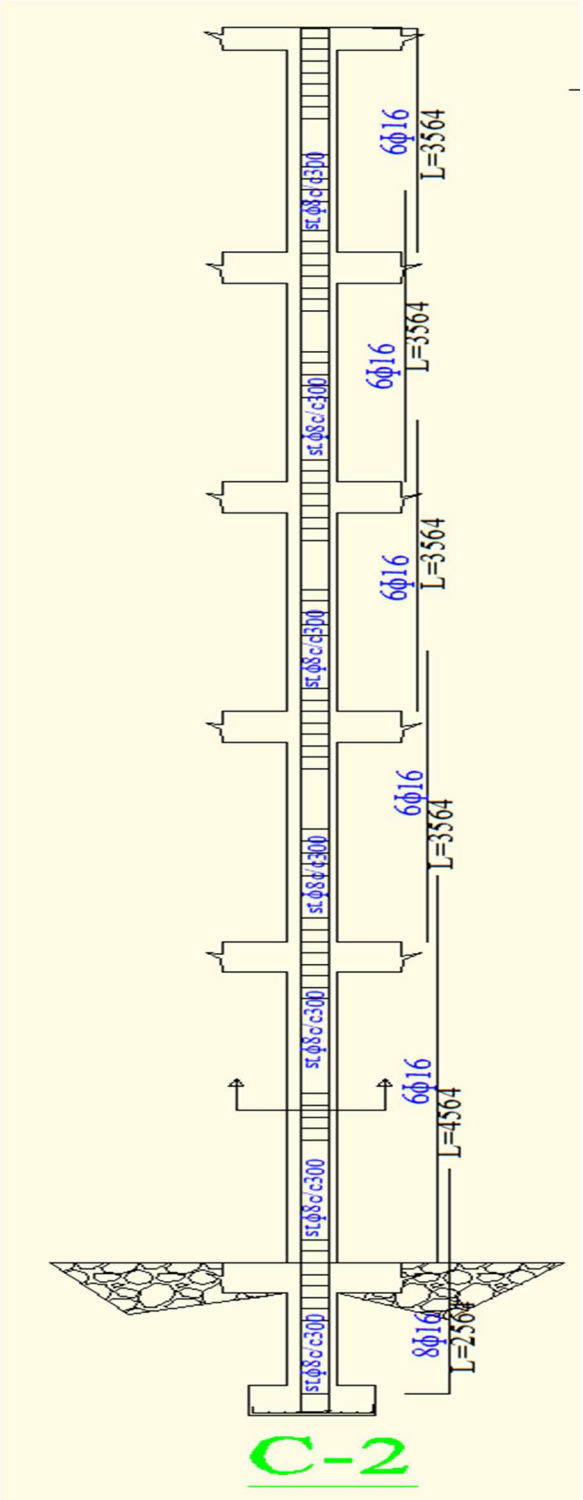
For shear reinforcement

Diameter of bar = max
- ¼*longitudinal bar diameter=16/**4** =4
- 6mm
- Diameter of stirrup = 8mm

From above take Ø8mm

Spacing of shear reinforcement= min
- 20*$\emptyset_{long}$, =20*16mm=320mm
- lesser dimension of column =300mm
- 400mm

Finally, provideØ8 /300mm


6ϕ16
L=3564
6ϕ16
L=3564
6ϕ16
L=3564
6ϕ16
L=3564
6ϕ16
L=4564
8ϕ16
L=2564
st.ϕ8@c/c300
st.ϕ8@c/c300
st.ϕ8@c/c300
st.ϕ8@c/c300
st.ϕ8@c/c300
st.ϕ8@c/c300
C-2

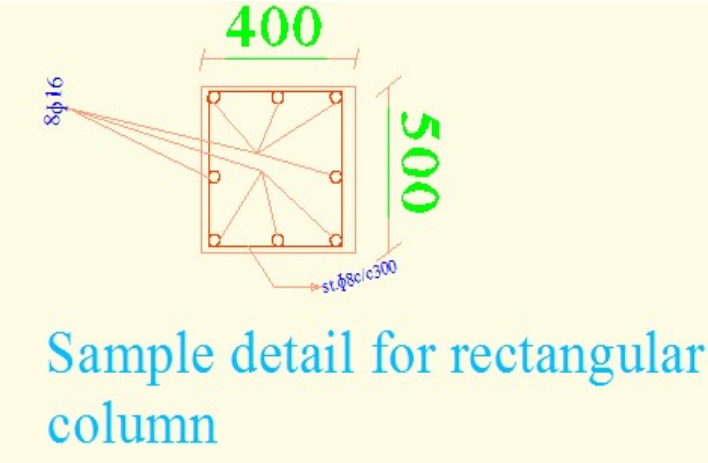


Figure 17 column detail

All the steps in the above calculation and values are summarized in the table below by using excel template.

In order to simplify the design procedure we categorize all column in categories based on the axial load applied. Each categories of column are designed for the maximum value of axial load in the range. The values

of axial load, Mx, My are obtained from ETABS 2016 analysis result as follow

footing (for sort)	Location	Footing	ground	1st	2nd	3rd	4th	gruop	Use axial load (col. Design)
1938.35	c1	1938.35	1885.12	1482.06	1059.91	664.06	274.3	1	1938.35
2950.89	c2	2950.89	2556.6	1982.62	1481.07	916.46	357.19	2	2950.89
2950.89	c3	2950.89	2165.85	1753.47	1333.02	817.88	300.55	2	2950.89
2950.89	c4	2950.89	2868.15	2224.49	1554.57	981.29	427.65	2	2950.89
3729.52	c5	3729.52	3632.29	2836.39	2107.31	1310.27	524.03	3	3928.44
2800.18	c6	2800.18	2715.58	2145.14	1637.02	1007.23	367.34	2	2950.89
3115.64	c7	3115.64	3026.05	2376.72	1638.21	1030.68	448.21	3	3928.44
4165.25	c8	4165.25	4064.63	3137.25	2257.81	1403.19	557.27	4	4842.56
2895.42	c9	2895.42	2764.14	2161.62	1627.19	1038.62	435.52	2	2950.89
1361.72	c10	1361.72	1260.85	992.86	711.75	483.91	260.91	1	1938.35
1596.62	c11	1596.62	1569.79	1180.72	824.3	517.16	211.88	1	1938.35
3729.95	c12	3729.95	3635.69	2884.82	2007.93	1250	504.2	3	3928.44
4842.56	c13	4842.56	4732.79	3771.12	2690.26	1660.05	617.39	4	4842.56
4693.37	c14	4693.37	4482.22	3529.89	2545.59	1578.06	594.35	5	4693.37
4681.17	c15	4681.17	4444.44	3449.63	2461.42	1534.64	602.19	4	4842.56
4075.41	c16	4075.41	3989.03	3081.76	2195.17	1359.2	516.99	4	4842.56

2697.11	c17	2697.11	2640.49	2096.97	1486.62	922.41	355.44	5	4693.37
3838.79	c18	3838.79	3748.74	3011.84	2171.57	1338.97	488.37	3	3928.44
4048.46	c19	4048.46	3962.22	3144.87	2255.99	1391.34	509.91	4	4842.56
3928.44	c20	3928.44	3841.05	3010.12	2162.67	1334.01	487.3	3	3928.44
2845.35	c21	2845.35	2788.46	2185.69	1573.36	970.82	357.69	5	4693.37

Table 13 Grouping columns based on having equal axial load

Column groups	Clumns	Governing column	Governing Axial load
1	C1,C10,C11	C1	1938.35
2	C2,C3,C4,C6,C9	C4	2950.95
3	C5,C7,C12,C18,C20	C20	3928.44
4	C8,C13,C15,C19	C13	4842.56
5	C14,C17,C21	C14	4693.38

2.2.13 Foundation design

Introduction

The foundation is the part of building structure which transmits load to, and into, the underlying soil or rock the foundation and its self-weight.

Purposes of foundations to distribute the load of the structure over a large bearing area so as to bring intensity of loading within the safe bearing capacity of the soil lying underneath.

- To load the bearing surface at a uniform rate so as to prevent unequal settlement.
- To prevent the lateral movement of the supporting material.
- To secure a level and firm bed for building operations.
- To increase the stability of the structure as a whole.
- The foundation should be design such that
- The soil below doesn't fail in shear
- The settlement is within the safe limit

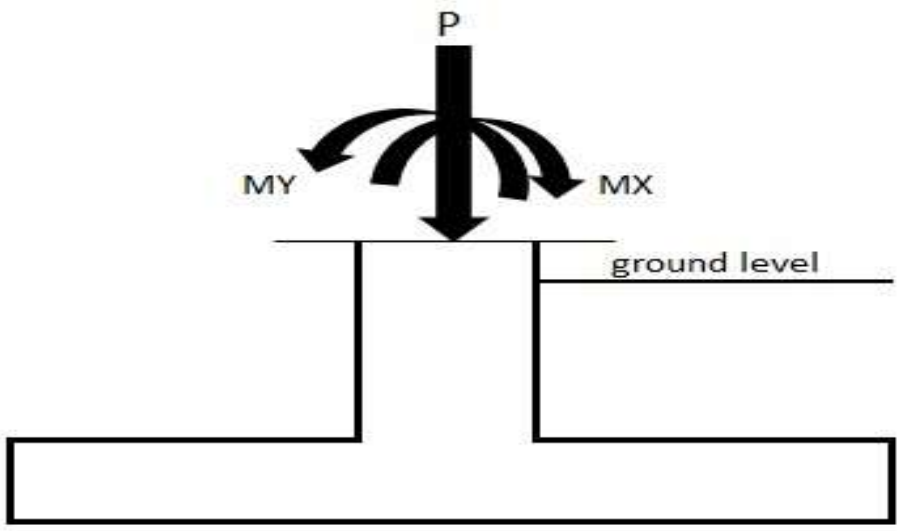
Column	POINT	LOAD	FZ	MX	MY
A1	1	COMB10	1410.55	4.06	4.57
A2	2	COMB10	2143.97	2.33	0.75
A3	3	COMB10	2230.44	0.01	3.78
A4	4	COMB10	2555.19	0.53	7.76
A5	5	COMB10	1947.96	4.99	1.81
B1	6	COMB10	1531.744	5.54	2.17
B2	7	COMB10	2550.04	0.53	5.71
B3	8	COMB10	2555.91	0.73	3.33
B4	9	COMB10	2972.99	4.31	3.63
B5	10	COMB10	2758.44	6.88	0.79
C1	11	COMB10	1607.59	2.23	1.6
C2	12	COMB10	2018.05	2.03	3.56
C3	13	COMB10	2062.78	5.56	4.11
C4	14	COMB10	2358.35	14.66	6.65
C5	15	COMB10	2905.91	6.07	0.73
D3	16	COMB10	955.86	14.66	5.54
D4	17	COMB10	2953.87	16.27	5.02
D5	18	COMB10	2817.65	1.83	1.85
E3	19	COMB10	1153.17	8.01	1.59
E4	20	COMB10	2918.95	6.46	1.73
E5	21	COMB10	2046.67	1.66	0.87

Table: The values of axial load, Mx, My are obtained from ETABS 2016 analysis result fom base reaction.

We have the footing of two type such as, F1$ F2

Footing 1 include the load from 500-1500 and

Footing 2 include the loads from 1500-3000



Bearing capacity

The soil must be capable of carrying the loads from the structure placed up on it without a shear failure and with the resulting settlements being tolerable for the structure. The design bearing resistance can be taken from the presumed design bearing pressure for different soils according to EBCS-7, 1995 ART 6.10.2 Table 6.3 or can be calculated analytically according to EBCS-7, 1995 ART 6.5.22 for drained as well as un-drained conditions. We have assumed type of soil to be cohesive clay stiff soil. And thus we take the corresponding bearing capacity from the presumed design bearing pressure from EBCS-7, 1995 ART 6.10.2 Table 6.3 page 805 which is 350$\boldsymbol{kPa}$

Sample footing design of footing 1 at the axis A1

Dimension of column is 500x400mm

From bearing capacity of the soil using the theoretical method

$\sigma_{Max} \leq Qmax$

$$\sigma_{Max/min} = \frac{p}{A} * \left(1 \pm \frac{6e_y}{B} \pm \frac{6e_x}{L}\right) \dots\dots\dots\dots 1$$

Now finding the eccentricity of each loadings

Eccentricity for service load

$ex = \frac{MY}{FZ}, ey = \frac{MX}{FZ}$

Mx = 4.06 $ My=4.57

$ey = \frac{4.06}{1410.55}$ =0.0029

$ex = \frac{4.57}{1410.55}$ = 0.0032

Since our column is square assume square foundation B=L

Substituting these values in equation1, the dimensions of the footing is calculated as fallows

$$\sigma_{All} = \frac{p}{A} * \left(1 \pm \frac{6e_y}{B} \pm \frac{6e_x}{B}\right) \dots\dots\dots\dots 1$$

At limiting condition σ_{Max} = Qmax

$$350 = \frac{1914.82}{B * B} * (1 + \frac{6 * 0.0182}{B} + \frac{6 * 0.0266}{B})$$

By rearranging the equation, we have the following simplified equation.

$350B^3$-1410.55B-51.623=0

By trial and error the value of B = 2.2m

$A=B^2=2.2*2.2=4.84m^2$

σmax=296.3Kpa, n = 265.84Kpa

Since x = 296.30< allowable(350Kpa)

So, the footing dimension is accepted

The stress at four corner of the footing

$$\sigma min = \frac{P}{A}\left(1 - 6 * \frac{ex}{B} - 6 * \frac{ey}{B}\right)$$

=265.84Kpa

$$\sigma int1 = \frac{P}{A}\left(1 + 6 * \frac{ex}{B} - 6 * \frac{ey}{B}\right)$$

=291.67Kpa

$$\sigma int2 = \frac{P}{A}\left(1 - 6 * \frac{ex}{B} + 6 * \frac{ey}{B}\right)$$

=291.20Kpa

$$\sigma max = \frac{P}{A}\left(1 - 6 * \frac{ex}{B} + 6 * \frac{ey}{B}\right)$$

=296.3Kpa

$\sigma aveg$ = (σmin + σ1 + σ2 + σ max) /4

=286.25Kpa

Depth determination

The critical section used for footing depth determination is similar to flat slab section.

These sections are

Wide beam shear section

Punching shear section

Bending moment section

Wide beam shear section

Critical section for shear is at distance d from the face of supports.

first assume the depth of footing pad to be

D=900mm Cover =50mm

Min φ=16

Depth for vrdc

The design shear at section Y-Y

$$Vyy = \sigma avge * B * (\frac{B}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000})$$

$dvrc = D - cover - min\phi/2,$

=900-50-16/2

=842

$$crdc = \frac{0.18}{\gamma}$$

=0.18/1.5=0.12

$$k = \min of \{1 + \sqrt{\frac{200}{dvrc}} = 1 + \sqrt{\frac{200}{842}} = 1$$

K=1.00

$$\rho min = min\begin{cases} 0.26\frac{ftcm}{400} = 0.26\frac{2.6}{400} = 0.001 \\ 0.02 \end{cases}$$

ρmin=0.001

$$Vyy = \sigma avge * B * (\frac{B}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000})$$

Vy-y =286.25*2.2(2.2/2-0.5/2-842/1000)

=31.49

The design shear at section X-X

$$Vxx = \sigma avge * L * (\frac{L}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000})$$

=286.25*2.2(2.2/2-0.5/2-842/1000)

Vxx=31.49

$$Vsd = \max \begin{matrix} 1. Vyy \\ 2. Vxx \end{matrix}$$

Vsd=31.49

$$VR = Crdc * K * [100 * \rho min * FCK]^{\frac{1}{3}} * B * dvrc$$

Vrdc=337.50

Vrdc>Vsd--------the depth is adequate for wide beam shear

Depth for vrdc min

The design shear at section Y-Y

$$Vyy = \sigma avge * B * (\frac{B}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000})$$

$$dvrmin = D - cover - min\phi/2$$

=700-50-16/2

=642

$$crdc = 0.18/_{\gamma}$$

=0.18/1.5=0.12

$$k = \min of \{1 + \sqrt{\frac{200}{dvrc}} = 1 + \sqrt{\frac{200}{642}} = 1$$

K=1.0008

$$\rho min = min \begin{cases} 0.26\frac{ftcm}{400} = 0.26\frac{2.6}{400} = 0.0014 \\ 0.02 \end{cases}$$

ρmin=0.0014

$$Vyy = \sigma avge * B * (\frac{B}{2} - \frac{col(H)}{2} - \frac{dvrmin}{1000})$$

Vy-y =286.25*2.2*(2.2/2-0.5/2-642/1000)

=130.99

The design shear at section X-X

$$Vxx = \sigma avge * L * (\frac{L}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000})$$

VX-X =130.99

$$Vsd = \max \begin{matrix} 1. Vyy \\ 2. Vxx \end{matrix}$$

Vsd=130.99

$$VRDC = Crdc * K * [100 * \rho min * FCK]^{\frac{1}{3}} * B * dvrmin$$

Vrdc=257.33

Vrdc>Vsd--------the depth is adequate for wide beam shear

The depth of footing for wide beam shear

$$d = \max \quad \frac{Dvrdc - cover - \frac{\phi}{2}}{Dvmin}$$

Dmax from 842m $ 642m is…dmax=842m

Punching shear

Critical section for shear is at distance 2d from the face of column

check punching shear at critical perimeter (column face)

$Ved, red = Fz - \sigma avg$ *(H+B)

Ved, red=1410.55-286.25*(0.5+0.5) =1124.3KN

With	*22*	*0.383972435*	cot θ	2.475	tanθ	0.404

Cw =1

$$V = 0.6 * [1 - \frac{FCK}{250}]$$

V=0.54*[1-25/250] =0.486

Z=0.9*d=0.9*842=757.8

U_0=perimeter of column

U_0 =2*((3*d/1000+H))+((3*d/1000+0.5))

= 2*((3*842/1000+0.5))+((3*842/1000+0.5))

=12.11

Vrd, max = cw* U_0*1000*z*Θ*fcd)/(cotΘ+tanΘ)/1000

= (1*12.11*1000*757.8**22*11.33/ (2.475+0.404)

=1610.74

Ved, red<Vrd, max ---1610.74>1124.3--------punching shear at the face of the column is **safe**

check punching shear at critical perimeter (1.5d distance from the column face)

$$Ved, red = Fz - \sigma avg * \left[3 * \frac{d}{1000} + H\right] * [3 * \frac{d}{1000} + B]$$

Ved, red = 680.26

U_0=perimeter of column

Uo=2∗[$Lpunching$+$Wpunching$]

d = 842

L Punching= [3*d/1000+H] ----[3*0.842+0.5] =3.026

W Punching= [3*d/100+H] ---[3*0.832+0.5] =3.026

Uo=2∗[3.026+3.026]]=12.11

$$Vrdc = crdc * k * \frac{[100 * \rho min * fck]1/3}{3} * Uo * d$$

$$Vrdc = 0.12 * 1 * \frac{[100 * 0.0014 * 25]1/3}{3} * 12.11 * 842$$

$Vrdc$= 1850.04

$$Vmin = 0.035 * k^{1.5} * Fck^{0.5} * Uo * d$$

$$Vmin = 0.035 * 1^{1.5} * 25^{0.5} * 12.11 * 642$$

Vmin=1360.56

Min of Vmin, Vrdc>>Ved, red----1850.04>680.26----- punching shear at the face of the column is safe check punching shear at critical perimeter (2d distance from the column face)

$$Ved,red = Fz - \sigma avg * \left[4 * \frac{d}{1000} + H\right] * [4 * \frac{d}{1000} + B]$$

$Ved, = -2872.16$

U0=perimeter of column

$$Uo = 2 * [Lpunching + Wpunch\ ]$$

L Punching=[4*d/100+H] ----[4*0.842+0.5] =3.868

W Punching=[4*d/100+H] ---[4*0.842+0.5] =3.868

$Uo = 2 * [3.868 + 3.868]$ =15.472

$$Vrdc=crdc* \text{ k} * \frac{[100*\rho min*fck]1/3}{3}*Uo*d$$

$Vrdc\ Vrdc = 0.12 * 1 * [100 * 0.0014 * 25](1/3) * 12.11 * 842$

Vrdc=1660.67

$Vmin = 0.035 * k^{1.5} * Fck^{0.5} * Uo * d$

$Vmin = 0.035 * 1^{1.5} * 25^{0.5} * 12.11 * 842$

Vmin=1784.40

Min of Vmin, Vrdc>>Ved, red----1660.67>-2872.16----- punching shear at the face of the column is safe

The depth of footing required for both wide beam and punching shear

D=d+cover/2=842+50+8=900

Reinforcement Calculation

parallel to longer direction

$$Msdx = \sigma avg * B * [\left(\frac{L}{2} - \frac{col(H)}{2}\right)^2]/B$$

$Msdx$=286.25∗2.2∗[($\frac{2.2}{2}$− $\frac{col(0.5)2)}{2}$]/2.45 = 122.09

sd=msd∗10^6/[Fcd∗1000∗d^2]

122.09∗10^6/[11.33∗1000∗842^2]

Sd =0.02

KZ=0.986

Msdx =122.09kNm

Kz,x	
SD	Kz
0.04	0.976
0.021	0.986
0.05	0.971

Kz,y	
SD	Kz
0.04	0.976
0.021	0.986
0.05	0.971

$ASc alculated = msdx * 106/(kz * d * Fyd)$

$ASc alculated$ = 122.09* 10^6/ (0.986 * 842 * 347.83)

AS calculated=422.79mm^2

$ASmin$ = 0.26 * $fctm/fyk$) * 1000 * d

$ASmin$ = (0.26 *2.565/400) * 1000 * 842

Asmin=1403.83mm2

$ASmax$ = 0.04 * 1000 * d

$ASmax$ = 0.04 * 1000 * 842

ASmax =33680mm2

AS, provided = AS calculated <= Asmin <=ASmax

As provided=1403.83 mm2

$ASc alculated = msdx * 10^6/(kz * d * Fyd)$

$Msdx = \sigma avg * L * [(\frac{B}{2} - \frac{COL(H)}{2})2]/L$

$Msdx$ = 286.25 * 2.2* [(2.2/2−(0.5)/ 2) 2]/2.45

Msdx=185.5Knm

Sd=$msd * 10^6/[Fcd * 1000 * d^2]$

=$185.50 * 106/[11.33 * 1000 * 842^2]$

Sd = 0.021

KZ=0.986

Kz, x	
Sd	Kz
0.04	0.976
0.021	0.986
0.05	0.971

Kz,y	
sd	Kz
0.04	0.976
0.021	0.986
0.05	0.971

$AScalculated = msdx * 10^6/(kz * d * Fyd)$

$AScalculated$ = 185.5.50 * 10^6/(0.986 * 842 * 347.83)

AS calculated=642.40mm^2

$ASmin$ = 0.26 * $fctm/fyk$) * 1000 * d

$ASmin$ = (0.26 *2.565/ 400) * 1000 * 842

Asmin=1403.82mm^2

$ASmax$ = 0.04 * 1000 * d

$ASmax$ = 0.04 * 1000 * 842

ASmax=33680mm^2

AS, provided = AS calculated <= Asmin <=ASmax

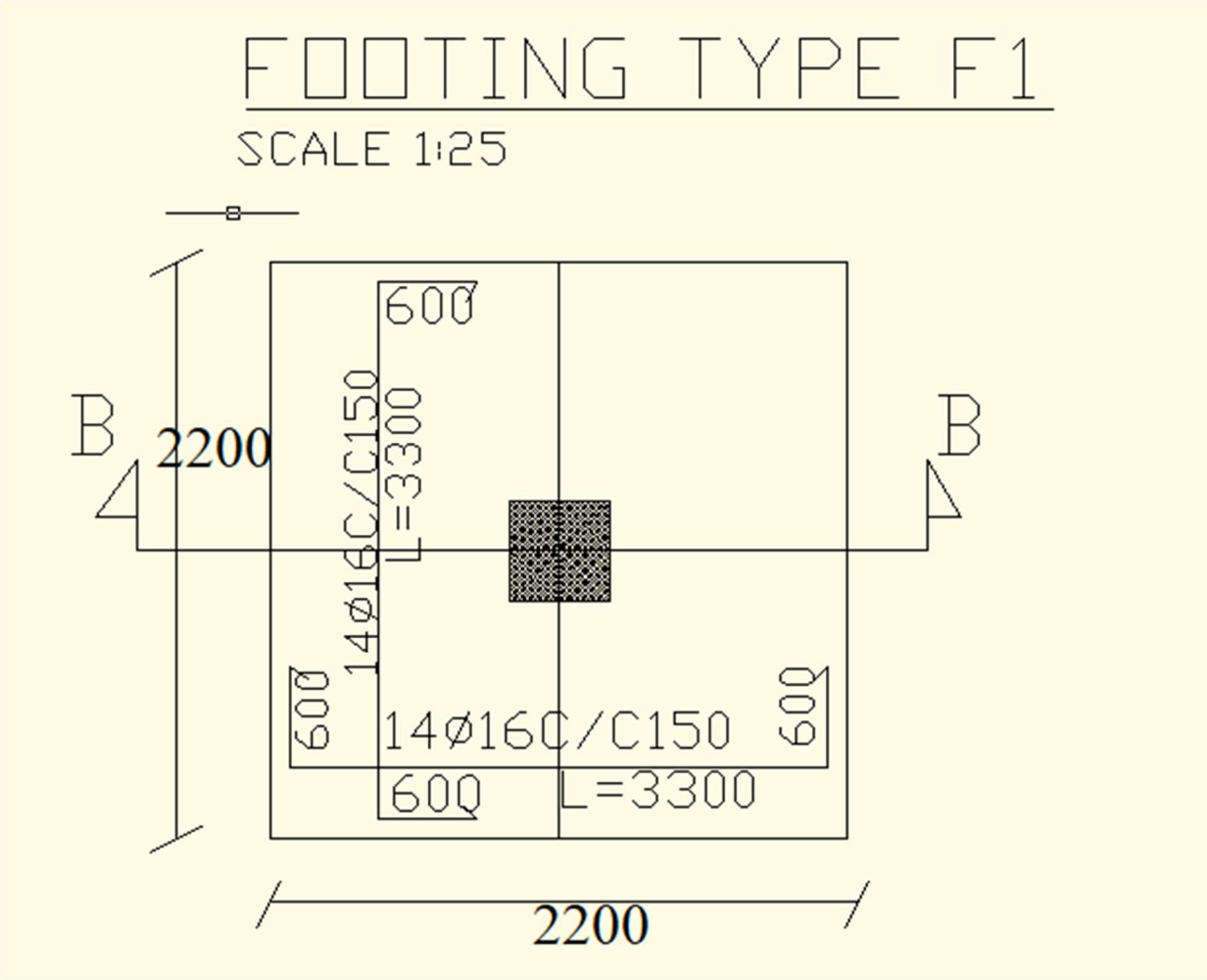
AS, provided =1403.82

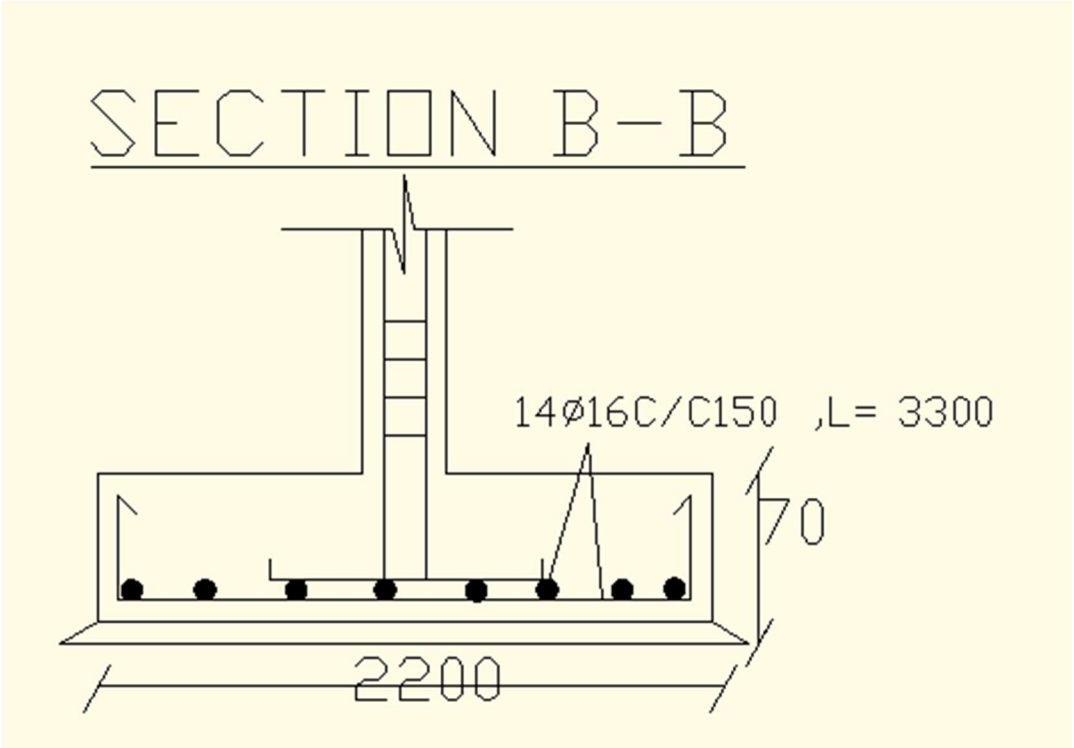
Spacing

Spacing= Min
- $\frac{b*as}{AS} = \frac{1000*3.14*8^2}{1403.82} = 143.15mm$
- $3 * d = 3 * 842 = 2526mm$
- 400

So we take the minimum spacing =150mm

Provide φ16 C/C 150mm




SECTION B-B
14Ø16C/C150 ,L= 3300
70
2200

Reference

1) Asmare, Betelihm. 2020. “Design of Columen.” Presented at the wolkite university, Ethiopia.
2) Bonsa, Mr. Abdi. 2020. “Design of Beam.” Presented at the Info-session for BSc Thesis/Project CEng5281, wolkite university, Ethiopia.
3) [1] “Design of G+4 building using cofficient and strip method,” final year project wolkite university, Ethiopia, 2008E.C.
4) “ES EN.” 1992.
5) “ES- EN Actions on Structures 1991-1-4.” 2015.
6) *ES EN 1992;2015 Art .7.42*. 1992. 1992;2015.
7) *ES EN 1992;Art 5.2.7*.
8) “ES EN Art 4.4.1.2.” 2015.
9) *ES EN Design Aid of Concrete Structures1992*. 2015. Addis Ababa, Ethiopia.
10) *ES EN Design of Concrete Structures*. 2015. Addis Ababa, Ethiopia.
11) “ES EN Design of Concrete Structures 1992-1-1.” 2010.
12) “ES-EN Actions on Structures 1991-1-1.” 2015.
13) Ethiopian ministry of construction. 2015. *ES EN 2 1992-2015*. Ethiopia.
14) Hibbeler. 2012. *Structural Analysis R.C*. 8th edition. united state of American.
15) kassimal, Aslam. *Structural Analysis*. 4th edition. 2011.
16) Mr. Abudeta. 2020a. “Earthquake Loading and Earthquake Resistant Structures.” Presented at the wolkite, Ethiopian.
17) Mr. desalegn. 2010E.C. “Beam Design.” Presented at the wolkite, Ethiopian.
18) Mr.mustefa. 2020b. “Design of Shallow Foundations.” Presented at the Info-session for BSc Thesis/Project CEng5281, wolkite university, Ethiopia.
19) “Sample Project G+10 Structural Buiding.” 2005.
20) 2012E.C. Presented at the wolkite university, Ethiopia.

Appendix

Appendix A Detail of roof analysis

Appendix A1; Terrain categories and terrain parameter

s(ES- EN Actions on Structures 1991-1-4 2015)

Table 4.1 — Terrain categories and terrain parameters

Terrain category	Z_0 m	Z_{min} m
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	1
II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle height	0.05	2
III Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as 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	1.0	10
The terrain categories are illustrated in Annex A.1.		

Appendix A2 External pressure coefficients for flat roofs

(ES-EN Actions on Structures 1991-1-1 2015)

Table 7.2-External pressure coefficients for flat roofs

Roof type		Zone							
		F		G		H		I	
		cpe,10	cpe,1	cpe,10	cpe,1	cpe,10	cpe,1	cpe,10	cpe,1
Sharp eaves		-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	+0,2	
								-0,2	
With Parapets	$h_p/h=0,025$	-1,6	-2,2	-1,1	-1,8	-0,7	-1,2	+0,2	
								-0,2	
	$h_p/h=0,05$	-1,4	-2,0	-0,9	-1,6	-0,7	-1,2	+0,2	
								-0,2	
	$h_p/h=0,10$	-1,2	-1,8	-0,8	-1,4	-0,7	-1,2	+0,2	
								-0,2	
Curved Eaves	r/h = 0,05	-1,0	-1,5	-1,2	-1,8	-0,4		+0,2	
								-0,2	
	r/h = 0,10	-0,7	-1,2	-0,8	-1,4	-0,3		+0,2	
								-0,2	
	r/h = 0,20	-0,5	-0,8	-0,5	-0,8	-0,3		+0,2	
								-0,2	
Mansard Eaves	α = 30°	-1,0	1,5	-1,0	-1,5	-0,3		+0,2	
								-0,2	
	α = 45°	-1,2	1,8	-1,3	-1,9	-0,4		+0,2	
								-0,2	
	α = 60°	-1,3	1,9	-1,3	-1,9	-0,5		+0,2	
								-0,2	

NOTE 1 For roofs with parapets or curved eaves, linear interpolation may be used for intermediate values of hp/h and r/h.

Appendix B; detail slab analysis

Appendix B1; Minimum cover

	Exposure Classes according to Table 4.1									
Corrosion										
	Carbonation-induced corrosion				Chloride-induced corrosion			Chloride-induced corrosion from sea-water		
	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
Indicative Strength Class	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	
Damage to Concrete										
	No risk	Freeze/Thaw Attack			Chemical Attack					
	X0	XF1	XF2	XF3	XA1	XA2	XA3			
Indicative Strength Class	C12/15	C30/37	C25/30	C30/37	C30/37		C35/45			

Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class [1)2)]	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Table 4.2: Minimum cover, $c_{min,b}$, requirements with regard to bond

Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)(see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $c_{min,b}$ should be increased by 5 mm.	

Minimum cover, (ES EN Design Aid of Concrete Structures1992 2015)

Table 4.4N: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Table 5.8: Minimum dimensions and axis distances for reinforced and prestressed concrete simply supported one-way and two-way solid slabs

Standard fire resistance	Minimum dimensions (mm)			
	slab thickness h_s (mm)	axis-distance *a*		
		one way	two way:	
			$l_y/l_x \leq 1,5$	$1,5 < l_y/l_x \leq 2$
1	2	3	4	5
REI 30	60	10*	10*	10*
REI 60	80	20	10*	15*
REI 90	100	30	15*	20
REI 120	120	40	20	25
REI 180	150	55	30	40
REI 240	175	65	40	50
l_x and l_y are the spans of a two-way slab (two directions at right angles) where l_y is the longer				

Appendix B2;Basic span/effective depth ratio

(ES EN Design of Concrete Structures 1992-1-1 2010)

Table 7.4N**:** Basic span/effective depth ratio (typical values for rectangular section for steel grade f_{yk} = 500 N/mm^2 and concrete class C30/35)

	Structural System	K	Basic span-effective depth ratio	
			Concrete highly stressed, ρ = 1.5%	Concrete lightly stressed, ρ = 0.5%
1.	Simply supported beam, one/two way simply supported slab	11.0	14	20
2.	End span of continuous beam or one-way continuous slab or two way spanning slab continuous over one long side	11.3	18	26
3.	Interior span of beam or one way or two way spanning slab	11.5	20	30
4.	Slab supported on columns without beam (flat slab) based on longer span	11.2	17	24
5.	Cantilever	0.4	6	8

The basic ratios are modified in particular cases as follows:

For flange section where the ratio of the flange width to the web width exceeds 3, the values should be multiplied by 0.8.

For beam and slabs, other than flat slab, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflection, the values should be multiplied by 7/span.

Where more tension reinforcement is provided ($A_{s,\ prov}$) than that calculated ($A_{s,\ req}$), multiply the values by = $\frac{A_{s,prov}}{A_{s,req}}$ (upper limit = 1.5).

Room Functions and their finishing materials

No	Functions	Finishing	Thickness	Unit wt (kN/m^3)
1	Kitchen	Ceramic	2 cm	21
2	Cafeteria	Terrazzo Tile	2 cm	23
3	Shop	Terrazzo Tile	2 cm	23
4	Corridor	Marble	3 cm	27
5	Retail	Terrazzo Tile	2 cm	23
6	Balcony	Marble	3 cm	27
7	Toilet	Ceramic	2 cm	21
8	Shower	Ceramic	2 cm	21
9	Landing	Marble	3 cm	27
10	Internet Café	Terrazzo Tile	2 cm	23
11	Bed Room	PVC Tile	2 cm	16
12	Janitor Room	Terrazzo Tile	2 cm	23

Dead Load Computation based on function rooms

Table 14 dead load computation

Function	Material	Thickness (m)	Unit Weight, γ (KN/m^2)	γ_t (KN/m^2)	Total Dead Load (KN/m^2)
Kitchen Toilet Shower	Ceramic	0.02	21	0.42	6.59
	Cement Screed	0.03	23	1.15	
	RC Slab	0.2	25	4.56	
	Plastering & Painting	0.02	23	0.46	
Cafeteria Shop Retail Internet Café Janitors Room	Terrazzo Tile	0.02	23	0.46	6.63
	Cement Screed	0.03	23	1.15	
	RC Slab	0.2	25	4.56	
	Plastering & Painting	0.02	23	0.46	
Corridor Balcony Landing	Marble	0.03	27	0.81	6.98
	Cement Screed	0.03	23	1.15	
	RC Slab	0.2	25	4.56	
	Plastering & Painting	0.02	23	0.46	
Bed Room	PVC	0.002	16	0.032	6.21
	Cement Screed	0.03	23	1.15	
	RC Slab	0.2	24	4.56	

	Plastering & Painting	0.02	23	0.046	

ES EN 1991-1-1:2015

Table 6.1 - Categories of use

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1)])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3:** Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores

[1)] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See ES EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3

NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex.

NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2

NOTE 3 See 6.3.2 for storage or industrial activity

ES EN 1991-1-1:2015

Note: Values for q_k and Q_k are given in Table 6.2 below. Where a range is given in this table, the value may set by the National annex. The recommended values, intended for separate application, are underlined. q intended for determination of general effects and Q_k for local effects. The National annex may define differ conditions of use of this Table.

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1.5 to 2.0	2.0 to 3.0
- Stairs	2.0 to 4.0	2.0 to 4.0
- Balconies	2.5 to 4.0	2.0 to 3.0
Category B	2.0 to 3.0	1.5 to 4.5
Category C		
- C1	2.0 to 3.0	3.0 to 4.0
- C2	3.0 to 4.0	2.5 to 7.0 (4.0)
- C3	3.0 to 5.0	4.0 to 7.0
- C4	4.5 to 5.0	3.5 to 7.0
- C5	5.0 to 7.5	3.5 to 4.5
Category D		
- D1	4.0 to 5.0	3.5 to 7.0 (4.0)
- D2	4.0 to 5.0	3.5 to 7.0

(2) Where necessary q_k and Q_k should be increased in the design (e.g. for stairs and balconies depending on the occupancy and on dimensions).

Table 15 Live Load for different functions based on EBCS-1

Function	Category		Live Load
Kitchen	A	General	2 KN/m^2
Cafeteria	C	C1	3 KN/m^2
Shop	D	D1	5 KN/m^2
Corridor	C	C3	5 KN/m^2
Retail	D	D1	5 KN/m^2
Balcony	A	Balconies	4 KN/m^2
Toilet	A	General	2 KN/m^2
Shower	A	General	2 KN/m^2
Landing	A	Satires	3 KN/m^2
Internet Café	C	C1	3 KN/m^2
Bed Room	A	General	2 KN/m^2
Janitor Room	D	D1	5 KN/m^2

Table 16 depth determination for roof

Nel	Ly (mm)	Lx (mm)	Span Ratio	K	Type of slab	Type of supporting Condition	d=l/N*K*F1*F2*F3	D=d+22
F1	2890	1500	1.93	0.4	Two Way	Cantilever	169.40	189.40
F2	5610	1500	3.74	0.4	One Way	Cantilever	169.40	189.40
F3	2800	1500	1.87	0.4	Two Way	Cantilever	169.40	189.40
F4	6640	1500	4.43	0.4	One Way	Cantilever	169.40	189.40
F5	6640	5610	1.18	11.5	Two Way	Interior	22.04	42.04
F6	6640	6520	1.02	11.5	Two Way	Interior	25.61	45.61
F7	6640	1500	4.43	0.4	One Way	Cantilever	169.40	189.40
F8	9170	5610	1.63	11.5	Two Way	Interior	22.04	42.04
F9	9170	6520	1.41	11.3	Two Way	End Span	26.06	46.06
F10	7980	5610	1.42	11.5	Two Way	Interior Span	22.04	42.04
F11	7980	6520	1.22	11.3	Two Way	End Span	26.06	46.06
F12	7980	166	48.07	0.4	One Way	Cantilever	18.75	38.75
F13	7980	5760	1.39	11.5	Two Way	Interior Span	22.63	42.63

F14	7980	1500	5.32	0.4	One Way	Cantilever	169.40	189.40
F15	10650	5610	1.90	11.5	Two Way	Interior	22.04	42.04
F16	10650	6520	1.63	11.5	Two Way	Interior	25.61	45.61
F17	10650	6320	1.69	11.5	Two Way	Interior	24.83	44.83
F18	10650	5760	1.85	11.5	Two Way	Interior	22.63	42.63
F19	3220	1500	2.15	0.4	Two Way	Cantilever	169.40	189.40
F20	5610	1500	3.74	0.4	Two Way	Cantilever	169.40	189.40
F21	6520	1500	4.35	0.4	One Way	Cantilever	169.40	189.40
F22	5760	1500	3.84	0.4	Two Way	Cantilever	169.40	189.40

Moment Analysis

Table 17 Moment Calculation for two way slab for roof

Moment Calculation for two way slab using coefficient method for roof floor											
Panel	Ly/Lx	Pd (KN/m)	$Lx^2(m^2)$	Bxs	bxf	bys	byf	M_{xs}(kN*m)	M_{xf}(kN*m)	M_{ys}(kN*m)	M_{yf}(kN*m)
P5	1.20	9.36	31.50	0.04	0.03	0.03	0.02	12.39	9.44	9.44	7.08
P6	1.00	9.36	42.50	0.03	0.02	0.03	0.02	12.33	9.55	12.73	9.55
P8	1.75	9.36	31.50	0.06	0.04	0.03	0.02	17.40	12.98	9.44	7.08
P9	1.40	9.36	42.50	0.04	0.03	0.04	0.03	15.52	11.94	14.72	11.14
P10	1.40	9.36	31.50	0.05	0.05	0.03	0.02	14.74	15.63	9.44	7.08
P11	1.20	9.36	42.50	0.04	0.03	0.03	0.02	16.71	12.73	12.73	9.55
P13	1.40	9.36	33.20	0.05	0.05	0.03	0.02	15.54	16.47	9.95	7.46
P15	2.00	9.36	31.50	0.06	0.05	0.03	0.02	18.58	14.15	9.44	7.08
P16	1.75	9.36	42.50	0.06	0.04	0.03	0.02	23.47	17.51	12.73	9.55
P17	1.75	9.36	39.90	0.06	0.04	0.03	0.02	22.04	16.44	11.95	8.96
P18	2.00	9.36	33.20	0.06	0.05	0.03	0.02	19.58	14.92	9.95	7.46

Table 18 moment cantilever slabs for roof

Moment Calculation for cantilever slabs for roof floor.					
Panel	Type of supportig Condition	$D.L_{ex}$	Lx	pd'=1.35 D.L +1. 5 L.L	MXS
P1	Two Way Cantilever		1.5	9.36	5.27
P2	One Way Cantilever		1.5	9.36	5.27
P3	Two Way Cantilever		1.5	9.36	5.27
P4	One Way Cantilever		1.5	9.36	5.27
P7	One Way Cantilever		1.5	9.36	5.27
P12	One Way Cantilever		1.66	9.36	5.83
P14	One Way Cantilever		1.5	9.36	5.27
P19	Two Way Cantilever		1.5	9.36	5.27
P20	Two Way Cantilever		1.5	9.36	5.27
P21	One Way Cantilever		1.5	9.36	5.27
P22	Two Way Cantilever		1.5	9.36	5.27
P23	Two Way Cantilever		1.5	9.36	5.27

Table 19 support moment adjustment between panels for roof

Support Moment adjustment between panels for roof fioor					
Panel	ML	MS	ms1-ms2	0.2*msla	moment distribute
P1&P4	5.27	5.27	0	1.054	Take average
P2&P5	12.4	5.27	7.13	2.48	moment distribute
P2&P6	12.3	5.27	7.03	2.46	moment distribute
P3&P7	5.27	5.27	0	1.054	Take average
P5&P8	17.4	12.4	5	3.48	moment distribute
P6&P9	15.51	12.3	3.21	3.102	moment distribute
P8&P10	17.4	14.74	2.66	3.48	Take average
P9&P11	16.71	15.51	1.2	3.342	Take average
P12&P17	22.04	5.83	16.21	4.408	moment distribute
P13&P18	19.58	15.55	4.03	3.916	moment distribute
P10&P15	18.58	14.74	3.84	3.716	moment distribute
P11&P16	23.5	16.71	6.79	4.7	moment distribute
P4&P19	5.27	5.27	0	1.054	Take average
P15&P20	18.58	5.27	13.31	3.716	moment distribute
P21&P16	23.5	5.27	18.23	4.7	moment distribute

P17&P21	22.04	5.27	16.77	4.408	moment distribute
P18&P22	19.58	5.27	14.31	3.916	moment distribute
P14&P19	5.27	5.27	0	1.054	Take average
P5&P6	12.73	9.4	3.33	2.546	moment distribute
P8&P9	14.72	9.4	5.32	2.944	moment distribute
P10&P11	12.73	9.43	3.3	2.546	moment distribute
P15&P16	12.73	9.43	3.3	2.546	moment distribute
P16&P17	12.73	11.95	0.78	2.546	Take average
P17&P18	11.95	9.95	2	2.39	Take average

Support moment adjustment for roof floor						
Panel	ML	MS	ms1-ms2	0.2*msla	moment distribute	Take average
P1&P4	5.27	5.27	0	1.054	Take average	5.27
P3&P7	5.27	5.27	0	1.054	Take average	5.27
P8&P10	17.4	14.74	2.66	3.48	Take average	16.07
P9&P11	16.71	15.51	1.2	3.342	Take average	16.11
P4&P19	5.27	5.27	0	1.054	Take average	5.27
P14&P19	5.27	5.27	0	1.054	Take average	5.27
P16&P17	12.73	11.95	0.78	2.546	Take average	12.34
P17&P18	11.95	9.95	2	2.39	Take average	10.95

Moment adjustment between panels

*T*able 20 support moment adjustment for roof

Support moment adjustment for roof fioor															
Panel	LY	LX	ML	MS	K1	K2	K1+K2	Df1	Df2	Δm	Δmdf1	Δmdf2	M1(adje)	M2(adje)	Panel
P2&P5	6.64	1.50	12.40	5.27	0.11	0.67	0.78	0.14	0.86	7.13	-1.03	6.10	11.37	11.37	P2&P5
P2&P6	6.64	1.50	12.30	5.27	0.11	0.67	0.78	0.14	0.86	7.03	-1.02	6.01	11.28	11.28	P2&P6
P5&P8	9.17	6.64	17.40	12.40	0.08	0.15	0.23	0.35	0.65	5.00	-1.76	3.24	15.64	15.64	P5&P8
P6&P9	9.17	6.64	15.51	12.30	0.08	0.15	0.23	0.35	0.65	3.21	-1.13	2.08	14.38	14.38	P6&P9
P12&P17	10.65	7.89	22.04	5.83	0.07	0.13	0.20	0.36	0.64	16.21	-5.79	10.42	16.25	16.25	P12&P17
P13&P18	10.65	7.89	19.58	15.55	0.07	0.13	0.20	0.36	0.64	4.03	-1.44	2.59	18.14	18.14	P13&P18
P10&P15	10.65	9.17	18.58	14.74	0.07	0.11	0.18	0.39	0.61	3.84	-1.51	2.33	17.07	17.07	P10&P15
P11&P16	10.65	6.64	23.50	16.71	0.07	0.15	0.22	0.32	0.68	6.79	-2.16	4.63	21.34	21.34	P11&P16
P15&P20	10.65	3.22	18.58	5.27	0.07	0.31	0.38	0.18	0.82	13.31	-2.46	10.85	16.12	16.12	P15&P20
P21&P16	10.65	3.22	23.50	5.27	0.07	0.31	0.38	0.18	0.82	18.23	-3.37	14.86	20.13	20.13	P21&P16
P17&P21	10.65	3.22	22.04	5.27	0.07	0.31	0.38	0.18	0.82	16.77	-3.10	13.67	18.94	18.94	P17&P21
P18&P22	10.65	3.22	19.58	5.27	0.07	0.31	0.38	0.18	0.82	14.31	-2.65	11.66	16.93	16.93	P18&P22
P5&P6	6.52	5.61	12.73	9.40	0.12	0.18	0.29	0.39	0.61	3.33	-1.31	2.02	11.42	11.42	P5&P6
P8&P9	6.52	5.61	14.72	9.40	0.12	0.18	0.29	0.39	0.61	5.32	-2.09	3.23	12.63	12.63	P8&P9
P10&P11	6.52	5.61	12.73	9.43	0.12	0.18	0.29	0.39	0.61	3.30	-1.29	2.01	11.44	11.44	P10&P11
P15&P16	6.52	5.61	12.73	9.43	0.12	0.18	0.29	0.39	0.61	3.30	-1.29	2.01	11.44	11.44	P15&P16

Moment adjustment between panels

Table 21 support moment adjustment

Panel	ML	MS	ms1-ms2	0.2*msla	moment distribute	Take average
P1&P4	5.27	5.27	0	1.054	Take average	5.27
P3&P7	5.27	5.27	0	1.054	Take average	5.27
P8&P10	17.4	14.74	2.66	3.48	Take average	16.07
P9&P11	16.71	15.51	1.2	3.342	Take average	16.11
P4&P19	5.27	5.27	0	1.054	Take average	5.27
P14&P19	5.27	5.27	0	1.054	Take average	5.27
P16&P17	12.73	11.95	0.78	2.546	Take average	12.34
P17&P18	11.95	9.95	2	2.39	Take average	10.95

Table 22 Span Moment Adjustment

Panel	Ly/Lx	Pd (KN/m)	$Lx^2(m^2)$	bxs	bxf	bys	byf	M_{xs}(kN*m)	M_{xf}(kN*m)	M_{ys}(kN*m)	M_{yf}(kN*m)
P5	1.20	9.36	31.50	0.04	0.03	0.03	0.02	12.39	9.44	9.44	7.08
P6	1.00	9.36	42.50	0.03	0.02	0.03	0.02	12.33	9.55	12.73	9.55
P8	1.75	9.36	31.50	0.06	0.04	0.03	0.02	17.40	12.98	9.44	7.08
P9	1.40	9.36	42.50	0.04	0.03	0.04	0.03	15.52	11.94	14.72	11.14
P10	1.40	9.36	31.50	0.05	0.05	0.03	0.02	14.74	15.63	9.44	7.08
P11	1.20	9.36	42.50	0.04	0.03	0.03	0.02	16.71	12.73	12.73	9.55
P13	1.40	9.36	33.20	0.05	0.05	0.03	0.02	15.54	16.47	9.95	7.46
P15	2.00	9.36	31.50	0.06	0.05	0.03	0.02	18.58	14.15	9.44	7.08
P16	1.75	9.36	42.50	0.06	0.04	.03	0.02	23.47	17.51	12.73	9.55
P17	1.75	9.36	39.90	0.06	0.04	0.03	0.02	22.04	16.44	11.95	8.96
P18	2.00	9.36	33.20	0.06	0.05	0.03	0.02	19.58	14.92	9.95	7.46

Table 23 Reinforcement design for roof

Reinforcement design for roof											
panel	moment	d	b	K	Z	ASmin	Asrequred	ASmax	as	Spacing Calculated=S	spacing provided=S
1mxs	5.27	174	1000	0.01	0.98	226.2	88.68	8000	113.04	499.73	400
2mxs	5.27	174	1000	0.01	0.99	226.2	88.01	8000	113.04	499.73	400
3mxs	5.27	174	1000	0.01	0.99	226.2	88.22	8000	113.04	499.73	400
4mxs	5.27	174	1000	0.01	0.99	226.2	87.76	8000	113.04	499.73	400
5mxs	15.64	174	1000	0.02	0.98	226.2	262.77	8000	113.04	430.19	400
mxf	9.43	174	1000	0.01	0.99	226.2	157.50	8000	113.04	499.73	400
mys	11.42	174	1000	0.02	0.99	226.2	191.18	8000	113.04	499.73	400
myf	7.08	174	1000	0.01	0.99	226.2	118.25	8000	113.04	499.73	400
6mxs	14.38	174	1000	0.02	0.99	226.2	238.96	8000	113.04	473.05	400
mxf	9.54	174	1000	0.01	0.98	226.2	160.62	8000	113.04	499.73	400
mys	11.42	174	1000	0.02	0.99	226.2	191.47	8000	113.04	499.73	400
myf	9.54	174	1000	0.01	0.99	226.2	159.94	8000	113.04	499.73	400
7mxs	5.27	174	1000	0.01	0.99	226.2	87.78	8000	113.04	499.73	400
8mxs	16.07	174	1000	0.02	0.98	226.2	270.57	8000	113.04	417.79	400
mxf	12.68	174	1000	0.02	0.99	226.2	212.40	8000	113.04	499.73	400
mys	12.63	174	1000	0.02	0.98	226.2	212.29	8000	113.04	499.73	400
myf	7.22	174	1000	0.01	0.99	226.2	120.94	8000	113.04	499.73	400
9mxs	16.11	174	1000	0.02	0.98	226.2	271.56	8000	113.04	416.25	400
mxf	11.94	174	1000	0.02	0.98	226.2	200.91	8000	113.04	499.73	400
mys	14.72	174	1000	0.02	0.99	226.2	246.43	8000	113.04	458.72	400
myf	11.94	174	1000	0.02	0.99	226.2	198.84	8000	113.04	499.73	400

10mxs	17.07	174	1000	0.02	0.97	226.2	289.29	8000	113.04	390.75	390
mxf	15.63	174	1000	0.02	0.98	226.2	263.61	8000	113.04	428.81	400
mys	11.44	174	1000	0.02	0.99	226.2	191.52	8000	113.04	499.73	400
myf	7.08	174	1000	0.01	0.99	226.2	118.26	8000	113.04	499.73	400
11mxs	21.34	174	1000	0.03	0.98	226.2	359.36	8000	113.04	314.56	310
mxf	17.50	174	1000	0.02	0.98	226.2	295.39	8000	113.04	382.68	380
mys	11.44	174	1000	0.02	0.98	226.2	192.84	8000	113.04	499.73	400
myf	9.59	174	1000	0.01	0.99	226.2	160.25	8000	113.04	499.73	400
12mxs	16.25	174	1000	0.02	0.99	226.2	270.91	8000	113.04	417.27	400
13mxs	18.14	174	1000	0.02	0.99	226.2	301.44	8000	113.04	375.00	370
mxf	17.07	174	1000	0.02	0.98	226.2	287.75	8000	113.04	499.73	400
mys	9.94	174	1000	0.01	0.98	226.2	167.00	8000	113.04	499.73	400
myf	7.98	174	1000	0.01	0.99	226.2	133.59	8000	113.04	499.73	400
14mxs	5.27	174	1000	0.01	0.99	226.2	87.80	8000	113.04	499.73	400
15mxs	17.07	174	1000	0.02	0.97	226.2	289.29	8000	113.04	390.75	390
mxf	14.36	174	1000	0.02	0.98	226.2	242.52	8000	113.04	466.11	400
mys	11.44	174	1000	0.02	0.99	226.2	191.73	8000	113.04	499.73	400
myf	7.43	174	1000	0.01	0.99	226.2	124.21	8000	113.04	499.73	400
16mxs	21.34	174	1000	0.03	0.98	226.2	360.56	8000	113.04	313.51	310

mxf	18.57	174	1000	0.02	0.98	226.2	313.30	8000	113.04	499.73	400
mys	12.34	174	1000	0.02	0.99	226.2	206.81	8000	113.04	499.73	400
myf	10.25	174	1000	0.01	0.99	226.2	171.25	8000	113.04	499.73	400
17mxs	18.94	174	1000	0.03	0.98	226.2	319.69	8000	113.04	353.59	400
mxf	17.76	174	1000	0.02	0.98	226.2	298.54	8000	113.04	378.64	370
mys	12.34	174	1000	0.02	0.99	226.2	206.46	8000	113.04	499.73	400
myf	9.77	174	1000	0.01	0.99	226.2	162.74	8000	113.04	499.73	400
18mxs	18.14	174	1000	0.02	0.99	226.2	301.44	8000	113.04	375.00	370
mxf	14.81	174	1000	0.02	0.98	226.2	249.36	8000	113.04	453.32	400
mys	10.95	174	1000	0.01	0.98	226.2	185.29	8000	113.04	499.73	400
myf	7.29	174	1000	0.01	0.98	226.2	122.87	8000	113.04	499.73	400
19mxs	5.27	174	1000	0.01	1.00	226.2	87.03	8000	113.04	499.73	400
20mxs	16.12	174	1000	0.02	1.00	226.2	266.22	8000	113.04	424.62	400
21mxs	20.13	174	1000	0.03	1.00	226.2	332.44	8000	113.04	340.03	340
22mxs	16.93	174	1000	0.02	1.00	226.2	279.59	8000	113.04	404.30	400

Table 24 depth determination for second floor

Panel	Ly (mm)	Lx (mm)	Span Ratio	K	Type of slab	Type of supporting Condition	d=l/N*K*F1*F2*F3	D=d+22
F1	2890	1500	1.93	0.4	Two Way	Cantilever	169.40	189.40
F2	5610	1500	3.74	0.4	One Way	Cantilever	169.40	189.40
F3	2800	1500	1.87	0.4	Two Way	Cantilever	169.40	189.40
F4	6640	1500	4.43	0.4	One Way	Cantilever	169.40	189.40
F5	6640	5610	1.18	11.5	Two Way	Interior	22.04	42.04
F6	6640	6520	1.02	11.5	Two Way	Interior	25.61	45.61
F7	6640	1500	4.43	0.4	One Way	Cantilever	169.40	189.40
F8	9170	5610	1.63	11.5	Two Way	Interior	22.04	42.04
F9	9170	6520	1.41	11.3	Two Way	End Span	26.06	46.06
F10	7980	5610	1.42	11.5	Two Way	Interior Span	22.04	42.04
F11	7980	6520	1.22	11.3	Two Way	End Span	26.06	46.06
F12	7980	166	48.07	0.4	One Way	Cantilever	18.75	38.75
F13	7980	5760	1.39	11.5	Two Way	Interior Span	22.63	42.63
F14	7980	1500	5.32	0.4	One Way	Cantilever	169.40	189.40
F15	10650	5610	1.90	11.5	Two Way	Interior	22.04	42.04
F16	10650	6520	1.63	11.5	Two Way	Interior	25.61	45.61
F17	10650	6320	1.69	11.5	Two Way	Interior	24.83	44.83
F18	10650	5760	1.85	11.5	Two Way	Interior	22.63	42.63
F19	3220	1500	2.15	0.4	Two Way	Cantilever	169.40	189.40
F20	5610	1500	3.74	0.4	Two Way	Cantilever	169.40	189.40

F21	6520	1500	4.35	0.4	One Way	Cantilever	169.40	189.40
F22	5760	1500	3.84	0.4	Two Way	Cantilever	169.40	189.40

Partition wall dead load computation

Dead Load of Partition Wall on First Flour Panels

Table 25 Dead load of HCB for first floor

pation wall								
Panel	wh		wt	WL	rHCB	DLW	Area	DLW/A
F1	2.81		0.40	4.39	14.00	69.08	4.33	15.94
F2	2.81		0.40	7.11	14.00	111.88	8.15	13.74
F3	2.81		0.40	4.33	14.00	68.14	4.20	16.22
F4	2.81		0.40	10.65	14.00	167.59	9.96	16.83
F5	2.81		0.40	1.30	14.00	20.46	37.25	0.55
F6	2.81		0.40	1.17	14.00	18.36	43.29	0.42
F7	2.81		0.40	13.16	14.00	207.09	9.96	20.79
F8	2.81		0.40	5.61	14.00	88.28	51.44	1.72
F9	2.81		0.40	10.22	14.00	160.82	59.89	2.69
F10	2.81		0.40	13.59	14.00	213.85	44.77	4.78
F11	2.81		0.40	6.52	14.00	102.60	52.03	1.97
F12	2.81		0.40	8.15	14.00	128.19	1.32	97.11
F13	2.81		0.40	5.76	14.00	90.64	45.97	1.97
F14	2.81		0.40	9.48	14.00	149.18	11.97	12.46
F15	2.81		0.40	5.61	14.00	88.28	59.75	1.48
F16	2.81		0.40	6.52	14.00	102.60	69.44	1.48
F17	2.81		0.40	10.65	14.00	167.59	67.30	2.49
F18	2.81		0.40	17.28	14.00	271.92	61.34	4.43
F19	2.81		0.40	1.50	14.00	23.60	4.83	4.89

Table 26 Dead load of plaster for first floor

Panel	ph	Pt	pL	rpla	DLp	Area	DLp/A	TDLPW
F1	2.81	0.03	4.39	23.00	8.51	4.33	1.96	17.90
F2	2.81	0.03	7.11	23.00	13.79	8.15	1.69	15.43
F3	2.81	0.03	4.33	23.00	8.40	4.20	2.00	18.22
F4	2.81	0.03	10.65	23.00	20.65	9.96	2.07	18.90
F5	2.81	0.03	1.30	23.00	2.52	37.25	0.07	0.62
F6	2.81	0.03	1.17	23.00	2.26	43.29	0.05	0.48
F7	2.81	0.03	13.16	23.00	25.52	9.96	2.56	23.35
F8	2.81	0.03	5.61	23.00	10.88	51.44	0.21	1.93
F9	2.81	0.03	10.22	23.00	19.82	59.89	0.33	3.02
F10	2.81	0.03	13.59	23.00	26.35	44.77	0.59	5.37
F11	2.81	0.03	6.52	23.00	12.64	52.03	0.24	2.21
F12	2.81	0.03	8.15	23.00	15.79	1.32	11.97	109.08
F13	2.81	0.03	5.76	23.00	11.17	45.97	0.24	2.21
F14	2.81	0.03	9.48	23.00	18.38	11.97	1.54	14.00
F15	2.81	0.03	5.61	23.00	10.88	59.75	0.18	1.66
F16	2.81	0.03	6.52	23.00	12.64	69.44	0.18	1.66
F17	2.81	0.03	10.65	23.00	20.65	67.30	0.31	2.80
F18	2.81	0.03	17.28	23.00	33.50	61.34	0.55	4.98
F19	2.81	0.03	1.50	23.00	2.91	4.83	0.60	5.49

Summary of Imposed load

$$Pd=Pd'+1.35\ D.L_{pw}$$

Where, pd'=1.35 D.L +1. 5 L.L

$D.L_{pw}$=Dead load of Partition wall

Table 27 imposed load for first floor

Panel	Pd'=(1.35 D.L+1.5L.L)(KN/m^2)	Partition wall Dead Load ($D.L_{pw}$)(KN/m^2)	Pd=Pd'+1.3 $D.L_{pw}$
F1	11.9	17.9	36.065
F2	11.9	15.4	32.69
F3	11.9	18.2	36.47
F4	13.45	18.9	38.965
F5	11.9	0.62	12.737
F6	11.9	0.47	12.5345
F7	11.9	23.35	43.4225
F8	13.45	1.92	16.042
F9	16.9	3	20.95
F10	13.45	5.36	20.686
F11	16.92	2.21	19.9035
F12	16.45	109.07	163.6945
F13	16.45	2.21	19.4335
F14	13.45	13.99	32.3365
F15	16.9	1.66	19.141
F16	16.9	1.66	19.141
F17	16.9	2.8	20.68
F18	16.9	4.98	23.623
F19	15.4	5.5	22.825
F20	15.4	14.3	34.705
F21	15.4		15.4

F22	15.4		15.4

Moment Analysis

Table 28 Moment for two way slab for first floor

Panel	Ly/Lx	Pd (KN/m)	$Lx^2(m^2)$	bxs	bxf	bys	byf	M_{xs}(kN*m)	M_{xf}(kN*m)	M_{ys}(kN*m)	M_{yf}(kN*m)
F5	1.20	12.74	31.50	0.04	0.03	0.03	0.02	16.85	12.84	12.84	9.63
F6	1.00	12.53	42.50	0.03	0.02	0.03	0.02	16.51	12.79	17.05	12.79
F8	1.75	16.04	31.50	0.06	0.04	0.03	0.02	29.81	22.23	16.17	12.13
F9	1.40	20.95	42.50	0.04	0.03	0.04	0.03	34.72	26.71	32.94	24.93
F10	1.40	20.69	31.50	0.05	0.05	0.03	0.02	32.58	34.54	20.85	15.64
F11	1.20	19.90	42.50	0.04	0.03	0.03	0.02	35.53	27.07	27.07	20.30
F13	1.40	19.43	33.20	0.05	0.05	0.03	0.02	32.26	34.20	20.65	15.48
F15	2.00	19.14	31.50	0.06	0.05	0.03	0.02	37.99	28.94	19.29	14.47
F16	1.75	19.14	42.50	0.06	0.04	0.03	0.02	48.00	35.79	26.03	19.52
F17	1.75	20.68	39.90	0.06	0.04	0.03	0.02	48.68	36.31	26.40	19.80
F18	2.00	23.62	33.20	0.06	0.05	0.03	0.02	49.41	37.65	25.10	18.82

Table 29 Moment Calculation for cantilever first floors

Moment Calculation for one and two way cantilever slabs.					
F1	Two Way Cantilever	17.03	1.50	36.07	45.83
F2	One Way Cantilever	17.03	1.50	32.69	43.93
F3	Two Way Cantilever	17.03	1.50	36.47	46.06
F4	One Way Cantilever	17.03	1.50	38.97	47.46
F7	One Way Cantilever	17.03	1.50	43.42	49.97
F12	One Way Cantilever	17.03	1.66	163.69	130.17
F14	One Way Cantilever	17.03	1.50	32.34	43.73
F19	Two Way Cantilever	17.03	1.50	22.83	38.38
F20	Two Way Cantilever	17.03	1.50	34.71	45.07
F21	One Way Cantilever	17.03	1.50	15.40	34.21
F22	Two Way Cantilever	17.03	1.50	15.40	34.21
F23	Two Way Cantilever	17.03	1.50	15.40	34.21

Table 30Support Moment adjustment between panels for first floor

Support Moment adjustment between panels for frist fioor							
Panel	ML	MS		ms1-ms2	0.2*msla	moment distribute	
F1&F4	47.46	45.83		1.63	9.49	Take average	
F2&F5	43.93	16.85		27.08	8.79	moment distribute	
F2&F6	43.93	16.51		27.42	8.79	moment distribute	
F3&F7	49.97	46.60		3.37	9.99	Take average	
F5&F8	29.81	16.85		12.96	5.96	moment distribute	
F6&F9	34.72	16.51		18.21	6.94	moment distribute	
F8&F10	32.58	29.81		2.77	6.52	Take average	
F9&F11	35.53	34.72		0.81	7.11	Take average	
F12&F17	141.04	48.67		92.37	28.21	moment distribute	
F13&F18	49.41	32.26		17.15	9.88	moment distribute	
F10&F15	38.00	32.58		5.42	7.60	Take average	
F11&F16	49.00	35.53		13.47	9.80	moment distribute	
F4&F19	47.46	38.38		9.08	9.49	Take average	
F15&F20	45.07	38.00		7.07	9.01	Take average	
F21&F16	49.00	34.21		14.79	9.80	moment distribute	
F17&F21	48.67	34.21		14.46	9.73	moment distribute	
F18&F22	49.41	34.21		15.20	9.88	moment distribute	
F14&F19	43.74	38.38		5.36	8.75	Take average	
F5&F6	17.05	12.84		4.21	3.41	moment distribute	
F8&F9	32.94	16.17		16.77	6.59	moment distribute	
F10&F11	27.10	20.85		6.25	5.42	moment distribute	
F15&16	26.03	19.03		7.00	5.21	moment distribute	
F16&F17	26.40	26.03		0.37	5.28	Take average	

F17&F18	26.40	25.10		1.30	5.28	Take average	

Moment adjustment between panel

Support moment adjustment for frist floor														
Panel	Ly	Lx	M1	M2	k1	k2	k1+k2	df1	df2	Δm	Δmdf1	Δmdf2	M1(adje)	M2(adje)
b/n s1 &c1	3.56	1.20	7.52	3.70	0.21	0.83	1.04	0.20	0.8	3.8	-0.8	3.0	6.7	6.7
F2&F5	6.64	1.50	43.93	16.85	0.11	0.67	0.78	0.14	0.86	27.08	-3.92	23.16	40.01	40.01
F2&F6	6.64	1.50	43.93	16.51	0.11	0.67	0.78	0.14	0.86	27.42	-3.97	23.45	39.96	39.96
F5&F8	9.17	6.64	29.81	16.85	0.08	0.15	0.23	0.35	0.65	12.96	-4.56	8.40	25.25	25.25
F6&F9	9.17	6.64	34.72	16.51	0.08	0.15	0.23	0.35	0.65	18.21	-6.41	11.80	28.31	28.31
F12&F17	10.65	7.89	141.04	48.67	0.07	0.13	0.20	0.36	0.64	92.37	-32.99	59.38	108.05	108.05
F13&F18	10.65	7.89	49.41	32.26	0.07	0.13	0.20	0.36	0.64	17.15	-6.13	11.02	43.28	43.28
F11&F16	10.65	7.89	49.00	35.53	0.07	0.13	0.20	0.36	0.64	13.47	-4.81	8.66	44.19	44.19
F21&F16	10.65	3.22	49.00	34.21	0.07	0.31	0.38	0.18	0.82	14.79	-2.73	12.06	46.27	46.27

F17&F21	10.65	3.22	48.67	34.21	0.07	0.31	0.38	0.18	0.82	14.46	-2.67	11.79	46.00	46.00
F18&F22	10.65	3.22	49.41	34.21	0.07	0.31	0.38	0.18	0.82	15.20	-2.81	12.39	46.60	46.60
F5&F6	6.52	5.61	17.05	12.84	0.12	0.18	0.29	0.39	0.61	4.21	-1.65	2.56	15.40	15.40
F8&F9	6.52	5.61	32.94	16.17	0.12	0.18	0.29	0.39	0.61	16.77	-6.58	10.19	26.36	26.36
F10&F11	6.52	5.61	27.10	20.85	0.12	0.18	0.29	0.39	0.61	6.25	-2.45	3.80	24.65	24.65
F15&16	6.52	5.61	26.03	19.30	0.12	0.18	0.29	0.39	0.61	6.73	-2.64	4.09	23.39	23.39

Moment adjustment between panels

Panel	ML	MS	ms1-ms2	0.2*mL	moment distribute	M(adje)
F1&F4	47.46	45.83	1.63	9.492	Take average	46.645
F3&F7	49.97	46.6	3.37	9.994	Take average	48.285
F8&F10	32.58	29.81	2.77	6.516	Take average	31.195
F9&F11	35.53	34.72	0.81	7.106	Take average	35.125
F10&F15	38	32.58	5.42	7.6	Take average	35.29
F4&F19	47.46	38.38	9.08	9.492	Take average	42.92
F15&F20	45.067	38	7.067	9.0134	Take average	41.5335
F14&F19	43.74	38.38	5.36	8.748	Take average	41.06
F16&F17	26.4	26.03	0.37	5.28	Take average	26.215
F17&F18	26.4	25.1	1.3	5.28	Take average	25.75

Table 31 Span Moment Adjustment for first floor

panel	LY/LX	mxf	myf	Δmxs	Δmys	cx1	cy1	cx2	cy2	Δmxf	Δmyf	mxf(adj)	myf(adj)
F16	1.75	35.79	19.52	2.73	-0.18	0.46	0.27	0.29	0.07	1.21	0.73	37.00	20.25
F17	1.75	36.31	19.08	2.67	0.18	0.46	0.27	0.29	0.07	1.28	0.74	37.59	19.82
F18	2.00	37.65	18.82	2.81	0.15	0.26	0.06	0.48	0.25	0.80	0.21	38.45	19.03

Table 32 design reinforisment for first floor

reinforcement design for first floor slab											
panel	moment	d	B	k	Z	ASmin	Asrequred	ASmax	as	Scalculated	Sprovided
1mxs	46.26	174	1000	0.06	0.95	226.2	802.47	8000	113.04	140.87	140
2mxs	40.84	174	1000	0.05	0.96	226.2	700.09	8000	113.04	161.47	155
3mxs	43.50	174	1000	0.06	0.98	226.2	735.58	8000	113.04	153.67	145
4mxs	46.26	174	1000	0.06	0.94	226.2	810.88	8000	113.04	139.40	140
5mxs	40.84	174	1000	0.05	0.94	226.2	718.47	8000	113.04	157.34	155
mxf	11.50	174	1000	0.02	0.92	226.2	207.42	8000	113.04	544.99	400
mys	25.55	174	1000	0.03	0.93	226.2	452.25	8000	113.04	249.95	250
myf	8.61	174	1000	0.01	1.00	226.2	142.19	8000	113.04	499.73	400
6mxs	43.50	174	1000	0.06	1.00	226.2	718.39	8000	113.04	157.35	145
mxf	25.96	174	1000	0.03	1.00	226.2	428.72	8000	113.04	263.67	250
mys	25.55	174	1000	0.03	1.00	226.2	421.95	8000	113.04	267.90	250
mxf	25.96	174	1000	0.03	1.00	226.2	428.72	8000	113.04	263.67	250
7mxs	46.60	174	1000	0.06	1.00	226.2	769.59	8000	113.04	146.88	135
8mxs	29.61	174	1000	0.04	1.00	226.2	489.00	8000	113.04	231.17	215
mxf	20.20	174	1000	0.03	1.00	226.2	333.60	8000	113.04	338.85	320
mys	29.00	174	1000	0.04	1.00	226.2	478.93	8000	113.04	236.03	220
myf	11.03	174	1000	0.01	1.00	226.2	182.16	8000	113.04	620.56	400
9mxs	44.20	174	1000	0.06	1.00	226.2	729.95	8000	113.04	154.86	145
mxf	31.00	174	1000	0.04	1.00	226.2	511.96	8000	113.04	220.80	205
mys	29.00	174	1000	0.04	1.00	226.2	478.93	8000	113.04	236.03	220
myf	28.94	174	1000	0.04	1.00	226.2	477.94	8000	113.04	236.52	220
10mxs	45.65	174	1000	0.06	1.00	226.2	753.90	8000	113.04	149.94	120
mxf	34.00	174	1000	0.04	1.00	226.2	561.50	8000	113.04	201.32	165
mys	24.04	174	1000	0.03	1.00	226.2	397.01	8000	113.04	284.73	270

myf	15.40	174	1000	0.02	1.00	226.2	254.33	8000	113.04	444.47	400
11mxs	61.78	174	1000	0.08	1.00	226.2	1020.28	8000	113.04	110.79	90
mxf	35.10	174	1000	0.05	1.00	226.2	579.67	8000	113.04	195.01	185
mys	24.40	174	1000	0.03	1.00	226.2	402.96	8000	113.04	280.52	265
myf	26.52	174	1000	0.04	1.00	226.2	437.97	8000	113.04	258.10	245
12mxs	105.50	174	1000	0.14	1.00	226.2	1742.30	8000	113.04	64.88	50
13mxs	49.10	174	1000	0.06	1.00	226.2	810.87	8000	113.04	139.41	110
mxf	49.52	174	1000	0.07	1.00	226.2	817.81	8000	113.04	138.22	110
mys	29.90	174	1000	0.04	1.00	226.2	493.79	8000	113.04	228.92	215
myf	22.42	174	1000	0.03	1.00	226.2	370.26	8000	113.04	305.30	290
14mxs	35.72	174	1000	0.05	1.00	226.2	589.91	8000	113.04	191.62	180
15mxs	49.35	174	1000	0.07	1.00	226.2	815.00	8000	113.04	138.70	100
mxf	41.50	174	1000	0.05	1.00	226.2	685.36	8000	113.04	164.93	135
mys	27.88	174	1000	0.04	1.00	226.2	460.43	8000	113.04	245.51	230
Myf	20.74	174	1000	0.03	1.00	226.2	342.52	8000	113.04	330.03	310
16mxs	66.30	174	1000	0.09	1.00	226.2	1094.93	8000	113.04	103.24	80
Mxf	51.55	174	1000	0.07	1.00	226.2	851.33	8000	113.04	132.78	105
Mys	30.61	174	1000	0.04	1.00	226.2	505.52	8000	113.04	223.61	210
Myf	28.11	174	1000	0.04	1.00	226.2	464.23	8000	113.04	243.50	230
17mxs	105.50	174	1000	0.14	1.00	226.2	1742.30	8000	113.04	64.88	50
Mxf	57.60	174	1000	0.08	1.00	226.2	951.25	8000	113.04	118.83	95
Mys	30.61	174	1000	0.04	1.00	226.2	505.52	8000	113.04	223.61	210
Myf	31.74	174	1000	0.04	1.00	226.2	524.18	8000	113.04	215.65	200
18mxs	53.65	174	1000	0.07	1.00	226.2	886.02	8000	113.04	127.58	100
Mxf	39.20	174	1000	0.05	1.00	226.2	647.38	8000	113.04	174.61	140
Mys	30.27	174	1000	0.04	1.00	226.2	499.90	8000	113.04	226.12	210
Myf	19.59	174	1000	0.03	1.00	226.2	323.52	8000	113.04	349.40	330

19mxs	46.78	174	1000	0.06	1.00	226.2	772.56	8000	113.04	146.32	135
20mxs	49.35	174	1000	0.07	1.00	226.2	815.00	8000	113.04	138.70	130
21mxs	66.30	174	1000	0.09	1.00	226.2	1094.93	8000	113.04	103.24	95
22mxs	53.65	174	1000	0.07	1.00	226.2	886.02	8000	113.04	127.58	120

Check Depth for Deflection: Serviceability Requirement

Table 33 Check Depth for Deflection: Serviceability Requirement for frist floor

panel	Moment	lx	d(mm)	B	k	z	ASmin	Asrequred	ASmax	ρ	ρo	F1	K	LX/d	d(m)	Check
1mxs	46.26	1500	0.174	1000	0.061	0.95	226.2	802.47	8000	0.046	0.4472	1.25	0.4	183.53	94.09	OK
2mxs	40.84	1500	0.174	1000	0.054	0.96	226.2	700.09	8000	0.040	0.4472	1.25	0.4	230.19	75.90	OK
3mxs	43.5	1500	0.174	1000	0.057	0.98	226.2	735.58	8000	0.042	0.4472	1.25	0.4	212.13	82.25	OK
4mxs	46.26	1500	0.174	1000	0.061	0.94	226.2	810.88	8000	0.047	0.4472	1.25	0.4	180.35	96.78	OK
5mxs	40.84	5610	0.174	1000	0.054	0.94	226.2	718.47	8000	0.041	0.4472	1.25	11.5	63.41	27.44	OK
mxf	11.5	5610	0.174	1000	0.015	0.92	226.2	207.42	8000	0.012	0.4472	1.25	11.5	45.39	3.83	OK
mys	25.55	5610	0.174	1000	0.034	0.93	226.2	452.25	8000	0.026	0.4472	1.25	11.5	134.21	12.96	OK
myf	8.61	5610	0.174	1000	0.011	1	226.2	142.19	8000	0.008	0.4472	1.25	11.5	810.07	2.15	OK
6mxs	43.5	6640	0.174	1000	0.057	1	226.2	718.39	8000	0.041	0.4472	1.25	11.5	63.42	27.44	OK
mxf	25.96	6640	0.174	1000	0.034	1	226.2	428.72	8000	0.025	0.4472	1.25	11.5	146.11	11.91	OK
mys	25.55	6640	0.174	1000	0.034	1	226.2	421.95	8000	0.024	0.4472	1.25	11.5	149.85	11.61	OK
mxf	25.96	6640	0.174	1000	0.034	1	226.2	428.72	8000	0.025	0.4472	1.25	11.5	146.11	11.91	OK
7mxs	46.6	1500	0.174	1000	0.062	1	226.2	769.59	8000	0.044	0.4472	1.25	0.4	196.79	84.17	OK
8mxs	29.61	5610	0.174	1000	0.039	1	226.2	489.00	8000	0.028	0.4472	1.25	11.5	11847.10	14.69	OK
mxf	20.2	5610	0.174	1000	0.027	1	226.2	333.60	8000	0.019	0.4472	1.25	11.3	213.23	8.16	OK
mys	29	5610	0.174	1000	0.038	1	226.2	478.93	8000	0.028	0.4472	1.25	11.3	120.35	14.46	OK
myf	11.03	5610	0.174	1000	0.015	1	226.2	182.16	8000	0.010	0.4472	1.25	11.3	544.66	3.19	OK
9mxs	44.2	6520	0.174	1000	0.058	1	226.2	729.95	8000	0.042	0.4472	1.25	11.3	60.69	28.67	OK
mxf	31	6520	0.174	1000	0.041	1	226.2	511.96	8000	0.029	0.4472	1.25	11.3	108.16	16.09	OK
mys	29	6520	0.174	1000	0.038	1	226.2	478.93	8000	0.028	0.4472	1.25	11.3	120.35	14.46	OK
myf	28.94	6520	0.174	1000	0.038	1	226.2	477.94	8000	0.027	0.4472	1.25	11.3	120.75	14.41	OK
10mxs	45.65	5610	0.174	1000	0.060	1	226.2	753.90	8000	0.043	0.4472	1.25	11.5	58.55	29.72	OK
mxf	34	5610	0.174	1000	0.045	1	226.2	561.50	8000	0.032	0.4472	1.25	11.5	94.85	18.34	OK
mys	24.04	5610	0.174	1000	0.032	1	226.2	397.01	8000	0.023	0.4472	1.25	11.5	165.02	10.54	OK
myf	15.4	5610	0.174	1000	0.020	1	226.2	254.33	8000	0.015	0.4472	1.25	11.5	331.22	5.25	OK

11mxs	61.78	6520	0.174	1000	0.082	1	226.2	1020.28	8000	0.059	0.4472	1.25	11.3	34.49	50.45	OK
mxf	35.1	6520	0.174	1000	0.046	1	226.2	579.67	8000	0.033	0.4472	1.25	11.3	88.52	19.66	OK
mys	24.4	6520	0.174	1000	0.032	1	226.2	402.96	8000	0.023	0.4472	1.25	11.3	158.38	10.99	OK
myf	26.52	6520	0.174	1000	0.035	1	226.2	437.97	8000	0.025	0.4472	1.25	11.3	138.78	12.54	OK
12mxs	105.5	166	0.174	1000	0.139	1	226.2	1742.30	8000	0.100	0.4472	1.25	0.4	46.20	65.89	OK
13mxs	49.1	5760	0.174	1000	0.065	1	226.2	810.87	8000	0.047	0.4472	1.25	11.5	5185.19	33.56	OK
mxf	49.52	5760	0.174	1000	0.065	1	226.2	817.81	8000	0.047	0.4472	1.25	11.5	51.73	34.04	OK
mys	29.9	5760	0.174	1000	0.040	1	226.2	493.79	8000	0.028	0.4472	1.25	11.5	113.66	14.92	OK
myf	22.42	5760	0.174	1000	0.030	1	226.2	370.26	8000	0.021	0.4472	1.25	11.5	1821.86	9.45	OK
14mxs	35.72	1500	0.174	1000	0.047	1	226.2	589.91	8000	0.034	0.4472	1.25	0.4	304.57	71.29	OK
15mxs	49.35	5610	0.174	1000	0.065	1	226.2	815.00	8000	0.047	0.4472	1.25	11.5	51.41	33.84	OK
mxf	41.5	5610	0.174	1000	0.055	1	226.2	685.36	8000	0.039	0.4472	1.25	11.5	68.54	25.39	OK
mys	27.88	5610	0.174	1000	0.037	1	226.2	460.43	8000	0.026	0.4472	1.25	11.5	130.43	13.34	OK
myf	20.74	5610	0.174	1000	0.027	1	226.2	342.52	8000	0.020	0.4472	1.25	11.5	208.21	8.36	OK
16mxs	66.3	6520	0.174	1000	0.088	1	226.2	1094.93	8000	0.063	0.4472	1.25	11.5	31.05	56.04	OK
mxf	51.55	6520	0.174	1000	0.068	1	226.2	851.33	8000	0.049	0.4472	1.25	11.5	47.78	36.42	OK
mys	30.61	6520	0.174	1000	0.040	1	226.2	505.52	8000	0.029	0.4472	1.25	11.5	112.33	15.49	OK
myf	28.11	6520	0.174	1000	0.037	1	226.2	464.23	8000	0.027	0.4472	1.25	11.5	128.73	13.52	OK
17mxs	105.5	6320	0.174	1000	0.139	1	226.2	1742.30	8000	0.100	0.4472	1.25	11.5	132.84	130.99	OK
mxf	57.6	6320	0.174	1000	0.076	1	226.2	951.25	8000	0.055	0.4472	1.25	11.5	39.58	43.96	OK
mys	30.61	6320	0.174	1000	0.040	1	226.2	505.52	8000	0.029	0.4472	1.25	11.5	112.33	15.49	OK
myf	31.74	6320	0.174	1000	0.042	1	226.2	524.18	8000	0.030	0.4472	1.25	11.5	105.98	16.42	OK
18mxs	53.65	5760	0.174	1000	0.071	1	226.2	886.02	8000	0.051	0.4472	1.25	11.5	44.67	38.96	OK
mxf	39.2	5760	0.174	1000	0.052	1	226.2	647.38	8000	0.037	0.4472	1.25	11.5	75.26	23.12	OK
mys	30.27	5760	0.174	1000	0.040	1	226.2	499.90	8000	0.029	0.4472	1.25	11.5	114.36	15.22	OK
myf	19.59	5760	0.174	1000	0.026	1	226.2	323.52	8000	0.019	0.4472	1.25	11.5	227.68	7.64	OK

19mxs	46.78	1500	0.174	1000	0.062	1	226.2	772.56	8000	0.044	0.4472	1.25	0.4	195.54	89.86	OK
20mxs	49.35	1500	0.174	1000	0.065	1	226.2	815.00	8000	0.047	0.4472	1.25	0.4	178.83	73.01	OK
21mxs	66.3	1500	0.174	1000	0.088	1	226.2	1094.93	8000	0.063	0.4472	1.25	0.4	108.00	61.13	OK
22mxs	53.65	1500	0.174	1000	0.071	1	226.2	886.02	8000	0.051	0.4472	1.25	0.4	155.36	119.97	OK

Table 34 depth determination for second floor

Panel	Ly (mm)	Lx (mm)	Span Ratio	K	Type of slab	Type of supporting Condition	d=l/N*K*F1*F2*F3	D=d+22
S1	2890	1500	1.93	0.4	Two Way	Cantilever	169.40	189.40
S2	5610	1500	3.74	0.4	One Way	Cantilever	169.40	189.40
S3	2800	1500	1.87	0.4	Two Way	Cantilever	169.40	189.40
S4	6640	1500	4.43	0.4	One Way	Cantilever	169.40	189.40
S5	6640	5610	1.18	11.5	Two Way	Interior	22.04	42.04
S6	6640	6520	1.02	11.5	Two Way	Interior	25.61	45.61
S7	6640	1500	4.43	0.4	One Way	Cantilever	169.40	189.40
S8	9170	5610	1.63	11.5	Two Way	Interior	22.04	42.04
S9	9170	6520	1.41	11.3	Two Way	End Span	26.06	46.06
S10	7980	5610	1.42	11.5	Two Way	Interior Span	22.04	42.04
S11	7980	6520	1.22	11.3	Two Way	End Span	26.06	46.06
S12	7980	166	48.07	0.4	One Way	Cantilever	18.75	38.75
S13	7980	5760	1.39	11.5	Two Way	Interior Span	22.63	42.63
S14	7980	1500	5.32	0.4	One Way	Cantilever	169.40	189.40
S15	10650	5610	1.90	11.5	Two Way	Interior	22.04	42.04

S16	10650	6520	1.63	11.5	Two Way	Interior	25.61	45.61
S17	10650	6320	1.69	11.5	Two Way	Interior	24.83	44.83
S18	10650	5760	1.85	11.5	Two Way	Interior	22.63	42.63
S19	3220	1500	2.15	0.4	Two Way	Cantilever	169.40	189.40
S20	5610	1500	3.74	0.4	Two Way	Cantilever	169.40	189.40
S21	6520	1500	4.35	0.4	One Way	Cantilever	169.40	189.40
S22	5760	1500	3.84	0.4	Two Way	Cantilever	169.40	189.40

Table 35 Dead load of HCB for second floor

Panel	wL	Wh	Wt	rHCB	DLW	Area	DLW/A
S1	4.39	2.81	0.40	14.00	69.08	4.33	15.94
S2	7.11	2.81	0.40	14.00	111.88	8.15	13.74
S3	4.33	2.81	0.40	14.00	68.14	4.20	16.22
S4	13.54	2.81	0.40	14.00	213.07	9.96	21.39
S5					0.00	37.25	0.00
S6	18.35	2.81	0.40	14.00	288.76	43.29	6.67
S7	9.42	2.81	0.40	14.00	148.23	9.96	14.88
S8	5.61	2.81	0.40	14.00	88.28	51.44	1.72
S9	22.21	2.81	0.40	14.00	349.50	59.89	5.84
S10					0.00	44.77	0.00
S11	7.98	2.81	0.40	14.00	125.57	52.03	2.41
S12	8.15	2.81	0.40	14.00	128.19	1.32	97.11
S13					0.00	45.97	0.00
S14	9.48	2.81	0.40	14.00	149.18	11.97	12.46
S15	5.61	2.81	0.40	14.00	88.28	59.75	1.48
S16	20.29	2.81	0.40	14.00	319.28	69.44	4.60
S17	16.97	2.81	0.40	14.00	267.04	67.30	3.97
S18					0.00	61.34	0.00
S19	1.50	2.81	0.40	14.00	23.60	4.83	4.89
S20	5.61	2.81	0.40	14.00	88.28	8.42	10.49
S21	6.52	2.81	0.40	14.00	102.60	9.78	10.49
S22	6.32	2.81	0.40	14.00	99.45	8.64	11.51
S23	5.76	2.81	0.40	14.00	90.64	8.64	10.49

Table 36 Dead load of plaster for second floor

Panel	wL	wh	pt	Rpla	DLP	Area	DLW/A	TDLPW
S1	4.39	2.81	0.03	23.00	8.51	4.33	1.96	17.90
S2	7.11	2.81	0.03	23.00	13.79	8.15	1.69	15.43
S3	4.33	2.81	0.03	23.00	8.40	4.20	2.00	18.22
S4	13.54	2.81	0.03	23.00	26.25	9.96	2.64	24.03
S5					0.00	37.25	0.00	0.00
S6	18.35	2.81	0.03	23.00	35.58	43.29	0.82	7.49
S7	9.42	2.81	0.03	23.00	18.26	9.96	1.83	16.72
S8	5.61	2.81	0.03	23.00	10.88	51.44	0.21	1.93
S9	22.21	2.81	0.03	23.00	43.06	59.89	0.72	6.55
S10				23.00	0.00	44.77	0.00	0.00
S11	7.98	2.81	0.03	23.00	15.47	52.03	0.30	2.71
S12	8.15	2.81	0.03	23.00	15.79	1.32	11.97	109.08
S13			0.03	23.00	0.00	45.97	0.00	0.00
S14	9.48	2.81	0.03	23.00	18.38	11.97	1.54	14.00
S15	5.61	2.81	0.03	23.00	10.88	59.75	0.18	1.66
S16	20.29	2.81	0.03	23.00	39.34	69.44	0.57	5.16
S17	16.97	2.81	0.03	23.00	32.90	67.30	0.49	4.46
S18					0.00	61.34	0.00	0.00
S19	1.50	2.81	0.03	23.00	2.91	4.83	0.60	5.49
S20	5.61	2.81	0.03	23.00	10.88	8.42	1.29	11.78
S21	6.52	2.81	0.03	23.00	12.64	9.78	1.29	11.78
S22	6.32	2.81	0.03	23.00	12.25	8.64	1.42	12.93
S23	5.76	2.81	0.03	23.00	11.17	8.64	1.29	11.78

Moment Analysis

Table 37 Moment Calculation for two way second floor

Table 38 Support Moment adjustment between panels for second floor

Panel	Pd'=(1.35 D.L+1.5L.L)(KN/m^2)	Partition wall Dead Load ($D.L_{pw}$)(KN/m^2)	Pd=Pd'+1.35$D.L_{pw}$
S1	13.45	17.9	37.615
S2	13.45	15.4	34.24
S3	16.45	18.22	41.047
S4	16.9	24	49.3
S5	13.45		13.45
S6	11.9	7.5	22.025
S7	11.9	16.7	34.445
S8	13.45	1.92	16.042
S9	16.91	6.55	25.7525
S10	16.45	0.8	17.53
S11	16.9	2.71	20.5585
S12	16.45	109.07	163.6945
S13	16.45		16.45
S14	16.45	13.99	35.3365
S15	16.9	1.66	19.141
S16	16.9	5.12	23.812
S17	16.9	4.45	22.9075
S18	16.9		16.9
S19	16.45	5.5	23.875
S20	16.45	11.78	32.353
S21	16.45	11.78	32.353

S22	16.45	12.9	33.865

Moment Calculation for cantilever slabs for second floor.

Panel	Type of supportig Condition	$D.L_{ex}$	Lx	pd'=1.35 D.L +1. 5 L.L	MXS
S1	Two Way Cantilever	17.03	1.5	37.62	46.70
S2	One Way Cantilever	17.03	1.5	34.24	44.81
S3	Two Way Cantilever	17.03	1.5	41.05	48.63
S4	One Way Cantilever	17.03	1.5	49.30	53.28
S7	One Way Cantilever	17.03	1.5	34.45	44.92
S12	One Way Cantilever	17.03	1.66	163.69	130.17
S14	One Way Cantilever	17.03	1.5	35.34	45.42
S19	Two Way Cantilever	17.03	1.5	23.88	38.97
S20	Two Way Cantilever	17.03	1.5	32.35	43.74
S21	One Way Cantilever	17.03	1.5	32.35	43.74
S22	Two Way Cantilever	17.03	1.5	33.87	44.59
S23	Two Way Cantilever	17.08	1.5	33.87	44.67

Support	Moment adjustment between panels for second fioor					
Panel	ML	MS		ms1-ms2	0.2*msla	moment distribute
S1&S4	53.28	46.7		6.58	10.66	Take average
S2&S5	44.81	17.79		27.02	8.96	moment distribute
S2&S6	44.81	29.02		15.79	8.96	moment distribute
S3&S7	48.64	44.92		3.72	9.73	Take average
S5&S8	29.81	17.79		12.02	5.96	moment distribute
S6&S9	42.68	29.02		13.66	8.54	moment distribute
S8&S10	29.81	27.61		2.2	5.96	Take average
S9&S11	42.68	36.7		5.98	8.54	Take average
S12&S17	141.04	53.93		87.11	28.21	moment distribute
S13&S18	35.35	27.31		8.04	7.07	moment distribute
S10&S15	37.98	27.61		10.37	7.60	moment distribute
S11&S16	59.71	36.7		23.01	11.94	moment distribute
S4&S19	53.28	38.97		14.31	10.66	moment distribute
S15&S20	43.74	37.985		5.755	8.75	Take average
S21&S16	59.71	43.74		15.97	11.94	moment distribute
S17&S21	53.93	43.74		10.19	10.79	moment distribute
S18&S22	53.93	43.74		10.19	10.79	moment distribute
S14&S19	45.43	38.97		6.46	9.09	Take average
S5&S6	29.02	13.56		15.46	5.80	moment distribute
S8&S9	40.5	16.17		24.33	8.10	moment distribute
S10&S11	28	17.67		10.33	5.60	moment distribute
S15&S16	32.38	19.29		13.09	6.48	moment distribute
S16&S17	32.38	29.25		3.13	6.48	Take average
S17&S18	29.25	17.95		11.3	5.85	moment distribute

Moment adjustment between panels

Support moment adjustment for second fioor															
Panel	LY	LX	M1	M2	K1	K2	K1+K2	df1	df2	Δm	Δmdf1	Δmdf2	M1(adje)	M2(adje)	
S2&S5	6.64	1.50	44.81	17.79	0.11	0.67	0.78	0.14	0.86	27.02	-3.91	23.11	40.9	40.90	S2&S5
S2&S6	6.64	1.50	44.81	29.02	0.11	0.67	0.78	0.14	0.86	15.79	-2.29	13.50	42.5	42.52	S2&S6
S5&S8	9.17	6.64	29.81	17.79	0.08	0.15	0.23	0.35	0.65	12.02	-4.23	7.79	25.6	25.58	S5&S8
S6&S9	9.17	6.64	42.68	29.02	0.08	0.15	0.23	0.35	0.65	13.66	-4.81	8.85	37.9	37.87	S6&S9
S12&S17	10.65	7.89	141.04	53.93	0.07	0.13	0.20	0.36	0.64	87.11	-31.11	56.00	109.9	109.93	S12&S17
S13&S18	10.65	7.89	35.35	27.31	0.07	0.13	0.20	0.36	0.64	8.04	-2.87	5.17	32.5	32.48	S13&S18
S10&S15	10.65	9.17	37.98	27.61	0.07	0.11	0.18	0.39	0.61	10.37	-4.07	6.30	33.9	33.91	S10&S15
S11&S16	10.65	6.64	59.71	36.70	0.07	0.15	0.22	0.32	0.68	23.01	-7.33	15.68	52.4	52.38	S11&S16
S4&S19	10.65	3.22	53.28	38.97	0.07	0.31	0.38	0.18	0.82	14.31	-2.65	11.66	50.6	50.63	S4&S19
S21&S16	10.65	3.22	59.71	43.74	0.07	0.31	0.38	0.18	0.82	15.97	-2.95	13.02	56.8	56.76	S21&S16
S17&S21	10.65	3.22	53.93	43.74	0.07	0.31	0.38	0.18	0.82	10.19	-1.88	8.31	52.0	52.05	S17&S21
S18&S22	10.65	3.22	53.93	43.74	0.07	0.31	0.38	0.18	0.82	10.19	-1.88	8.31	52.0	52.05	S18&S22
S5&S6	6.52	5.61	29.02	13.56	0.12	0.18	0.29	0.39	0.61	15.46	-6.06	9.40	23.0	22.96	S5&S6
S8&S9	6.52	5.61	40.50	16.17	0.12	0.18	0.29	0.39	0.61	24.33	-9.54	14.79	31.0	30.96	S8&S9
S10&S11	6.52	5.61	28.00	17.67	0.12	0.18	0.29	0.39	0.61	10.33	-4.05	6.28	23.9	23.95	S10&S11
S15&S16	6.52	5.61	32.38	19.29	0.12	0.18	0.29	0.39	0.61	13.09	-5.13	7.96	27.2	27.25	S15&S16
S17&S18	6.32	5.76	29.25	17.95	0.12	0.17	0.29	0.41	0.59	11.3	-4.59	6.71	24.7	24.66	S17&S18

Moment adjustment between panels

Panel	ML	MS	ms1-ms2	0.2*msla	moment distribute	M(adje)
S1&S4	53.28	46.7	6.58	10.656	Take average	49.99
s3&S7	48.64	44.92	3.72	9.728	Take average	46.78
S8&S10	29.81	27.61	2.2	5.962	Take average	28.71
S9&S11	42.68	36.7	5.98	8.536	Take average	39.69
S15&S20	43.74	37.985	5.755	8.748	Take average	40.8625
S14&S19	45.43	38.97	6.46	9.086	Take average	42.2
S16&S17	32.38	29.25	3.13	6.476	Take average	30.815

Table 39 Span Moment Adjustment for second floor

panel	LY/LX	Mxf	Myf	Δmxs	Δmys	Cx1	Cy1	Cx2	Cy2	Δmxf	Δmyf	mxf(adj)	myf(adj)
s9	1.40	32.83	30.65	2.98	9.54	0.40	0.33	0.32	0.11	4.19	2.04	37.02	32.69
s16	1.60	44.53	24.29	2.95	1.56	0.44	0.29	0.30	0.08	1.77	0.98	46.30	25.27
s17	1.70	40.21	21.93	31.11	4.59	0.46	0.27	0.29	0.07	15.65	8.80	55.86	30.73

Table 40 Reinforcement design for second floor

Reinforcement design for second floor

panel	moment	d	B	K	Z	ASmin	Asrequred	ASmax	as	Scalculated	Sprovided
1mxs	49.99	174	1000	0.07	0.95	226.2	869.16	6960	113.04	130.06	130
2mxs	40.89	174	1000	0.05	0.98	226.2	686.30	6960	113.04	164.71	160
3mxs	42.52	174	1000	0.06	0.97	226.2	722.07	6960	113.04	156.55	155
4mxs	53.28	174	1000	0.07	0.99	226.2	890.59	6960	113.04	126.93	125
5mxs	40.89	174	1000	0.05	0.95	226.2	710.94	6960	113.04	159.00	155
mxf	13.55	174	1000	0.02	0.98	226.2	227.42	6960	113.04	499.73	400
mys	22.96	174	1000	0.03	0.97	226.2	389.90	6960	113.04	289.92	285
myf	10.17	174	1000	0.01	0.99	226.2	169.99	6960	113.04	499.73	400
6mxs	42.52	174	1000	0.06	0.95	226.2	740.94	6960	113.04	152.56	150
mxf	22.47	174	1000	0.03	0.97	226.2	381.35	6960	113.04	296.42	290
mys	22.96	174	1000	0.03	0.97	226.2	389.90	6960	113.04	289.92	285
myf	22.47	174	1000	0.03	0.97	226.2	381.35	6960	113.04	296.42	295
7mxs	46.78	174	1000	0.06	0.94	226.2	819.99	6960	113.04	137.85	30
8mxs	28.71	174	1000	0.04	0.97	226.2	491.16	6960	113.04	230.15	230
mxf	22.13	174	1000	0.03	0.97	226.2	375.41	6960	113.04	301.11	300
mys	30.96	174	1000	0.04	0.96	226.2	531.20	6960	113.04	212.80	210
myf	12.13	174	1000	0.02	0.99	226.2	203.24	6960	113.04	499.73	400
9mxs	39.70	174	1000	0.05	0.95	226.2	689.14	6960	113.04	164.03	160
mxf	37.02	174	1000	0.05	0.95	226.2	640.30	6960	113.04	176.54	175
mys	30.96	174	1000	0.04	0.96	226.2	531.20	6960	113.04	212.80	210
myf	32.69	174	1000	0.04	0.96	226.2	562.16	6960	113.04	201.08	200
10mxs	33.91	174	1000	0.04	0.96	226.2	584.08	6960	113.04	193.53	190
mxf	29.27	174	1000	0.04	0.96	226.2	501.10	6960	113.04	225.58	220

mys	23.95	174	1000	0.03	0.97	226.2	407.23	6960	113.04	277.58	270
myf	13.25	174	1000	0.02	0.98	226.2	222.31	6960	113.04	508.49	400
11mxs	52.38	174	1000	0.07	0.93	226.2	925.46	6960	113.04	122.14	120
mxf	28.00	174	1000	0.04	0.97	226.2	478.57	6960	113.04	236.20	230
mys	23.95	174	1000	0.03	0.97	226.2	407.23	6960	113.04	277.58	270
myf	21.00	174	1000	0.03	0.97	226.2	355.74	6960	113.04	317.76	315
12mxs	109.93	174	1000	0.15	0.85	226.2	2137.91	6960	113.04	52.87	50
13mxs	32.48	174	1000	0.04	0.96	226.2	558.40	6960	113.04	202.44	200
mxf	28.94	174	1000	0.04	0.97	226.2	495.24	6960	113.04	228.25	220
mys	17.48	174	1000	0.02	0.98	226.2	294.81	6960	113.04	383.43	380
myf	13.11	174	1000	0.02	0.98	226.2	219.92	6960	113.04	514.00	400
14mxs	45.42	174	1000	0.06	0.94	226.2	794.65	6960	113.04	142.25	140
15mxs	40.86	174	1000	0.05	0.95	226.2	710.39	6960	113.04	159.12	155
mxf	28.94	174	1000	0.04	0.97	226.2	495.24	6960	113.04	228.25	220
mys	27.25	174	1000	0.04	0.97	226.2	465.30	6960	113.04	242.94	240
myf	14.47	174	1000	0.02	0.98	226.2	243.14	6960	113.04	464.92	400
16mxs	56.76	174	1000	0.07	0.93	226.2	1009.23	6960	113.04	112.01	110
mxf	46.33	174	1000	0.06	0.94	226.2	811.60	6960	113.04	139.28	130
mys	30.82	174	1000	0.04	0.96	226.2	528.70	6960	113.04	213.81	210
myf	25.30	174	1000	0.03	0.97	226.2	430.92	6960	113.04	262.32	260
17mxs	109.93	174	1000	0.15	0.85	226.2	2137.91	6960	113.04	52.87	50
mxf	55.86	174	1000	0.07	0.93	226.2	991.92	6960	113.04	113.96	110
mys	30.82	174	1000	0.04	0.96	226.2	528.70	6960	113.04	213.81	210
myf	30.82	174	1000	0.04	0.96	226.2	528.70	6960	113.04	213.81	210
18mxs	52.04	174	1000	0.07	0.94	226.2	919.01	6960	113.04	123.00	120
mxf	26.43	174	1000	0.03	0.97	226.2	450.82	6960	113.04	250.74	250
mys	24.66	174	1000	0.03	0.97	226.2	419.68	6960	113.04	269.35	265

myf	13.47	174	1000	0.02	0.98	226.2	226.06	6960	113.04	499.73	400
19mxs	50.63	174	1000	0.07	0.94	226.2	892.31	6960	113.04	126.68	120
20mxs	40.86	174	1000	0.05	0.95	226.2	710.39	6960	113.04	159.12	155
21mxs	56.76	174	1000	0.07	0.93	226.2	1009.23	6960	113.04	112.01	110
22mxs	52.04	174	1000	0.07	0.94	226.2	919.01	6960	113.04	123.00	120

Table 41 Check Depth for Deflection: Serviceability Requirement for second floor

panel	moment	Lx	D	B	k	z	ASmin	Asrequred	ASmax	ρ	ρo	F1	K1	LX/d	d	Check
1mxs	49.99	1500	0.174	1000	0.07	0.95	226.2	869.16	6960	0.05	0.4472	1.25	0.4	19.05	91.39	OK
2mxs	40.89	1500	0.174	1000	0.05	0.98	226.2	686.30	6960	0.04	0.4472	1.25	0.4	27.59	63.72	OK
3mxs	42.52	1500	0.174	1000	0.06	0.97	226.2	722.07	6960	0.04	0.4472	1.25	0.4	25.49	68.70	OK
4mxs	53.28	1500	0.174	1000	0.07	0.99	226.2	890.59	6960	0.05	0.4472	1.25	11.5	52.70	33.02	OK
5mxs	40.89	5610	0.174	1000	0.05	0.95	226.2	710.94	6960	0.04	0.4472	1.25	11.5	75.07	23.18	OK
mxf	13.55	5610	0.174	1000	0.02	0.98	226.2	227.42	6960	0.01	0.4472	1.25	11.5	426.79	4.08	OK
mys	22.96	5610	0.174	1000	0.03	0.97	226.2	389.90	6960	0.02	0.4472	1.25	11.5	189.04	9.20	OK
myf	10.17	5610	0.174	1000	0.01	0.99	226.2	169.99	6960	0.01	0.4472	1.25	11.5	660.47	2.63	OK
6mxs	42.52	6640	0.174	1000	0.06	0.95	226.2	740.94	6960	0.04	0.4472	1.25	11.5	70.38	24.72	OK
mxf	22.47	6640	0.174	1000	0.03	0.97	226.2	381.35	6960	0.02	0.4472	1.25	11.5	195.52	8.90	OK
mys	22.96	6640	0.174	1000	0.03	0.97	226.2	389.90	6960	0.02	0.4472	1.25	11.5	189.04	9.20	OK
myf	22.47	6640	0.174	1000	0.03	0.97	226.2	381.35	6960	0.02	0.4472	1.25	0.4	68.01	25.85	OK
7mxs	46.78	1500	0.174	1000	0.06	0.94	226.2	819.99	6960	0.05	0.4472	1.25	11.5	60.04	28.98	OK
8mxs	28.71	5610	0.174	1000	0.04	0.97	226.2	491.16	6960	0.03	0.4472	1.25	11.3	130.60	13.32	OK
mxf	22.13	5610	0.174	1000	0.03	0.97	226.2	375.41	6960	0.02	0.4472	1.25	11.3	196.75	8.84	OK
mys	30.96	5610	0.174	1000	0.04	0.96	226.2	531.20	6960	0.03	0.4472	1.25	11.3	115.80	15.03	OK
myf	12.13	5610	0.174	1000	0.02	0.99	226.2	203.24	6960	0.01	0.4472	1.25	11.3	496.52	3.50	OK
9mxs	39.7	6520	0.174	1000	0.05	0.95	226.2	689.14	6960	0.04	0.4472	1.25	11.3	77.43	22.47	OK
mxf	37.02	6520	0.174	1000	0.05	0.95	226.2	640.30	6960	0.04	0.4472	1.25	11.3	86.80	20.05	OK
mys	30.96	6520	0.174	1000	0.04	0.96	226.2	531.20	6960	0.03	0.4472	1.25	11.3	115.80	15.03	OK
myf	32.69	6520	0.174	1000	0.04	0.96	226.2	562.16	6960	0.03	0.4472	1.25	11.5	108.02	16.11	OK
10mxs	33.91	5610	0.174	1000	0.04	0.96	226.2	584.08	6960	0.03	0.4472	1.25	11.5	101.83	17.09	OK
mxf	29.27	5610	0.174	1000	0.04	0.96	226.2	501.10	6960	0.03	0.4472	1.25	11.5	128.89	13.50	OK
mys	23.95	5610	0.174	1000	0.03	0.97	226.2	407.23	6960	0.02	0.4472	1.25	11.5	176.94	9.83	OK
myf	13.25	5610	0.174	1000	0.02	0.98	226.2	222.31	6960	0.01	0.4472	1.25	11.3	433.96	4.01	OK

11mxs	52.38	6520	0.174	1000	0.07	0.93	226.2	925.46	6960	0.05	0.4472	1.25	11.3	48.72	35.71	OK
mxf	28	6520	0.174	1000	0.04	0.97	226.2	478.57	6960	0.03	0.4472	1.25	11.3	135.90	12.80	OK
mys	23.95	6520	0.174	1000	0.03	0.97	226.2	407.23	6960	0.02	0.4472	1.25	11.3	1738.66	10.01	OK
myf	21	6520	0.174	1000	0.03	0.97	226.2	355.74	6960	0.02	0.4472	1.25	0.4	75.58	23.23	OK
12mxs	109.93	166	0.174	1000	0.15	0.85	226.2	2137.91	6960	0.12	0.4472	1.25	11.5	123.32	141.09	OK
13mxs	32.48	5760	0.174	1000	0.04	0.96	226.2	558.40	6960	0.03	0.4472	1.25	11.5	109.14	15.94	OK
mxf	28.94	5760	0.174	1000	0.04	0.97	226.2	495.24	6960	0.03	0.4472	1.25	11.5	131.24	13.26	OK
mys	17.48	5760	0.174	1000	0.02	0.98	226.2	294.81	6960	0.02	0.4472	1.25	11.5	288.71	6.03	OK
myf	13.11	5760	0.174	1000	0.02	0.98	226.2	219.92	6960	0.01	0.4472	1.25	0.4	156.12	111.45	OK
14mxs	45.42	1500	0.174	1000	0.06	0.94	226.2	794.65	6960	0.05	0.4472	1.25	11.5	630.73	27.59	OK
15mxs	40.86	5610	0.174	1000	0.05	0.95	226.2	710.39	6960	0.04	0.4472	1.25	11.5	75.16	23.15	OK
mxf	28.94	5610	0.174	1000	0.04	0.97	226.2	495.24	6960	0.03	0.4472	1.25	11.5	131.24	13.26	OK
mys	27.25	5610	0.174	1000	0.04	0.97	226.2	465.30	6960	0.03	0.4472	1.25	11.5	144.38	12.05	OK
myf	14.47	5610	0.174	1000	0.02	0.98	226.2	243.14	6960	0.01	0.4472	1.25	11.5	38.60	4.51	OK
16mxs	56.76	6520	0.174	1000	0.07	0.93	226.2	1009.23	6960	0.06	0.4472	1.25	11.5	431.92	40.28	OK
mxf	46.33	6520	0.174	1000	0.06	0.94	226.2	811.60	6960	0.05	0.4472	1.25	11.5	61.02	28.52	OK
mys	30.82	6520	0.174	1000	0.04	0.96	226.2	528.70	6960	0.03	0.4472	1.25	11.5	118.71	14.66	OK
myf	25.3	6520	0.174	1000	0.03	0.97	226.2	430.92	6960	0.02	0.4472	1.25	11.5	1623.35	10.72	OK
17mxs	109.93	6320	0.174	1000	0.15	0.85	226.2	2137.91	6960	0.12	0.4472	1.25	11.5	123.32	141.09	OK
mxf	55.86	6320	0.174	1000	0.07	0.93	226.2	991.92	6960	0.06	0.4472	1.25	11.5	444.03	39.19	OK
mys	30.82	6320	0.174	1000	0.04	0.96	226.2	528.70	6960	0.03	0.4472	1.25	11.5	1187.11	14.66	OK
Myf	30.82	6320	0.174	1000	0.04	0.96	226.2	528.70	6960	0.03	0.4472	1.25	11.5	118.71	14.66	OK
18mxs	52.04	5760	0.174	1000	0.07	0.94	226.2	919.01	6960	0.05	0.4472	1.25	11.5	50.14	34.70	OK
Mxf	26.43	5760	0.174	1000	0.03	0.97	226.2	450.82	6960	0.03	0.4472	1.25	11.5	151.53	11.48	OK
Mys	24.66	5760	0.174	1000	0.03	0.97	226.2	419.68	6960	0.02	0.4472	1.25	11.5	169.01	10.30	OK
Myf	13.47	5760	0.174	1000	0.02	0.98	226.2	226.06	6960	0.01	0.4472	1.25	0.4	149.80	116.16	OK
19mxs	50.63	1500	0.174	1000	0.07	0.94	226.2	892.31	6960	0.05	0.4472	1.25	0.4	18.27	95.16	OK

20mxs	40.86	1500	0.174	1000	0.05	0.95	226.2	710.39	6960	0.04	0.4472	1.25	0.4	26.14	65.56	OK
21mxs	56.76	1500	0.174	1000	0.07	0.93	226.2	1009.23	6960	0.06	0.4472	1.25	0.4	15.02	158.19	OK
22mxs	52.04	1500	0.174	1000	0.07	0.94	226.2	919.01	6960	0.05	0.4472	1.25	0.4	17.44	97.70	OK

Table 42 *depth determination for third & fourth floor*

Panel	Ly (mm)	Lx (mm)	Span Ratio	**Type of slab**	**Type of supporting Condition**	K	d=l/N*K*F1*F2*F3	D=d+22
T1	2890	1500	1.93	Two way	Cantilever	0.4	169.40	189.40
T2	5610	1500	3.74	One way	Cantilever	0.4	169.40	189.40
T3	2800	1500	1.87	Two way	Cantilever	0.4	169.40	189.40
T4	6640	1500	4.43	One way	Cantilever	0.4	169.40	189.40
T5	6640	5610	1.18	Two way	Interior	11.5	22.04	42.04
T6	6640	6520	1.02	Two way	Interior	11.5	25.61	45.61
T7	6640	1500	4.43	One way	Cantilever	0.4	169.40	189.40
T8	9170	5610	1.63	Two way	Interior	11.5	22.04	42.04
T9	9170	6520	1.41	Two way	End Span	11.3	26.06	46.06

T10	7980	5610	1.42	Two way	Interior Span	11.5	22.04	42.04
T11	7980	6520	1.22	Two way	End Span	11.3	26.06	46.06
T12	7980	166	48.07	One way	Cantilever	0.4	18.75	38.75
T13	7980	5760	1.39	Two way	Interior Span	11.5	22.63	42.63
T14	7980	1500	5.32	One way	Cantilever	0.4	169.40	189.40
T15	10650	5610	1.90	Two way	Interior	11.5	22.04	42.04
T16	10650	6520	1.63	Two way	Interior	11.5	25.61	45.61
T17	10650	6320	1.69	Two way	Interior	11.5	24.83	44.83
T18	10650	5760	1.85	Two way	Interior	11.5	22.63	42.63
T19	3220	1500	2.15	One way	Cantilever	0.4	169.40	189.40
T20	5610	1500	3.74	One way	Cantilever	0.4	169.40	189.40
T21	6520	1500	4.35	One way	Cantilever	0.4	169.40	189.40

T22	5760	1500	3.84	One way	Cantilever	0.4	169.40	189.40

Partition wall dead load computation

Dead Load of Partition Wall on third& flour Panels

Table 43 Dead load of HCB for third &fourth floor

Panel	Wl	Wh	Wt	rHCB	DLW	Area	DLW/A
T1	4.39	2.81	0.40	14.00	69.08	4.33	15.94
T2	7.11	2.81	0.40	14.00	111.88	8.15	13.74
T3	4.33	2.81	0.40	14.00	68.14	4.20	16.22
T4	13.54	2.81	0.40	14.00	213.07	9.96	21.39
T5							
T6	18.35	2.81	0.40	14.00	288.76	43.29	6.67
T7	9.42	2.81	0.40	14.00	148.23	9.96	14.88
T8	7.72	2.81	0.40	14.00	121.48	51.44	2.36
T9	22.21	2.81	0.40	14.00	349.50	59.89	5.84
T10	2.25	2.81	0.40	14.00	35.41	44.77	0.79
T11	22.02	2.81	0.40	14.00	346.51	52.03	6.66
T12	8.15	2.81	0.40	14.00	128.19	1.32	97.11
T13							
T14	9.48	2.81	0.40	14.00	149.18	11.97	12.46
T15	3.34	2.81	0.40	14.00	52.56	59.75	0.88
T16	34.34	2.81	0.40	14.00	540.37	69.44	7.78
T17	33.94	2.81	0.40	14.00	534.08	67.30	7.94
T18	22.17	2.81	0.40	14.00	348.87	61.34	5.69
T19	4.39	2.81	0.40	14.00	69.08	4.83	14.30
T20	7.11	2.81	0.40	14.00	111.88	8.42	13.30

T21	16.04	2.81	0.40	14.00	252.41	9.78	25.81
T22	15.64	2.81	0.40	14.00	246.11	8.64	28.49
T23	8.76	2.81	0.40	14.00	137.85	8.64	15.95

Table 44 Dead load of plaster for third & fourth floor

Panel	Wl	Wh	Pt	rpla	DLP	Area	DLW/A	TDLPW
T1	4.39	2.81	0.03	23.00	8.51	4.33	1.96	15.94
T2	7.11	2.81	0.03	23.00	13.79	8.15	1.69	13.74
T3	4.33	2.81	0.03	23.00	8.40	4.20	2.00	16.22
T4	13.54	2.81	0.03	23.00	26.25	9.96	2.64	21.39
T5							#DIV/0!	0.00
T6	18.35	**2.81**	0.03	23.00	35.58	43.29	0.82	6.67
T7	9.42	2.81	0.03	23.00	18.26	9.96	1.83	14.88
T8	7.72	2.81	0.03	23.00	14.97	51.44	0.29	2.36
T9	22.21	2.81	0.03	23.00	43.06	59.89	0.72	5.84
T10	2.25	2.81	0.03	23.00	4.36	44.77	0.10	0.79
T11	22.02	2.81	0.03	23.00	42.69	52.03	0.82	6.66
T12	8.15	2.81	0.03	23.00	15.79	1.32	11.97	97.11
T13								0.00
T14	9.48	2.81	0.03	23.00	18.38	11.97	1.54	12.46
T15	3.34	2.81	0.03	23.00	6.48	59.75	0.11	0.88
T16	34.34	2.81	0.03	23.00	66.58	69.44	0.96	7.78
T17	33.94	2.81	0.03	23.00	65.81	67.30	0.98	7.94
T18	22.17	2.81	0.03	23.00	42.99	61.34	0.70	5.69
T19	4.39	2.81	0.03	23.00	8.51	4.83	1.76	14.30
T20	7.11	2.81	0.03	23.00	13.79	8.42	1.64	13.30
T21	16.04	2.81	0.03	23.00	31.10	9.78	3.18	25.81
T22	15.64	2.81	0.03	23.00	30.32	8.64	3.51	28.49
T23	8.76	2.81	0.03	23.00	16.98	8.64	1.97	15.95

Summary of Imposed load

Table 45 imposed load third &fourth floor

Panel	Pd'=(1.35 D.L+1.5L.L)(KN/m^2)	Partition wall Dead Load ($D.L_{pw}$)(KN/m^2)	Pd=Pd'+1.3 $D.L_{pw}$
T1	11.4	15.9	32.865
T2	16.45	13.7	34.945
T3	16.45	16.2	38.32
T4	11.9	21.4	40.79
T5	11.4		11.4
T6	16.45	6.67	25.4545
T7	16.45	14.88	36.538
T8	11.4	2.36	14.586
T9	16.45	5.83	24.3205
T10	11.4	6.66	20.391
T11	16.9	6.6	25.81
T12	16.9	97.1	147.985
T13	11.4	12.4	28.14
T14	16.9	0.88	18.088
T15	16.9	7.8	27.43
T16	16.9	7.9	27.565
T17	15.4	7.8	25.93
T18	16.9	5.7	24.595
T19	15.4	14.3	34.705
T20	15.4	13.3	33.355
T21	15.4	25.8	50.23
T22	15.4	28.5	53.875

Moment Analysis

Table 46Moment Calculation for two way for third &fourth floor

Panel	Ly/Lx	Pd (KN/m)	$Lx^2(m^2)$	bxs	bxf	bys	Byf	M_{xs}(kN*m)	M_{xf}(kN*m)	M_{ys}(kN*m)	M_{yf}(kN*m)
T5	1.20	11.40	31.50	0.04	0.03	0.03	0.02	15.08	11.49	11.49	8.62
T6	1.00	25.45	42.50	0.03	0.02	0.03	0.02	33.54	25.96	34.62	25.96
T8	1.75	14.59	31.50	0.06	0.04	0.03	0.02	27.11	20.22	14.70	11.03
T9	1.40	24.32	42.50	0.04	0.03	0.04	0.03	40.31	31.01	38.24	28.94
T10	1.40	20.39	31.50	0.05	0.05	0.03	0.02	32.12	34.04	20.55	15.42
T11	1.20	25.81	42.50	0.04	0.03	0.03	0.02	46.07	35.10	35.10	26.33
T13	1.40	28.14	33.20	0.05	0.05	0.03	0.02	46.71	49.52	29.90	22.42
T15	2.00	27.43	31.50	0.06	0.05	0.03	0.02	54.43	41.47	27.65	20.74
T16	1.75	27.57	42.50	0.06	0.04	0.03	0.02	69.12	51.55	37.49	28.12
T17	1.75	25.93	39.90	0.06	0.04	0.03	0.02	61.04	45.52	33.11	24.83
T18	2.00	24.60	33.20	0.06	0.05	0.03	0.02	51.44	39.19	26.13	19.60

Table 47 Moment Calculation for cantilever third &fourth floor

Moment Calculation for cantilever slabs for thrid floor.					
Panel	Type of supportig Condition	$D.L_{ex}$	Lx	pd'=1.35 D.L +1. 5 L.L	MXS
T1	Two Way Cantilever	17.03	1.5	32.87	44.03
T2	One Way Cantilever	17.03	1.5	34.95	45.20
T3	Two Way Cantilever	17.03	1.5	38.32	47.10
T4	One Way Cantilever	17.03	1.5	40.79	48.49
T7	One Way Cantilever	17.03	1.5	36.54	46.10
T12	One Way Cantilever	17.03	1.66	147.99	120.39
T14	One Way Cantilever	17.03	1.5	18.09	35.72
T19	Two Way Cantilever	17.03	1.5	34.71	45.07
T20	Two Way Cantilever	17.03	1.5	33.36	44.31
T21	One Way Cantilever	17.03	1.5	50.23	53.80
T22	Two Way Cantilever	17.03	1.5	53.88	55.85
T23	Two Way Cantilever	17.03	1.5	36.93	46.32

Table 48 Support Moment adjustment between panels for third &fourth floor

Panel	ML	MS		ms1-ms2	0.2*msla	moment distribute
T1&T4	48.49	44.03		4.46	9.70	Take average
T2&T5	45.20	15.08		30.12	9.04	moment distribute
T2&T6	45.20	33.50		11.70	9.04	moment distribute
T3&T7	47.10	46.10		1.00	9.42	Take average
T5&T8	27.11	15.08		12.03	5.42	moment distribute
T6&T9	40.31	33.50		6.81	8.06	Take average
T8&T10	32.11	27.11		5.00	6.42	Take average
T9&T11	46.71	40.31		6.40	9.34	moment distribute
T12&T17	130.21	61.04		69.17	26.04	moment distribute
T13&T18	51.44	46.71		4.73	10.29	Take average
T10&T15	54.40	32.11		22.29	10.88	moment distribute
T11&T16	69.12	46.07		23.05	13.82	moment distribute
T4&T19	48.49	45.07		3.42	9.70	Take average
T15&T20	54.40	44.31		10.09	10.88	Take average
T21&T16	69.12	53.80		15.32	13.82	moment distribute
T17&T21	61.04	53.80		7.24	12.21	Take average
T18&T22	55.85	51.44		4.41	11.17	Take average
T14&T19	45.07	35.72		9.35	9.01	moment distribute
T5&T6	34.62	11.50		23.12	6.92	moment distribute
T8&T9	38.24	14.70		23.54	7.65	moment distribute
T10&T11	26.32	20.51		5.81	5.26	moment distribute
T15&T16	28.11	27.65		0.46	5.62	Take average
T16&T17	33.11	28.11		5.00	6.62	Take average
T17&T18	33.11	26.13		6.98	6.62	moment distribute

Table 49 Moment adjustment between panels

Support moment adjustment for thrid fioor															
Panel	LY	LX	**M1**	M2	K1	K2	K1+K2	Df1	Df2	Δm	Δmdf1	Δmdf2	M1(adje)	M2(adje)	
T2&T5	6.64	1.50	45.20	15.08	0.11	0.67	0.78	0.14	0.86	30.12	-4.36	25.76	40.84	40.84	T2&T5
T2&T6	6.64	1.50	45.20	33.50	0.11	0.67	0.78	0.14	0.86	11.70	-1.70	10.00	43.50	43.50	T2&T6
T5&T8	9.17	6.64	27.11	15.08	0.08	0.15	0.23	0.35	0.65	12.03	-4.23	7.80	22.88	22.88	T5&T8
`	9.17	7.98	46.71	40.31	0.08	0.13	0.21	0.39	0.61	6.40	-2.53	3.87	44.18	44.18	T9&T11
T12&T17	10.65	7.89	130.21	61.04	0.07	0.13	0.20	0.36	0.64	69.17	-24.71	44.46	105.50	105.50	T12&T17
T10&T15	10.65	9.17	54.40	32.11	0.07	0.11	0.18	0.39	0.61	22.29	-8.75	13.54	45.65	45.65	T10&T15
T11&T16	10.65	6.64	69.12	46.07	0.07	0.15	0.22	0.32	0.68	23.05	-7.34	15.71	61.78	61.78	T11&T16
T21&T16	10.65	3.22	69.12	53.80	0.07	0.31	0.38	0.18	0.82	15.32	-2.83	12.49	66.29	66.29	T21&T16
T14&T19	10.65	3.22	45.07	35.72	0.07	0.31	0.38	0.18	0.82	9.35	-1.73	7.62	43.34	43.34	T14&T19
T5&T6	6.52	5.61	34.62	11.50	0.12	0.18	0.29	0.39	0.61	23.12	-9.07	14.05	25.55	25.55	T5&T6
T8&T9	6.52	5.61	38.24	14.70	0.12	0.18	0.29	0.39	0.61	23.54	-9.23	14.31	29.01	29.01	T8&T9
T10&T11	6.52	5.61	26.32	20.51	0.12	0.18	0.29	0.39	0.61	5.81	-2.28	3.53	24.04	24.04	T10&T11
T17&T18	6.32	5.76	33.11	26.13	0.12	0.17	0.29	0.41	0.59	6.98	-2.83	4.15	30.28	30.28	T17&T18

Table 50 Moment adjustment between panels

Panel	ML	MS	ms1-ms2	0.2*msla	moment distribute	M(adje)
T1&T4	48.49	44.03	4.46	9.698	Take average	46.26
T3&T7	47.1	46.1	1	9.42	Take average	46.6
T6&T9	40.31	33.5	6.81	8.062	Take average	36.905
T8&T10	32.11	27.11	5	6.422	Take average	29.61
T13&T18	51.44	46.71	4.73	10.288	Take average	49.075
T4&T19	48.49	45.07	3.42	9.698	Take average	46.78
T15&T20	54.4	44.31	10.09	10.88	Take average	49.355
T17&T21	61.04	53.8	7.24	12.208	Take average	57.42
T18&T22	55.85	51.44	4.41	11.17	Take average	53.645
T15&T16	28.11	27.65	0.46	5.622	Take average	27.88
T16&T17	33.11	28.11	5	6.622	Take average	30.61

Table 51 Span Moment Adjustment for third &fourth floor

Panel	LY/LX	mxf	Myf	Δmxs	Δmys	Cx1	Cy1	Cx2	Cy2	Δmxf	Δmyf	mxf(adj)	myf(adj)
T15	1.90	41.50	20.74	5.05	-0.23	0.48	0.25	0.26	0.06	2.37	1.25	43.87	21.99
T16	1.60	51.55	28.11	2.82	-2.50	0.44	0.29	0.30	0.08	0.51	0.61	52.06	28.72
T17	1.70	45.50	24.83	24.71	2.50	0.46	0.27	0.29	0.07	12.10	6.91	57.60	31.74

Table 52 Reinforcement Design for Third Floor and 4th floor

Reinforcement Design For Third Floor and 4th floor											
Panel	moment	d	B	k	z	ASmin	Asrequred	ASmax	as	Scalculated	Sprovided
1mxs	46.26	174	1000	0.06	0.95	226.20	804.26	6960	113.04	140.55	140
2mxs	40.84	174	1000	0.05	0.99	226.20	683.75	6960	113.04	165.32	160
3mxs	43.50	174	1000	0.06	0.97	226.20	741.15	6960	113.04	152.52	150
4mxs	48.49	174	1000	0.06	0.99	226.20	809.00	6960	113.04	139.73	135
5mxs	40.84	174	1000	0.05	0.95	226.20	712.62	6960	113.04	158.63	155
Mxf	11.50	174	1000	0.02	0.97	226.20	196.04	6960	113.04	499.73	400
Mys	25.55	174	1000	0.03	0.97	226.20	435.32	6960	113.04	259.67	255
Myf	8.61	174	1000	0.01	0.97	226.20	146.77	6960	113.04	499.73	400
6mxs	43.50	174	1000	0.06	0.94	226.20	762.31	6960	113.04	148.29	140
Mxf	25.96	174	1000	0.03	0.96	226.20	444.63	6960	113.04	254.23	250
Mys	25.55	174	1000	0.03	0.98	226.20	432.38	6960	113.04	261.44	260
Myf	25.96	174	1000	0.03	0.96	226.20	444.28	6960	113.04	254.44	250
7mxs	46.60	174	1000	0.06	0.99	226.20	779.74	6960	113.04	144.97	140
8mxs	29.61	174	1000	0.04	0.95	226.20	517.17	6960	113.04	218.58	215
Mxf	20.20	174	1000	0.03	0.97	226.20	345.67	6960	113.04	327.01	325
Mys	29.00	174	1000	0.04	0.96	226.20	496.30	6960	113.04	227.76	225
Myf	11.03	174	1000	0.01	0.97	226.20	188.75	6960	113.04	499.73	400
9mxs	44.20	174	1000	0.06	0.94	226.20	773.55	6960	113.04	146.13	140
Mxf	28.94	174	1000	0.04	0.96	226.20	498.54	6960	113.04	226.74	225
Mys	29.00	174	1000	0.04	0.97	226.20	493.15	6960	113.04	229.22	225
Myf	28.94	174	1000	0.04	0.98	226.20	486.83	6960	113.04	232.19	230
10mxs	45.65	174	1000	0.06	0.92	226.20	817.74	6960	113.04	138.23	135
Mxf	34.00	174	1000	0.04	0.96	226.20	586.56	6960	113.04	192.72	190

Mys	24.04	174	1000	0.03	0.97	226.20	408.80	6960	113.04	276.51	270
Myf	15.40	174	1000	0.02	0.97	226.20	262.71	6960	113.04	430.28	400
11mxs	61.78	174	1000	0.08	0.86	226.20	1191.23	6960	113.04	94.89	90
Mxf	35.10	174	1000	0.05	0.94	226.20	617.27	6960	113.04	183.13	180
Mys	24.04	174	1000	0.03	0.94	226.20	423.02	6960	113.04	267.22	260
Myf	26.52	174	1000	0.04	0.96	226.20	454.39	6960	113.04	248.77	240
12mxs	105.50	174	1000	0.14	0.97	226.20	1790.36	6960	113.04	63.14	60
13mxs	49.10	174	1000	0.06	0.96	226.20	847.76	6960	113.04	133.34	130
Mxf	49.52	174	1000	0.07	0.94	226.20	871.17	6960	113.04	129.76	125
Mys	29.90	174	1000	0.04	0.95	226.20	521.99	6960	113.04	216.55	215
Myf	22.42	174	1000	0.03	0.97	226.20	383.14	6960	113.04	295.04	290
14mxs	35.72	174	1000	0.05	0.97	226.20	605.85	6960	113.04	186.58	180
15mxs	49.35	174	1000	0.07	0.92	226.20	890.09	6960	113.04	127.00	120
Mxf	43.87	174	1000	0.06	0.94	226.20	774.75	6960	113.04	145.90	145
Mys	27.88	174	1000	0.04	0.96	226.20	478.14	6960	113.04	236.42	230
Myf	22.00	174	1000	0.03	0.97	226.20	376.37	6960	113.04	300.34	300
16mxs	66.30	174	1000	0.09	0.86	226.20	1278.39	6960	113.04	88.42	85
Mxf	52.06	174	1000	0.07	0.93	226.20	926.80	6960	113.04	121.97	120
Mys	30.61	174	1000	0.04	0.96	226.20	524.72	6960	113.04	215.43	215
Myf	28.72	174	1000	0.04	0.96	226.20	493.27	6960	113.04	229.16	225
17mxs	105.50	174	1000	0.14	0.93	226.20	1867.41	6960	113.04	60.53	60
Mxf	57.60	174	1000	0.08	0.95	226.20	999.18	6960	113.04	113.13	110
Mys	30.27	174	1000	0.04	0.97	226.20	516.12	6960	113.04	219.02	215
Myf	31.74	174	1000	0.04	0.98	226.20	536.72	6960	113.04	210.61	210
18mxs	53.65	174	1000	0.07	0.94	226.20	940.42	6960	113.04	120.20	120
Mxf	39.20	174	1000	0.05	0.94	226.20	689.61	6960	113.04	163.92	160
Mys	26.13	174	1000	0.03	0.92	226.20	471.29	6960	113.04	239.85	230

myf	19.59	174	1000	0.03	0.93	226.20	346.75	6960	113.04	325.99	320
19mxs	46.78	174	1000	0.06	1.00	226.20	772.56	6960	113.04	146.32	145
20mxs	49.35	174	1000	0.07	1.00	226.20	815.00	6960	113.04	138.70	135
21mxs	66.30	174	1000	0.09	1.00	226.20	1094.93	6960	113.04	103.24	100
22mxs	53.65	174	1000	0.07	1.00	226.20	886.02	6960	113.04	127.58	125

Appendix C Lateral Load Analysis

Table 53 ***Computation of weight of structure along with center of mass ground floor***

Computation of weight of structure along with center of mass								
FOUNDATION					Moment arm		Moment	
column on	Depth d(m)	Width W(m)	height h(m)	Weight W(KN)	X(m)	Y(m)	Mx(KNm)	My(KNm)
Axis-A1	0.4	0.4	4	16	0	34.44	0	551.04
A2	0.4	0.4	4	16	0	27.8	0	444.8
A3	0.4	0.4	4	16	0	18.63	0	298.08
A4	0.4	0.4	4	16	0	10.65	0	170.4
A5	0.4	0.4	4	16	0	0	0	0
Axis-B1	0.4	0.4	4	16	5.61	34.44	89.76	551.04
B2	0.4	0.4	4	16	5.61	27.8	89.76	444.8
B3	0.4	0.4	4	16	5.61	18.63	89.76	298.08
B4	0.4	0.4	4	16	5.61	10.56	89.76	168.96
B5	0.4	0.4	4	16	5.61	0	89.76	0
Axis-c1	0.4	0.4	4	16	12.13	34.44	194.08	551.04
C2	0.4	0.4	4	16	12.13	27.8	194.08	444.8
C3	0.4	0.4	4	16	12.13	18.63	194.08	298.08
C4	0.4	0.4	4	16	12.13	10.65	194.08	170.4
C5	0.4	0.4	4	16	12.13	0	194.08	0
Axis-D3	0.4	0.4	4	16	18.65	18.63	298.4	298.08

D4	0.4	0.4	4	16	18.65	10.65	298.4	170.4
D5	0.4	0.4	4	16	18.65	0	298.4	0
Axis-E3	0.4	0.4	4	16	24.22	18.63	387.52	298.08
E4	0.4	0.4	4	16	24.22	10.65	387.52	170.4
E5	0.4	0.4	4	16	24.22	0	387.52	0
Total Weight due to foundation column=				**336**	SUM =		**3089.44**	**5328.48**
				369.6				

MY = 5328.48 **Xm** = 14.41688312

Mx = 3089.44 **Ym** = 8.358874459

GROUND FLOOR CENTER OF MASS								
GRADE					Moment arm		Moment	
BEAM	Length L(m)	Width W(m)	Depth d(m)	Weight W(KN)	X(m)	Y(m)	Mx(KNm)	My(KNm)
Axis-A	37.44	0.5	0.4	179.712	0	18.62	3346.24	0.00
	2.5	0.5	0.4	12	0	-0.8	-9.60	0.00
Axis-B	37.44	0.5	0.4	179.712	5.61	18.62	3346.24	1008.18
	2.5	0.5	0.4	12	5.61	-0.8	-9.60	67.32
Axis-C	37.44	0.5	0.4	179.712	12.13	18.62	3346.24	2179.91
	2.5	0.5	0.4	12	12.13	-0.8	-9.60	145.56
Axis-D	20.13	0.5	0.4	96.624	18.65	9.315	900.05	1802.04
	1.6	0.5	0.4	7.68	18.65	-0.8	-6.14	143.23
Axis-E	20.13	0.5	0.4	96.624	24.41	9.315	900.05	2358.59

	2.5	0.5	0.4	12	24.41	-0.8	-9.60	292.92
Axis-1	17.82	0.5	0.4	85.536	8.2	34.44	2945.86	701.40
Axis-2	17.82	0.5	0.4	85.536	8.2	27.8	2377.90	701.40
Axis-3	28.8	0.5	0.4	138.24	13.7	18.63	2575.41	1893.89
Axis-4	28.8	0.5	0.4	138.24	13.7	10.65	1472.26	1893.89
Axis-5	28.8	0.5	0.4	138.24	13.7	0	0.00	1893.89
Total Weight due to beam=				**1373.86**	SUM =		**21165.70**	**14936.65**

					Moment arm		Moment	
Wall	Length L(m)	Thicknesst(m)	height h(m)	Weight W(KN)	X(m)	Y(m)	Mx(KNm)	My(KNm)
Axis-A'	35.94	0.4	3.85	774.87	2.7	18.24	14133.56314	2092.14
AXis-B	0.61	0.4	3.6	12.30	5.62	35.48	436.32	69.11
	4.89	0.4	3.6	98.58	5.62	31.14	3069.86	554.03
AXis-B'	2.73	0.3	3.85	44.14	9.70	30.72	1356.11	428.20
	10.36	0.4	3.6	208.86	8.94	17.46	3646.65	1867.19
AXis-B"	3.34	0.4	3.85	72.01	7.18	3.34	240.51	517.03
AXis-C	8.37	0.4	3.6	168.74	12.14	23.24	3921.50	2048.49
AXis-C'	9.1	0.4	3.85	196.20	17.40	16.84	3303.94	3413.81
AXis-D	4.93	0.4	3.6	99.39	18.46	5.34	530.74	1834.72
	0.83	0.4	3.6	16.73	18.46	11.90	199.12	308.89
AXis-E	19.02	0.4	3.6	383.44	27.10	9.52	3650.38	10391.31
AXis-5'	2.75	0.4	3.85	59.29	12.12	-3.00	-177.87	718.59

AXis-5	1.23	0.4	3.6	24.80	-1.46	0.00	0.00	-36.20
	7.84	0.4	3.6	158.05	2.82	0.00	0.00	445.71
AXis-4'	18.51	0.4	3.85	399.08	7.18	6.88	2745.64	2865.36
AXis-3	14.11	0.4	3.6	284.46	-1.46	18.84	5359.18	-415.31
	2	0.4	3.6	40.32	2.82	18.84	759.63	113.70
AXis-2	16.52	0.4	3.6	333.04	-1.46	28.02	9331.87	-486.24
	6.85	0.4	3.6	138.10	2.82	28.02	3869.45	389.43
	4.29	0.4	3.6	86.49	8.88	28.02	2423.35	768.00
AXis-1	4.6	0.4	3.85	99.18	7.44	36.28	3598.11	737.87
Total weight due to wall =				3489.20	SUM =		**62398.04**	**27857.84**

		Length L(m)	DL (KN/m)	Weight W(KN)	Moment arm		Moment	
Staircase					X(m)	Y(m)	Mx(KNm)	My(KNm)
Side staircase (right)	Landing	1.66	14.9	21.23	5.34	5.34	113.38	113.38
	Flight	6.72	18.2	120.22	5.34	5.34	641.98	641.98
	landing	2.3	14.9	29.42	5.34	5.34	157.09	157.09

Total Weight due to staircase =	170.87	SUM =	**912.44**	**912.44**

					Moment arm		Moment	
column on axis	Length L(m)	Width W(m)	height h(m)	Weight W(KN)	X(m)	Y(m)	Mx(KNm)	My(KNm)
A-1	0.4	0.4	4	15.36	0	17.23	264.65	4
A-2	0.4	0.4	4	15.36	0	13.91	213.66	0
A-3	0.4	0.4	4	15.36	0	9.32	143.16	0
A-4	0.4	0.4	4	15.36	0	5.33	81.87	0
A-5	0.4	0.4	4	15.36	0	0	0	0
B-1	0.4	0.4	4	15.36	2.81	17.23	264.65	43.16
B-2	0.4	0.4	4	15.36	2.81	13.91	213.66	43.16
B-3	0.4	0.4	4	15.36	2.81	9.32	143.16	43.16
B-4	0.4	0.4	4	15.36	2.81	5.33	81.87	43.16
B-5	0.4	0.4	4	15.36	2.81	0	0	43.16
C-1	0.4	0.4	4	15.36	6.07	17.23	264.65	93.24
C-2	0.4	0.4	4	15.36	6.07	13.91	213.66	93.24
C-3	0.4	0.4	4	15.36	6.07	9.32	143.16	93.24
C-4	0.4	0.4	4	15.36	6.07	5.33	81.87	93.24
C-5	0.4	0.4	4	15.36	6.07	0	0	93.24
D-3	0.4	0.4	4	15.36	9.23	9.32	143.16	141.77
D-4	0.4	0.4	4	15.36	9.23	5.33	81.87	141.77
D-5	0.4	0.4	4	15.36	9.23	0	0	141.77
E-3	0.4	0.4	4	15.36	12.11	9.32	143.16	186.01
E-4	0.4	0.4	4	15.36	12.11	5.33	81.87	186.01
E-5	0.4	0.4	4	15.36	12.11	0	0	186.01

Total Weight due to foundation column=	**322.56**	SUM =	**2560.08**	**1669.34**
Total Weight of ground floor =	**5692.48**			

MY = 50704.75 **Xm = 15.83**

Mx = 90125.71 **Ym = 8.91**

					Moment arm		Moment	
panel	Length L(m)	Width W(m)	Dead load	Weight W(KN)	X(m)	Y(m)	My(KNm)	Mx(KNm)
F-1	1.5	2.89	6.59	28.57	-0.75	35.19	-21.43	1005.30
F-2	5.61	1.5	6.59	55.45	2.8	35.19	155.27	1951.46
	6.52	1.5	6.59	64.45	8.87	35.19	571.67	2268.00
F-3	2.8	1.5	6.59	27.68	13.53	35.19	374.48	973.99
F-4	6.64	1.5	6.63	66.03	-0.75	31.12	-49.53	2055.00
	9.17	1.5	6.63	91.20	-0.75	23.22	-68.40	2117.56
	7.98	1.5	6.63	79.36	-0.75	14.64	-59.52	1161.85
	10.65	1.5	6.63	105.91	-0.73	5.33	-77.32	564.52
F-5	6.64	5.61	6.59	245.48	2.8	31.12	687.34	7639.34
F-6	6.64	6.52	6.59	285.30	8.87	31.12	2530.61	8878.52
F-7	6.64	2.8	6.59	122.52	13.53	31.12	1657.71	3812.86

F-8	9.17	5.61	6.63	341.07	2.8	23.22	955.00	7919.69
F-9	9.17	6.52	6.98	417.32	8.87	23.22	3701.66	9690.24
F-10	7.98	5.61	6.63	296.81	2.8	14.64	831.07	4345.31
F-11	7.98	6.52	6.98	363.17	8.87	14.64	3221.29	5316.76
F-12	7.98	1.66	6.63	87.83	15.39	14.64	1351.65	1285.78
F-13	7.98	5.76	6.63	304.75	21.53	14.64	6561.19	4461.49
F-14	7.98	1.5	6.98		25.16	14.64	0.00	0.00
	10.65	1.5	6.98	111.51	25.16	5.33	2805.48	594.32
F-15	10.65	5.61	6.63	396.12	2.8	5.33	1109.13	2111.32
F-16	10.65	6.52	6.63	460.37	8.87	5.33	4083.52	2453.79
F-17	10.65	6.52	6.98	484.68	15.39	5.33	7459.18	2583.33
F-18	10.65	5.76	6.63	406.71	21.53	5.33	8756.48	2167.77
F-19	1.5	1.5	6.98	15.71	-0.75	-0.75	-11.78	-11.78
	1.5	1.5	6.98	15.71	21.53	-0.75	338.13	-11.78
F-20	5.61	1.5	6.98	58.74	2.8	-0.75	164.46	-44.05
F-21	6.52	1.5	6.98	68.26	8.87	-0.75	605.51	-51.20
	6.52	1.5	6.98	68.26	15.39	-0.75	1050.59	-51.20
F-22	5.76	1.5	6.98	60.31	21.53	-0.75	1298.41	-45.23
Total weigth due to slab=				**5129.27**	SUM =		**48285.15**	**75194.16**

Beam on axis	Length L(m)	Width W(m)	Depth d(m)	Weight W(KN)	X(m)	Y(m)	My(KNm)
1	16.43	0.5	0.4	78.864	6.1	34.4	481.07
2	16.43	0.5	0.4	78.864	6.1	27.8	481.07

3	27.41	0.5	0.4	131.568	12.2	18.63	1605.13
4	27.41	0.5	0.4	131.568	12.2	10.65	1605.13
5	27.41	0.5	0.4	131.568	12.2	0	1605.13
A	39.16	0.5	0.4	187.968	0	15.04	0.00
B	39.16	0.5	0.4	187.968	5.61	15.04	1054.50
C	39.16	0.5	0.4	187.968	12.13	15.04	2280.05
D	21.85	0.5	0.4	104.88	18.65	7.72	1956.01
E	21.85	0.5	0.4	104.88	24.41	7.72	2560.12
Total Weight due to beam=				**1194.528**	SUM=		**13628.21**

					Moment arm		Moment	
column on axis	depth(m)	Width W(m)	height h(m)	Weight W(KN)	X(m)	Y(m)	My(KNm)	Mx(KNm)
A1	0.4	0.5	3	14.4	0	34.44	0.00	495.94
A2	0.4	0.5	3	14.4	0	27.8	0.00	400.32
A3	0.4	0.5	3	14.4	0	18.63	0.00	268.27
A4	0.4	0.5	3	14.4	0	10.65	0.00	153.36
B1	0.4	0.5	3	14.4	5.61	34.44	80.78	495.94
B2	0.4	0.5	3	14.4	5.61	27.8	80.78	400.32
B3	0.4	0.5	3	14.4	5.61	18.63	80.78	268.27
B4	0.4	0.5	3	14.4	2.81	10.65	40.46	153.36
B5	0.4	0.5	3	14.4	5.61	0	80.78	0.00
C1	0.4	0.5	3	14.4	12.13	34.44	174.67	495.94
C2	0.4	0.5	3	14.4	12.13	27.8	174.67	400.32
C3	0.4	0.5	3	14.4	12.13	18.63	174.67	268.27
C4	0.4	0.5	3	14.4	12.13	10.65	174.67	153.36
C5	0.4	0.5	3	14.4	12.13	0	174.67	0.00
D3	0.4	0.5	3	14.4	18.65	18.63	268.56	268.27
D4	0.4	0.5	3	14.4	18.65	10.65	268.56	153.36
D5	0.4	0.5	3	14.4	18.65	0	268.56	0.00
E3	0.4	0.5	3	14.4	24.41	18.63	351.50	268.27
E4	0.4	0.5	3	14.4	24.41	10.65	351.50	153.36
E5	0.4	0.5	3	14.4	24.41	0	351.50	0.00

Total Weight due to foundation column=	=	259.2	SUM =	3097.15	4260.38

Staircase		Length L(m)	DL (KN/m)	Weight W(KN)	Moment arm		Moment
					X(m)	Y(m)	Mx(KNm)
Side staircase (right)	Landing	1.66	14.9	21.23	5.34	5.34	113.38
	Flight	6.72	18.2	120.22	5.34	5.34	641.98
	Landing	2.3	14.9	29.42	5.34	5.34	157.09
Total Weight due to staircase =				170.87	SUM =		**912.44**

Axis	Length L(m)	Width W(m)	height h(m)	Weight W(KN)	X(m)	Y(m)	My(KNm)
A	39.67	0.4	2.8	1066.33	-3.2	16.64	-3412.25
B	7.07	0.4	2.8	190.04	5.62	32.54	1068.03
B	8.45	0.4	2.8	227.14	8.92	22.98	2026.05
C	7.56	0.4	2.8	203.21	12.14	16.62	2467.00
C	8.67	0.4	2.8	233.05	14.84	30.12	3458.46
D	10.37	0.4	2.8	278.75	17.96	9.94	5006.27
E	22.03	0.4	2.8	592.17	24.2	9.42	14330.43
1	4.06	0.3	2.8	81.85	5.98	36.48	489.46
1	3.2	0.3	2.8	64.51	10.98	32.96	708.34
1	5.32	0.3	2.8	107.25	8.6	29.76	922.36
2	5.4	0.4	2.8	145.15	9.84	27.82	1428.30
2	10.71	0.4	2.8	287.88	2.88	23.28	829.11

3	3.7	0.4	2.8	99.46	19.66	19.78	1955.30
4	8.6	0.4	2.8	231.17	14.66	10.66	3388.92
4	8.6	0.4	2.8	231.17	23.26	5.06	5376.97
5	1.66	0.4	2.8	44.62	12.1	0	539.91
	Total weight due to wall			**3856.61**	SUM =		**40582.66**

Total weight of frist floor = **10610.47686**

MY = 139807.7599 Xm = 13.04948742

Mx = 138461.2843 Ym = 13.176388

Table 54

					Moment arm		Moment	
Beam on axis	Length L(m)	Width W(m)	Depth d(m)	Weight W(KN)	X(m)	Y(m)	My(KNm)	Mx(KNm)
1	16.43	0.5	0.4	82.15	6.1	34.4	501.12	2825.96
2	16.43	0.5	0.4	82.15	6.1	27.8	501.12	2283.77
3	27.41	0.5	0.4	137.05	12.2	18.63	1672.01	2553.24
4	27.41	0.5	0.4	137.05	12.2	10.65	1672.01	1459.58
5	27.41	0.5	0.4	137.05	12.2	0	1672.01	0.00
A	39.16	0.5	0.4	195.8	0	15.04	0.00	2944.83
B	39.16	0.5	0.4	195.8	5.61	15.04	1098.44	2944.83
C	39.16	0.5	0.4	195.8	12.13	15.04	2375.05	2944.83
D	21.85	0.5	0.4	109.25	18.65	7.72	2037.51	843.41
E	21.85	0.5	0.4	109.25	24.41	7.72	2666.79	843.41
Total Weight due to beam=				**1381.35**	SUM=		**14196.06**	**19643.87**

					Moment arm		Moment	
Panel	Length L(m)	Width W(m)	Dead load(KN/m2)	Weight W(KN)	X(m)	Y(m)	Mx(KNm)	My(KNm)
S1	1.5	2.89	6.63	28.74	-0.75	35.19	1011.40	-21.56
S2	5.61	1.5	6.63	55.79	2.8	35.19	1963.30	156.22
	6.52	1.5	6.63	64.84	8.87	35.19	2281.77	575.14
S3	2.8	1.5	6.63	27.85	13.53	35.19	979.90	376.76
S4	6.64	1.5	6.98	69.52	-0.75	31.12	2163.49	-52.14
	9.17	1.5	6.98	96.01	-0.75	23.22	2229.35	-72.01
	7.98	1.5	6.98	83.55	-0.75	14.64	1223.18	-62.66
	10.65	1.5	6.98	111.51	-0.73	5.33	594.32	-81.40
S5	6.64	5.61	6.63	246.97	2.8	31.12	7685.71	691.52
S6	6.64	6.52	6.59	285.30	8.87	31.12	8878.52	2530.61
S7	6.64	2.8	6.59	122.52	13.53	31.12	3812.86	1657.71
S8	9.17	5.61	6.63	341.07	2.8	23.22	7919.69	955.00
S9	9.17	6.52	6.63	396.40	8.87	23.22	9204.34	3516.04
S10	7.98	5.61	6.98	312.48	2.8	14.64	4574.70	874.94
S11	7.98	6.52	6.98	363.17	8.87	14.64	5316.76	3221.29
S12	7.98	1.66	6.63	87.83	15.39	14.64	1285.78	1351.65
S13	7.98	5.76			21.53	14.64	0.00	0.00
S14	7.98	1.5	6.63	79.36	25.16	14.64	1161.85	1996.73
	10.65	1.5	6.63	105.91	25.16	5.33	564.52	2664.80
S15	10.65	5.61	6.63	396.12	2.8	5.33	2111.32	1109.13
S16	10.65	6.52	6.63	460.37	8.87	5.33	2453.79	4083.52
S17	10.65	6.52	6.63	460.37	15.39	5.33	2453.79	7085.15

S18	10.65	5.76	6.63	406.71	21.53	5.33	2167.77	8756.48
S19	1.5	1.5	6.98	15.71	-0.75	-0.75	-11.78	-11.78
	1.5	1.5	6.98	15.71	21.53	-0.75	-11.78	338.13
S20	5.61	1.5	6.98	58.74	2.8	-0.75	-44.05	164.46
S21	6.52	1.5	6.98	68.26	8.87	-0.75	-51.20	605.51
	6.52	1.5	6.98	68.26	15.39	-0.75	-51.20	1050.59
S22	5.76	1.5	6.98	60.31	21.53	-0.75	-45.23	1298.41
	Total weight due to slab=			4889.37		Sum=	71822.87	44758.24

					Moment arm		Moment	
column on axis	depth(m)	Width W(m)	height h(m)	Weight W(KN)	X(m)	Y(m)	My(KNm)	Mx(KNm)
A1	0.4	0.5	3	15	0	34.44	198.49	0
A2	0.4	0.5	3	15	0	27.8	160.24	0
A3	0.4	0.5	3	15	0	18.63	107.37	0
A4	0.4	0.5	3	15	0	10.65	61.4	0
B1	0.4	0.5	3	15	5.61	34.44	198.49	32.37
B2	0.4	0.5	3	15	5.61	27.8	160.24	32.37
B3	0.4	0.5	3	15	5.61	18.63	107.37	32.37
B4	0.4	0.5	3	15	2.81	10.65	61.4	32.37
B5	0.4	0.5	3	15	5.61	0	0	32.37
C1	0.4	0.5	3	15	12.13	34.44	198.49	69.93
C2	0.4	0.5	3	15	12.13	27.8	160.24	69.93
C3	0.4	0.5	3	15	12.13	18.63	107.37	69.93
C4	0.4	0.5	3	15	12.13	10.65	61.4	69.93
C5	0.4	0.5	3	15	12.13	0	0	69.93

D3	0.4	0.5	3	15	18.65	18.63	107.37	106.33
D4	0.4	0.5	3	15	18.65	10.65	61.4	106.33
D5	0.4	0.5	3	15	18.65	0	0	106.33
E3	0.4	0.5	3	15	24.41	18.63	107.37	139.51
E4	0.4	0.5	3	15	24.41	10.65	61.4	139.51
E5	0.4	0.5	3	15	24.41	0	0	139.51
Total Weight dueto foundationcolumn=		t due to	column	=	**300**	SUM =	**1920.04**	**1249.02**

		Length L(m)	DL (KN/m)	Weight W(KN)	Moment arm		Moment	
staircase-					X(m)	Y(m)	Mx(KNm)	My(KNm)
Side staircase (right)	landing	1.66	14.9	21.2314	5.34	5.34	113.38	113.38
	flight	6.72	18.2	120.2208	5.34	5.34	641.98	641.98
	Landing	2.3	14.9	29.417	5.34	5.34	157.09	157.09
Total Weight due to staircase =				170.8692	SUM =		**912.44**	**912.44**

					Moment arm		Moment	
Axis	Length L(m)	Width W(m)	height h(m)	Weight W(KN)	X(m)	Y(m)	Mx(KNm)	My(KNm)
A'	36.91	0.4	2.8	578.7488	-3.2	16.64	9630.38	-1852.00
B	17.48	0.4	2.8	274.0864	5.62	16.2	4440.20	1540.37

B'	6.7	0.4	2.8	105.056	8.92	22.98	2414.19	937.10
C	14.08	0.4	2.8	220.7744	12.14	17.48	3859.14	2680.20
C'	13.64	0.4	2.8	213.8752	14.84	30.12	6441.92	3173.91
D	9.21	0.4	2.8	144.4128	17.96	8.76	1265.06	2593.65
E'	18.78	0.4	2.8	294.4704	24.2	9.42	2773.91	7126.18
1'	4.06	0.3	2.8	47.7456	5.98	36.48	1741.76	285.52
1"	3.2	0.3	2.8	37.632	10.98	32.96	1240.35	413.20
1"'	5.32	0.3	2.8	62.5632	8.6	28.76	1799.32	538.04
2	2.51	0.4	2.8	39.3568	9.84	27.82	1094.91	387.27
3	13.24	0.4	2.8	207.6032	12.4	19.34	4015.05	2574.28
4	10.95	0.4	2.8	171.696	9.04	10.66	1830.28	1552.13
4'	15	0.4	2.8	235.2	12.1	7.72	1815.74	2845.92
5'	9.05	0.4	2.8	141.904	12.1	-3.2	-454.09	1717.04
	Total weight due to wall=			**2775.1248**		Sum=	**43908.10**	**26512.82**

Total weight of second floor= 9516.72

MY = 93076.39 **Xm** = 13.95

Mx = 132759.51 **Ym** = 9.78

Table 55 thrid and forth floor determine the center of mass

					Moment arm		Moment	
panel	Length L(m)	Width W(m)	Dead load	Weight W(KN)	X(m)	Y(m)	Mx(KNm)	Mx(KNm)
T1	1.5	2.89	6.21	26.92	-0.75	35.19	947.33	-20.19
T2	5.61	1.5	6.63	55.79	2.8	35.19	1963.30	156.22
	6.52	1.5	6.63	64.84	8.87	35.19	2281.77	575.14
T3	2.8	1.5	6.63	27.85	13.53	35.19	979.90	376.76
T4	6.64	1.5	6.59	65.64	-0.75	31.12	2042.60	-49.23
	9.17	1.5	6.59	90.65	-0.75	23.22	2104.79	-67.98
	7.98	1.5	6.59	78.88	-0.75	14.64	1154.84	-59.16
	10.65	1.5	6.59	105.28	-0.73	5.33	561.12	-76.85
T5	6.64	5.61	6.21	231.32	2.8	31.12	7198.83	647.71
T6	6.64	6.52	6.63	287.03	8.87	31.12	8932.41	2545.97
T7	6.64	2.8	6.63	123.26	13.53	31.12	3836.01	1667.77

T8	9.17	5.61	6.21	319.47	2.8	23.22	7417.99	894.50
T9	9.17	6.52	6.98	417.32	8.87	23.22	9690.24	3701.66
T10	7.98	5.61	6.21	278.01	2.8	14.64	4070.04	778.42
T11	7.98	6.52	6.98	363.17	8.87	14.64	5316.76	3221.29
T12	7.98	1.66	6.98	92.46	15.39	14.64	1353.65	1423.00
T13	7.98	5.76	6.21	285.44	21.53	14.64	4178.86	6145.55
S14	7.98	1.5	6.63	79.36	25.16	14.64	1161.85	1996.73
	10.65	1.5	6.63	105.91	25.16	5.33	564.52	2664.80
T15	10.65	5.61	6.98	417.03	2.8	5.33	2222.77	1167.69
T16	10.65	6.52	6.98	484.68	8.87	5.33	2583.33	4299.09
T17	10.65	6.52	6.98	484.68	15.39	5.33	2583.33	7459.18
T18	10.65	5.76	6.98	428.18	21.53	5.33	2282.21	9218.74
T19	1.5	1.5	6.98	15.71	-0.75	-0.75	-11.78	-11.78
	1.5	1.5	6.98	15.71	21.53	-0.75	-11.78	338.13
T20	5.61	1.5	6.98	58.74	2.8	-0.75	-44.05	164.46
T21	6.52	1.5	6.98	68.26	8.87	-0.75	-51.20	605.51
	6.52	1.5	6.98	68.26	15.39	-0.75	-51.20	1050.59
T22	5.76	1.5	6.98	60.31	21.53	-0.75	-45.23	1298.41
Total weigth due to slab=				**5200.151155**	SUM =		**75213.21**	**52112.12**

					Moment arm		Moment	
Beam on axis	Length L(m)	Width W(m)	Depth d(m)	Weight W(KN)	X(m)	Y(m)	My(KNm)	Mx(KNm)

1	16.43	0.5	0.4	82.15	6.1	34.4	501.12	2825.96
2	16.43	0.5	0.4	82.15	6.1	27.8	501.12	2283.77
3	27.41	0.5	0.4	137.05	12.2	18.63	1672.01	2553.24
4	27.41	0.5	0.4	137.05	12.2	10.65	1672.01	1459.58
5	27.41	0.5	0.4	137.05	12.2	0	1672.01	0.00
A	39.16	0.5	0.4	195.8	0	15.04	0.00	2944.83
B	39.16	0.5	0.4	195.8	5.61	15.04	1098.44	2944.83
C	39.16	0.5	0.4	195.8	12.13	15.04	2375.05	2944.83
D	21.85	0.5	0.4	109.25	18.65	7.72	2037.51	843.41
E	21.85	0.5	0.4	109.25	24.41	3.86	2666.79	421.71
Total Weight due to beam=				**1381.35**	SUM=		**14196.06**	**19222.17**

column on axis	depth(m)	Width W(m)	height h(m)	Weight W(KN)	X(m)	Y(m)	My(KNm)	Mx(KNm)
A1	0.4	0.5	3	15	0	34.44	0	516.6
A2	0.4	0.5	3	15	0	27.8	0	417
A3	0.4	0.5	3	15	0	18.63	0	279.45
A4	0.4	0.5	3	15	0	10.65	0	159.75
B1	0.4	0.5	3	15	5.61	34.44	84.15	516.6
B2	0.4	0.5	3	15	5.61	27.8	84.15	417
B3	0.4	0.5	3	15	5.61	18.63	84.15	279.45
B4	0.4	0.5	3	15	2.81	10.65	42.15	159.75
B5	0.4	0.5	3	15	5.61	0	84.15	0
C1	0.4	0.5	3	15	12.13	34.44	181.95	516.6
C2	0.4	0.5	3	15	12.13	27.8	181.95	417
C3	0.4	0.5	3	15	12.13	18.63	181.95	279.45
C4	0.4	0.5	3	15	12.13	10.65	181.95	159.75
C5	0.4	0.5	3	15	12.13	0	181.95	0
D3	0.4	0.5	3	15	18.65	18.63	279.75	279.45
D4	0.4	0.5	3	15	18.65	10.65	279.75	159.75
D5	0.4	0.5	3	15	18.65	0	279.75	0
E3	0.4	0.5	3	15	24.41	18.63	366.15	279.45
E4	0.4	0.5	3	15	24.41	10.65	366.15	159.75
E5	0.4	0.5	3	15	24.41	0	366.15	0
Total Weight due to foundation column=	t due to column		=	**300**		SUM =	**3226.2**	**4996.8**

Staircase		Length L(m)	DL (KN/m)	Weight W(KN)	Moment arm X(m)	Moment arm Y(m)	Moment Mx(KNm)	Moment My(KNm)
Side staircase (right)	Landing	1.66	14.9	21.23	5.34	5.34	113.38	113.38
	Flight	6.72	18.2	120.22	5.34	5.34	641.98	641.98
	Landing	2.3	14.9	29.42	5.34	5.34	157.09	157.09
Total Weight due to staircase =				170.87	SUM =		**912.44**	**912.44**

Wall					Moment arm		Moment	
AXIS	Length L(m)	Thicknesst(m)	height h(m)	Weight W(KN)	X(m)	Y(m)	Mx(KNm)	My(KNm)
A	4.74	0.4	2.8	74.32	2.7	23.56	1751.05	200.67
	36.91	0.4	2.8	578.75	1.5	3.95	2286.06	868.12
	2.35	0.4	2.8	36.85	1.34	8.84	325.74	49.38
B	13.93	0.3	2.8	163.82	5.88	17.76	2909.39	963.24
	12.58	0.3	2.8	147.94	9	15.57	2303.44	1331.47
C	12.55	0.4	2.8	196.78	24.26	23.24	4573.26	4773.98
	8.67	0.3	2.8	101.96	28.26	16.12	1643.58	2881.37
	8.36	0.3	2.8	98.31	24.26	7.3	717.69	2385.09
D	9.05	0.3	2.8	106.43	36.92	3.88	412.94	3929.32
	2.85	0.3	2.8	33.52	34.76	-1.68	-56.31	1165.02
	2.17	0.4	2.8	34.03	46.28	14.66	498.82	1574.70

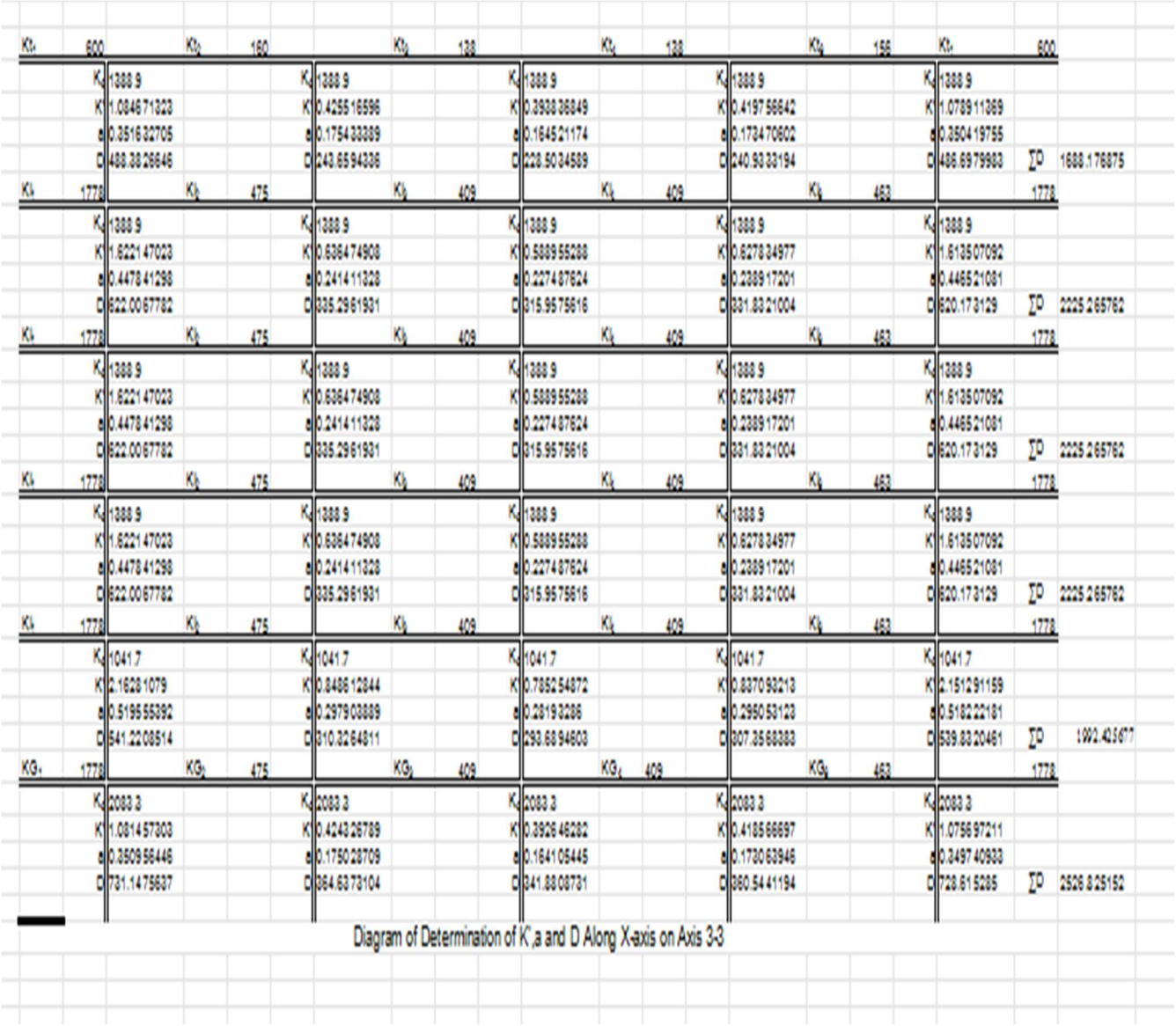
E	16.82	0.4	2.8	263.74	53.84	14.66	3866.39	14199.63
Axis-5	6.62	0.3	2.8	77.85	3.16	-0.2	-15.57	246.01
	22.46	0.3	2.8	264.13	2.76	-0.2	-52.83	729.00
Axis-4	22.46	0.4	2.8	352.17	2.48	10.46	3683.73	873.39
	7.5	0.4	2.8	117.60	5.6	10.46	1230.10	658.56
Axis-3	13.71	0.3	2.8	161.23	2.28	14.4	2321.71	367.60
	3.75	0.3	2.8	44.10	5.64	18.9	833.49	248.72
Axis-2	14.41	0.3	2.8	169.46	2.28	27.56	4670.36	386.37
	6.83	0.3	2..8	5.74	5.64	27.56	158.19	32.37
Axis-1	4.06	0.4	2.8	63.66	12.16	36.34	2313.43	774.12
Total weight due to wall =				**3092.39**	SUM =		**36374.66**	**52837.77**

Total weight of third floor= 10144.76

$$= \frac{\mathbf{MY}}{\mathbf{130081.2957}} = \frac{\mathbf{Xm}}{\mathbf{12.80686735}}$$

$$= \frac{\mathbf{Mx}}{\mathbf{129922.5643}} = \frac{\mathbf{Ym}}{\mathbf{12.82251399}}$$

Table 56 D- values on each axise



K_1	600		K_2	160		K_3	138		K_4	138		K_5	156	K_6	600		
	K_d	1388.9		K_d	1388.9		K_d	418.87		K_d	1388.9		K_d	1388.9			
	K	1.084671323		K	0.425516596		K	1.30589443		K	0.419756642		K	1.078911369			
	a	0.351632705		a	0.175433389		a	0.395020004		a	0.173470602		a	0.350419755			
	D	488.3826646		D	243.6594336		D	165.462029		D	240.9333194		D	486.6979983	∑D	1625.135445	
K_1	1778		K_2	475		K_3	409		K_4	409		K_5	463		1778		
	K_d	1388.9		K_d	1388.9		K_d	418.87		K_d	1388.9		K_d	1388.9			
	K	1.622147023		K	0.636474908		K	1.952873206		K	0.627834977		K	1.613507092			
	a	0.447841298		a	0.241411328		a	0.494038919		a	0.238917201		a	0.446521081			
	D	622.0067782		D	335.2961931		D	206.9380821		D	331.8321004		D	620.173129	∑D	2116.246283	
K_1	1778		K_2	475		K_3	409		K_4	409		K_5	463		1778		
	K_d	1388.9		K_d	1388.9		K_d	418.87		K_d	1388.9		K_d	1388.9			
	K	1.622147023		K	0.636474908		K	1.952873206		K	0.627834977		K	1.613507092			
	a	0.447841298		a	0.241411328		a	0.494038919		a	0.238917201		a	0.446521081			
	D	622.0067782		D	335.2961931		D	206.9380821		D	331.8321004		D	620.173129	∑D	2116.246283	
K_1	1778		K_2	475		K_3	409		K_4	409		K_5	463		1778		
	K_d	1388.9		K_d	1388.9		K_d	418.87		K_d	1388.9		K_d	1388.9			
	K	1.622147023		K	0.636474908		K	1.952873206		K	0.627834977		K	1.613507092			
	a	0.447841298		a	0.241411328		a	0.494038919		a	0.238917201		a	0.446521081			
	D	622.0067782		D	335.2961931		D	206.9380821		D	331.8321004		D	620.173129	∑D	2116.246283	
K_1	1778		K_2	475		K_3	409		K_4	409		K_5	463		1778		
	K_d	1041.7		K_d	1041.7		K_d	314.16		K_d	1041.7		K_d	1041.7			
	K	2.16281079		K	0.848612844		K	2.60376878		K	0.837083213		K	2.151291159			
	a	0.519555392		a	0.297903889		a	0.565573317		a	0.295053123		a	0.518222181			
	D	541.2208514		D	310.3264811		D	177.6805133		D	307.3568383		D	539.8320461	∑D	1876.41673	
KG_1	1778		KG_2	475		KG_3	409		KG_4	409		KG_5	463		1778		
	K_d	2083.3		K_d	2083.3		K_d	628.32		K_d	2083.3		K_d	2083.3			
	K	1.081457303		K	0.424326789		K	1.30188439		K	0.418566697		K	1.075687211			
	a	0.350956446		a	0.175028709		a	0.394285274		a	0.173063946		a	0.349740933			
	D	731.1475637		D	364.6373104		D	247.7373231		D	360.5441194		D	728.615285	∑D	2432.681602	

Diagram of Determination of K',a and D Along X-axis on Axis4-4

Kl_1	600		Kl_2	160		Kl_3	138		Kl_4	138		Kl_5	156	Kl_6	600		
	K_d	418.87		K_d	1388.9		K_d	1388.9		K_d	1388.9		K_d	418.87			
	K	3.596581278		K	0.425516596		K	0.393836849		K	0.419756642		K	3.577482274			
	a	0.642638979		a	0.175433389		a	0.164521174		a	0.173470602		a	0.641415266			
	D	269.1821891		D	243.6594336		D	228.5034589		D	240.9333194		D	268.6696123	∑D	1250.948013	
Kl_1	1778		Kl_2	475		Kl_3	409		Kl_4	409		Kl_6	463		1778		
	K_d	418.87		K_d	1388.9		K_d	1388.9		K_d	1388.9		K_d	418.87			
	K	5.378757132		K	0.636474908		K	0.588955288		K	0.627834977		K	5.350108626			
	a	0.728951643		a	0.241411328		a	0.227487624		a	0.238917201		a	0.727895178			
	D	305.3359746		D	335.2961931		D	315.9575616		D	331.8321004		D	304.8934532	∑D	1593.315283	
Kl_1	1778		Kl_2	475		Kl_3	409		Kl_4	409		Kl_6	463		1778		
	K_d	418.87		K_d	1388.9		K_d	1388.9		K_d	1388.9		K_d	418.87			
	K	5.378757132		K	0.636474908		K	0.588955288		K	0.627834977		K	5.350108626			
	a	0.728951643		a	0.241411328		a	0.227487624		a	0.238917201		a	0.727895178			
	D	305.3359746		D	335.2961931		D	315.9575616		D	331.8321004		D	304.8934532	∑D	1593.315283	
Kl_1	1778		Kl_2	475		Kl_3	409		Kl_4	409		Kl_6	463		1778		
	K_d	418.87		K_d	1388.9		K_d	1388.9		K_d	1388.9		K_d	418.87			
	K	5.378757132		K	0.636474908		K	0.588955288		K	0.627834977		K	5.350108626			
	a	0.728951643		a	0.241411328		a	0.227487624		a	0.238917201		a	0.727895178			
	D	305.3359746		D	335.2961931		D	315.9575616		D	331.8321004		D	304.8934532	∑D	1593.315283	
Kl_1	1778		Kl_2	475		Kl_3	409		Kl_4	409		Kl_6	463		1778		
	K_d	314.16		K_d	1041.7		K_d	1041.7		K_d	1041.7		K_d	314.16			
	K	7.171504966		K	0.848612844		K	0.785254872		K	0.837093213		K	7.133307869			
	a	0.781933281		a	0.297903889		a	0.28193286		a	0.295053123		a	0.781021287			
	D	245.6521594		D	310.3264811		D	293.6894603		D	307.3568383		D	245.3656476	∑D	1402.390387	
KG_1	1778		KG_2	475		KG_3	409		KG_4	409		KG_6	463		1778		
	K_d	628.32		K_d	2083.3		K_d	2083.3		K_d	2083.3		K_d	628.32			
	K	3.585752483		K	0.424326789		K	0.392646282		K	0.418566697		K	3.566653934			
	a	0.641946183		a	0.175028709		a	0.164105445		a	0.173063946		a	0.640717741			
	D	403.3476254		D	364.6373104		D	341.8808731		D	360.5441194		D	402.5757711	∑D	1872.985699	

Diagram of Determination of K',a and D Along X-axis on Axis 5-5

Kt_{12}	600		Kt_{11}	85		Kt_{10}	113		Kt_9	309		Kt_8	136	Kt_7	600		
	K_d	1388.9		K_d	418.87		K_d	1388.9		K_d	1388.9		K_d	1388.9			
	K	0.976672187		K	0.933463843		K	0.376916985		K	0.409676723		K	1.049751602			
	a	0.328108749		a	0.318212152		a	0.158573895		a	0.170013147		a	0.344208886			
	D	455.7102412		D	133.2885242		D	220.2492829		D	236.1312597		D	478.071722	∑D	1523.44603	
Kl_{12}	1778		Kl_{11}	250		Kl_{10}	334		Kl_9	291		Kl_8	402		1778		
	K_d	1388.9		K_d	418.87		K_d	1388.9		K_d	1388.9		K_d	1388.9			
	K	1.460148319		K	1.394227326		K	0.4499964		K	0.498956008		K	1.569587443			
	a	0.421990095		a	0.410764275		a	0.18367227		a	0.199665783		a	0.439711162			
	D	586.1020434		D	172.0568318		D	255.1024157		D	277.3158062		D	610.7148332	∑D	1901.29193	
Kl_{12}	1778		Kl_{11}	250		Kl_{10}	334		Kl_9	291		Kl_8	402		1778		
	K_d	1388.9		K_d	418.87		K_d	1388.9		K_d	1388.9		K_d	1388.9			
	K	1.460148319		K	1.394227326		K	0.4499964		K	0.498956008		K	1.569587443			
	a	0.421990095		a	0.410764275		a	0.18367227		a	0.199665783		a	0.439711162			
	D	586.1020434		D	172.0568318		D	255.1024157		D	277.3158062		D	610.7148332	∑D	1901.29193	
Kl_{12}	1778		Kl_{11}	250		Kl_{10}	334		Kl_9	291		Kl_8	402		1778		
	K_d	1388.9		K_d	418.87		K_d	1388.9		K_d	1388.9		K_d	1388.9			
	K	1.460148319		K	1.394227326		K	0.4499964		K	0.498956008		K	1.569587443			
	a	0.421990095		a	0.410764275		a	0.18367227		a	0.199665783		a	0.439711162			
	D	586.1020434		D	172.0568318		D	255.1024157		D	277.3158062		D	610.7148332	∑D	1901.29193	
Kl_{12}	1778		Kl_{11}	250		Kl_{10}	334		Kl_9	291		Kl_8	402		1778		
	K_d	1041.7		K_d	314.16		K_d	1041.7		K_d	1041.7		K_d	1041.7			
	K	1.946817702		K	1.858925388		K	0.599980801		K	0.665258712		K	2.092733033			
	a	0.493262636		a	0.481720998		a	0.23076355		a	0.249603803		a	0.511328986			
	D	513.8316875		D	151.3374687		D	240.3863905		D	260.0122821		D	532.651405	∑D	1698.219234	
KG_{12}	1778		KG_{11}	250		KG_{10}	334		KG_9	291		KG_8	803		2640		
	K_d	2083.3		K_d	628.32		K_d	2083.3		K_d	2083.3		K_d	2083.3			
	K	0.973455575		K	0.929462694		K	0.3000048		K	0.332645322		K	1.046416743			
	a	0.327381913		a	0.317280946		a	0.130436597		a	0.14260433		a	0.343491003			
	D	682.0347399		D	199.3539638		D	271.7385633		D	297.0875998		D	715.5948067	∑D	2165.809674	

Diagram of Determination of K',a and D Along Y-axis on Axis C-C

Kt_{12}	600		Kt_{11}	85		Kt_{10}	113			
	K_c	1388.9		K_c	1388.9		K_c	1388.9		
	K'	0.976672187		K'	0.281517748		K'	0.160918713		
	a	0.328108749		a	0.123390558		a	0.074467731		
	D	455.7102412		D	171.3771459		D	103.4282311	∑D	730.5156182
Kk_2	1778		Kk_1	250		Kk_0	334			
	K_c	1388.9		K_c	1388.9		K_c	1388.9		
	K'	1.460148319		K'	0.420476636		K'	0.240478076		
	a	0.421990095		a	0.173716451		a	0.107333376		
	D	586.1020434		D	241.2747933		D	149.0753262	∑D	976.4521628
Kk_2	1778		Kk_1	250		Kk_0	334			
	K_c	1388.9		K_c	1388.9		K_c	1388.9		
	K'	1.460148319		K'	0.420476636		K'	0.240478076		
	a	0.421990095		a	0.173716451		a	0.107333376		
	D	586.1020434		D	241.2747933		D	149.0753262	∑D	976.4521628
Kk_2	1778		Kk_1	250		Kk_0	334			
	K_c	1388.9		K_c	1388.9		K_c	1388.9		
	K'	1.460148319		K'	0.420476636		K'	0.240478076		
	a	0.421990095		a	0.173716451		a	0.107333376		
	D	586.1020434		D	241.2747933		D	149.0753262	∑D	976.4521628
Kk_2	1778		Kk_1	250		Kk_0	334			
	K_c	1041.7		K_c	1041.7		K_c	1041.7		
	K'	1.946817702		K'	0.56062205		K'	0.32062974		
	a	0.493262636		a	0.218939792		a	0.138164971		
	D	513.8316875		D	228.0695809		D	143.9264499	∑D	885.8277183
KG_{12}	1778		KG_{11}	250		KG_{10}	334			
	K_c	2083.3		K_c	2083.3		K_c	2083.3		
	K'	0.973455575		K'	0.280324485		K'	0.160322565		
	a	0.327381913		a	0.12293184		a	0.074212327		
	D	682.0347399		D	256.1039027		D	154.6065414	∑D	1092.745184

Diagram of Determination of K',a and D Along Y-axis on Axis D-D

Table 57 Correction factors *in x -and y- axise*

story	Axis	Dx	y'	Dx*y'2	Jx	Jy	ΣDx	ey1	ey2	αX1	αX2	αXd
	5	1872.99	-18.63	650070.1746	1285227.977	538420.3618	9707.819	3.86	0.115	0.62	0.99	0.99
	4	2432.68	-7.98	154914.1628	1285227.977	538420.3618	9707.819	3.86	0.115	0.84	1.00	1.00
	3	2526.83	0	0	1285227.977	538420.3618	9707.819	3.86	0.115	1.00	1.00	1.00
	2	1437.67	9.17	120891.7525	1285227.977	538420.3618	9707.819	3.86	0.115	1.19	1.01	1.19
	1	1437.66	15.81	359351.8867	1285227.977	538420.3618	9707.819	3.86	0.115	1.32	1.01	1.32
1st	ΣDx	9707.82	Jx	1285227.977				3.86	0.115			
	5	1402.39	-18.63	486737.1738	988789.8547	420968.3418	7561.713	3.61	-0.13	0.64	1.01	1.01
	4	1876.42	-7.98	119490.9851	988789.8547	420968.3418	7561.713	3.61	-0.13	0.85	1.01	1.01
	3	1992.43	0	0	988789.8547	420968.3418	7561.713	3.61	-0.13	1.00	1.00	1.00
	2	1145.24	9.17	96301.97184	988789.8547	420968.3418	7561.713	3.61	-0.13	1.18	0.99	1.18
	1	1145.24	15.81	286259.724	988789.8547	420968.3418	7561.713	3.61	-0.13	1.31	0.99	1.31
2nd	ΣDx	7561.71	Jx	988789.8547				3.61	-0.13			
	5	1593.31	-18.63	553001.0955	1113090.624	470299.2859	8481.3426	3.70	-0.04	0.63	1.00	1.00
	4	2116.25	-7.98	134763.3918	1113090.624	470299.2859	8481.3426	3.70	-0.04	0.84	1.00	1.00
	3	2225.27	0	0	1113090.624	470299.2859	8481.3426	3.70	-0.04	1.00	1.00	1.00
	2	1273.26	9.17	107067.0328	1113090.624	470299.2859	8481.3426	3.70	-0.04	1.18	1.00	1.18
	1	1273.26	15.81	318259.1039	1113090.624	470299.2859	8481.3426	3.70	-0.04	1.31	1.00	1.31
3rd	ΣDx	8481.34	Jx	1113090.624				3.70	-0.04			
	5	1593.31	-18.63	553001.0955	1113090.624	470299.2859	8481.3426	3.70	-0.04	0.63	1.00	1.00

	4	2116.25	-7.98	134763.3918	1113090.624	470299.2859	8481.3426	3.70	-0.04	0.84	1.00	1.00
	3	2225.27	0	0	1113090.624	470299.2859	8481.3426	3.70	-0.04	1.00	1.00	1.00
	2	1273.26	9.17	107067.0328	1113090.624	470299.2859	8481.3426	3.70	-0.04	1.18	1.00	1.18
	1	1273.26	15.81	318259.1039	1113090.624	470299.2859	8481.3426	3.70	-0.04	1.31	1.00	1.31
4th	ΣDx	8481.34	Jx	1113090.624				3.70	-0.04			
	5	1593.31	-18.63	553001.0955	1113090.624	470299.2859	8481.3426	3.70	-0.04	0.63	1.00	1.00
	4	2116.25	-7.98	134763.3918	1113090.624	470299.2859	8481.3426	3.70	-0.04	0.84	1.00	1.00
	3	2225.27	0	0	1113090.624	470299.2859	8481.3426	3.70	-0.04	1.00	1.00	1.00
	2	1273.26	9.17	107067.0328	1113090.624	470299.2859	8481.3426	3.70	-0.04	1.18	1.00	1.18
	1	1273.26	15.81	318259.1039	1113090.624	470299.2859	8481.3426	3.70	-0.04	1.31	1.00	1.31
5th	ΣDx	8481.34	Jx	1113090.624				3.70	-0.04			
	5	1250.95	-18.63	434175.8481	858530.6835	374256.7599	6485.357	3.85	0.114	0.62	0.99	0.99
	4	1625.14	-7.98	103489.2469	858530.6835	374256.7599	6485.357	3.85	0.114	0.84	1.00	1.00
	3	1688.18	0	0	858530.6835	374256.7599	6485.357	3.85	0.114	1.00	1.00	1.00
	2	960.55	9.17	80771.25654	858530.6835	374256.7599	6485.357	3.85	0.114	1.19	1.01	1.19
	1	960.55	15.81	240094.332	858530.6835	374256.7599	6485.357	3.85	0.114	1.32	1.01	1.32
6th	ΣDx	6485.36	Jx	858530.6835				3.85	0.114			

Story	Axis	Dy	x'	Dy*x'2	Jy	Jx	Σdy	ex1	ex2	αy1	αy2	αyd
	A	1928.46	-12.13	283747.92	538420.36	1285227.98	8208.22	-2.75	-5.50	1.15	1.30	1.30
	B	2222.56	-6.52	94481.90	538420.36	1285227.98	8208.22	-2.75	-5.50	1.08	1.16	1.16
	C	2165.81	0.00	0.00	538420.36	1285227.98	8208.22	-2.75	-5.50	1.00	1.00	1.00
	D	1092.75	6.32	43646.86	538420.36	1285227.98	8208.22	-2.75	-5.50	0.92	0.84	0.92
	E	798.65	12.08	116543.68	538420.36	1285227.98	8208.22	-2.75	-5.50	0.85	0.70	0.85
1st	ΣDY	8208.22	Jy	538420.36				-2.75	-5.50			
	A	1500.97	-12.13	220848.07	420968.34	988789.85	6471.81	-2.75	-5.49	1.15	1.31	1.31
	B	1774.95	-6.52	75453.83	420968.34	988789.85	6471.81	-2.75	-5.49	1.08	1.16	1.16
	C	1698.22	0.00	0.00	420968.34	988789.85	6471.81	-2.75	-5.49	1.00	1.00	1.00
	D	885.83	6.32	35382.10	420968.34	988789.85	6471.81	-2.75	-5.49	0.92	0.84	0.92
	E	611.85	12.08	89284.34	420968.34	988789.85	6471.81	-2.75	-5.49	0.85	0.70	0.85
2nd	ΣDY	6471.81	Jy	420968.34				-2.75	-5.49			
	A	1680.83	-12.13	247312.12	470299.29	1113090.62	7215.86	-2.75	-5.50	1.15	1.30	1.30
	B	1970.51	-6.52	83767.17	470299.29	1113090.62	7215.86	-2.75	-5.50	1.08	1.16	1.16
	C	1901.29	0.00	0.00	470299.29	1113090.62	7215.86	-2.75	-5.50	1.00	1.00	1.00
	D	976.45	6.32	39001.84	470299.29	1113090.62	7215.86	-2.75	-5.50	0.92	0.84	0.92
	E	686.77	12.08	100218.17	470299.29	1113090.62	7215.86	-2.75	-5.50	0.85	0.70	0.85
3rd	ΣDY	7215.86	Jy	470299.29				-2.75	-5.50			
	A	1680.83	-12.13	247312.12	470299.29	1113090.62	7215.86	-2.75	-5.50	1.15	1.30	1.30
	B	1970.51	-6.52	83767.17	470299.29	1113090.62	7215.86	-2.75	-5.50	1.08	1.16	1.16
	C	1901.29	0.00	0.00	470299.29	1113090.62	7215.86	-2.75	-5.50	1.00	1.00	1.00
	D	976.45	6.32	39001.84	470299.29	1113090.62	7215.86	-2.75	-5.50	0.92	0.84	0.92

	E	686.77	12.08	100218.17	470299.29	1113090.62	7215.86	-2.75	-5.50	0.85	0.70	0.85
4th	ΣDY	7215.86	Jy	470299.29				-2.75	-5.50			
	A	1680.83	-12.13	247312.12	470299.29	1113090.62	7215.86	-2.75	-5.50	1.15	1.30	1.30
	B	1970.51	-6.52	83767.17	470299.29	1113090.62	7215.86	-2.75	-5.50	1.08	1.16	1.16
	C	1901.29	0.00	0.00	470299.29	1113090.62	7215.86	-2.75	-5.50	1.00	1.00	1.00
	D	976.45	6.32	39001.84	470299.29	1113090.62	7215.86	-2.75	-5.50	0.92	0.84	0.92
	E	686.77	12.08	100218.17	470299.29	1113090.62	7215.86	-2.75	-5.50	0.85	0.70	0.85
5th	ΣDY	7215.86	Jy	470299.29				-2.75	-5.50			
	A	1364.77	-12.13	200808.03	374256.76	858530.68	5714.02	-2.91	-5.65	1.16	1.32	1.32
	B	1561.53	-6.52	66381.26	374256.76	858530.68	5714.02	-2.91	-5.65	1.09	1.17	1.17
	C	1523.45	0.00	0.00	374256.76	858530.68	5714.02	-2.91	-5.65	1.00	1.00	1.00
	D	730.52	6.32	29178.52	374256.76	858530.68	5714.02	-2.91	-5.65	0.91	0.83	0.91
	E	533.76	12.08	77888.95	374256.76	858530.68	5714.02	-2.91	-5.65	0.84	0.68	0.84
6th	ΣDY	5714.02	Jy	374256.76				-2.91	-5.65			

Appendix D; Frame Analysis

ETABS 2016 OUTPUT ANALYSIS

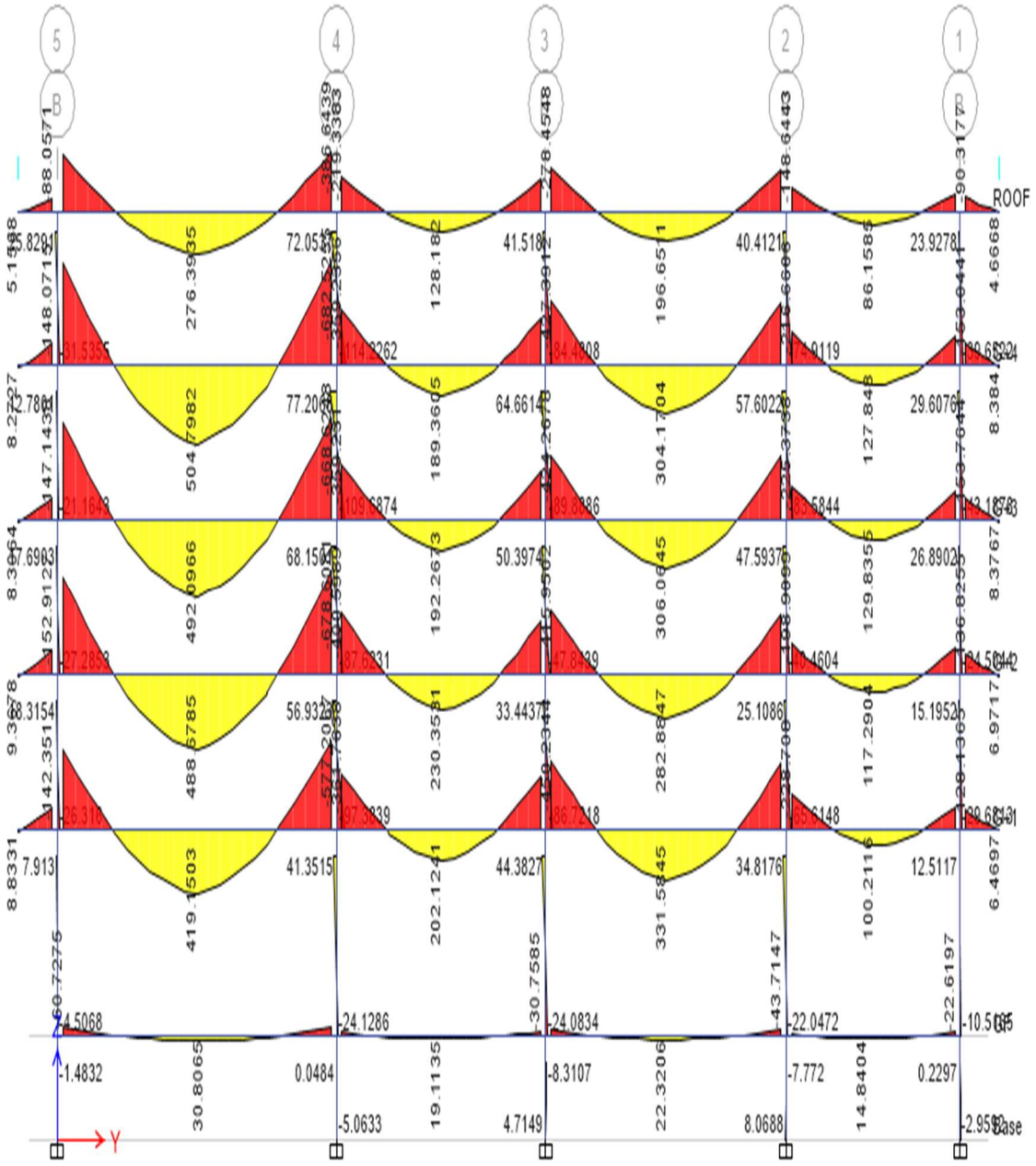


Figure 18moment @ axis B-B

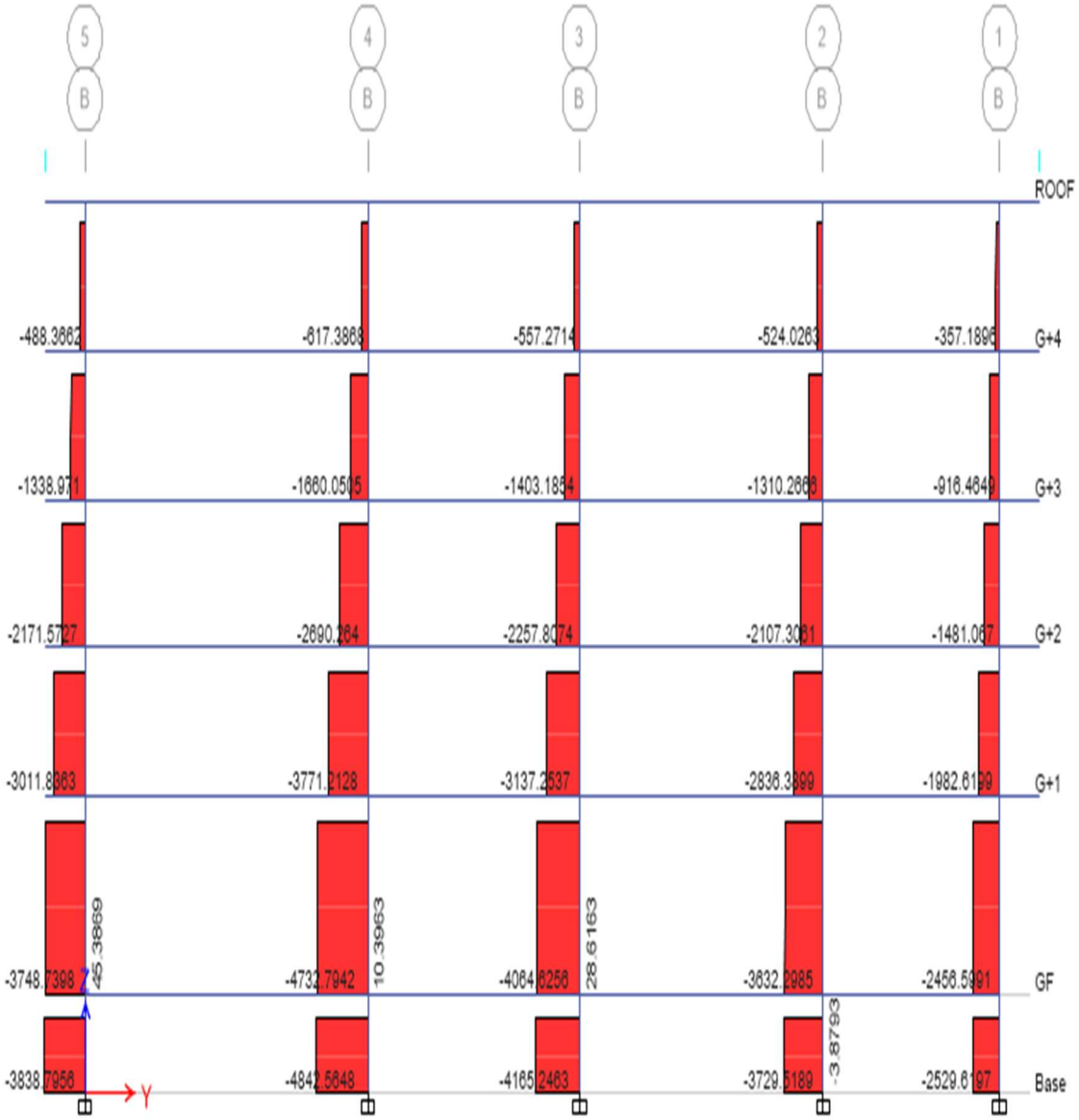


Figure 19Axial Force @ axis B-B

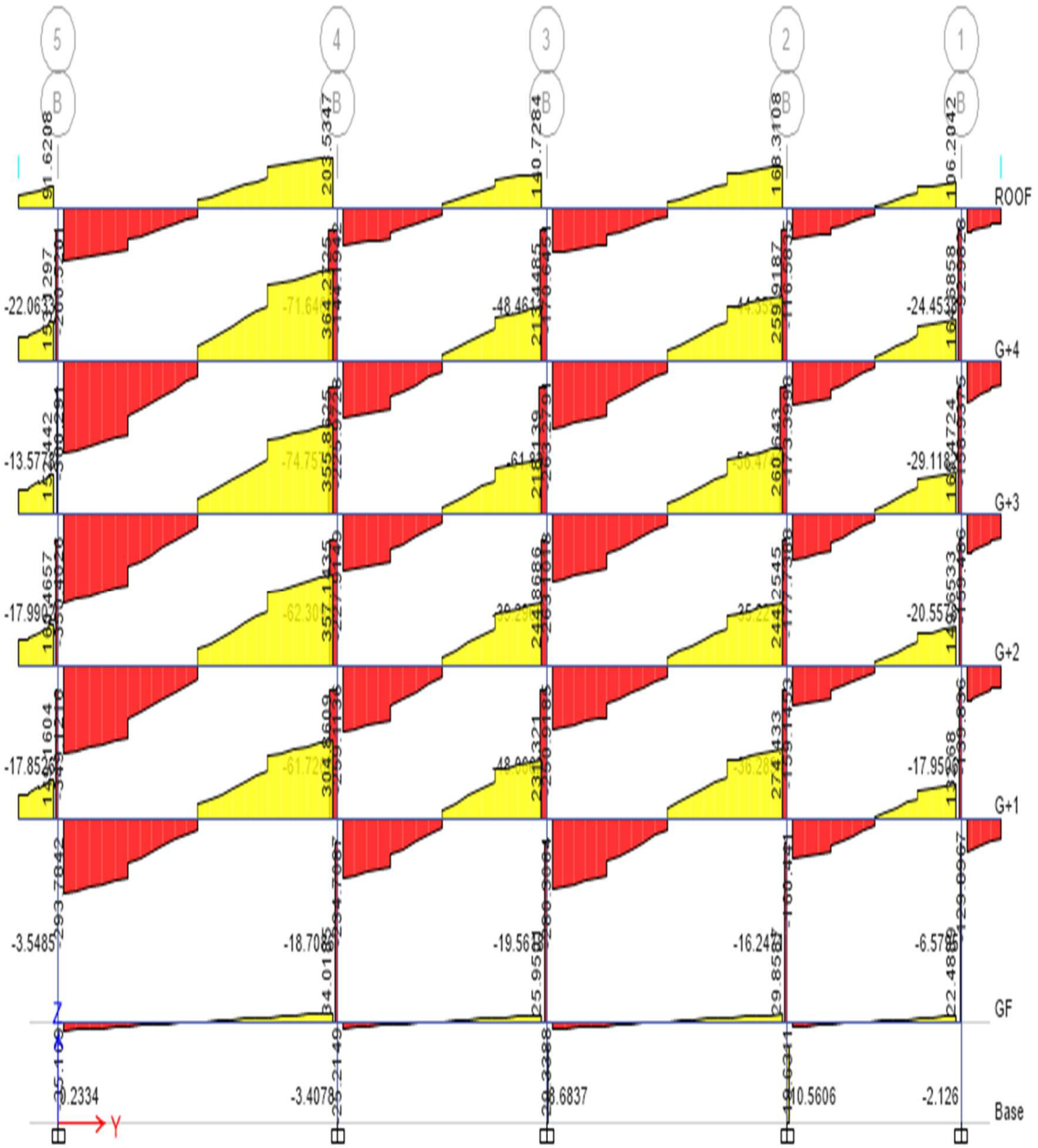


Figure 20 Shear force @ axis B-B



Figure 21Shear force @ axis 1-1

Moment @ axis 2-2

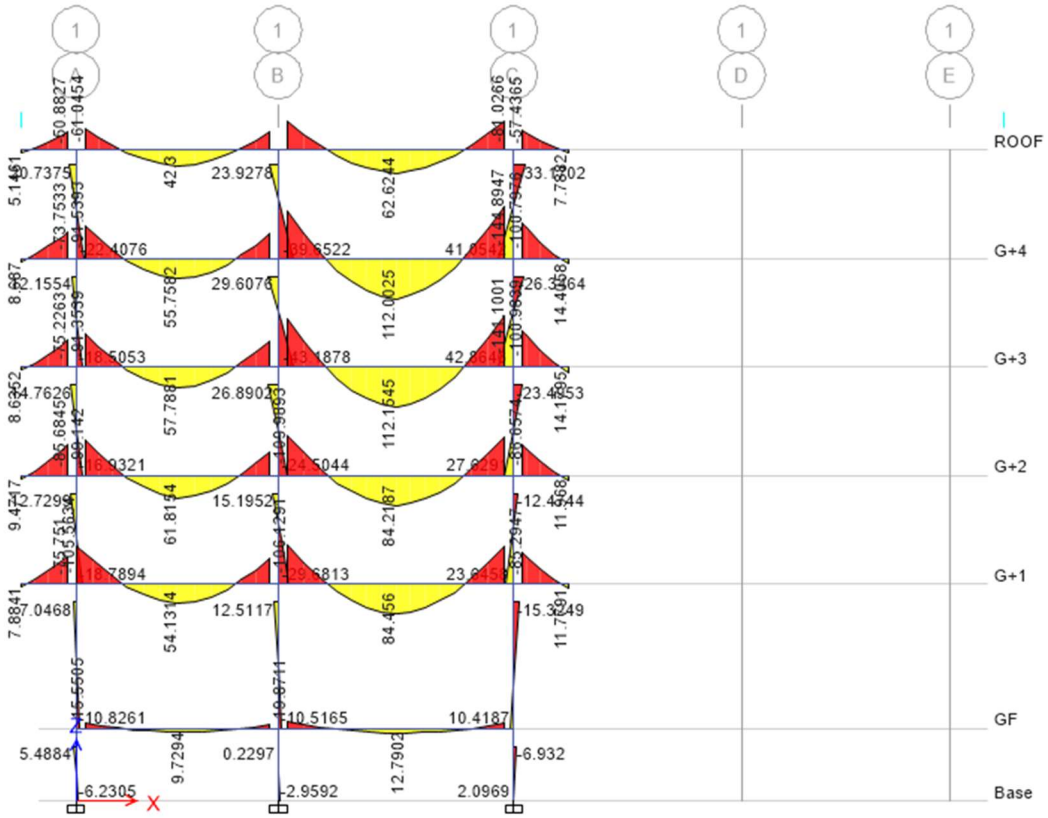


Figure 22 Moment @ axis 1-1

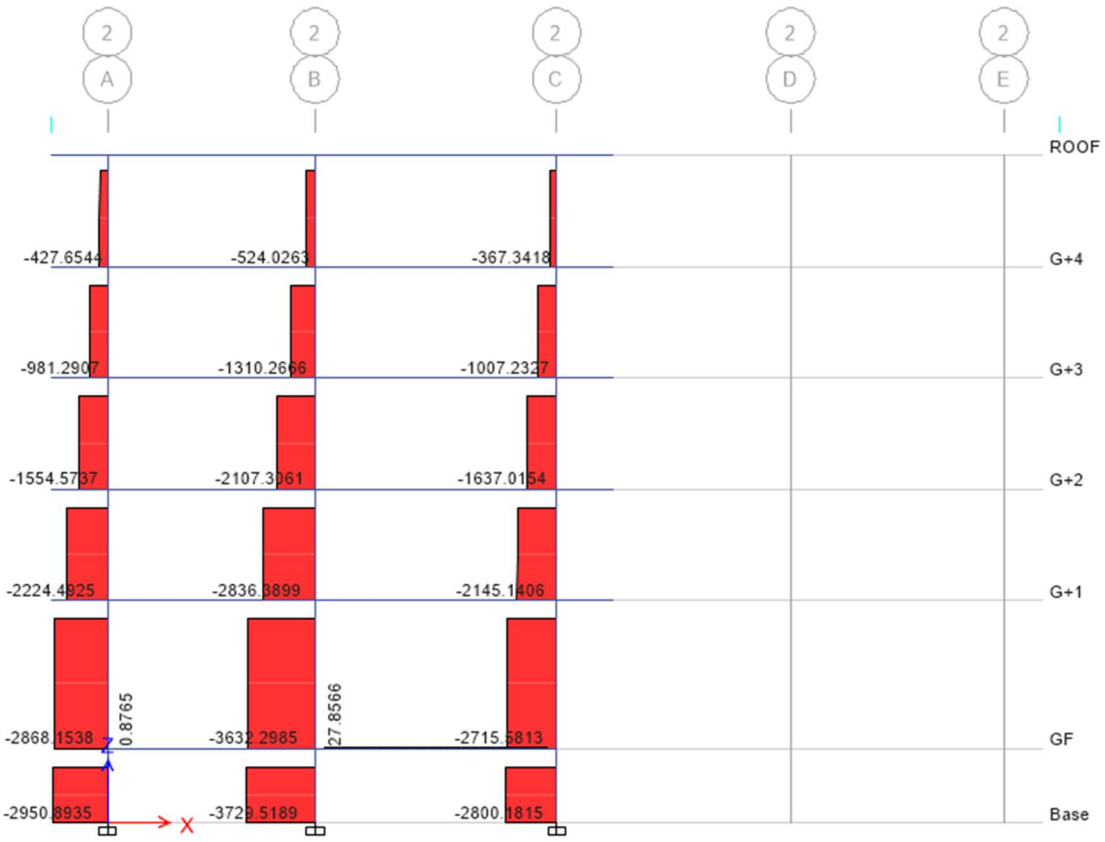


Figure 23 Axial force @ 2-2

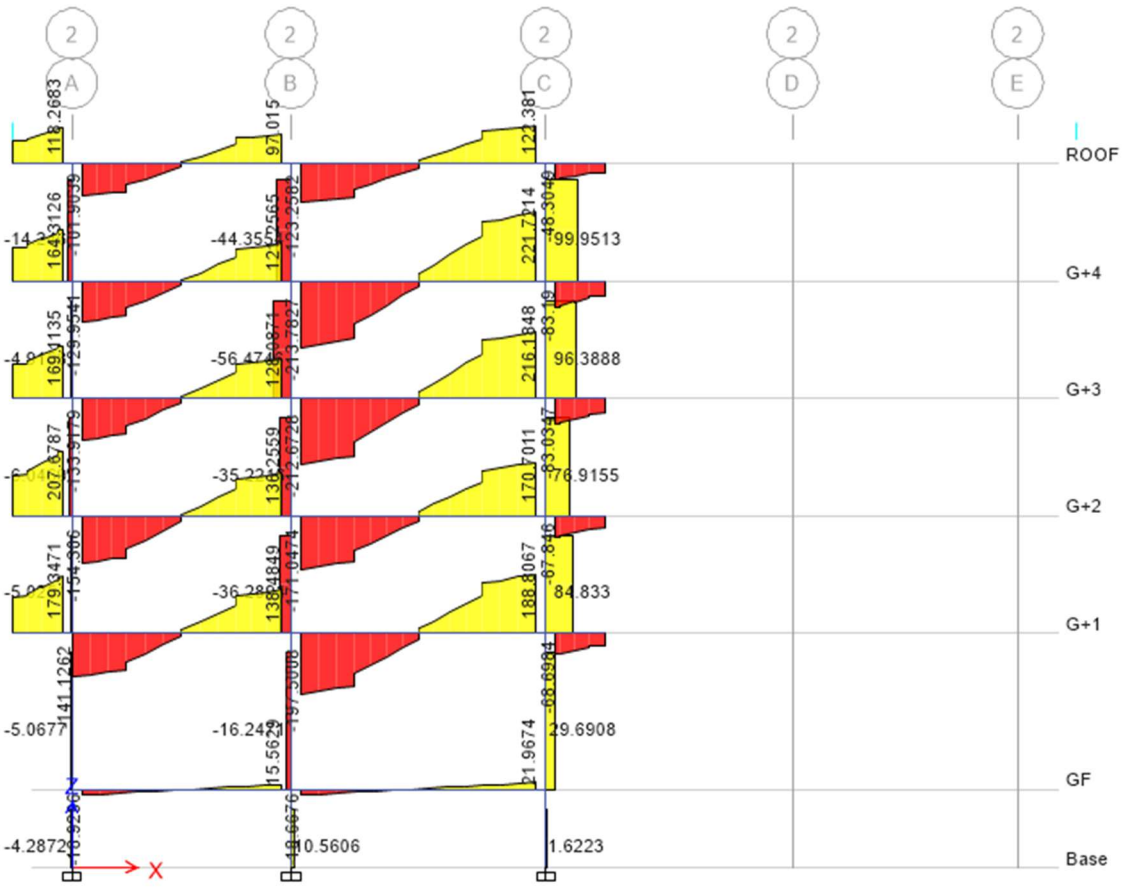


Figure 24 Shear @ axis 2-2

Appendeix E beam design

Check the reinforcement weather single or double reinforcement

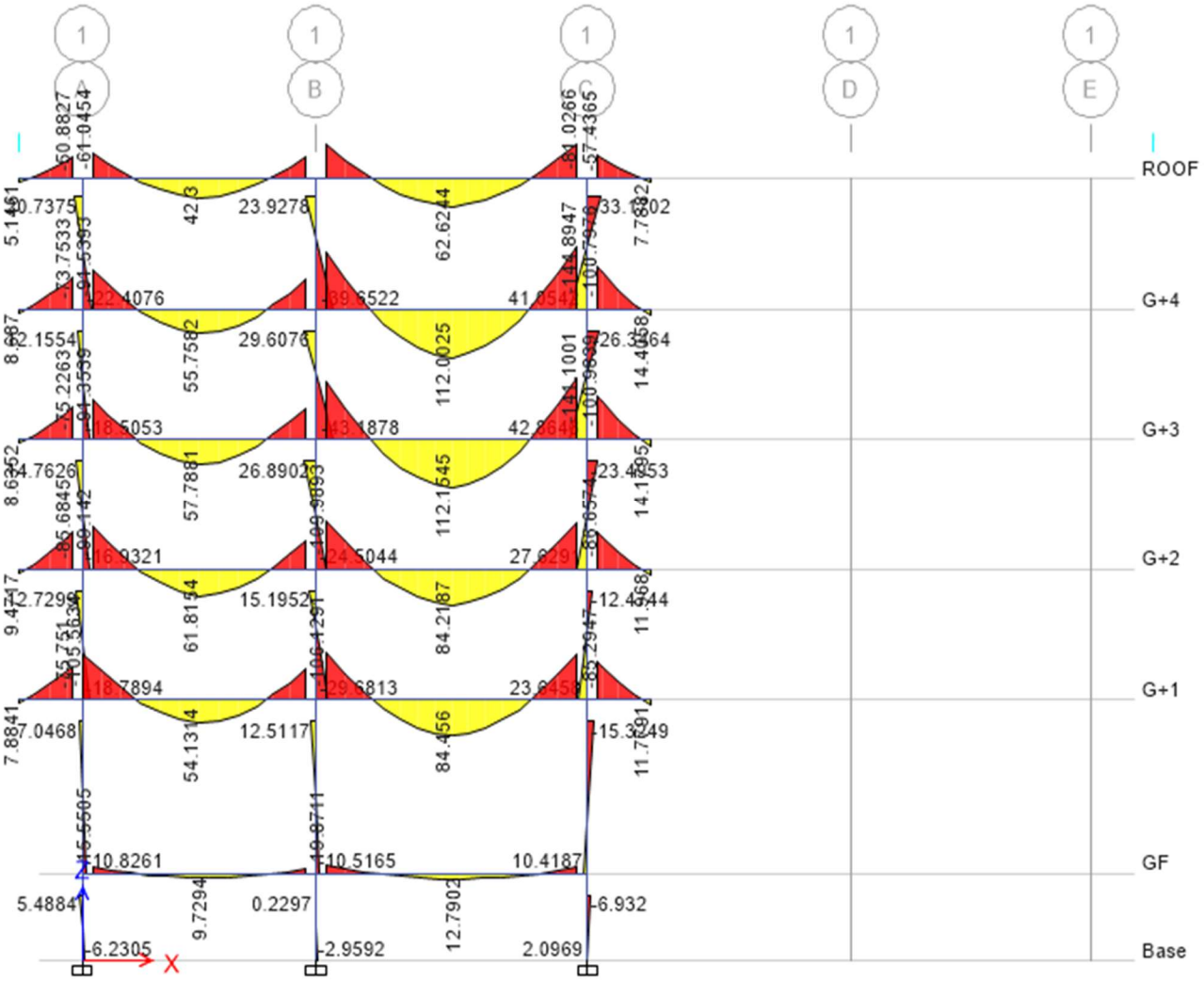


Table 58reinforcement weather single or double reinforcement

Axis 1

ground floor beam design

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	10.82	400	500	0.0054	497.601	single	286	8000	62.5149	286	1.42317	2Ø16
span	A-B	9.73	400	500	0.0049	497.844	single	286	8000	56.1898	286	1.42296	2Ø16
support	B	10.52	400	500	0.0053	497.668	single	286	8000	60.7734	286	1.42296	2Ø16
span	B-C	12.8	400	500	0.0064	497.16	single	286	8000	74.0205	286	1.42296	2Ø16
support	C	10.42	400	500	0.0052	497.69	single	286	8000	60.193	286	1.42296	2Ø16

Axis 1

1st floor beam design

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	12.37	400	500	0.0062	497.256	single	286	8000	71.52	286	1.42296	2Ø16
span	A-B	50.64	400	500	0.0253	488.566	single	286	8000	297.994	297.994	1.48263	2Ø16
support	B	31.46	400	500	0.0157	492.96	single	286	8000	183.478	286	1.42296	2Ø16
span	B-C	82.4	400	500	0.0412	481.107	single	286	8000	492.406	492.406	2.44991	2Ø16
support	C	10.2	400	500	0.0051	497.739	single	286	8000	58.9164	286	1.42296	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	17.04	400	500	0.0085	496.212	single	286	8000	98.728	286	1.42317	2Ø16
span	A-B	57.9	400	500	0.0290	486.882	single	286	8000	341.895	341.895	1.70131	2Ø16
support	B	26.75	400	500	0.0134	494.027	single	286	8000	155.672	286	1.42317	2Ø16
span	B-C	82.95	400	500	0.0415	480.975	single	286	8000	495.829	495.829	2.4673	3Ø16
support	C	10.47	400	500	0.0052	497.679	single	286	8000	60.4832	286	1.42317	2Ø16

3rd floor

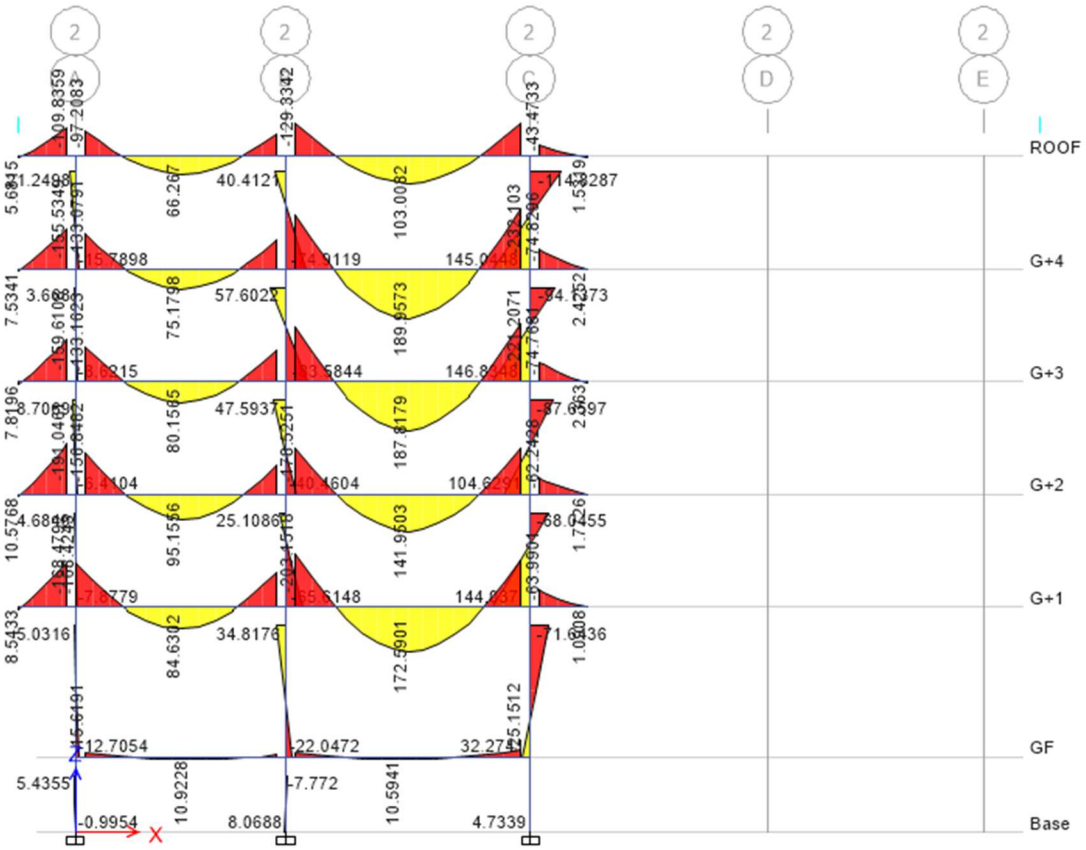
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	26.31	400	500	0.0132	494.126	single	286	8000	153.081	286	1.42317	2Ø16
span	A-B	54.86	400	500	0.0274	487.588	single	286	8000	323.475	323.475	1.60965	2Ø16
support	B	40.03	400	500	0.0200	491.007	single	286	8000	234.388	286	1.42317	2Ø16
span	B-C	109.64	400	500	0.0548	474.511	single	286	8000	664.294	664.294	3.3056	4Ø16
support	C	12.16	400	500	0.0061	497.303	single	286	8000	70.2992	286	1.42317	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	27.27	400	500	0.0136	493.909	single	286	8000	158.736	286	1.42317	2Ø16
span	A-B	53.12	400	500	0.0266	487.992	single	286	8000	312.956	312.956	1.5573	2Ø16
support	B	39.04	400	500	0.0195	491.233	single	286	8000	228.486	286	1.42317	2Ø16
span	B-C	109.52	400	500	0.0548	474.541	single	286	8000	663.526	663.526	3.30178	4Ø16
support	C	12.36	400	500	0.0062	497.258	single	286	8000	71.4619	286	1.42317	2Ø16

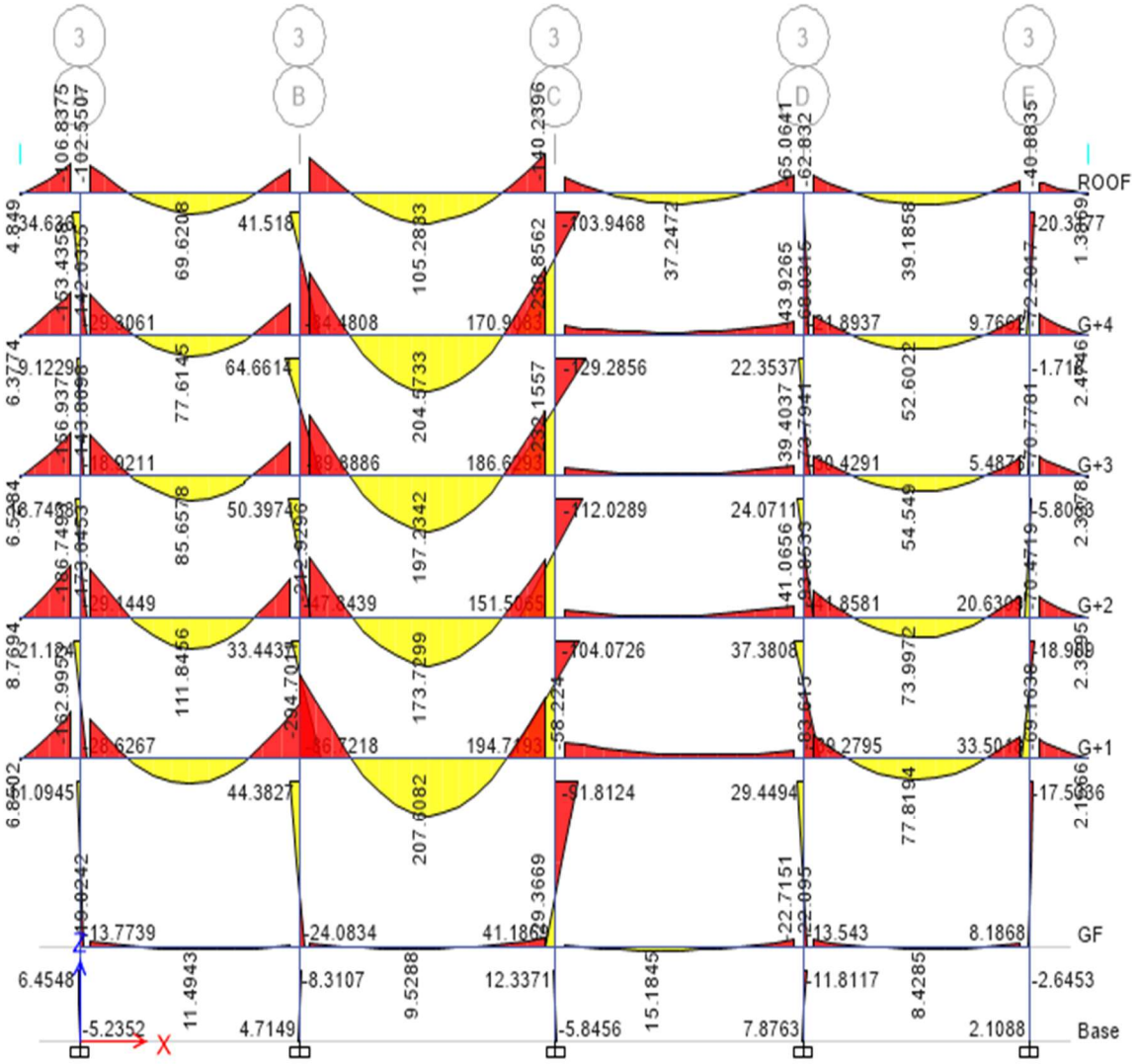
roof floor

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	A	60.82	300	400	0.0634	376.223	single	171.6	4800	464.771	464.771	2.31276	3Ø16
span	A-B	42.36	300	400	0.0441	383.765	single	171.6	4800	317.343	317.343	1.57913	2Ø16
support	B	80.46	300	400	0.0838	367.826	single	171.6	4800	628.891	628.891	3.12943	3Ø16
span	B-C	62.83	300	400	0.0654	375.382	single	171.6	4800	481.207	481.207	2.39454	2Ø16
support	C	7.76	300	400	0.0081	397.126	single	171.6	4800	56.1787	171.6	0.8539	2Ø16


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C
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ROOF
G+4
G+3
G+2
G+1
GF
Base
X

Axis 2												
ground floor beam design												
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	12.7	400	500	0.0064	497.182	single	286	8000	73.4389	286	1.42317	2Ø16
span	A-B	10.92	400	500	0.0055	497.579	single	286	8000	63.0955	286	1.42317	2Ø16
support	B	22.04	400	500	0.0110	495.089	single	286	8000	127.987	286	1.42317	2Ø16
span	B-C	10.59	400	500	0.0053	497.653	single	286	8000	61.1797	286	1.42317	2Ø16
support	C	32.27	400	500	0.0161	492.776	single	286	8000	188.273	286	1.42317	2Ø16
Axis 2												
1st floor beam design												
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	7.87	400	500	0.0039	498.258	single	286	8000	45.4107	286	1.42317	2Ø16
span	A-B	84.63	400	500	0.0423	480.574	single	286	8000	506.293	506.293	2.51937	3Ø16
support	B	65.61	400	500	0.0328	485.08	single	286	8000	388.861	388.861	1.93502	2Ø16
span	B-C	172.59	400	500	0.0863	458.473	single	286	8000	1082.28	1082.28	5.38555	6Ø16
support	C	144.83	400	500	0.0724	465.693	single	286	8000	894.122	894.122	4.44925	5Ø16

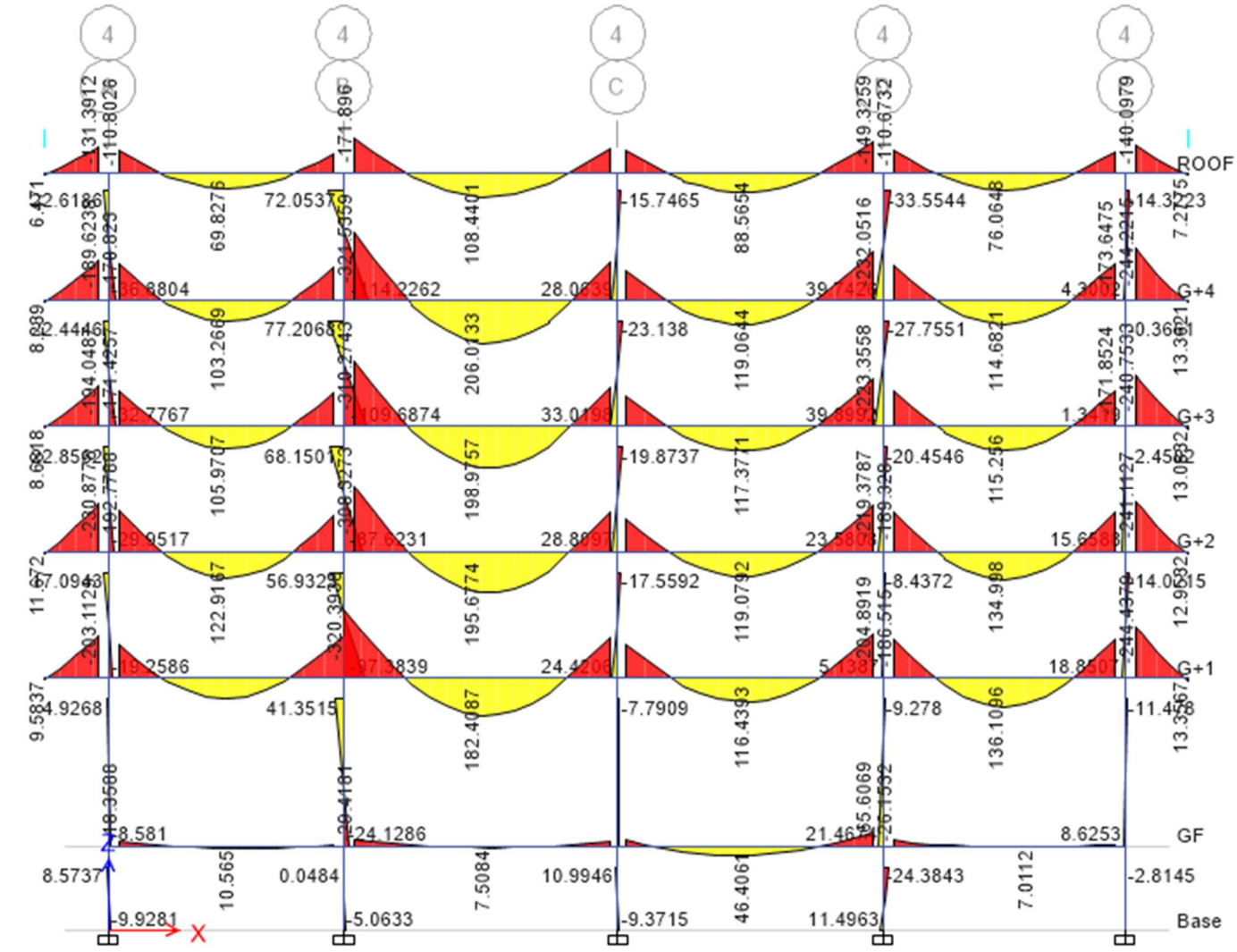
Axis 2												
2nd floor beam design												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	6.41	400	500	0.0032	498.582	single	286	8000	36.9623	286	1.42317	2Ø16
span	A-B	95.15	400	500	0.0476	478.043	single	286	8000	572.242	572.242	2.84754	3Ø16
support	B	40.46	400	500	0.0202	490.908	single	286	8000	236.954	286	1.42317	2Ø16
span	B-C	141.95	400	500	0.0710	466.428	single	286	8000	874.961	874.961	4.3539	5Ø16
support	C	104.63	400	500	0.0523	475.739	single	286	8000	632.303	632.303	3.14641	3Ø16
Axis 2													
3rd floor beam design													
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	8.62	400	500	0.0043	498.091	single	286	8000	49.755	286	1.42317	2Ø16
span	A-B	80.15	400	500	0.0401	481.643	single	286	8000	478.428	478.428	2.38071	3Ø16
support	B	83.58	400	500	0.0418	480.825	single	286	8000	499.751	499.751	2.48682	3Ø16
span	B-C	187.82	400	500	0.0939	454.404	single	286	8000	1188.33	1188.33	5.91327	6Ø16
support	C	146.83	400	500	0.0734	465.181	single	286	8000	907.467	907.467	4.51566	5Ø16
Axis 2													
4th floor beam design													
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	15.79	400	500	0.0079	496.492	single	286	8000	91.4341	286	1.42317	2Ø16
span	A-B	75.18	400	500	0.0376	482.823	single	286	8000	447.664	447.664	2.22763	2Ø16
support	B	74.91	400	500	0.0375	482.887	single	286	8000	445.997	445.997	2.21933	3Ø16
span	B-C	189.96	400	500	0.0950	453.826	single	286	8000	1203.4	1203.4	5.98827	6Ø16
support	C	145.04	400	500	0.0725	465.639	single	286	8000	895.522	895.522	4.45622	5Ø16
Axis 2													
Roof floor beam design													
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	15.79	400	500	0.0079	496.492	single	286	8000	91.4341	286	1.42317	2Ø16
span	A-B	66.27	400	500	0.0331	484.925	single	286	8000	392.899	392.899	1.95511	2Ø16
support	B	74.91	400	500	0.0375	482.887	single	286	8000	445.997	445.997	2.21933	3Ø16
span	B-C	103	400	500	0.0515	476.137	single	286	8000	621.933	621.933	3.09481	3Ø16
support	C	43.47	400	500	0.0217	490.218	single	286	8000	254.94	286	1.42317	2Ø16


3
B
C
D
E
ROOF
G+4
G+3
G+2
G+1
GF
Base
X

Axis 3												
Ground floor beam design												
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	13.78	400	500	0.0069	496.941	single	286	8000	79.7227	286	0.91083	2Ø16
span	A-B	11.49	400	500	0.0057	497.452	single	286	8000	66.4059	286	0.91083	2Ø16
support	B	24.08	400	500	0.0120	494.63	single	286	8000	139.963	286	0.91083	2Ø16
span	B-C	9.53	400	500	0.0048	497.889	single	286	8000	55.0299	286	0.91083	2Ø16
support	C	41.18	400	500	0.0206	490.743	single	286	8000	241.251	286	0.91083	2Ø16
span	C-D	15.18	400	500	0.0076	496.628	single	286	8000	87.8776	286	0.91083	2Ø16
support	D	13.54	400	500	0.0068	496.995	single	286	8000	78.3258	286	0.91083	2Ø16
span	D-E	8.42	400	500	0.0042	498.135	single	286	8000	48.5962	286	0.91083	2Ø16

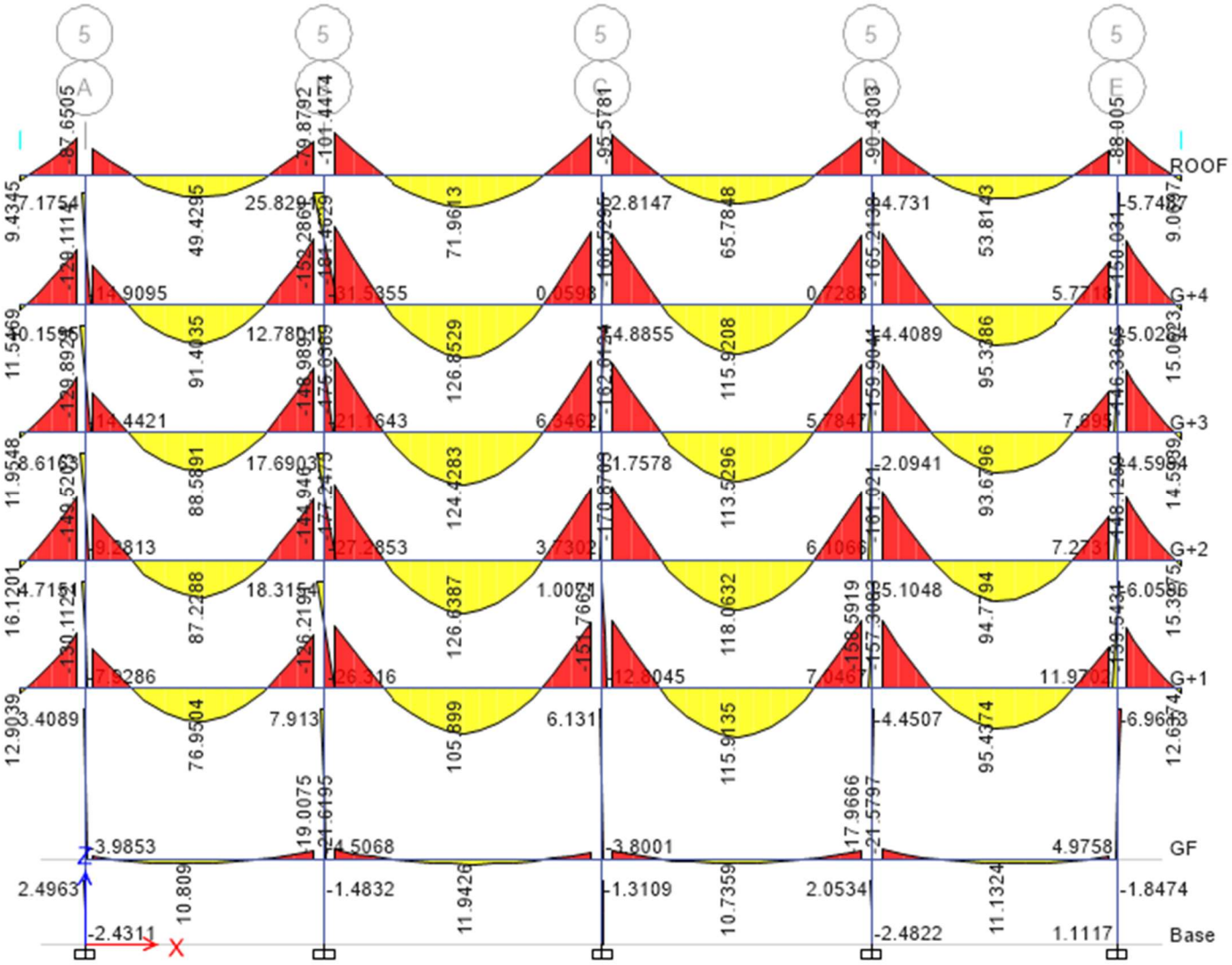
Axis 3												
1ST floor beam design												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	28.62	400	500	0.0143	493.604	single	286	8000	166.697	286	0.91083	2Ø16
span	A-B	44.38	400	500	0.0222	490.009	single	286	8000	260.388	286	0.91083	2Ø16
support	B	86.76	400	500	0.0434	480.064	single	286	8000	519.587	519.587	1.65474	2Ø16
span	B-C	207.6	400	500	0.1038	448.995	single	286	8000	1329.3	1329.3	4.23344	4Ø16
support	C	194.7	400	500	0.0974	452.539	single	286	8000	1236.94	1236.94	3.93929	2Ø16
span	C-D	0	400	500	0.0000	500	single	286	8000	0	286	0.91083	2Ø16
support	D	39.3	400	500	0.0197	491.174	single	286	8000	230.036	286	0.91083	2Ø16
span	D-E	77.82	400	500	0.0389	482.197	single	286	8000	463.986	463.986	1.47766	2Ø16
support	E	33.5	400	500	0.0168	492.496	single	286	8000	195.56	286	0.91083	2Ø16
Axis 3													
2ND floor beam design													
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	29.14	400	500	0.0146	493.486	single	286	8000	169.767	286	0.91083	2Ø16
span	A-B	111.84	400	500	0.0559	473.97	single	286	8000	678.397	678.397	2.1605	2Ø16
support	B	47.84	400	500	0.0239	489.213	single	286	8000	281.146	286	0.91083	2Ø16
span	B-C	173.73	400	500	0.0869	458.172	single	286	8000	1090.15	1090.15	3.4718	4Ø16
support	C	50.6	400	500	0.0253	488.575	single	286	8000	297.753	297.753	0.94826	2Ø16
span	C-D	0	400	500	0.0000	500	single	286	8000	0	286	0.91083	2Ø16
support	D	41.85	400	500	0.0209	490.59	single	286	8000	245.253	286	0.91083	2Ø16
span	D-E	73.99	400	500	0.0370	483.105	single	286	8000	440.321	440.321	1.4023	2Ø16
support	E	20.63	400	500	0.0103	495.406	single	286	8000	119.722	286	0.91083	2Ø16
Axis 3													
3RD floor beam design													
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	18.9	400	500	0.0095	495.795	single	286	8000	109.597	286	0.91083	2Ø16
span	A-B	85.65	400	500	0.0428	480.33	single	286	8000	512.656	512.656	1.63266	2Ø16
support	B	89.88	400	500	0.0449	479.314	single	286	8000	539.114	539.114	1.71692	2Ø16
span	B-C	197.23	400	500	0.0986	451.849	single	286	8000	1254.92	1254.92	3.99657	4Ø16
support	C	29.3	400	500	0.0147	493.45	single	286	8000	170.711	286	0.91083	2Ø16
span	C-D	0	400	500	0.0000	500	single	286	8000	0	286	0.91083	2Ø16
support	D	30.43	400	500	0.0152	493.194	single	286	8000	177.387	286	0.91083	2Ø16
span	D-E	54.55	400	500	0.0273	487.66	single	286	8000	321.599	321.599	1.0242	2Ø16
support	E	5.48	400	500	0.0027	498.788	single	286	8000	31.5866	286	0.91083	2Ø16

Axis 3													
4th floor beam design													
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	29.3	400	500	0.0147	493.45	single	286	8000	170.711	286	1.42317	2Ø16
span	A-B	77.61	400	500	0.0388	482.247	single	286	8000	462.686	462.686	2.30238	2Ø16
support	B	84.48	400	500	0.0422	480.61	single	286	8000	505.358	505.358	2.51472	3Ø16
span	B-C	204.57	400	500	0.1023	449.833	single	286	8000	1307.46	1307.46	6.50607	7Ø16
support	C	38.3	400	500	0.0192	491.402	single	286	8000	224.078	286	1.42317	2Ø16
span	C-D	0	400	500	0.0000	500	single	286	8000	0	286	1.42317	2Ø16
support	D	24.9	400	500	0.0125	494.445	single	286	8000	144.784	286	1.42317	2Ø16
span	D-E	52.61	400	500	0.0263	488.11	single	286	8000	309.876	309.876	1.54198	2Ø16
support	E	9.76	400	500	0.0049	497.837	single	286	8000	56.3638	286	1.42317	2Ø16
Axis 3													
Roof floor beam design													
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	106.84	400	500	0.0534	475.198	single	286	8000	646.393	646.393	3.21653	2Ø16
span	A-B	69.62	400	500	0.0348	484.137	single	286	8000	413.432	413.432	2.05728	2Ø16
support	B	84.48	400	500	0.0422	480.61	single	286	8000	505.358	505.358	2.51472	2Ø16
span	B-C	105.28	400	500	0.0526	475.58	single	286	8000	636.444	636.444	3.16702	3Ø16
support	C	140.23	400	500	0.0701	466.866	single	286	8000	863.548	863.548	4.29711	4Ø16
span	C-D	37.24	400	500	0.0186	491.644	single	286	8000	217.769	286	1.42317	2Ø16
support	D	65.04	400	500	0.0325	485.213	single	286	8000	385.377	385.377	1.91768	2Ø16
span	D-E	39.18	400	500	0.0196	491.201	single	286	8000	229.321	286	1.42317	2Ø16
support	E	40.88	400	500	0.0204	490.812	single	286	8000	239.46	286	1.42317	2Ø20


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C
ROOF
G+4
G+3
G+2
G+1
GF
Base
-131.3912
-110.8026
-171.896
-149.3259
-110.6732
-140.0979
-15.7465
88.5654
-33.5544
76.0648
-23.138
119.0644
-27.7551
114.6821
-17.5592
119.0792
-8.4372
134.988
-7.7909
116.4393
-9.278
136.1086
-11.4578
-24.1286
21.4677
8.6253
8.5737
10.565
0.0484
7.5084
10.9946
46.4061
-24.3843
7.0112
-2.8145
-9.9281
-5.0633
-9.3715
11.4963
X
Z

Axis 4												
Ground floor beam design												
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	8.58	400	500	0.0043	498.1	single	286	8000	49.5232	286	0.91083	2Ø16
span	A-B	10.56	400	500	0.0053	497.659	single	286	8000	61.0056	286	0.91083	2Ø16
support	B	24.12	400	500	0.0121	494.621	single	286	8000	140.198	286	0.91083	2Ø16
span	B-C	7.5	400	500	0.0038	498.34	single	286	8000	43.2687	286	0.91083	2Ø16
support	C	10.99	400	500	0.0055	497.563	single	286	8000	63.502	286	0.91083	2Ø16
span	C-D	46.4	400	500	0.0232	489.544	single	286	8000	272.498	286	0.91083	2Ø16
support	D	21.46	400	500	0.0107	495.22	single	286	8000	124.586	286	0.91083	2Ø16
span	D-E	7.01	400	500	0.0035	498.449	single	286	8000	40.433	286	0.91083	2Ø16
support	E	8.62	400	500	0.0043	498.091	single	286	8000	49.755	286	0.91083	2Ø16
Axis 4													
1st floor beam design													
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	19.25	400	500	0.0096	495.716	single	286	8000	111.644	286	0.91083	2Ø16
span	A-B	10.56	400	500	0.0053	497.659	single	286	8000	61.0056	286	0.91083	2Ø16
support	B	97.38	400	500	0.0487	477.503	single	286	8000	586.315	586.315	1.86725	2Ø16
span	B-C	182.4	400	500	0.0912	455.862	single	286	8000	1150.35	1150.35	3.66353	4Ø16
support	C	24.42	400	500	0.0122	494.553	single	286	8000	141.962	286	0.91083	2Ø16
span	C-D	116.43	400	500	0.0582	472.837	single	286	8000	707.932	707.932	2.25456	2Ø16
support	D	5.13	400	500	0.0026	498.866	single	286	8000	29.5646	286	0.91083	2Ø16
span	D-E	136.11	400	500	0.0681	467.911	single	286	8000	836.304	836.304	2.66339	3Ø16
support	E	18.85	400	500	0.0094	495.806	single	286	8000	109.304	286	0.91083	2Ø16
Axis 4													
2nd floor beam design													
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	29.95	400	500	0.0150	493.303	single	286	8000	174.551	286	0.91083	2Ø16
span	A-B	122.91	400	500	0.0615	471.227	single	286	8000	749.885	749.885	2.38817	3Ø16
support	B	87.62	400	500	0.0438	479.857	single	286	8000	524.963	524.963	1.67186	2Ø16
span	B-C	195.7	400	500	0.0979	452.267	single	286	8000	1244.04	1244.04	3.96191	4Ø16
support	C	28.88	400	500	0.0144	493.545	single	286	8000	168.232	286	0.91083	2Ø16
span	C-D	119.07	400	500	0.0595	472.183	single	286	8000	724.987	724.987	2.30888	2Ø16
support	D	23.58	400	500	0.0118	494.742	single	286	8000	137.026	286	0.91083	2Ø16
span	D-E	134.99	400	500	0.0675	468.195	single	286	8000	828.921	828.921	2.63987	3Ø16
support	E	15.65	400	500	0.0078	496.523	single	286	8000	90.6176	286	0.91083	2Ø16

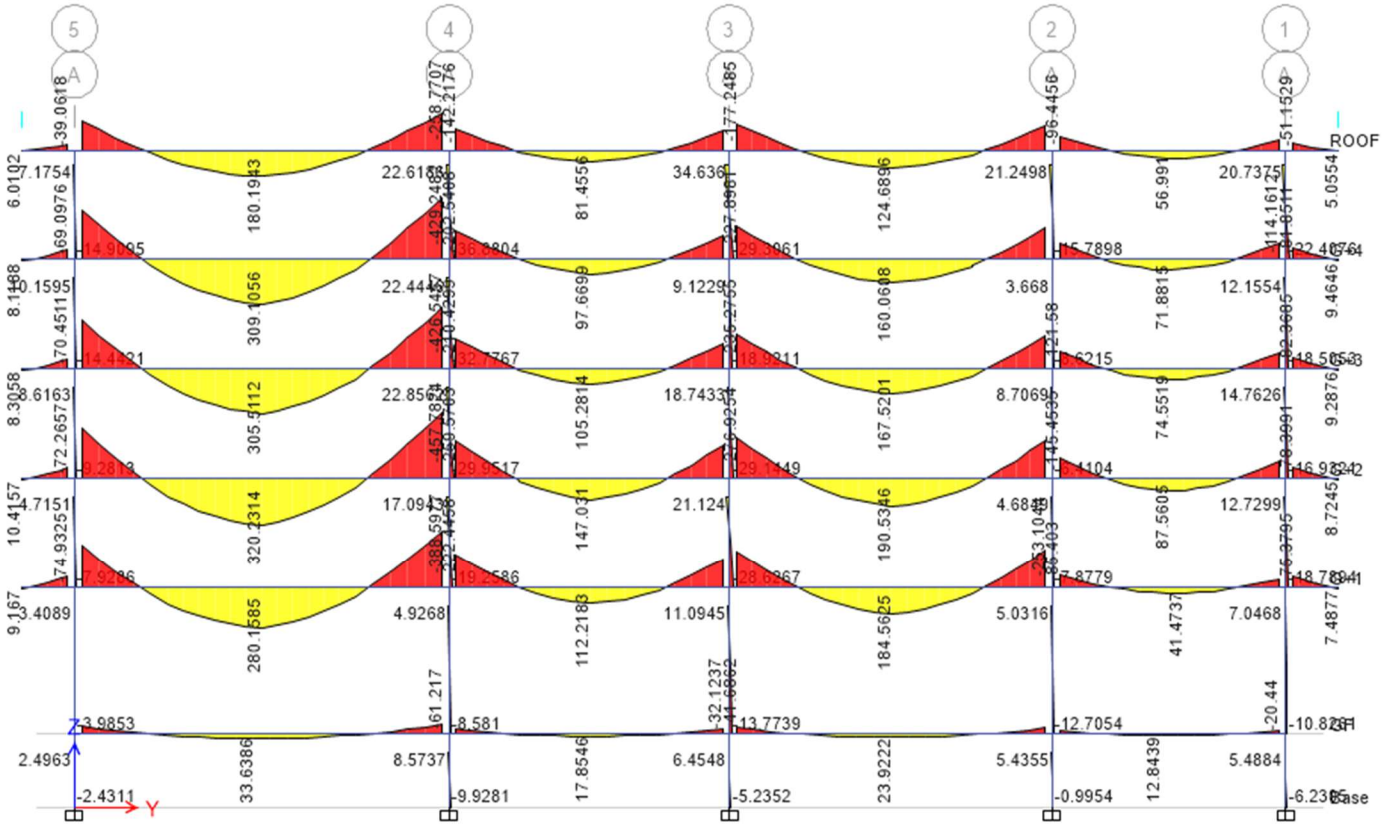
Axis 4												
3rd floor beam design												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	32.77	400	500	0.0164	492.662	single	286	8000	191.234	286	0.91083	2Ø16
span	A-B	105.97	400	500	0.0530	475.411	single	286	8000	640.843	640.843	2.0409	2Ø16
support	B	109.68	400	500	0.0548	474.502	single	286	8000	664.55	664.55	2.1164	2Ø16
span	B-C	198.97	400	500	0.0995	451.373	single	286	8000	1267.33	1267.33	4.03608	2Ø16
support	C	33.03	400	500	0.0165	492.603	single	286	8000	192.774	286	0.91083	2Ø16
span	C-D	117.37	400	500	0.0587	472.604	single	286	8000	713.999	713.999	2.27388	2Ø16
support	D	38.89	400	500	0.0194	491.267	single	286	8000	227.592	286	0.91083	2Ø16
span	D-E	115.25	400	500	0.0576	473.129	single	286	8000	700.325	700.325	2.23033	2Ø16
support	E	1.34	400	500	0.0007	499.704	single	286	8000	7.70956	286	0.91083	2Ø16
Axis 4													
4th floor beam design													
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	36.88	400	500	0.0184	491.726	single	286	8000	215.628	286	0.91083	2Ø16
span	A-B	103.26	400	500	0.0516	476.073	single	286	8000	623.586	623.586	1.98594	2Ø16
support	B	114.22	400	500	0.0571	473.383	single	286	8000	693.693	693.693	2.20921	2Ø16
span	B-C	206.02	400	500	0.1030	449.433	single	286	8000	1317.9	1317.9	4.19713	2Ø16
support	C	28.06	400	500	0.0140	493.731	single	286	8000	163.394	286	0.91083	2Ø16
span	C-D	119.06	400	500	0.0595	472.185	single	286	8000	724.922	724.922	2.30867	2Ø16
support	D	39.74	400	500	0.0199	491.073	single	286	8000	232.659	286	0.91083	2Ø20
span	D-E	114.68	400	500	0.0573	473.27	single	286	8000	696.654	696.654	2.21864	3Ø20
support	E	4.3	400	500	0.0022	499.05	single	286	8000	24.7721	286	0.91083	2Ø20
Axis 4													
Roof floor beam design													
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	131.32	400	500	0.0657	469.12	single	286	8000	804.793	804.793	2.56304	3Ø16
span	A-B	69.82	400	500	0.0349	484.09	single	286	8000	414.66	414.66	1.32057	2Ø16
support	B	171.89	400	500	0.0859	458.659	single	286	8000	1077.45	1077.45	3.43138	3Ø16
span	B-C	108.44	400	500	0.0542	474.806	single	286	8000	656.616	656.616	2.09113	2Ø16
support	C	28.06	400	500	0.0140	493.731	single	286	8000	163.394	286	0.91083	2Ø16
span	C-D	88.56	400	500	0.0443	479.632	single	286	8000	530.845	530.845	1.69059	2Ø16
support	D	148.32	400	500	0.0742	464.799	single	286	8000	917.43	917.43	2.92175	3Ø16
span	D-E	76.06	400	500	0.0380	482.615	single	286	8000	453.099	453.099	1.44299	2Ø16
support	E	140.09	400	500	0.0700	466.902	single	286	8000	862.62	862.62	2.7472	2Ø16


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G+3
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G+1
GF
Base

Axis 5													
Ground floor beam design													
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	3.98	400	500	0.0020	499.12	single	286	8000	22.9253	286	0.91083	2Ø16
span	A-B	10.8	400	500	0.0054	497.606	single	286	8000	62.3988	286	0.91083	2Ø16
support	B	4.5	400	500	0.0023	499.005	single	286	8000	25.9266	286	0.91083	2Ø16
span	B-C	11.94	400	500	0.0060	497.352	single	286	8000	69.0206	286	0.91083	2Ø16
support	C	3.8	400	500	0.0019	499.16	single	286	8000	21.8868	286	0.91083	2Ø16
span	C-D	10.73	400	500	0.0054	497.621	single	286	8000	61.9924	286	0.91083	2Ø16
support	D	21.57	400	500	0.0108	495.195	single	286	8000	125.231	286	0.91083	2Ø16
span	D-E	11.13	400	500	0.0056	497.532	single	286	8000	64.3149	286	0.91083	2Ø16
support	E	4.97	400	500	0.0025	498.901	single	286	8000	28.6404	286	0.91083	2Ø16

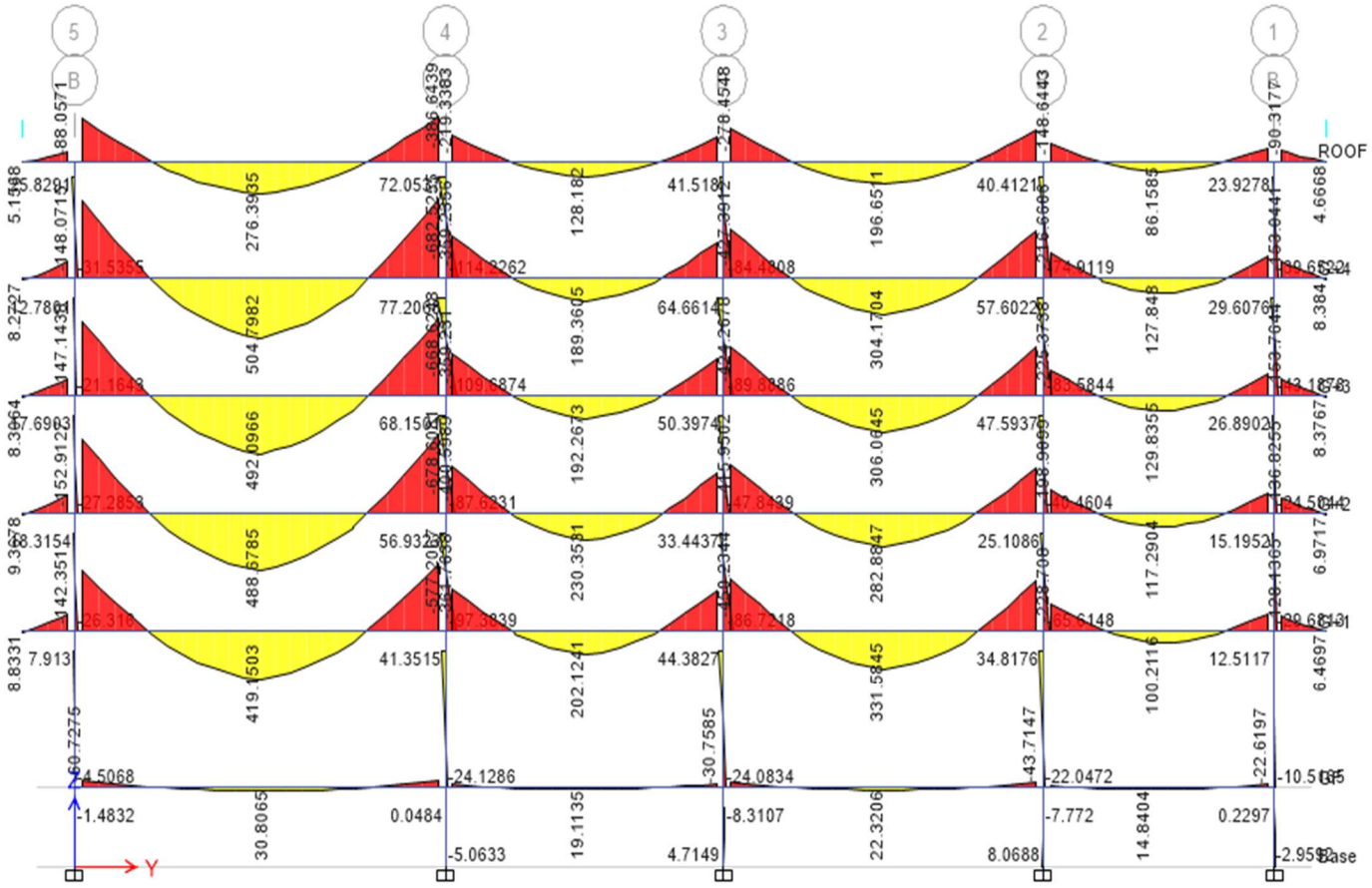
Axis 5												
1st floor beam design												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	7.92	400	500	0.0040	498.247	single	286	8000	45.7003	286	0.91083	2Ø16
span	A-B	76.95	400	500	0.0385	482.404	single	286	8000	458.602	458.602	1.46052	2Ø16
support	B	26.31	400	500	0.0132	494.126	single	286	8000	153.081	286	0.91083	2Ø16
span	B-C	105.89	400	500	0.0529	475.431	single	286	8000	640.333	640.333	2.03928	2Ø16
support	C	12.8	400	500	0.0064	497.16	single	286	8000	74.0205	286	0.91083	2Ø16
span	C-D	115.91	400	500	0.0580	472.966	single	286	8000	704.578	704.578	2.24388	2Ø16
support	D	7.05	400	500	0.0035	498.44	single	286	8000	40.6644	286	0.91083	2Ø16
span	D-E	95.43	400	500	0.0477	477.976	single	286	8000	574.007	574.007	1.82805	2Ø16
support	E	11.93	400	500	0.0060	497.354	single	286	8000	68.9625	286	0.91083	2Ø16
Axis 5												
2nd floor beam design												
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	9.28	400	500	0.0046	497.944	single	286	8000	53.5803	286	0.91083	2Ø16
span	A-B	87.22	400	500	0.0436	479.953	single	286	8000	522.462	522.462	1.66389	2Ø16
support	B	27.28	400	500	0.0136	493.907	single	286	8000	158.795	286	0.91083	2Ø16
span	B-C	126.63	400	500	0.0633	470.298	single	286	8000	774.108	774.108	2.46531	2Ø16
support	C	3.73	400	500	0.0019	499.176	single	286	8000	21.4829	286	0.91083	2Ø16
span	C-D	118.06	400	500	0.0590	472.433	single	286	8000	718.456	718.456	2.28808	2Ø16
support	D	6.11	400	500	0.0031	498.648	single	286	8000	35.2277	286	0.91083	2Ø16
span	D-E	94.77	400	500	0.0474	478.135	single	286	8000	569.847	569.847	1.8148	2Ø20
support	E	7.27	400	500	0.0036	498.391	single	286	8000	41.9375	286	0.91083	2Ø20
Axis 5												
3rd floor beam design												
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	14.44	400	500	0.0072	496.794	single	286	8000	83.5659	286	0.91083	2Ø16
span	A-B	85.88	400	500	0.0429	480.275	single	286	8000	514.091	514.091	1.63723	2Ø16
support	B	21.16	400	500	0.0106	495.287	single	286	8000	122.828	286	0.91083	2Ø16
span	B-C	124.43	400	500	0.0622	470.848	single	286	8000	759.77	759.77	2.41965	2Ø16
support	C	6.34	400	500	0.0032	498.597	single	286	8000	36.5576	286	0.91083	2Ø16
span	C-D	111.53	400	500	0.0558	474.047	single	286	8000	676.408	676.408	2.15416	2Ø16
support	D	5.78	400	500	0.0029	498.722	single	286	8000	33.3202	286	0.91083	2Ø16
span	D-E	93.68	400	500	0.0468	478.399	single	286	8000	562.982	562.982	1.79294	2Ø16
support	E	7.69	400	500	0.0038	498.298	single	286	8000	44.3686	286	0.91083	2Ø16

Axis 5												
4th floor beam design												
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	14.9	400	500	0.0075	496.691	single	286	8000	86.2458	286	0.91083	2Ø16
span	A-B	91.4	400	500	0.0457	478.949	single	286	8000	548.65	548.65	1.74729	2Ø16
support	B	31.53	400	500	0.0158	492.944	single	286	8000	183.893	286	0.91083	2Ø16
span	B-C	126.83	400	500	0.0634	470.248	single	286	8000	775.413	775.413	2.46947	2Ø16
support	C	0.06	400	500	0.0000	499.987	single	286	8000	0.34501	286	0.91083	2Ø16
span	C-D	115.92	400	500	0.0580	472.963	single	286	8000	704.643	704.643	2.24408	2Ø16
support	D	0.73	400	500	0.0004	499.839	single	286	8000	4.19885	286	0.91083	2Ø16
span	D-E	95.33	400	500	0.0477	478	single	286	8000	573.376	573.376	1.82604	2Ø16
support	E	5.77	400	500	0.0029	498.724	single	286	8000	33.2624	286	0.91083	2Ø16
Axis 5												
Roof floor beam design												
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	87.65	400	500	0.0438	479.85	single	286	8000	525.151	525.151	1.67246	2Ø16
span	A-B	49.42	400	500	0.0247	488.848	single	286	8000	290.648	290.648	0.92563	2Ø16
support	B	79.87	400	500	0.0399	481.71	single	286	8000	476.69	476.69	1.51812	2Ø16
span	B-C	71.96	400	500	0.0360	483.585	single	286	8000	427.815	427.815	1.36247	2Ø16
support	C	95.57	400	500	0.0478	477.942	single	286	8000	574.89	574.89	1.83086	2Ø16
span	C-D	65.78	400	500	0.0329	485.04	single	286	8000	389.901	389.901	1.24172	2Ø16
support	D	90.43	400	500	0.0452	479.182	single	286	8000	542.563	542.563	1.72791	2Ø16
span	D-E	53.81	400	500	0.0269	487.832	single	286	8000	317.125	317.125	1.00995	2Ø16
support	E	88	400	500	0.0440	479.766	single	286	8000	527.34	527.34	1.67943	2Ø16


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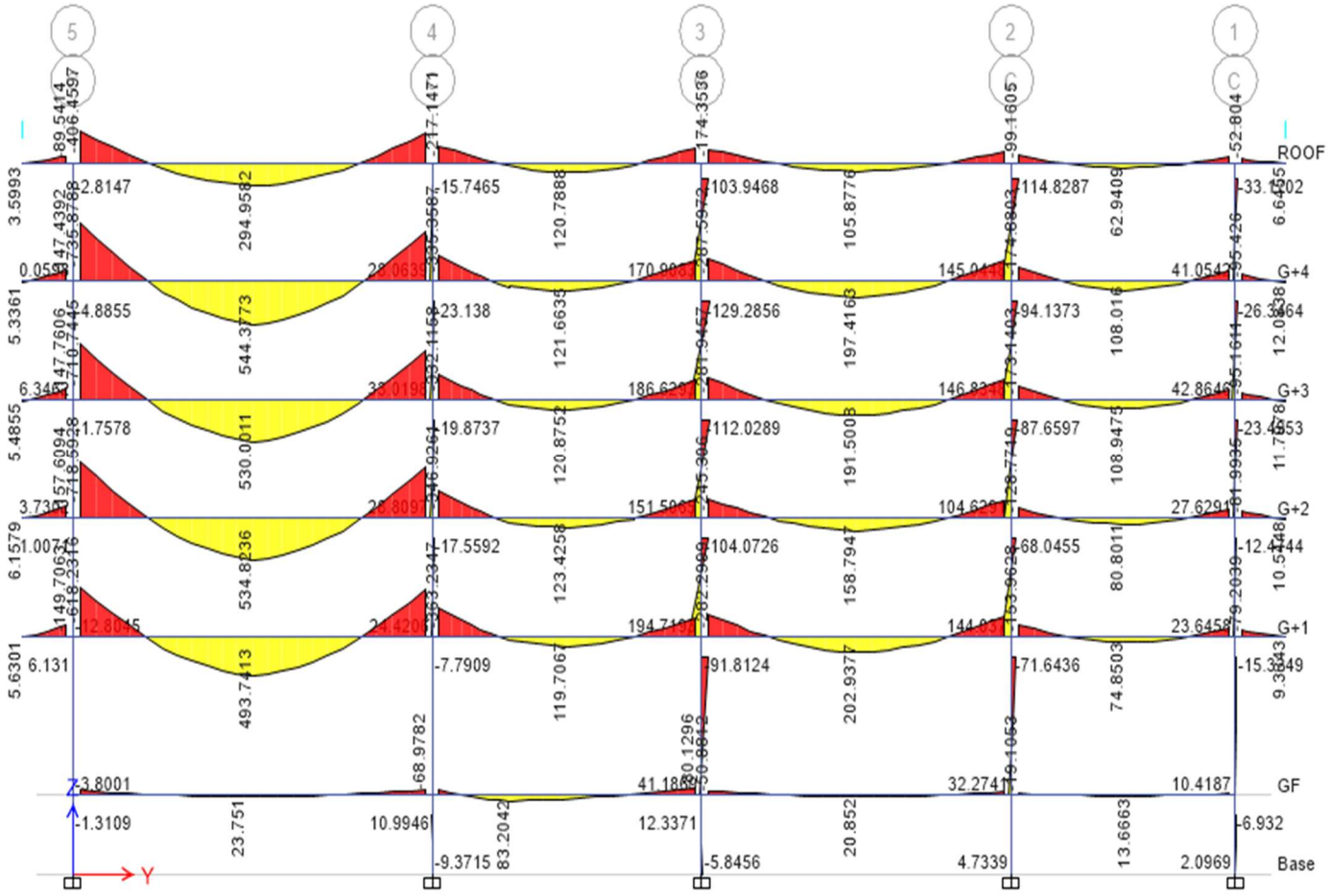
Axis A												
ground floor beam												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	5	3.98	400	500	0.0020	499.12	single	286	8000	22.9253	286	0.91083	2Ø16
span	5-4	33.63	400	500	0.0168	492.467	single	286	8000	196.33	286	0.91083	2Ø16
support	4	8.58	400	500	0.0043	498.1	single	286	8000	49.5232	286	0.91083	2Ø16
span	4-3	17.85	400	500	0.0089	496.03	single	286	8000	103.459	286	0.91083	2Ø16
support	3	13.77	400	500	0.0069	496.943	single	286	8000	79.6645	286	0.91083	2Ø16
span	3-2	23.92	400	500	0.0120	494.666	single	286	8000	139.023	286	0.91083	2Ø16
support	2	12.7	400	500	0.0064	497.182	single	286	8000	73.4389	286	0.91083	2Ø16
span	2-1	12.84	400	500	0.0064	497.151	single	286	8000	74.2531	286	0.91083	2Ø16
support	1	10.82	400	500	0.0054	497.601	single	286	8000	62.5149	286	0.91083	2Ø16
Axis A													
1ST floor beam													
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	5	7.92	400	500	0.0040	498.247	single	286	8000	45.7003	286	0.91083	2Ø16
span	5-4	280.16	400	500	0.1401	427.749	single	286	8000	1883.02	1883.02	5.99687	6Ø16
support	4	19.25	400	500	0.0096	495.716	single	286	8000	111.644	286	0.91083	2Ø16
span	4-3	112.21	400	500	0.0561	473.879	single	286	8000	680.772	680.772	2.16806	2Ø16
support	3	28.62	400	500	0.0143	493.604	single	286	8000	166.697	286	0.91083	2Ø16
span	3-2	184.56	400	500	0.0923	455.282	single	286	8000	1165.45	1165.45	3.71163	4Ø16
support	2	7.88	400	500	0.0039	498.255	single	286	8000	45.4687	286	0.91083	2Ø16
span	2-1	41.47	400	500	0.0207	490.677	single	286	8000	242.983	286	0.91083	2Ø16
support	1	18.78	400	500	0.0094	495.822	single	286	8000	108.895	286	0.91083	2Ø16
Axis A													
2ND floor beam													
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	5	9.28	400	500	0.0046	497.944	single	286	8000	53.5803	286	0.91083	2Ø16
span	5-4	320.23	400	500	0.1601	414.847	single	286	8000	2219.28	2219.28	7.06776	7Ø16
support	4	29.95	400	500	0.0150	493.303	single	286	8000	174.551	286	0.91083	2Ø16
span	4-3	147.3	400	500	0.0737	465.06	single	286	8000	910.608	910.608	2.90002	3Ø16
support	3	29.14	400	500	0.0146	493.486	single	286	8000	169.767	286	0.91083	2Ø16
span	3-2	190.53	400	500	0.0953	453.672	single	286	8000	1207.42	1207.42	3.8453	4Ø16
support	2	6.41	400	500	0.0032	498.582	single	286	8000	36.9623	286	0.91083	2Ø16
span	2-1	87.56	400	500	0.0438	479.872	single	286	8000	524.588	524.588	1.67066	2Ø16
support	1	16.93	400	500	0.0085	496.236	single	286	8000	98.0858	286	0.91083	2Ø16

Axis A												
3RD floor beam												
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	5	14.44	400	500	0.0072	496.794	single	286	8000	83.5659	286	0.91083	2Ø16
span	5-4	305.51	400	500	0.1528	419.701	single	286	8000	2092.78	2092.78	6.6649	7Ø16
support	4	32.77	400	500	0.0164	492.662	single	286	8000	191.234	286	0.91083	2Ø16
span	4-3	105.28	400	500	0.0526	475.58	single	286	8000	636.444	636.444	2.02689	2Ø16
support	3	18.92	400	500	0.0095	495.79	single	286	8000	109.714	286	0.91083	2Ø16
span	3-2	167.52	400	500	0.0838	459.811	single	286	8000	1047.43	1047.43	3.33577	3Ø16
support	2	6.62	400	500	0.0033	498.535	single	286	8000	38.1768	286	0.91083	2Ø16
span	2-1	74.55	400	500	0.0373	482.973	single	286	8000	443.775	443.775	1.4133	2Ø16
support	1	18.5	400	500	0.0093	495.885	single	286	8000	107.258	286	0.91083	2Ø16
Axis A												
4TH floor beam												
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	5	14.9	400	500	0.0075	496.691	single	286	8000	86.2458	286	0.91083	2Ø16
span	5-4	309.1	400	500	0.1546	418.53	single	286	8000	2123.29	2123.29	6.76208	7Ø16
support	4	36.88	400	500	0.0184	491.726	single	286	8000	215.628	286	0.91083	2Ø16
span	4-3	97.67	400	500	0.0488	477.433	single	286	8000	588.148	588.148	1.87308	2Ø16
support	3	15.79	400	500	0.0079	496.492	single	286	8000	91.4341	286	0.91083	2Ø16
span	3-2	160.06	400	500	0.0800	461.763	single	286	8000	996.557	996.557	3.17375	3Ø16
support	2	15.79	400	500	0.0079	496.492	single	286	8000	91.4341	286	0.91083	2Ø16
span	2-1	71.88	400	500	0.0359	483.604	single	286	8000	427.323	427.323	1.3609	2Ø16
support	1	22.4	400	500	0.0112	495.008	single	286	8000	130.099	286	0.91083	2Ø16
Axis A												
Roof floor beam												
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	5	14.9	400	500	0.0075	496.691	single	286	8000	86.2458	286	0.91083	2Ø16
span	5-4	180.19	400	500	0.0901	456.453	single	286	8000	1134.94	1134.94	3.61446	4Ø16
support	4	36.88	400	500	0.0184	491.726	single	286	8000	215.628	286	0.91083	2Ø16
span	4-3	81.45	400	500	0.0407	481.333	single	286	8000	486.5	486.5	1.54936	2Ø16
support	3	15.79	400	500	0.0079	496.492	single	286	8000	91.4341	286	0.91083	2Ø16
span	3-2	124.69	400	500	0.0623	470.783	single	286	8000	761.463	761.463	2.42504	2Ø16
support	2	15.79	400	500	0.0079	496.492	single	286	8000	91.4341	286	0.91083	2Ø16
span	2-1	56.99	400	500	0.0285	487.093	single	286	8000	336.375	336.375	1.07126	2Ø16
support	1	22.4	400	500	0.0112	495.008	single	286	8000	130.099	286	0.91083	2Ø16


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Axis B												
ground floor beam												
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	5	4.5	400	500	0.0023	499.005	single	286	8000	25.9266	286	1.42317	2Ø16
span	5-4	30.8	400	500	0.0154	493.11	single	286	8000	179.575	286	1.42317	2Ø16
support	4	24.13	400	500	0.0121	494.618	single	286	8000	140.257	286	1.42317	2Ø16
span	4-3	19.11	400	500	0.0096	495.748	single	286	8000	110.825	286	1.42317	2Ø16
support	3	24.08	400	500	0.0120	494.63	single	286	8000	139.963	286	1.42317	2Ø16
span	3-2	22.32	400	500	0.0112	495.026	single	286	8000	129.63	286	1.42317	2Ø16
support	2	22.05	400	500	0.0110	495.087	single	286	8000	128.046	286	1.42317	2Ø16
span	2-1	14.84	400	500	0.0074	496.704	single	286	8000	85.8962	286	1.42317	2Ø16
support	1	10.5	400	500	0.0053	497.673	single	286	8000	60.6573	286	1.42317	2Ø16
Axis B													
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	5	37.79	400	500	0.0189	491.519	single	286	8000	221.042	286	1.42317	2Ø16
span	5-4	443.92	400	500	0.2220	366.319	double	286	8000	3484.04	3484.04	17.337	18Ø16
support	4	90.58	400	500	0.0453	479.146	single	286	8000	543.503	543.503	2.70454	3Ø16
span	4-3	205.37	400	500	0.1027	449.612	single	286	8000	1313.22	1313.22	6.53472	7Ø16
support	3	213.72	400	500	0.1069	447.292	single	286	8000	1373.7	1373.7	6.83569	7Ø16
span	3-2	350.993	400	500	0.1755	404.211	double	286	8000	2496.48	2496.48	12.4228	13Ø16
support	2	135.03	400	500	0.0675	468.185	single	286	8000	829.184	829.184	4.12612	4Ø16
span	2-1	97.73	400	500	0.0489	477.418	single	286	8000	588.527	588.527	2.92858	3Ø16
support	1	2.54	400	500	0.0013	499.439	single	286	8000	14.6214	286	1.42317	2Ø16
Axis B													
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	5	77.4	400	500	0.0387	482.297	single	286	8000	461.386	461.386	2.29591	2Ø16
span	5-4	515.66	400	500	0.2578	324.942	double	286	8000	4562.43	4562.43	22.7032	23Ø16
support	4	163.4	400	500	0.0817	460.891	single	286	8000	1019.28	1019.28	5.07203	5Ø16
span	4-3	235.95	400	500	0.1180	440.976	single	286	8000	1538.31	1538.31	7.65479	8Ø16
support	3	212.1	400	500	0.1061	447.744	single	286	8000	1361.91	1361.91	6.77702	7Ø16
span	3-2	99.06	400	500	0.0495	477.096	single	286	8000	596.94	596.94	2.97044	3Ø16
support	2	116.16	400	500	0.0581	472.904	single	286	8000	706.19	706.19	3.51408	4Ø16
span	2-1	114.54	400	500	0.0573	473.304	single	286	8000	695.752	695.752	3.46214	4Ø16
support	1	2.36	400	500	0.0012	499.479	single	286	8000	13.5842	286	1.42317	2Ø16

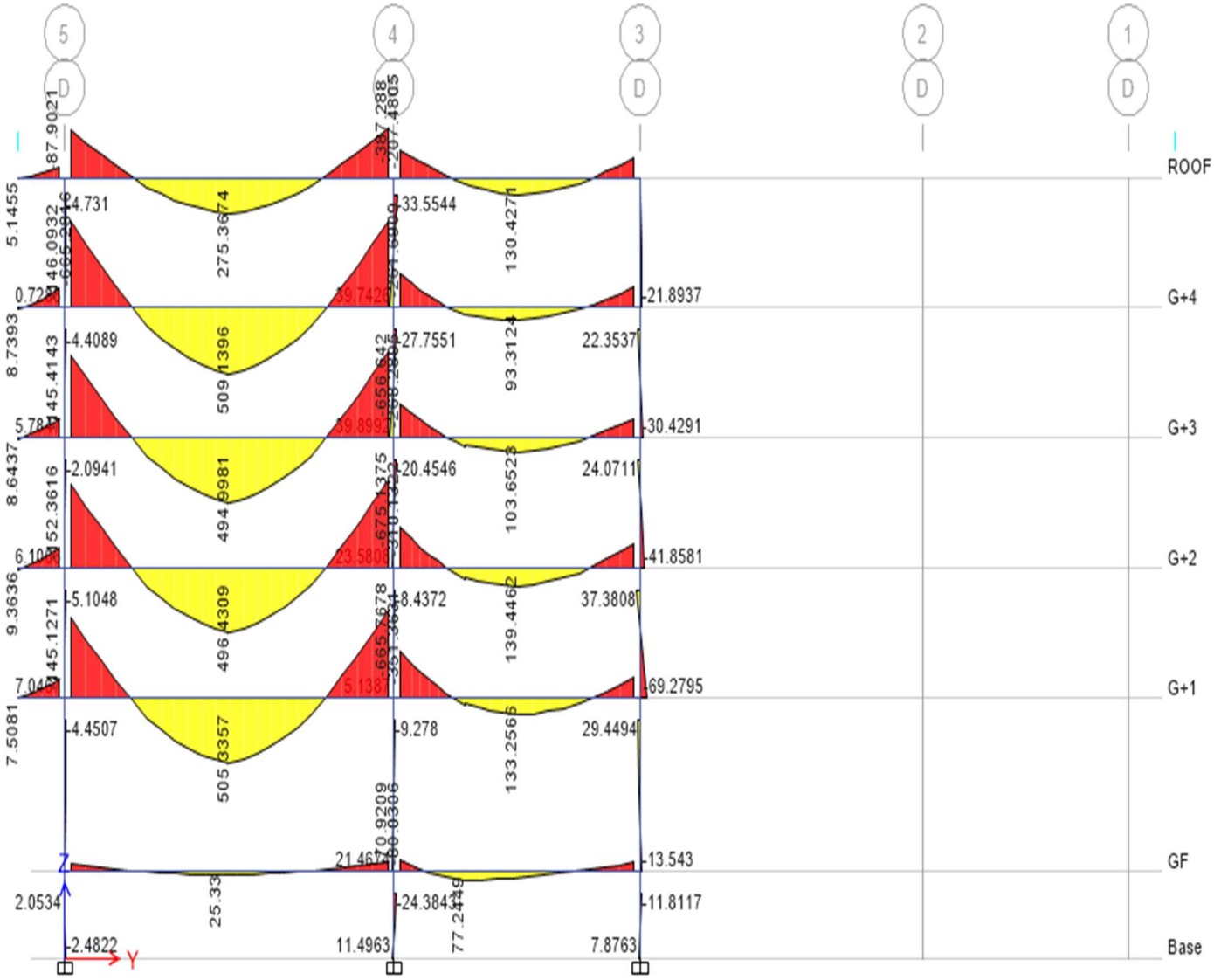
Axis B												
3rd floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	5	76.4	400	500	0.0382	482.534	single	286	8000	455.201	455.201	2.26513	3Ø16
span	5-4	518.45	400	500	0.2592	322.859	double	286	8000	4616.7	4616.7	22.9732	23Ø16
support	4	47.4	400	500	0.0237	489.314	single	286	8000	278.502	286	1.42317	2Ø16
span	4-3	196.38	400	500	0.0982	452.081	single	286	8000	1248.87	1248.87	6.21454	7Ø16
support	3	23.8	400	500	0.0119	494.693	single	286	8000	138.318	286	1.42317	2Ø16
span	3-2	223.45	400	500	0.1117	444.552	single	286	8000	1445.09	1445.09	7.19094	8Ø16
support	2	63.66	400	500	0.0318	485.537	single	286	8000	376.949	376.949	1.87574	2Ø16
span	2-1	125.76	400	500	0.0629	470.516	single	286	8000	768.434	768.434	3.82381	4Ø16
support	1	2.75	400	500	0.0014	499.393	single	286	8000	15.8317	286	1.42317	2Ø16
Axis B												
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	5	32.71	400	500	0.0164	492.676	single	286	8000	190.878	286	1.42317	2Ø16
span	5-4	534.23	400	500	0.2671	309.731	double	286	8000	4958.86	4958.86	24.6759	25Ø16
support	4	106.98	400	500	0.0535	475.164	single	286	8000	647.287	647.287	3.22098	3Ø16
span	4-3	191.16	400	500	0.0956	453.501	single	286	8000	1211.87	1211.87	6.03041	6Ø16
support	3	72.99	400	500	0.0365	483.342	single	286	8000	434.157	434.157	2.16042	2Ø16
span	3-2	322.6	400	500	0.1613	414.052	single	286	8000	2239.99	2239.99	11.1465	11Ø16
support	2	62.33	400	500	0.0312	485.848	single	286	8000	368.837	368.837	1.83538	2Ø16
span	2-1	121.92	400	500	0.0610	471.474	single	286	8000	743.456	743.456	3.69952	4Ø16
support	1	2.87	400	500	0.0014	499.366	single	286	8000	16.5235	286	1.42317	2Ø16
Axis B												
top tie beam												
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	5	88.49	400	500	0.0442	479.649	single	286	8000	530.407	530.407	2.63936	3Ø16
span	5-4	275.05	400	500	0.1375	429.328	single	286	8000	1841.88	1841.88	9.16538	9Ø16
support	4	38.71	400	500	0.0194	491.309	single	286	8000	226.52	286	1.42317	2Ø16
span	4-3	129.29	400	500	0.0646	469.631	single	286	8000	791.491	791.491	3.93855	4Ø16
support	3	27.9	400	500	0.0140	493.767	single	286	8000	162.45	286	1.42317	2Ø16
span	3-2	195.97	400	500	0.0980	452.193	single	286	8000	1245.96	1245.96	6.20003	6Ø16
support	2	14.7	400	500	0.0074	496.735	single	286	8000	85.0805	286	1.42317	2Ø16
span	2-1	87.12	400	500	0.0436	479.977	single	286	8000	521.837	521.837	2.59672	3Ø16
support	1	4.6	400	500	0.0023	498.983	single	286	8000	26.5039	286	1.42317	2Ø16


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Axis c												
grade beam design												
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	5	3.8	400	500	0.0019	499.16	single	286	8000	21.8868	286	0.91083	2Ø16
span	5-4	23.75	400	500	0.0119	494.704	single	286	8000	138.024	286	0.91083	2Ø16
support	4	10.99	400	500	0.0055	497.563	single	286	8000	63.502	286	0.91083	2Ø16
span	4-3	83.2	400	500	0.0416	480.916	single	286	8000	497.385	497.385	1.58403	2Ø16
support	3	41.18	400	500	0.0206	490.743	single	286	8000	241.251	286	0.91083	2Ø16
span	3-2	20.85	400	500	0.0104	495.357	single	286	8000	121.011	286	0.91083	2Ø16
support	2	32.27	400	500	0.0161	492.776	single	286	8000	188.273	286	0.91083	2Ø16
span	2-1	13.67	400	500	0.0068	496.966	single	286	8000	79.0824	286	0.91083	2Ø16
support	1	10.42	400	500	0.0052	497.69	single	286	8000	60.193	286	0.91083	2Ø16

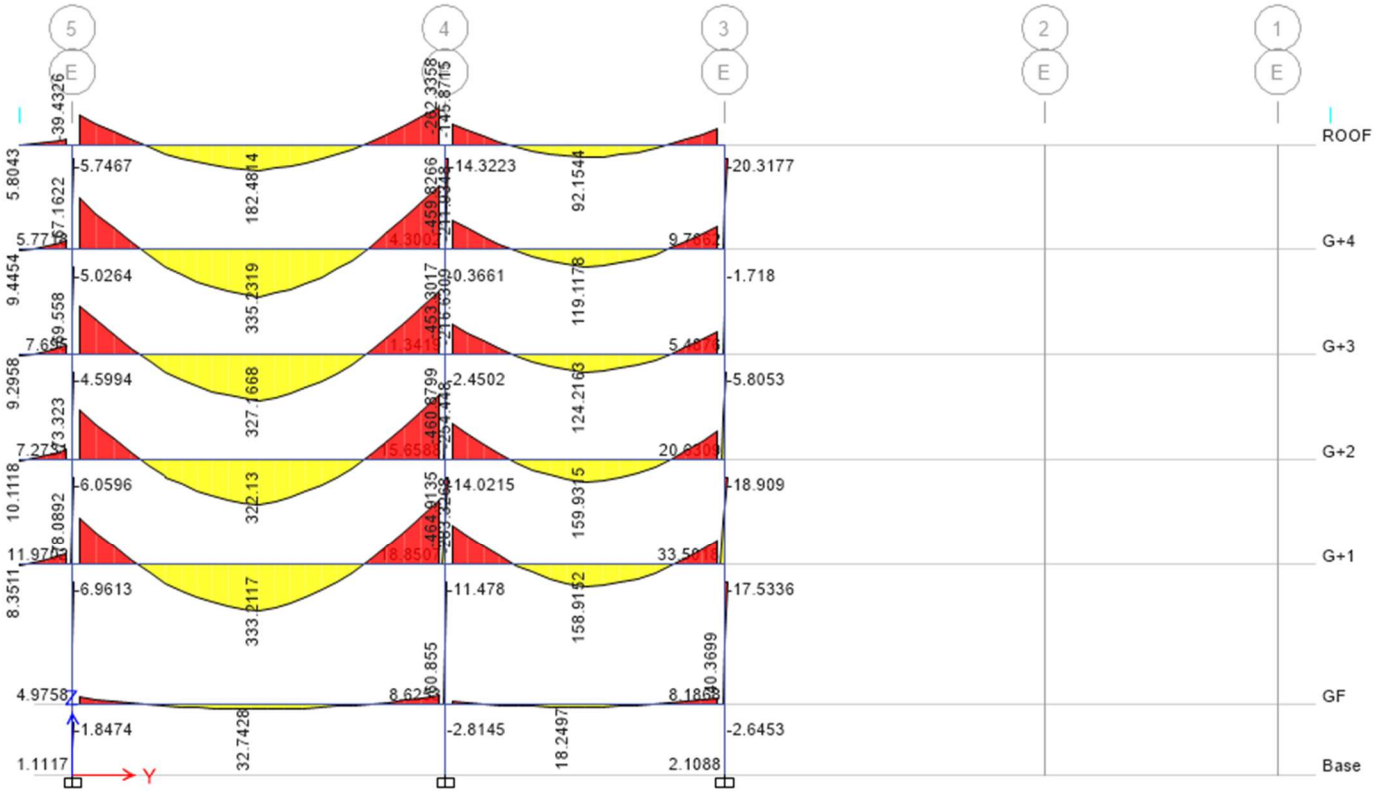
Axis c												
1st floor beam design												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	5	12.8	400	500	0.0064	497.16	single	286	8000	74.0205	286	0.91083	2Ø16
span	5-4	493.74	400	500	0.2469	339.634	double	286	8000	4179.5	4179.5	13.3105	14Ø16
support	4	24.42	400	500	0.0122	494.553	single	286	8000	141.962	286	0.91083	2Ø16
span	4-3	119.71	400	500	0.0599	472.024	single	286	8000	729.129	729.129	2.32207	3Ø16
support	3	194.72	400	500	0.0974	452.534	single	286	8000	1237.08	1237.08	3.93974	4Ø16
span	3-2	202.94	400	500	0.1015	450.283	single	286	8000	1295.75	1295.75	4.12658	4Ø16
support	2	144.93	400	500	0.0725	465.667	single	286	8000	894.788	894.788	2.84964	3Ø16
span	2-1	74.85	400	500	0.0374	482.902	single	286	8000	445.627	445.627	1.41919	2Ø16
support	1	23.64	400	500	0.0118	494.729	single	286	8000	137.378	286	0.91083	2Ø16
Axis c												
2nd floor beam design												
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	5	157.61	400	500	0.0788	462.4	single	286	8000	979.95	979.95	3.12086	3Ø16
span	5-4	534.82	400	500	0.2674	309.183	double	286	8000	4973.13	4973.13	15.838	16Ø16
support	4	28.81	400	500	0.0144	493.561	single	286	8000	167.819	286	0.91083	2Ø16
span	4-3	123.42	400	500	0.0617	471.1	single	286	8000	753.2	753.2	2.39873	3Ø16
support	3	151.58	400	500	0.0758	463.96	single	286	8000	939.289	939.289	2.99137	3Ø16
span	3-2	158.79	400	500	0.0794	462.093	single	286	8000	987.942	987.942	3.14631	3Ø16
support	2	104.62	400	500	0.0523	475.741	single	286	8000	632.24	632.24	2.0135	2Ø16
span	2-1	80.8	400	500	0.0404	481.488	single	286	8000	482.463	482.463	1.53651	2Ø16
support	1	27.63	400	500	0.0138	493.828	single	286	8000	160.858	286	0.91083	2Ø16
Axis c												
3rd floor beam design												
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	5	147.76	400	500	0.0739	464.942	single	286	8000	913.683	913.683	2.90982	3Ø16
span	5-4	530	400	500	0.2650	313.517	double	286	8000	4860.19	4860.19	15.4783	15Ø16
support	4	33.02	400	500	0.0165	492.606	single	286	8000	192.715	286	0.91083	3Ø16
span	4-3	120.87	400	500	0.0604	471.735	single	286	8000	736.645	736.645	2.346	3Ø16
support	3	186.63	400	500	0.0933	454.725	single	286	8000	1179.97	1179.97	3.75786	4Ø16
span	3-2	191.5	400	500	0.0958	453.409	single	286	8000	1214.27	1214.27	3.86711	4Ø16
support	2	146.83	400	500	0.0734	465.181	single	286	8000	907.467	907.467	2.89002	3Ø16
span	2-1	108.95	400	500	0.0545	474.681	single	286	8000	659.878	659.878	2.10152	2Ø16
support	1	42.86	400	500	0.0214	490.358	single	286	8000	251.291	286	0.91083	2Ø16

Axis c												
4th floor beam design												
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	5	147.44	400	500	0.0737	465.024	single	286	8000	911.543	911.543	2.903	3Ø16
span	5-4	544.78	400	500	0.2724	299.03	double	286	8000	5237.74	5237.74	16.6807	15Ø16
support	4	28.06	400	500	0.0140	493.731	single	286	8000	163.394	286	0.91083	2Ø16
span	4-3	121.66	400	500	0.0608	471.539	single	286	8000	741.768	741.768	2.36232	3Ø16
support	3	170.98	400	500	0.0855	458.899	single	286	8000	1071.19	1071.19	3.41143	4Ø16
span	3-2	197.41	400	500	0.0987	451.8	single	286	8000	1256.21	1256.21	4.00066	4Ø16
support	2	145.04	400	500	0.0725	465.639	single	286	8000	895.522	895.522	2.85198	3Ø16
span	2-1	108.01	400	500	0.0540	474.911	single	286	8000	653.867	653.867	2.08238	2Ø16
support	1	41.05	400	500	0.0205	490.773	single	286	8000	240.475	286	0.91083	2Ø16
Axis c												
roof floor beam design												
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	5	89.54	400	500	0.0448	479.396	single	286	8000	536.983	536.983	1.71014	2Ø16
span	5-4	294.95	400	500	0.1475	423.099	single	286	8000	2004.21	2004.21	6.38285	7Ø16
support	4	15.75	400	500	0.0079	496.501	single	286	8000	91.2008	286	0.91083	2Ø16
span	4-3	120.79	400	500	0.0604	471.755	single	286	8000	736.126	736.126	2.34435	3Ø16
support	3	103.95	400	500	0.0520	475.905	single	286	8000	627.975	627.975	1.99992	2Ø16
span	3-2	105.88	400	500	0.0529	475.433	single	286	8000	640.269	640.269	2.03907	2Ø16
support	2	114.83	400	500	0.0574	473.233	single	286	8000	697.619	697.619	2.22172	3Ø16
span	2-1	62.94	400	500	0.0315	485.705	single	286	8000	372.556	372.556	1.18648	2Ø16
support	1	33.12	400	500	0.0166	492.583	single	286	8000	193.308	286	0.91083	2Ø16


5
4
3
2
1
D
ROOF
G+4
G+3
G+2
G+1
GF
Base
-87.9021
-207.4805
-4.731
-33.5544
275.3674
130.4271
5.1455
0.7225
-21.8937
-4.4089
-27.7551
22.3537
509.1396
93.3124
8.7393
5.7811
-30.4291
-2.0941
-20.4546
24.0711
494.9981
103.6523
8.6437
6.1007
-41.8581
-5.1048
-8.4372
37.3808
496.4309
139.4462
9.3636
7.0447
-69.2795
-4.4507
-9.278
29.4494
505.3357
133.2566
7.5081
-13.543
2.0534
-24.3843
-11.8117
-2.4822
11.4963
7.8763
Z
Y

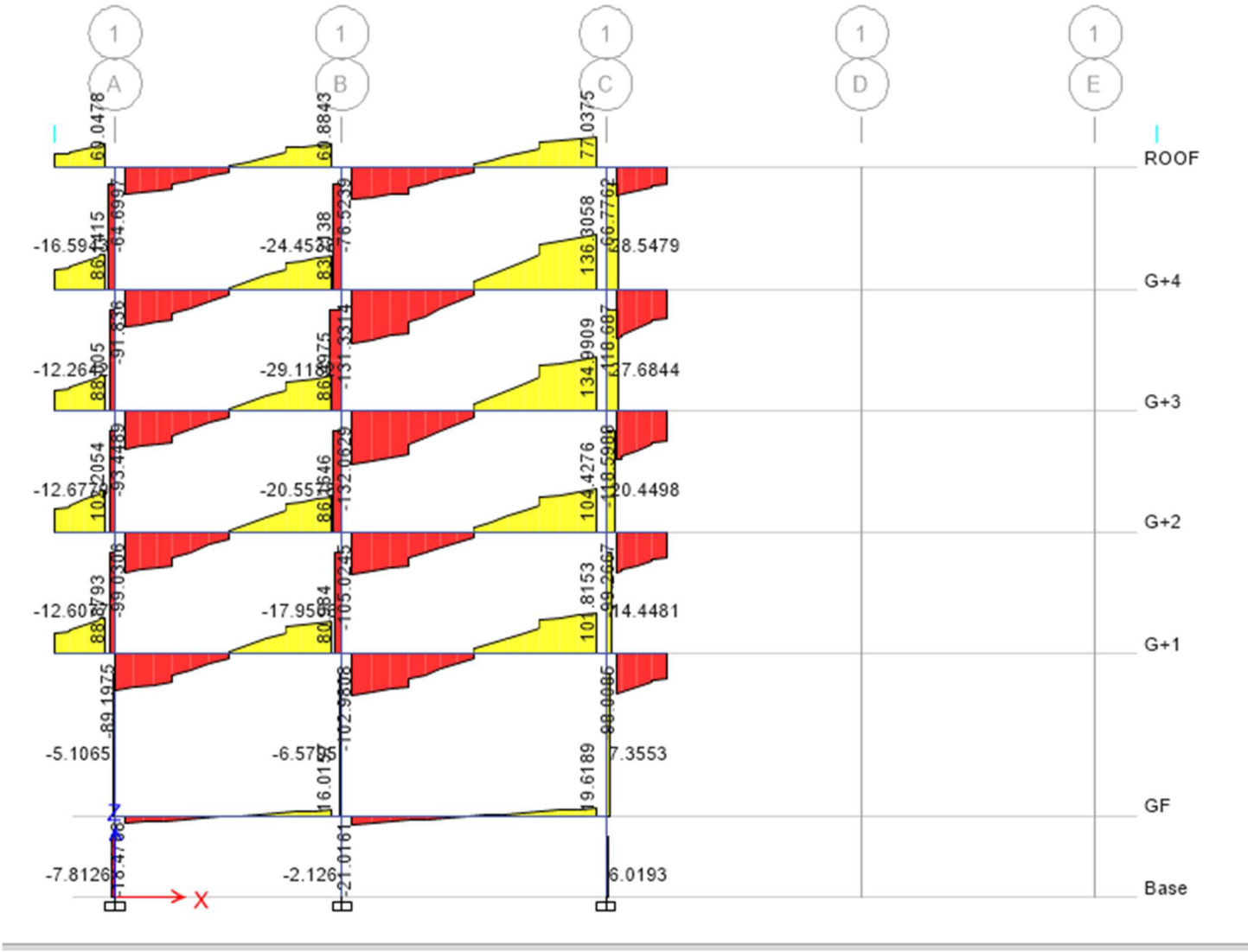
Axis D												
ground floor beam design												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	A	1.002	400	500	0.0005	499.779	single	286	8000	5.76405	286	0.91083	2Ø16
span	A-B	25.33	400	500	0.0127	494.348	single	286	8000	147.313	286	0.91083	2Ø16
support	B	21.46	400	500	0.0107	495.22	single	286	8000	124.586	286	0.91083	2Ø16
span	B-C	77.24	400	500	0.0386	482.335	single	286	8000	460.396	460.396	1.46623	2Ø16
support	C	13.54	400	500	0.0068	496.995	single	286	8000	78.3258	286	0.91083	2Ø16
Axis D													
1st floor beam design													
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	145.13	400	500	0.0726	465.616	single	286	8000	896.122	896.122	2.85389	3Ø16
span	A-B	505.33	400	500	0.2527	332.194	double	286	8000	4373.43	4373.43	13.9281	14Ø16
support	B	5.14	400	500	0.0026	498.863	single	286	8000	29.6223	286	0.91083	2Ø16
span	B-C	133.25	400	500	0.0666	468.634	single	286	8000	817.469	817.469	2.6034	3Ø16
support	C	29.45	400	500	0.0147	493.416	single	286	8000	171.597	286	0.91083	2Ø16
Axis D													
2nd floor beam design													
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	152.36	400	500	0.0762	463.759	single	286	8000	944.532	944.532	3.00806	3Ø16
span	A-B	496.43	400	500	0.2482	337.963	double	286	8000	4223.05	4223.05	13.4492	14Ø16
support	B	23.58	400	500	0.0118	494.742	single	286	8000	137.026	286	0.91083	2Ø16
span	B-C	139.45	400	500	0.0697	467.064	single	286	8000	858.38	858.38	2.73369	3Ø16
support	C	37.38	400	500	0.0187	491.612	single	286	8000	218.602	286	0.91083	2Ø16
Axis D													
3rd floor beam design													
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	145.41	400	500	0.0727	465.545	single	286	8000	897.989	897.989	2.85984	3Ø16
span	A-B	494.99	400	500	0.2475	338.862	double	286	8000	4199.64	4199.64	13.3746	14Ø16
support	B	39.99	400	500	0.0200	491.016	single	286	8000	234.15	286	0.91083	2Ø16
span	B-C	103.65	400	500	0.0518	475.978	single	286	8000	626.066	626.066	1.99384	2Ø16
support	C	24.07	400	500	0.0120	494.632	single	286	8000	139.905	286	0.91083	2Ø16
Axis D													
4th floor beam design													
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	146.1	400	500	0.0731	465.368	single	286	8000	902.592	902.592	2.8745	3Ø16
span	A-B	509.14	400	500	0.2546	329.596	double	286	8000	4441.13	4441.13	14.1437	15Ø16
support	B	39.74	400	500	0.0199	491.073	single	286	8000	232.659	286	0.91083	2Ø16
span	B-C	93.31	400	500	0.0467	478.488	single	286	8000	560.654	560.654	1.78552	2Ø16
support	C	22.35	400	500	0.0112	495.019	single	286	8000	129.806	286	0.91083	2Ø16

Axis D												
roof floor beam design												
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	87.9	400	500	0.0440	479.79	single	286	8000	526.715	526.715	1.67744	3Ø16
span	A-B	275.37	400	500	0.1377	429.23	single	286	8000	1844.44	1844.44	5.87402	14Ø16
support	B	387.3	400	500	0.1937	390.627	double	286	8000	2850.51	2850.51	9.07807	2Ø16
span	B-C	130.43	400	500	0.0652	469.344	single	286	8000	798.958	798.958	2.54445	2Ø16
support	C	22.35	400	500	0.0112	495.019	single	286	8000	129.806	286	0.91083	2Ø16



Axis E													
ground floor beam design													
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	4.97	400	500	0.0025	498.901	single	286	8000	28.6404	286	0.91083	2Ø16
span	A-B	32.74	400	500	0.0164	492.669	single	286	8000	191.056	286	0.91083	2Ø16
support	B	8.62	400	500	0.0043	498.091	single	286	8000	49.755	286	0.91083	2Ø16
span	B-C	18.25	400	500	0.0091	495.941	single	286	8000	105.796	286	0.91083	2Ø16
support	C	8.18	400	500	0.0041	498.189	single	286	8000	47.206	286	0.91083	2Ø16

Axis E												
1st floor beam design												
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	78.08	400	500	0.0390	482.135	single	286	8000	465.595	465.595	1.48279	2Ø16
span	A-B	333.21	400	500	0.1666	410.446	single	286	8000	2334	2334	7.43311	8Ø16
support	B	18.85	400	500	0.0094	495.806	single	286	8000	109.304	286	0.91083	2Ø16
span	B-C	158.91	400	500	0.0795	462.062	single	286	8000	988.756	988.756	3.1489	3Ø16
support	C	33.58	400	500	0.0168	492.478	single	286	8000	196.034	286	0.91083	2Ø16
Axis E													
2nd floor beam design													
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	73.32	400	500	0.0367	483.264	single	286	8000	436.191	436.191	1.38914	2Ø16
span	A-B	322.13	400	500	0.1611	414.21	single	286	8000	2235.88	2235.88	7.12063	7Ø16
support	B	15.66	400	500	0.0078	496.521	single	286	8000	90.676	286	0.91083	2Ø16
span	B-C	159.93	400	500	0.0800	461.796	single	286	8000	995.674	995.674	3.17094	3Ø16
support	C	20.63	400	500	0.0103	495.406	single	286	8000	119.722	286	0.91083	2Ø16
Axis E													
3rd floor beam design													
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	69.56	400	500	0.0348	484.151	single	286	8000	413.063	413.063	1.31549	2Ø16
span	A-B	327.17	400	500	0.1636	412.509	single	286	8000	2280.23	2280.23	7.26187	7Ø16
support	B	1.37	400	500	0.0007	499.698	single	286	8000	7.88227	286	0.91083	2Ø16
span	B-C	124.21	400	500	0.0621	470.903	single	286	8000	758.338	758.338	2.41509	3Ø16
support	C	5.48	400	500	0.0027	498.788	single	286	8000	31.5866	286	0.91083	2Ø16
Axis E													
4th floor beam design													
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	67.16	400	500	0.0336	484.716	single	286	8000	398.347	398.347	1.26862	2Ø16
span	A-B	335.23	400	500	0.1676	409.75	double	286	8000	2352.13	2352.13	7.49087	8Ø16
support	B	4.3	400	500	0.0022	499.05	single	286	8000	24.7721	286	0.91083	2Ø16
span	B-C	119.12	400	500	0.0596	472.17	single	286	8000	725.311	725.311	2.30991	3Ø16
support	C	9.78	400	500	0.0049	497.833	single	286	8000	56.4798	286	0.91083	2Ø16
Axis E													
Roof floor beam design													
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	39.43	400	500	0.0197	491.144	single	286	8000	230.811	286	0.91083	2Ø16
span	A-B	182.48	400	500	0.0912	455.84	single	286	8000	1150.91	1150.91	3.66531	4Ø16
support	B	4.3	400	500	0.0022	499.05	single	286	8000	24.7721	286	0.91083	2Ø16
span	B-C	92.15	400	500	0.0461	478.768	single	286	8000	553.361	553.361	1.7623	2Ø16
support	C	9.78	400	500	0.0049	497.833	single	286	8000	56.4798	286	0.91083	2Ø16

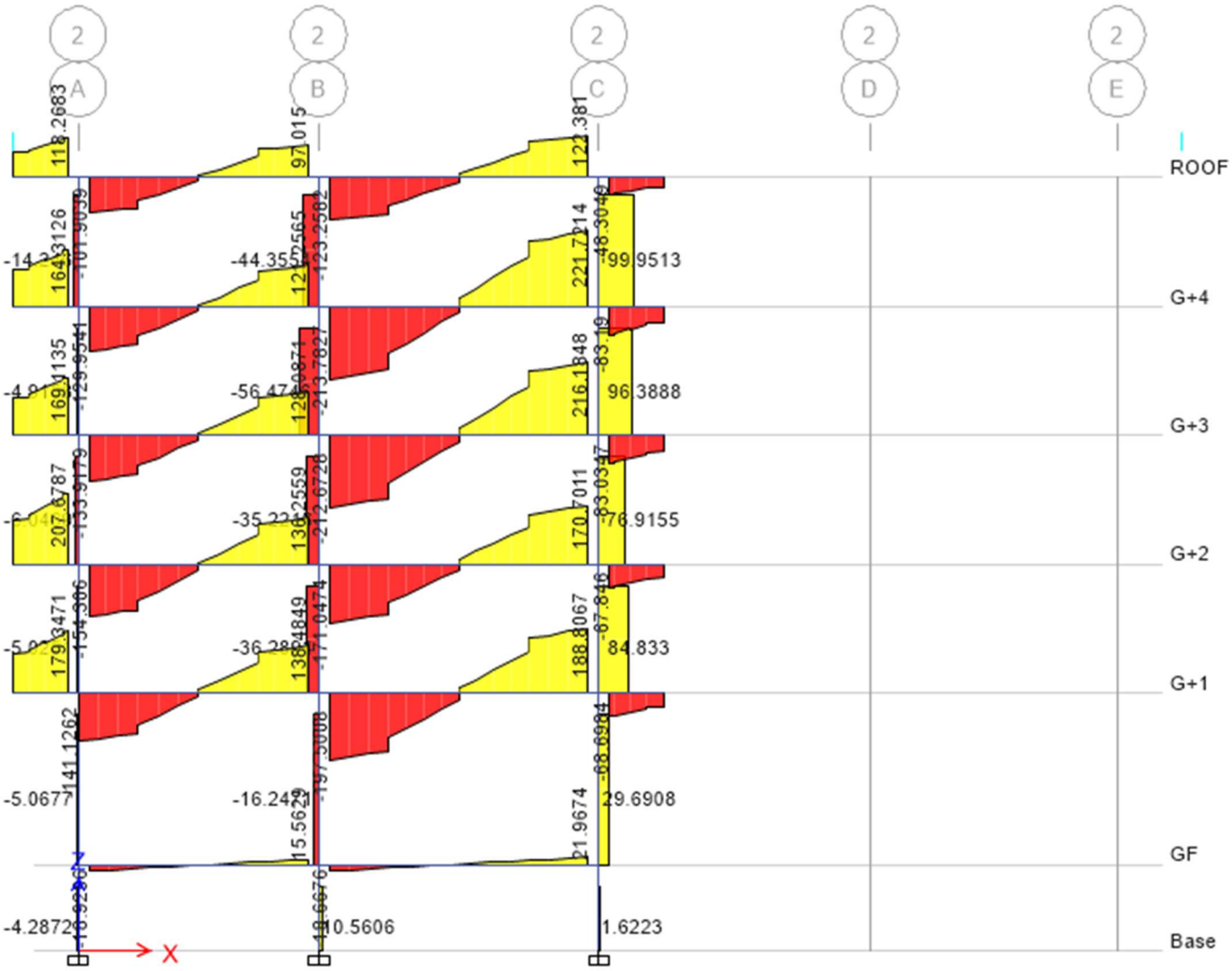

1
A
1
B
1
C
1
D
1
E
ROOF
G+4
G+3
G+2
G+1
GF
Base
-16.594
-24.452
136.3058
8.5479
-12.264
-29.118
134.9909
7.6844
-12.672
-20.557
104.4276
20.4498
-12.608
-17.954
101.8153
14.4481
-5.1065
-6.577
19.6189
7.3553
-7.8126
-2.126
6.0193
X

Beam Shear check at axis 1

axis 1							
			Roof				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	64.7	50.15	337.5	551.3	219.4	337.5	
B-C	77.03	61.68	337.5	448.2	219.4	337.5	

axis 1							
			G+4				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	91.83	74.38	337.5	371.7	219.4	337.5	
B-C	5.71	5.71	337.5	4841.6	219.4	337.5	

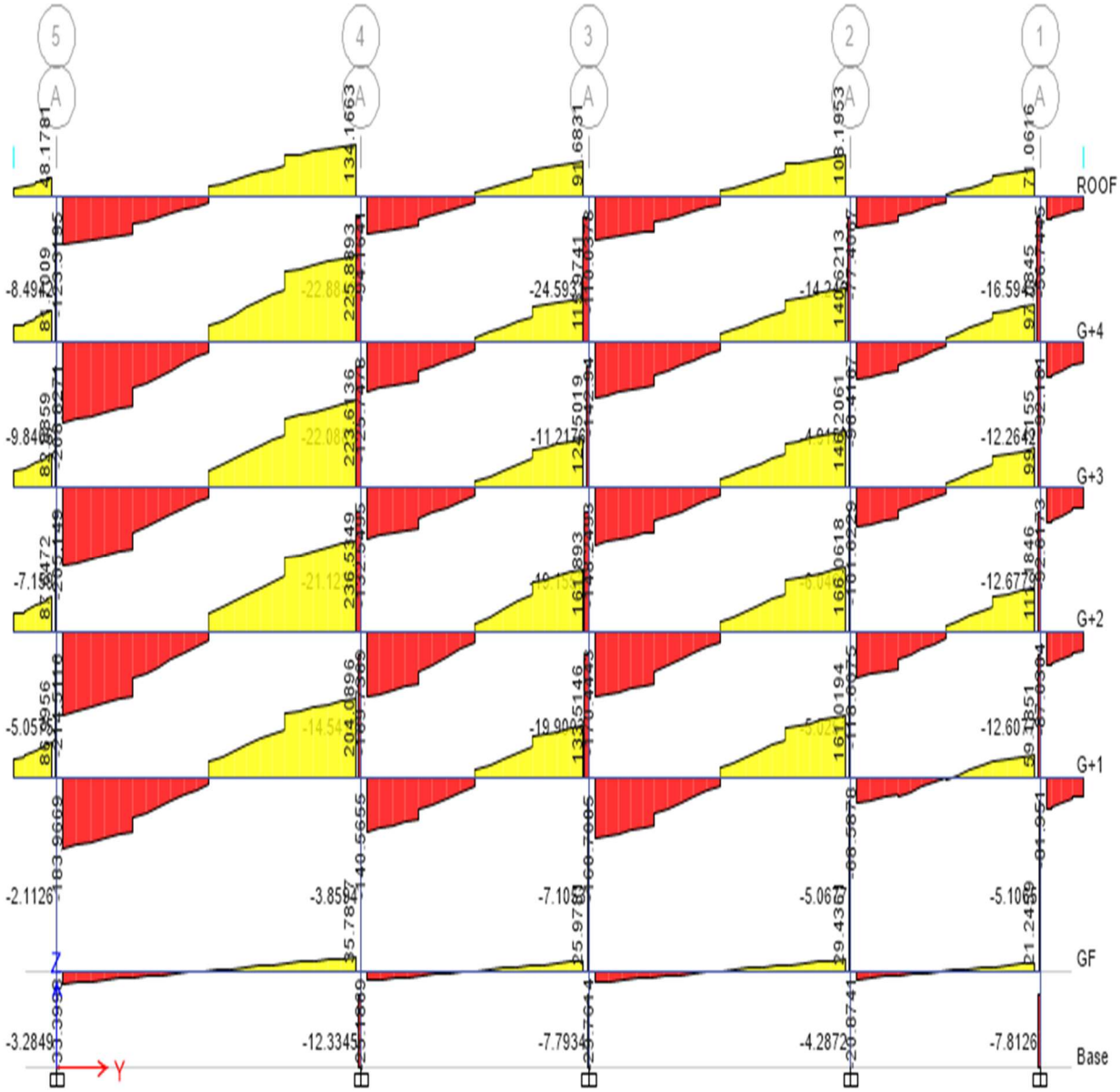
axis 1							
			G+3				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	93.44	72.59	337.5	380.8	219.4	337.5	
B-C	134.9	108.2	337.5	255.5	219.4	255.5	
axis 1							
			G+2				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	99.03	77.53	337.5	356.6	219.4	337.5	
B-C	105.02	84.08	337.5	328.8	219.4	328.8	
axis 1							
			G+1				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	89.2	69.45	337.5	398.1	219.4	337.5	
B-C	102.98	82.5	337.5	335.1	219.4	335.1	
axis 1							
			GF				
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	18.47	14.47	337.5	1910.5	219.4	337.5	
B-C	21.02	16.96	337.5	1630.1	219.4	337.5	



Beam Shear check at axis 2

axis 2							
ROOF							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	101.9	78.81	337.5	350.8	219.4	337.5	
B-C	123.25	98.69	337.5	280.1	219.4	280.1	
axis 2							
G+4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	129.9	100.75	337.5	274.4	219.4	274.4	
B-C	221.72	178.17	337.5	155.2	219.4	155.2	
axis 2							
G+3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	133.9	103.49	337.5	267.1	219.4	267.1	
B-C	216.18	173.3	337.5	159.5	219.4	159.5	
axis 2							
G+2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	154.3	120.58	337.5	229.3	219.4	229.3	
B-C	171.05	136.88	337.5	202.0	219.4	202.0	
axis 2							
G+1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	141.12	108.68	337.5	254.4	219.4	254.4	
B-C	197.5	158.87	337.5	174.0	219.4	174.0	
axis							
GF							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
A-B	18.92	14.92	337.5	1852.9	219.4	337.5	
B-C	21.96	17.9	337.5	1544.4	219.4	337.5	

Beam Shear check at axis A



axis A							
ROOF							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
5-4	134.7	-23.72	337.5	-1165.5	219.4	-1165.5	
4-3	94.6	78.87	337.5	350.5	219.4	337.5	
3-2	110.04	94.62	337.5	292.2	219.4	292.2	
2-1	77.41	13.07	337.5	2115.2	219.4	337.5	
axis A							
G+4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
5-4	225.83	-40.7	337.5	-679.3	219.4	-679.3	
4-3	125.75	105.86	337.5	261.2	219.4	261.2	
3-2	142.94	122.91	337.5	224.9	219.4	224.9	
2-1	96.41	23.84	337.5	1159.6	219.4	337.5	
axis A							
G+3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
5-4	223.61	-39.31	337.5	-703.3	219.4	-703.3	
4-3	132.54	111.31	337.5	248.4	219.4	248.4	
3-2	148.24	127.44	337.5	216.9	219.4	216.9	
2-1	101.02	14.28	337.5	1936.0	219.4	337.5	
axis A							
G+2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
5-4	236.53	-40.04	337.5	-690.5	219.4	-690.5	
4-3	163.73	136.94	337.5	201.9	219.4	201.9	
3-2	161.02	138.29	337.5	199.9	219.4	199.9	
2-1	118.01	18.69	337.5	1479.2	219.4	337.5	
axis A							
G+1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
5-4	204.1	-33.86	337.5	-816.5	219.4	-816.5	
4-3	140.56	118.01	337.5	234.3	219.4	234.3	
3-2	161.7	146.67	337.5	188.5	219.4	188.5	
2-1	88.58	24.55	337.5	1126.1	219.4	337.5	
axis A							
GF							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	remark
5-4	35.78	-6.64	337.5	-4163.5	219.4	-4163.5	
4-3	25.97	21.76	337.5	1270.5	219.4	337.5	
3-2	29.76	25.58	337.5	1080.8	219.4	337.5	
2-1	21.24	2.99	337.5	9246.0	219.4	337.5	

Appendix F; design of Column

Table 59 column design for group 1 (C1,C10;C11) select the maximum axial load value (C1)

Effective length determinaton

		column size		height	effective length	
	Column	H	B	L	L0x	L0y
	Foundation	0.5	0.4	4	3.00	3.00
c1	Ground	0.5	0.4	3	3.00	3.00
	1st	0.5	0.4	3	3.00	3.00
	2nd	0.5	0.4	3	3.00	3.00
	3rd	0.5	0.4	3	3.00	3.00
	4th	0.5	0.4	3	3.00	3.00

Table 60 Check second order effect for C1

		axial load	m2-2miner axises		m3-3 major axises		First order moment calculation including geometric imperfaction						check slendernees		
		P	Mx-xT	MX-XB	MY-YT	MY-YB	M01,X	M02,X	M0e,X	M01,y	M02,y	M0e,y	λ	λlim	2nd order effect
	Foundation	1938.35	5.49	6.23	6.23	5.49	44.26	45.00	44.70	44.26	45.00	44.70	1.88	13.63	Neglected
	ground	1885.13	7.05	10.82	10.82	7.05	44.75	48.52	47.01	44.75	48.52	47.01	1.88	17.70	Neglected
c1	1st	1482.1	2.73	18.79	18.7	12.73	32.37	48.43	42.01	42.37	48.34	45.95	1.88	29.60	Neglected
	2nd	1059.91	4.76	16.93	16.93	14.76	25.96	38.13	33.26	35.96	38.13	37.26	1.88	31.95	Neglected
	3rd	664.05	5.1	18.5	18.51	12.16	18.38	31.78	26.42	25.44	31.79	29.25	1.88	40.52	Neglected
	4th	274.29	0.74	22.41	22.4	20.73	6.23	27.90	19.23	26.22	27.89	27.22	1.88	73.79	Neglected

Table 61 Longitudinal Reinforcement for C1

	Column b\n elevation	NED	Msd,x	Msd,y	Vsd	μsd,X	μsd,y	ω	Ø	As,req	As,min	As,max	As	Provide
	Foundation	1938.35	45.00	45.00	0.86	0.04	0.04	0.24	16	1563.66	400	16000	200.96	7.78
c1	ground	1885.13	48.52	48.52	0.83	0.04	0.04	0.24	16	1563.66	400	16000	200.96	7.78
	1st	1482.1	48.43	48.34	0.65	0.04	0.04	0.25	16	1628.81	400	16000	200.96	8.11
	2nd	1059.91	38.13	38.13	0.47	0.03	0.03	0.26	16	1693.96	400	16000	200.96	8.43
	3rd	664.05	31.78	31.79	0.29	0.03	0.03	0.39	16	2540.94	400	16000	200.96	12.64
	4th	274.29	27.90	27.89	0.12	0.02	0.02	0.3	16	1954.57	400	16000	200.96	9.73

Table 62 Lateral Reinforcement calculation for C1

		Under normal Length				
	Column	Ø	Spacing	Provide	Ø	Spacing
	Foundation	8	300	Provide Ø8 C/C300mm	8	300
	ground	8	300	Provide Ø8 C/C300mm	8	300
c1	1st	8	300	Provide Ø8 C/C300mm	8	300
	2nd	8	300	Provide Ø8 C/C300mm	8	300
	3rd	8	300	Provide Ø8 C/C300mm	8	300
	4th	8	300	Provide Ø8 C/C300mm	8	300

Table 63 column design for group 2 (C2,C3;C4,C6,C7) select the maximum axial load value (C4)

Effective length determinaton

	column size		height	effective length	
column	H	b	L	L0x	L0y
Foundation	0.5	0.4	4	3.00	3.00
ground	0.5	0.4	3	3.00	3.00
1st	0.5	0.4	3	3.00	3.00
2nd	0.5	0.4	3	3.00	3.00
3rd	0.5	0.4	3	3.00	3.00
4th	0.5	0.4	3	3.00	3.00

Table 64 Check seconde order effect for C4

		axial load	m2-2miner axises			m3-3 maj	First order moment calculation including geometric imperfaction						check slendernees		
		P	Mx-xT	MX-XB	MY-YT	MY-YB	M01,X	M02,X	M0e,X	M01,y	M02,y	M0e,y	λ	λlim	2nd order
	Foundatio	2950.89	5.44	0.99	5.44	0.99	M01,Xmin	M0e,Xmax	m0ex	M01,y-ym	M01max,y	m0ey	λmax	λlim	Neglected
	ground	2868.15	5.03	12.71	5.03	12.71	60.01	64.46	62.68	60.01	64.46	62.68	1.88	-51.21	Neglected
c4	1st	2224.49	4.68	7.88	4.68	7.88	62.39	70.07	67.00	62.39	70.07	67.00	1.88	17.85	Neglected
	2nd	1554.57	8.7	6.41	8.71	6.41	49.17	52.37	51.09	49.17	52.37	51.09	1.88	17.19	Neglected
	3rd	981.29	6.6	3.62	3.67	6.62	37.50	39.79	38.88	37.50	39.80	38.88	1.88	6.37	Neglected
	4th	427.65	1.25	15.79	21.25	15.79	23.25	26.23	25.03	23.30	26.25	25.07	1.88	26.81	Neglected

Table 65 Longtudnal Reinforcement calculation C4

	Column b\n elevation	NED	Msd,x	Msd,y	Vsd	μsd,X	μsd,y	ω	Ø	As,req	As,min	As,max	As	Provide
	Foundation	2950.89	5.44	5.44	1.30	0.005	0.005	0.52	16	3387.92	400	16000	200.96	16.86
c4	ground	2868.15	12.71	12.71	1.27	0.011	0.011	0.5	16	3257.62	400	16000	200.96	16.21
	1st	2224.49	7.88	7.88	0.98	0.007	0.007	0.28	16	1824.27	400	16000	200.96	9.08
	2nd	1554.57	8.71	8.71	0.69	0.008	0.008	0.25	16	1628.81	400	16000	200.96	8.11
	3rd	981.29	6.62	6.62	0.43	0.006	0.006	0.24	16	1563.66	400	16000	200.96	7.78
	4th	427.65	15.79	21.25	0.19	0.014	0.019	0.39	16	2540.94	400	16000	200.96	12.64

Table 66 Lateral Reinforcement calculation C4

	Under normal Length					
	Column	Ø	Spacing	Provide	Ø	Spacing
	Foundation	8	300	Provide Ø8 C/C300mm	8	300
	ground	8	300	Provide Ø8 C/C300mm	8	300
c4	1st	8	300	Provide Ø8 C/C300mm	8	300
	2nd	8	300	Provide Ø8 C/C300mm	8	300
	3rd	8	300	Provide Ø8 C/C300mm	8	300

	4th	8	300	Provide Ø8 C/C300mm	8	300

Table 67 column design for group 3 (C5,C7;C12,C18,C20) select the maximum axial load value (C20)

Effective length determinaton

		column size		height	effective length	
	column	H	B	L	L0x	L0y
	Foundation	0.5	0.4	4	3.00	3.00
c20	ground	0.5	0.4	3	3.00	3.00
	1st	0.5	0.4	3	3.00	3.00
	2nd	0.5	0.4	3	3.00	3.00
	3rd	0.5	0.4	3	3.00	3.00
	4th	0.5	0.4	3	3.00	3.00

Table 68 Check seconde order effect (C20)

		axial load	m2-2miner axises		m3-3 major axises		First order moment calculation including geometric imperfaction						check slendernees		
		P	Mx-xT	MX-XB	MY-YT	MY-YB	M01,X	M02,X	M0e,X	M01,y	M02,y	M0e,y	λ	λlim	2nd order
	Foundatic	4165.24	8.31	4.71	8.31	4.71	M01,Xmim	M0e,Xmax	m0ex	M01,y-ymi	M01max,y	m0ey	λmax	λlim	Neglected
	ground	4064.63	44.38	24.08	44.38	24.08	88.01	91.61	90.17	88.01	91.61	90.17	1.88	-0.73	Neglected
c20	1st	3137.25	33.44	86.72	33.44	86.72	105.37	125.67	117.55	105.37	125.67	117.55	1.88	-1.64	Neglected
	2nd	2257.19	50.39	47.84	50.39	47.84	96.19	149.47	128.15	96.19	149.47	128.15	1.88	17.20	Neglected
	3rd	1403.19	64.66	89.89	64.66	89.89	92.98	95.53	94.51	92.98	95.53	94.51	1.88	9.98	Neglected
	4th	557.27	41.52	84.48	41.52	84.48	92.72	117.95	107.86	92.72	117.95	107.86	1.88	19.19	Neglected

Table 69 Longtudnal Reinforcement calculation (C20)

	Column b\n elevation	NED	Msd,x	Msd,y	Vsd	μsd,X	μsd,y	ω	ø	As,req	As,min	As,max	as	Provide
	Foundation	4165.24	91.61	91.61	1.84	0.08	0.08	0.38	16	2475.79	400	16000	200.96	12.32
c2	ground	4064.63	125.67	125.67	1.79	0.11	0.11	0.4	16	2606.10	400	16000	200.96	12.97
	1st	3137.25	149.47	149.47	1.38	0.13	0.13	0.23	16	1498.50	400	16000	200.96	7.46
	2nd	2257.19	95.53	95.53	1.00	0.08	0.08	0.16	16	1042.44	400	16000	200.96	5.19
	3rd	1403.19	117.95	117.95	0.62	0.10	0.10	0.18	16	1172.74	400	16000	200.96	5.84
	4th	557.27	95.63	95.63	0.25	0.08	0.08	0.15	16	977.29	400	16000	200.96	4.86

Table 70 Lateral Reinforcement calculation (C20)

	Under normal Length					
	Column	Ø	Spacing	Provide	Ø	Spacing
	Foundation	8	300	Provide Ø8 C/C300mm	8	300
	ground	8	300	Provide Ø8 C/C300mm	8	300
c20	1st	8	300	Provide Ø8 C/C300mm	8	300
	2nd	8	300	Provide Ø8 C/C300mm	8	300
	3rd	8	300	Provide Ø8 C/C300mm	8	300
	4th	8	300	Provide Ø8 C/C300mm	8	300

Table 71 column design for group 4 (C8,C13;C15,C19) select the maximum axial load value (C13)

Effective length determinaton

		column size		height	effective length	
	column	H	B	L	L0x	L0y
	Foundation	0.5	0.4	4	3.00	3.00
c13	ground	0.5	0.4	3	3.00	3.00
	1st	0.5	0.4	3	3.00	3.00
	2nd	0.5	0.4	3	3.00	3.00
	3rd	0.5	0.4	3	3.00	3.00
	4th	0.5	0.4	3	3.00	3.00

Table 72 Check seconde order effect(C13)

		axial load	m2-2minor axises			m3-3 maj	First order moment calculation including geometric imperfaction						check slendernees		
		P	Mx-xT	MX-XB	MY-YT	MY-YB	M01,X	M02,X	M0e,X	M01,y	M02,y	M0e,y	λ	λlim	2nd order effect
	Foundatio	4842.56	0.05	5.06	0.04	5.06	M01,Xmin	M0e,Xma	m0ex	M01,y-ym	M01max,y	m0ey	λmax	λlim	Neglected
	ground	4732.79	41.35	24.13	4.35	24.13	96.90	101.91	99.91	96.89	101.91	99.90	1.88	17.83	Neglected
c13	1st	3771.21	56.93	97.62	56.93	97.38	118.79	136.01	129.12	99.01	118.79	110.87	1.88	16.19	Neglected
	2nd	2690.26	68.15	67.62	68.15	87.62	132.35	173.04	156.77	132.35	172.80	156.62	1.88	13.33	Neglected
	3rd	1660.05	77.21	109.69	77.2	109.69	121.43	121.96	121.74	121.96	141.43	133.64	1.88	13.03	Neglected
	4th	617.37	72.05	114.23	72.05	114.23	110.41	142.89	129.90	110.40	142.89	129.90	1.88	17.92	Neglected

Table 73 *Longitudinal Reinforcement calculation*(C13)

	Column b	NED	Msd,x	Msd,y	Vsd	μsd,X	μsd,y	ω	Ø	As,req	As,min	As,max	as	Provide
	Foundatio	4842.56	101.91	101.91	2.14	0.09	0.09	0.2	16	1303.05	400	16000	200.96	6.48
c13	ground	4732.79	136.01	118.79	2.09	0.12	0.10	0.31	16	2019.72	400	16000	200.96	10.05
	1st	3771.21	173.04	172.80	1.66	0.15	0.15	0.36	16	2345.49	400	16000	200.96	11.67
	2nd	2690.26	121.96	141.43	1.19	0.11	0.12	0.29	16	1889.42	400	16000	200.96	9.40
	3rd	1660.05	142.89	142.89	0.73	0.13	0.13	0.26	16	1693.96	400	16000	200.96	8.43
	4th	617.37	126.58	126.58	0.27	0.11	0.11	0.16	16	1042.44	400	16000	200.96	5.19

Table 74 Lateral Reinforcement calculation(C13)

	Under normal Length					
	Column	Ø	Spacing	Provide	Ø	Spacing
	Foundation	8	300	Provide Ø8 C/C300mm	8	300
	ground	8	300	Provide Ø8 C/C300mm	8	300
c13	1st	8	300	Provide Ø8 C/C300mm	8	300
	2nd	8	300	Provide Ø8 C/C300mm	8	300

	3rd	8	300	Provide Ø8 C/C300mm	8	300
	4th	8	300	Provide Ø8 C/C300mm	8	300

Table 75 column design for group 5 (C14, C17; C21) select the maximum axial load value (C14)

Effective length determinaton

		column size		height	effective length	
	column	H	b	L	L0x	L0y
	Foundation	0.4	0.4	4	3.00	3.00
c14	ground	0.4	0.4	3	3.00	3.00
	1st	0.4	0.4	3	3.00	3.00
	2nd	0.4	0.4	3	3.00	3.00
	3rd	0.4	0.4	3	3.00	3.00
	4th	0.4	0.4	3	3.00	3.00

Table 76Check seconde order effect(C14)

		axial load	m2-2miner axises		m3-3 major axises		First order moment calculation including geometric imperfaction						check slendernees		
		P	Mx-xT	MX-XB	MY-YT	MY-YB	M01,X	M02,X	M0e,X	M01,y	M02,y	M0e,y	λ	λlim	2nd order
	Foundatic	4048.46	10.99	9.37	10.99	9.37	M01,Xmin	M0e,Xmax	m0ex	M01,y-ym	M01max,y	m0ey	λmax	λlim	Neglected
	ground	3962.22	7.79	0	7.79	68.98	90.34	91.96	91.31	90.34	91.96	91.31	1.88	6.073027	Neglected
c14	1st	3144.87	17.56	24.42	17.56	24.42	79.24	87.03	83.92	87.03	148.22	123.75	1.88	12	Neglected
	2nd	2255.99	19.87	28.86	19.87	26.81	80.46	87.32	84.57	80.46	87.32	84.57	1.88	56	Neglected
	3rd	1391.34	23.14	33.01	23.14	33.02	64.99	73.98	70.38	64.99	71.93	69.15	1.88	14	Neglected
	4th	509.91	15.75	28.06	15.75	23.06	50.97	60.84	56.89	50.97	60.85	56.89	1.88	89	Neglected

Table 77 *Longtudnal Reinforcement calculation*(C14)

	Column b	NED	Msd,x	Msd,y	Vsd	μsd,X	μsd,y	ω	Ø	As,req	As,min	As,max	as	Provide
	Foundatio	4048.46	91.96	91.96	1.79	0.08	0.08	0.38	16	2475.79	400	16000	200.96	12.32
c14	ground	3962.22	87.03	148.22	1.75	0.08	0.13	0.38	16	2475.79	400	16000	200.96	12.32
	1st	3144.87	87.32	87.32	1.39	0.08	0.08	0.5	16	3257.62	400	16000	200.96	16.21
	2nd	2255.99	73.98	71.93	1.00	0.07	0.06	0.3	16	1954.57	400	16000	200.96	9.73
	3rd	1391.34	60.84	60.85	0.61	0.05	0.05	0.28	16	1824.27	400	16000	200.96	9.08
	4th	509.91	38.26	33.26	0.23	0.03	0.03	0.16	16	1042.44	400	16000	200.96	5.19

Table 78 Lateral Reinforcement calculation(C14)

	Under normal Length					
	Column	Ø	Spacing	Provide	Ø	Spacing
	Foundation	8	300	Provide Ø8 C/C300mm	8	300
	ground	8	300	Provide Ø8 C/C300mm	8	300
c14	1st	8	300	Provide Ø8 C/C300mm	8	300
	2nd	8	300	Provide Ø8 C/C300mm	8	300
	3rd	8	300	Provide Ø8 C/C300mm	8	300
	4th	8	300	Provide Ø8 C/C300mm	8	300