



Civil Engineering Department

Wolkite University, College of Engineering and Technology

Civil Engineering Department

Structural Design of B+G+5 Mixed-Use Building in Addis Ababa

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

Engineering has traditionally been taught by specialists as a series of separate courses, and it has been assumed that graduates will be able to design the knowledge and understanding gained from these project as required to undertake real world design projects.

A final year course at Wolkite university of Civil Engineering, thesis that design different types of building. It will develop various fundamental courses such as reinforcement concrete, structural design and foundation engineering to name a few. This project is a structural design of a B+G+5 mixed use building. The proposed building is located in Addis Ababa city. This report mainly focuses on the project of a B+G+5 mixed use (such shop, cafeteria and residential on this project) design to be this building safe and economical.

On this project the analysis and designing set ups on chapter, each chapters will contains objective, general introduction, design procedures, code provisions or requirements and design solution. Design of the project will be well interpreted in this document able to perform design of roof, design of slab, design of frame structure and design of foundation on its own chapter. The design philosophy adopt for the project will the limit state design for all aspects or parts of the structure according to Ethiopian Building Code of Standards in European Norms (ES EN).

Finally this document has discussion and conclusion by the person who did this project, also reference and appendix.

Acknowledgement

Our greatest thank from the depth of heart is to God for giving us courage, strength and mainly health throughout school time and full help provided by him for successful accomplishment of this final project for the B.Sc. Degree in civil engineering.

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TABLE OF CONTENT

CONTENT	PAGE NO
DECLARATION	i
Summary	ii
Acknowledgement	iii
LIST OF FIGURE.....	xi
ABBREVIATION	xiii
1 Introduction.....	1
1.1 General	1
1.1. Background	1
1.2. Objectives	1
1.3.1 Specific objective	1
1.3. Building types	2
1.4. Design criterion for building.....	3
1.5. Methodology	3
1.5.1. Study Area	4
1.5.2. Data Collections.....	4
1.6. Material Specification.....	5
1.7. Geotechnical data.....	6
1.8. Data Analysis and Design.....	7
2. WIND LOAD ANALYSIS AND ROOF DESIGN	8
2.1 WIND LOAD ANALYSIS.....	8
2.1.1 CASE1 WIND DIRECTION ($\Theta=0^\circ$).....	10
2.1.2 CASE 2 WIND DIRECTION ($\Theta=90^\circ$).....	14
2.2 DESIGN AND ANALYSIS OF EGA SHEET	16
2.2.1 LOAD COMBINATION	17
2.2.2 CALCULATION OF MOMENT AND REACTION.....	19
2.2.3 MOMENT FOR SERVICEABILITY.....	20
2.2.4 CHECK THE MOMENT CAPACITY.....	20

2.2.5 CHECK FOR DEFLECTION	21
2.3 DESIGN OF PURLIN	21
2.3.1 SELECTION OF SECTION	22
2.3.2 CHECK THE CROSS SECTION	22
2.3.3 DESIGN CAPACITY OF THE SECTION.....	23
2.4 TRUSS ANALYSIS AND DESIGN.....	24
2.4.1 CHECK SECTION CAPACITY IN TENSION FOR RAFTER	26
2.4.2 CHECK SECTION CAPACITY IN COMPRESSION FOR RAFTER	26
2.4.3 CHECK SECTION CAPACITY IN TENSION FOR VERTICAL AND DIAGONAL MEMBERS	27
2.4.4 CHECK SECTION CAPACITY IN COMPRESSION FOR VERTICAL AND DIAGONAL MEMBERS	28
3. SLAB DESIGN.....	30
3.1 SOLID SLAB ANALYSIS AND DESIGN	30
3.1.1 CLASSIFICATION OF PANELS	30
3.1.2 DESIGN OF TWO WAY SLABS USING COEFFICIENT METHOD	32
3.1.3 DEPTH DETERMINATION	32
Nominal cover.....	32
3.1.4 SLAB DEAD LOAD AND DESIGN LOAD CALCULATION	37
3.1.5 DESIGN MOMENT	39
3.1.6 SLAB DESIGN WITH STRIP METHOD	42
3.1.7 MOMENT ADJUSTMENT	48
3.1.8 SUPPORT MOMENT ADJUSTMENT	48
3.1.9 FILED MOMENT ADJUSTMENT.....	49
3.1.10 REINFORCEMENT	50
3.2 RIBBED SLAB DESIGN.....	59
3.2.1 DETERMINATION OF DESIGN LOAD	60
3.2.2 ANALYSIS AND DESIGN OF RIBS (JOISTS).....	62
3.2.3 REINFORCEMENT DESIGN.....	64

3.3 LOAD TRANSFER.....	69
3.3.1 FOR SOLID SLAB	69
3.3.2 Load transfer for ribbed slab to the Main beam	71
3.3 STAIR CASE DESIGN AND ANALYSIS	73
3.3.1 GEOMETRICAL CONFIGURATIONS	74
3.3.2 DEPTH FOR DEFLECTION.....	74
3.3.3 DETERMINATION OF LOADS	75
3.3.4 DETERMINATION OF MOMENT	79
3.3.5 DESIGN OF STAIRCASE FOR FLEXURE.....	80
3.3.6 DESIGN OF STAIRCASE FOR SHEAR.....	84
3.3.7 DESIGN OF STAIR 2.....	85
3.3.8 DESIGN STAIR 3.....	87
3.3.9 STAIR DESIGN FOR BASEMENT	90
4.0 DESIGN FOR EARTHQUAKE RESISTANCE.....	94
4.1 EARTH QUAKE ANALYSIS	94
4.1.1 DESIGN SPECTRUM FOR ELASTIC ANALYSIS	95
4.1.2 DETERMINATION OF CENTER OF MASS	96
4.1.3 STORY SHEAR DETERMINATION.....	96
5.0 FRAME ANALYSIS	98
5.1 ANALYSIS OF 3D FRAME.....	98
5.2 DESIGN AND ANALYSIS OF BEAM.....	100
5.2.1 FLEXURAL DESIGN OF THE BEAM.....	104
5.2.2 CHECK THE DEPTH FOR DEFLECTION (SERVICEABILITY REQUIREMENT).....	107
5.2.3 SHEAR DESIGN FOR THE BEAM	108
5.2.4 DESIGN FOR OTHER SECTION OF THE BEAM.....	110
5.3 ANALYSIS AND DESIGN OF COLUMN.....	114
5.3.1 DETERMINATION OF EFFECTIVE DEPTH.....	119
5.3.2 DESIGN EFFECTS.....	120
5.3.3 CHECKING FOR SLENDERNESS.....	121

6. FOUNDATION ANALYSES AND DESIGN	125
6.1 INTRODUCTION	125
6.2 DESIGN OF ISOLATED FOOTING	125
6.2.1 DEPTH DETERMINATION	129
6.2.2 REINFORCEMENT DESIGN.....	132
7.1 DESIGN OF SHEAR WALL.....	139
CONCLUSION.....	143
RECOMMENDATION	144
REFERENCE.....	145
APPENDIX.....	146

List of Table

Table 2. 1 cpe values in 0 degree	12
Table 2. 2 values for internal pressure coefficient	13
Table 2. 3 calculation of peak wind velocity	13
Table 2. 4 calculation of net wind pressure	14
Table 2. 5 values of cpe in 90 degree.....	15
Table 2. 6 Calculation of net wind pressure in 90 degree	16
Table 2. 7 tension and compression effect on the truss.....	25
Table 3. 1 classification of panels.....	31
Table 3. 2 structural class to find depth of the slab.....	34
Table 3. 3 depth determination	36
Table 3. 4 design load determination	38
Table 3. 5 Moment result	40
Table 3. 6 sample support moment adjustment.....	48
Table 3. 7 adjusted filed moment.....	49
Table 3. 8 slab filed reinforcement	52
Table 3. 9 3rd floor slab moment.....	56
Table 3. 10 3rd floor slab reinforcement.....	57
Table 3. 11 sample design load for ribbed slab.....	62
Table 3. 12 load transfer coefficient	69
Table 3. 13 load transfer from slab to beam by using coefficient method.....	70
Table 3. 14 Load transfer for 3rd floor solid slab	70
Table 3. 15 load transfer for ribbed slab	72
Table 4. 1 Type 1 elastic response spectra for ground type A	95
Table 4. 2 Center of mass of the Building	96
Table 4. 3 Story Shear.....	97
Table 5. 1 beam reinforcement	110
Table 5. 2 Internal forces on the column	116
Table 5. 3 the geometry of the columns and the beam.....	118
Table 5. 4 column design exl sample.....	123

Table 6. 1 axial load and moments	126
Table 6. 2 footing check.....	127
Table 6. 3 wide beam and punching shear check for isolated footing	132

List of Table Appendix

Table A. 1 depth determination for G and 1st floor solid slab.....	146
Table A. 2 slab design load.....	147
Table A. 3 adjusted support moment	149
Table A. 4 Adjusted filed moment.....	151
Table A. 5 Reinforcement for G and 1st solid slab.....	151
Table A. 6 slab depth for 3rd floor.....	154
Table A. 7 design load for 3rd floor sab	155
Table A. 8 moment adjustment for 3rd floor slab.....	156
Table A. 9 support reinforcement for 3rd floor slab	158
Table A. 10 Ribbed slab design load	159
Table A. 11 mass calculation for each floor	163
Table A. 12 beam reinforcement.....	187
Table A. 13 design of column.....	189
Table A. 14 foundation reaction and moment.....	202

LIST OF FIGURE

Figure 2. 1 Dou-pitch Roof Truss layouts.....	10
Figure 2. 2 rear view of the roof	10
Figure 2. 3 analysis of external pressure coefficient Cpe in 0 degree.....	11
Figure 2. 4 analysis of cpe in 90 degree.....	15
Figure 2. 5 EGA-500 Cross section	17
Figure 2. 6 analysis of EGA sheet for load combo5	19
Figure 2. 7 analysis of EGA sheet for load combo 2	19
Figure 2. 8 analysis of EGA sheet for load combo 6	20
Figure 2. 9 analysis of purlin	21
Figure 2. 10 SAP result for shear	21
Figure 2. 11 vertical deflection	24
Figure 2. 12 truss layout.....	24
Figure 2. 13 SAP 2000 result for roof analysis.....	25
Figure 2. 14 Sectional view of RT42	27
Figure 2. 15 Sectional view of RT32	29
Figure 3. 1 Layout of floor plan.....	30
Figure 3. 2 slab sectional view.....	37
Figure 3. 3 slab section for PVC tiles floor finish	37
Figure 3. 4 shows the type of slab and its support condition	39
Figure 3. 5 Loading condition for maximum span moment.....	41
Figure 3. 6 Loading condition for maximum support moment and maximum shear.....	42
Figure 3. 7 Panel layout for S-2	42
Figure 3. 8 load distribution on strip.....	43
Figure 3. 9 layout for strong band.....	45
Figure 3. 10 slab layout from 3rd floor.....	54
Figure 3. 11 layout of HCB for ribbed slab	60
Figure 3. 12 slab layout from 4th -5th floor.....	60
Figure 3. 13 flanged section of the ribbed slab	64
Figure 3. 14 Definition of l_o , for calculation of effective flange width.....	65
Figure 3. 15 plane view of stair one.....	73
Figure 3. 16 Sectional view of stair one.....	73
Figure 3. 17 flight and thread details	76
Figure 3. 18 Moment and shear for flight 1 for B to G.....	79

Figure 3. 19 Moment and shear for flight 2 for B to G.....	80
Figure 5. 1 building 3d model.....	98
Figure 5. 2 beam result.....	101
Figure 5. 3 5th floor beam shear force diagram.....	103
Figure 5. 4 5th floor beam moment diagram	104
Figure 5. 5 axial force result	114
Figure 5. 6 moment result	115
Figure 5. 7 2D and 3D column View.....	120
Figure 6. 1 stress distribution for isolated footing	129
Figure 6. 2 average stress distribution for isolated footing.....	129
Figure 6. 3 punching shear resistance layout	130
Figure 6. 4 wide beam shear resistance layout.....	131
Figure 6. 5 reinforcement detail.....	138

ABBREVIATION

Yc -concrete factor of safety	Ys -steel factor of safety
ES -elastic modules of steel	fyk -characteristic strength of steel
fyd -design strength of steel	We -external wind pressure
Wi -internal wind pressure	Wnet -net wind pressure
C(ze) -exposure coefficient	Cpe -external pressure coefficient
Cpi -internal wind pressure coefficient	Vref -reference wind velocity
ρ -density of air	Gk - dead load
qk -live load	Wk -wind load
Pd -Design Load	P -load
V –shear	M -moment
Lx -shorter direction of slab	Ly -longer direction of slab
T -Vibration Period	q-Behavior factor
Fb-base shear	VRD,c -Ultimate Shear Force
Vc -Shear Force resisted by a concrete	bw -web width
Qult -Ultimate net bearing capacity	fs -Factor of Safety
NED -Axial force	Ac -Area of concrete
S –Spacing	Sd(T) -Design Spectrum at period T
λ -Reduction factor	I -importance factor
deff -effective depth	HCB -Hollow Concrete Block
As -Area of topping reinforcement	Pi -interior Slab, i
Si –slab with hole, I	Φ -reinforcement steel diameter
Madj -Adjusted Moment	C20/25 25MPa -grade of concrete
S400 -400MPa grade of steel	Fe430 275MPa yield strength of
VED Shear force exerted by the external action	XC -Exposure class
fck -compressive characteristic strength of concrete	
fctm -tensile characteristic strength of concrete	

f_{cd} -compressive design strength of concrete

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

α_x and α_y -support bending moment coefficients for rectangular slabs supported on four sides in the X and Y

M_{xs} and M_{ys} -support bending moments in the X and Y

α_{xf} and α_{yf} -field bending moment coefficients for rectangular slabs supported on four sides in the X and Y

M_{xf} and M_{yf} -field bending moments in the X and Y

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

ρ_0 -the reference reinforcement ratio

ρ -the required tension reinforcement ratio

$A_{s,max}$ -maximum required area of steel

ΔM -Support Moment difference between two adjacent slabs

C -Span Moment factors for adjustment

$\mu_{sd,s}$ -ultimate limit state for single reinforcement

S -Site Coefficient for soil characteristic

1 Introduction

1.1 General

Building structures are solids, which are composed of architectural and structural parts. The structural part of the building supports the body of the building preventing it from any collapse or failure. Therefore, structural design involves the determination of the different sections of the skeletal part of the building to make it stable and sustainable throughout its design life.

A structural design is executed in such a way that the building will remain fit with appropriate degrees of reliability and in an economic way. It should sustain all the actions and influences during execution and use. Therefore, structural design focuses on structural safety and serviceability with due durability. It must also optimize the cost expended in building the structure and maintenance.

This structural design is executed based on the Ethiopian Standard based on Euro Norms (ES EN 2015). This code follows the Limit State design approach. Limit state is a state beyond which the structure no longer satisfies the design performance requirements.

1.1. Background

General information about the project

- ✦ This project is B+G+5 Mixed used building,
- ✦ The building allocated on Addis Ababa,
- ✦ The height of the building is +20.69 m.
- ✦ Type of roof is duo pitch.
- ✦ The total area of the building 1051.86 m².

1.2. Objectives

The main objective of the project is to carry out a structural design and analysis of a B+G+5 mixed use building. The prime objective of design is structural safety and serviceability. In case the structure fails, it must be in such a way it will minimize risks and loss.

The other objective of thesis is to enable students to experience real life engineering problem solving, design, teamwork, project execution and management. To satisfy program requirements, the projects must have certain components such as problem definition, research, scheduling, solution analysis, design and communication of results.

1.3.1 Specific objective

It needs specific tasks to accomplish the structural design and analysis of our building.

- ✦ Roof design
- ✦ Floor slab design
- ✦ Building circulation design its stair case design
- ✦ Lateral load analysis its wind load and earthquake load
- ✦ Frame analysis using ETABS, SAP & frame element design
- ✦ Foundation design

1.3. Building types

There are many types of buildings which can be classified based on their purpose: Residential buildings that provide the facility for private houses, apartments, Educational building: buildings which function for learning process. Institutional buildings which are built for the purpose of health, recoveries etc. Assembly buildings which are used for accommodation of many peoples together egg. Recreation, cinema halls, conference halls. Business building used for a business purpose like banks, private libraries etc. Industrial building are used for fabricating or manufacturing purposes like brick industry, cement factory, steel industry etc. Storage building: buildings which are used for storage purposes such as ware house. Hazard building are used for storing dangerous materials or explosive materials such as gases, acids, explosives etc. This project used for mixed use building.

To design a given structure first analyze the structure.

Analysis of structure:-It is the analysis of a given structure by modelling of the loads and the structural frames to obtain internal forces (i.e. axial, shear, torsional, or stresses), deflections, and verifies that no unstable failure can occur.

Structural design:-Structural design can be defined as a mixture of art and science, combining the engineer's feeling for the behavior of a structure with a sound knowledge of the principles of statics, dynamics, mechanics of material, and structural analysis, to design safe, economical and durable structure that will serve its intended purpose. In other word structural design involves proportioning the dimension of the member, internal reinforcement (number and sizes of reinforcing bars) of RC structures and selecting an appropriate section for those of wood and steel structure elements to withstand an imposed load over it.

Once the structural form has been determined, the actual design begins with those elements that are subjected to the primary loads the structure is intended to carry, and proceeds in sequence to the various supporting members until it reaches to the foundation. The analysis and design of a building is starts at the building roof then sequentially it ends at the foundation. Therefore, design of a building generally involves the design of the following elements of the building according to their design steps;

- ✦ Design of roof
- ✦ Design of floor slabs

- ✦ Design of stair case
- ✦ Design of beams, columns and shear wall
- ✦ Design of footing

To analyze and design a structure, it is necessary to establish criteria for determining whether a given structure is acceptable for use in a specified circumstance or for use directly as a design objective that must be met. Safety, stability, strength, serviceability, economy and aesthetics.

1.4. Design criterion for building

The severe conditions, which can be expect to occur in the lifetime of the building, include

- ✦ Determined situation, which refer to the conditions of normal use;
- ✦ Transient situations, which refer to temporary conditions applicable to the structural, e.g. during execution or repair;
- ✦ Seismic situations, which refer to exceptional conditions applicable to the structure when it is subjected to seismic event;
- ✦ Accidental situation, which refer to exceptional conditions applicable to the structure or to its exposure, e.g. to fire, explosion, impact

These and other considerations are included in this document.

1.5. Methodology

Overall framework of this project is illustrated in Figure 1. Next, the structural design of the building will be conducted using Ethiopian building code 2015. This included analysis and design of roof, slab, staircase, beams, columns and footing.

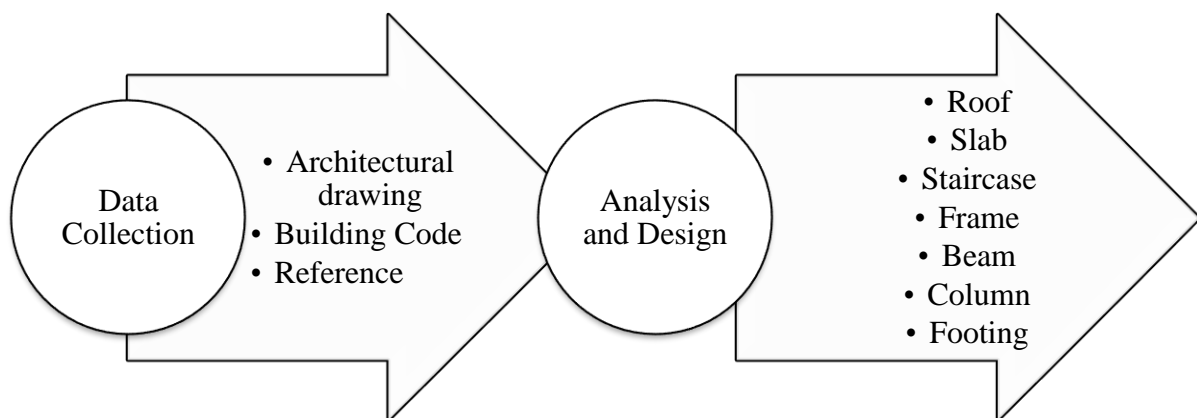


Figure 1. 1- The overall framework of this Project

1.5.1. Study Area

The project allocated on Addis Ababa, the effect of study area on structural analysis I simply identified. The earthquake is zone 2 on the new code describe on(ES EN 8. 2015 n.d.). The building is near to main asphalt road it effect on the material selection and consideration.

1.5.2. Data Collections

The architectural drawing was received from the Civil Engineering Department B.Sc. thesis/project Coordinator. The building used to carry out the structural design is limit state and building codes used on EN 1992 (Euro code 2, EC2) deals with the design of reinforced concrete structures –buildings, bridges and other civil engineering works.

The ES EN codes serve at least four distinct functions;

- ✦ Ensure adequate structural safety, by specifying certain essential minimum requirements for design.
- ✦ Render the task of the designer relatively simple; often, the results of sophisticated analyses are made available in the form of a simple formula or chart.
- ✦ Ensure a measure of consistency among different designers.
- ✦ Have some legal validity, in that they protect the structural designer from any liability due to structural failures that are caused by inadequate supervision and/or faulty material and construction

Table-1.1 EN ES determination

EN No	Euro code	The structural Euro codes
EN 1990	ES EN 0	Basis of structural design
EN 1991	ES EN 1	Actions on structures
	Part 1-1	General actions – Densities, self-weight and imposed loads
	Part 1-2	General actions on structures exposed to fire
	Part 1-3	General actions – Snow loads
	Part 1-4	General actions – Wind loads
	Part 1-5	General actions – Thermal actions
	Part 1-6	Actions during execution
	Part 1-7	Accidental actions from impact and explosions
	Part 2	Traffic loads on bridges
	Part 3	Actions induced by cranes and machinery
	Part 4	Actions in silos and tanks
EN 1992	ES EN 2	Design of concrete structures
	Part 1-1	General rules and rules for buildings
	Part 1-2	General rules – structural fire design
	Part 2	Reinforced and pre stresses concrete bridges
	Part 3	Liquid retaining and containing structures
EN 1993	ES EN 3	Design of steel structures
EN 1994	ES EN 4	Design of composite steel and concrete structures
EN 1995	ES EN 5	Design of timber structures
EN 1996	ES EN 6	Design of masonry structures
EN 1997	ES EN 7	Geotechnical design
EN 1998	ES EN 8	Design of structures for earthquake resistance
EN 1999	ES EN 9	Design of aluminum alloy structures
	Part 1-1	Densities, self-weight, impose loaded for buildings
	Part 1-2	Actions on structures exposed to fire
	Part 1-4	Wind actions
	Part 4	Soils and tanks

1.6. Material Specification

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

Concrete

✦ Class I workmanship and ordinary loading condition is used

- ✦ Ordinary Reinforced concrete class C-25/30, which is C-25(super-structure) and C-30(sub-structure).
- ✦ Unit weight of concrete, $\gamma_c = 25\text{KN/m}^3$
- ✦ Partial safety factor for concrete $\gamma_c = 1.5$
- ✦ Characteristics of compressive strength of concrete,

$$f_{ck} = 0.8 * 25 = 20\text{Mpa}$$

$$f_{cd} = 0.85 * \frac{20}{1.5} = 11.33\text{Mpa}$$

Where: f_{ck} - Characteristic of compressive strength of concrete.

γ_c - Partial Safety factor for ordinary loading=1.5.

$$f_{ctd} = \frac{f_{ctk}}{\gamma_c}$$

Where: f_{ctd} - Design of tensile strength of concrete.

f_{ctk} -Characteristic tensile strength of concrete.

γ_c - Partial Safety factor for ordinary loading=1.5.

Steel

- ✦ S-400 deformed bars
- ✦ Partial safety factor $\gamma_s = 1.15$
- ✦ Characteristic strength, $f_{yk} = 400\text{N/mm}^2$
- ✦ Design strength,

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{400}{1.15} = 347.83\text{Mpa}$$

Where: f_{yk} - Grade of steel, S-400

γ_s - Reinforcing steel factor =1.15 Mpa for ordinary loading and class I

- ✦ Secant modulus of elasticity E_{cm} (GPa)=200 Gpa
- ✦ Modulus of elasticity (E_s)= 200Gpa

Safety factors for load

- ✦ 1.35 for dead load
- ✦ 1.50 for live load

1.7. Geotechnical data

For a reason, a geotechnical report could not be found, the allowable bearing capacity of the site will be based on recommendations of EBCS 7 and geotechnical report of a site next to our proposed project. The bearing capacity is 300kpa.(ES EN 1997, Part 1. 2015)

1.8. Data Analysis and Design

The overall method to analyze and design a structure, it is necessary to establish criteria's or requirements for determining whether a given structure is acceptable for use in a specified circumstance or for use directly as a design objective that must be met. The overall method depends up on(Kasahun , 2020)

- ✦ Safety
- ✦ Durability,
- ✦ Economy, and
- ✦ Aesthetics appearance.

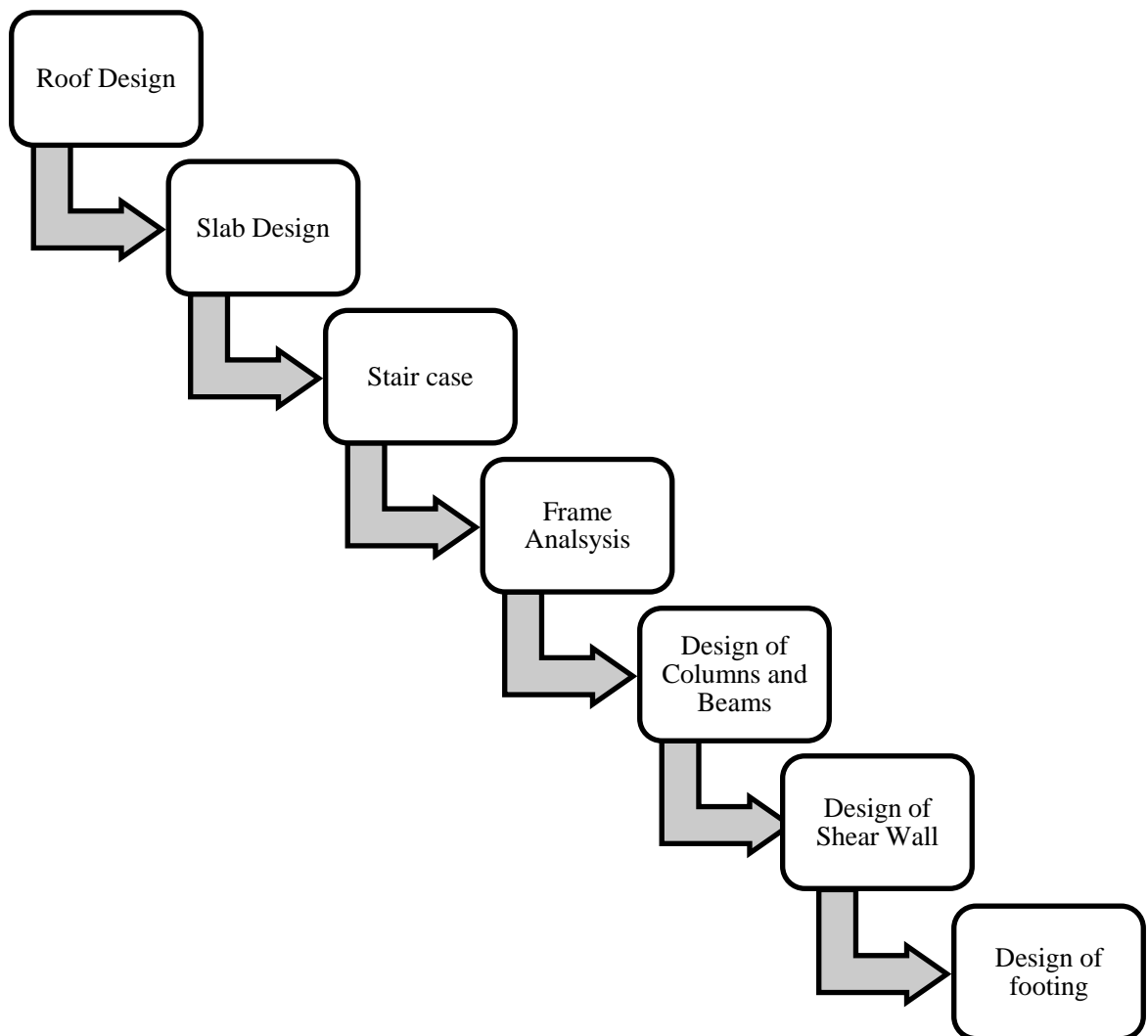


Figure 1.2- Overall framework of design and analysis

2. WIND LOAD ANALYSIS AND ROOF DESIGN

Wind as a moving air has an effect on building structures. We use the knowledge of fluid Mechanics to understand the effect of wind on building structures. Wind actions fluctuate with Time, hence its effect on different situations and structures should be carefully analyzed.

Wind act directly on the external surfaces of enclosed structures, through porosity of the external Surface and internal surface through opening. Wind pressure act on areas of the surfaces producing forces perpendicular to the surface of structure or on individual cladding components.

The effect of wind on structures is significant on light and dynamic structures. It does have considerable effect on vertically standing walls, columns and beams etc. therefore; its effect can be easily studied on roof structures such as truss and flat slabs.

Windstorm damage on buildings, bridges and other civil structures is a fact in the world. In the world history there are many structures failed by wind, especially roofed structures are highly exposed to wind damage, so to overcome these wind problem civil engineering experts studied wind effect on roof, building wall and bridge structures. The best solution that experts decided was considering the wind effect on the structure during design. Wind effect on structure is suction and push forward effect (pressure). This chapter focused on roof design to resist wind load. Main points included in this chapter:

- ✓ Detail introduction of the analysis of wind load based on ES EN 1991 2015
- ✓ Analysis and design of roof

2.1 WIND LOAD ANALYSIS

Wind load on the structure depends on many factors such as:

- ✓ wind velocity direction
- ✓ The height of the structure
- ✓ Topographic location of the structure
- ✓ Shape of the structure
- ✓ Terrain category
- ✓ The roughness of the surrounding

Action of the wind loads on structures is represented either as a wind pressure or as a suction force. Wind pressure on the structure may be external wind or internal wind Pressure. External wind pressure W_{ex} 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.

Given formulas

$$W_{ex} = q_p(z) * C_{pe} \dots \dots \dots \text{ES EN 1991:2014, (5.1)}$$

$$W_{ix} = q_p(z) * C_{pi} \dots \dots \dots \text{ES EN 1991:2014, (5.2)}$$

Where: $q_p(z)$ = peak mean wind pressure

C_{pe} = External wind pressure coefficient

C_{pi} = Internal wind pressure coefficient

ρ = is the air density.

$$q_p(z) = (1 + 7I_v(z)) * \frac{1}{2} * \rho * V^2(m) \dots \dots \dots \text{(ES EN 1991:2014 (4.8) n.d.)}$$

$$I_v(z) = \delta v / V_m(z) \dots \dots \dots \text{ES EN 1991:2014(4.7)}$$

$$\delta v = K_r * V_b * K_1 \dots \dots \dots \text{ES EN 1991:2014(4.6)}$$

$$V_b = C_{dir} * C_{seas} * V_{b,0} \dots \dots \dots \text{ES EN 1991:2014(4.1)}$$

$$V_m(z) = C_r(z) * C_o(z) * V_b \dots \dots \dots \text{ES EN 1991:2014(4.3)}$$

$$K_r = 0.19 * \left(\frac{Z_o}{Z_{ou}}\right)^{0.07} \dots \dots \dots \text{ES EN 1991:2014(4.5)}$$

$$C_r(z) = K_r * \ln\left(\frac{Z}{Z_o}\right) \dots \dots \dots \text{ES EN 1991:2014(4.4)}$$

$$V_{dir} = 1.0, C_{sea} = 1.0$$

$$V_{b0} = 25 \text{ m/s}$$

$$K_1 = 1.0$$

$$Z_o = 0.3 \text{ m}, Z_{ou} = 0.05, C_o(z) = 1 \text{ terrain category II}$$

Takes into account the variation of mean wind velocity at the site of the structure due to:

- ✓ The height above ground level
- ✓ The roughness of the terrain depending on the wind direction

$$C_r(z) = C_r(z_{min}), \text{ for } Z < Z_{min}$$

$$K_T \ln\left(\frac{Z}{Z_o}\right), \text{ for } Z_{min} < Z \leq 200 \text{ m}$$

Where: K_T : is the terrain factor

Z_o : is the roughness length

Z_{min} : is the minimum height

Note: Z_0, Z_{min} depends on the terrain category. Recommended values are given in table ES EN 1991 table 4.1 depending on five representative terrain categories.



Figure 2. 1 Dou-pitch Roof Truss layouts

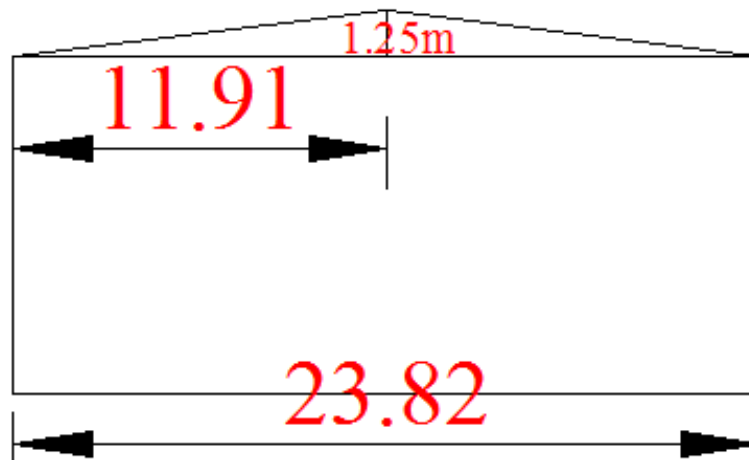


Figure 2. 2 rear view of the roof

Take pitch angle from -45° to 75° (ESEN 1991-4 n.d.)

$$\beta = \tan^{-1}(\text{pitch height/length of the pitch})$$

So we denote the angles as β

$$\beta = \tan^{-1}(\text{pitch height/length of the pitch})$$

$$\beta = \tan^{-1}(1.25/11.91) = 5.99^\circ$$

$$\beta = 5.99^\circ$$

2.1.1 CASE1 WIND DIRECTION ($\Theta=0^\circ$)

Given Data:

Width of roof, $d=23.82\text{m}$

Cross wind dimension, $b= 35.00\text{m}$

Height of the building at roof level, $h=19.44\text{m}$

$$e=\min \{b, 2h\}$$

$$e=\min \{35\text{m}, 2*19.44\text{m}=38.88\text{m}, e=35\text{m}\}$$

$$\frac{e}{4}=10.995\text{m and } \frac{e}{10}=3.5\text{m}$$

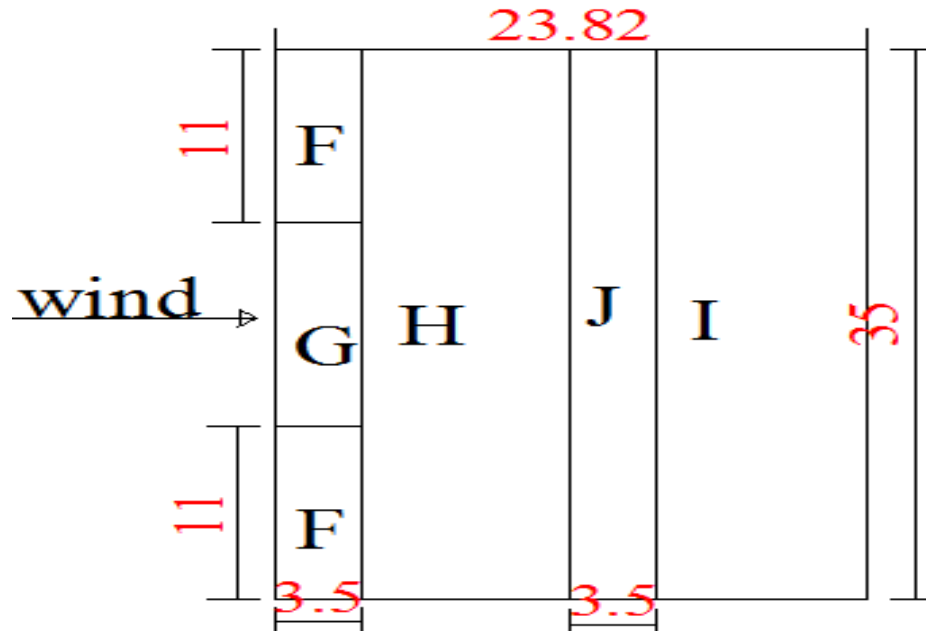


Figure 2. 3 analysis of external pressure coefficient C_{pe} in 0 degree

Area of zone:

$$\text{Area of F}=3.5*10.995=38.4825\text{m}^2$$

$$\text{Area of G}=13.01*3.5=45.535\text{m}^2$$

$$\text{Area of H}=8.41*35=294.35\text{m}^2$$

$$\text{Area of J}=3.5*35=122.5\text{m}^2$$

$$\text{Area of I}=8.41*35=294.35\text{m}^2$$

Now calculate external exposure coefficient depending on the area. Since all areas of the zones are greater than 10m^2 , $C_{pe} = C_{pe10}$ but the value should be read according to pitch angle.

For all of the zones pitch angle is given to be $\beta=5.99^\circ$

Exposure coefficient for duo pitch roof of each zone for wind direction ($\theta=0^\circ$). The pitch angle is between 5° and 15° , so interpolation is needed to obtain exposure coefficient of each zone.

1. Calculation of external pressure coefficient

Table 2. 1 cpe values in 0 degree

zone for wind direction $\theta=0^\circ$										
Pitch angle	F		G		H		J		I	
B	cpe10	cpe1	cpe10	cpe1	cpe10	cpe1	cpe10	cpe1	cpe10	cpe1
5°	-1.7	-2.5	-1.2	-2	-0.6	-1.2	-0.3		-0.3	
5.99°	-1.23	-2.13	-0.95	-1.18	-0.42	-0.7	-0.28	-0.33	-0.62	-0.85
15°	-0.9	-2	-0.8	-1.5	-0.3		-0.3	-0.4	-1	-1.5

2. Calculation of internal pressure coefficient

The internal pressure coefficient can be calculated by using three cases(ES EN 1991:2014 7.29 n.d.)

1. If there is no opening we use worst condition use $C_{pi} = -0.3$ or 0.2
2. By using opening ratio if there is opening but not dominant area
3. By using dominant area

Since, we use opening ratio use the table read the value of C_{pi}(ES EN 1991:2014 7.29 n.d.)

The dominant area is <2 so we use opening ratio. See the following excel result

Table 2. 2 values for internal pressure coefficient

FLOOR	width(cm)	hieght(cm)	area(cm ²)	∑area	FLOOR2	width(cm)3	hieght(cm)	area(cm ²)	∑area6	FLOOR7	area(cm ²)8	∑area9
FRONT					LEFT SIDE					REAR SIDE		
G	840	225	189000		G	495	180	89100		1ST	308320	
1st	66	115	7590		1st	495	180	89100		2ND	308320	
2nd	840	225	189000		2nd	495	180	89100	552600	3RD	308320	1730440
3rd	240	225	54000	547590	3rd	495	180	89100		4TH	308320	
4th	240	225	54000		4th	495	180	89100		5TH	482220	
5th	240	225	54000		5th	495	180	89100		G	14940	
					G	150	60	9000				
					1st	150	60	9000				
RIGHT SIDE	∑area											
	374373											
max. opening		1730440	1474563									
ratio		1.173527										
dominant area, so use opening ratio μ												
where cpe is -ve/∑area of all openings												
towarded rear	μ		0.46008	h=21.99	d=35.2	h/d	1.6007	h/d>1	cpi=0.15			
towarded front	μ		0.82914	h=21.99	d=35.2	h/d	1.6007	h/d>1	cpi=-0.35			
towarded right	μ		0.8832	h=21.99	d=24.85	h/d	1.130059	h/d>1	cpi=-0.35			
towarded left	μ		0.82758	h=21.99	d=24.85	h/d	1.130059	h/d>1	cpi=-0.35			

The two comparative value of Cpi

- ✓ 0.15
- ✓ -0.35

4. Calculation of peak wind velocity

Take Vdir = 1, csea = 1, k1 = 1, Vb0 = $\frac{25m}{sec}$ and Zo = 0.3m.....ES EN 1991 table 4.1

Table 2. 3 calculation of peak wind velocity

Vdir	Csea	Vb0	Vb(re. wind velocity)			K1(turb, factor)	δv(sta. deva. turbul)	ρ(air density)
1	1	25	25			1	5.38473328	1.25
Z0	Zou	kr(trra, fact.)	cr(rough.)	co(oroгра.)	Vm(z)(mean win	Iv(turb.inten)	qp(z)(peak velo	
0.3	0.05	0.21538	0.66565	1	16.641	0.323577	0.565	

✓ $W_i = q_p(z) * C_{pi}$

- ✓ Net wind pressure =we-wi

Table 2. 4 calculation of net wind pressure

cpe	-1.23	-0.95	-0.42	-0.28	-0.62	
cpil	0.15	0.15	0.15	0.15	0.15	
cpil	-0.35	-0.35	-0.35	-0.35	-0.35	
qp(z)	0.565	0.565	0.565	0.565	0.565	suction
We	-0.695	-0.5368	-0.2373	-0.0158	-0.3503	-0.7798
Wil	0.08475	0.08475	0.08475	0.08475	0.08475	
Wi2	-0.1978	-0.1978	-0.1978	-0.1978	-0.1978	pressure
Wnet1	-0.7798	-0.6216	-0.3221	-0.1006	-0.4351	0.182
Wnet2	-0.4972	-0.339	-0.0395	0.182	-0.1525	

- ✓ Wnet1(suction) =-0.77975KN/m²
- ✓ Wnet2(pressure)=0.182KN/m²

2.1.2 CASE 2 WIND DIRECTION (Θ=90°)

Geometric Data:

Width of roof, d=23.82m

Cross wind dimension, b= 35m

Height of the building at roof level, h=21.99m

$e = \min \{b, 2h\}$

$e = \{35, 2 \times 21.99 = 43.98\}$, $e = 35\text{m}$

$\frac{e}{2} = 17.5\text{m}$, $\frac{e}{4} = 8.75\text{m}$ and $\frac{e}{10} = 3.5\text{m}$

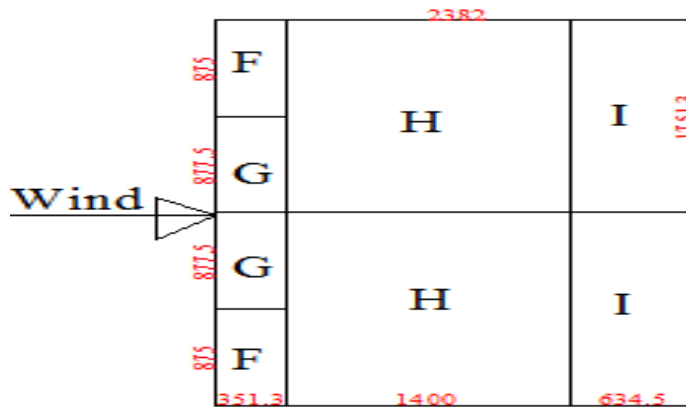


Figure 2. 4 analysis of cpe in 90 degree

Area of zone

Area of F=8.75*3.5=30.625m²

Area of G=8.75*3.5=30.625m²

Area of H=14*17.5=245m²

Area of I=17.5*6.32=110.6m²

Now calculate external exposure coefficient depending on zonal area F, G, H&I pitch angle $\beta=5.99^\circ$

Table 2. 5 values of cpe in 90 degree

	ZONE FOR WIND DIRECTION $\theta=90$							
Pitch angle	F		G		H		I	
Area	30.625		30.625		245		110.6	
B	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1
5°	-1.6	-2.2	-1.3	-2.0	-0.7	-1.2	-0.6	
5.99°	-1.57	-2.18	-1.3	-2.0	-0.69	-1.2	-0.59	
15°	-1.3	-2.0	-1.3	-2.0	-0.6	-1.2	-0.5	
Cpe	-1.57		-1.3		-0.69		-0.59	

1. Calculate the external pressure

Table 2. 6 Calculation of net wind pressure in 90 degree

C _{pe}	-1.57	-1.3	-0.69	-0.59
C _{pi1}	0.15	0.15	0.15	0.15
C _{pi2}	-0.35	-0.35	-0.35	-0.35
q _{p(z)}	0.565	0.565	0.565	0.565
w _e	-0.88705	-0.7345	-0.38985	-0.33335
W _{i1}	0.08475	0.08475	0.08475	0.08475
W _{i2}	-0.19775	-0.19775	-0.19775	-0.19775
W _{net1}	-0.9718	-0.81925	-0.4746	-0.4181
W _{net2}	-0.6893	-0.53675	-0.1921	-0.1356

The critical value of the wind effect in the duo-pitched roof

$$W_{\text{net(Suction)}} = -0.9718 \text{ KN/m}^2$$

$$W_{\text{net(Pressure)}} = 0.182 \text{ KN/m}^2$$

2.2 DESIGN AND ANALYSIS OF EGA SHEET

As determined in the above analysis of wind load for duo-pitch roof $W_{\text{net(Suction)}} = -0.9718 \text{ KN/m}^2$ and $W_{\text{net(Pressure)}} = 0.182 \text{ KN/m}^2$.

Selection EGA sheet

EGA sheet is the top cover of the roof and selected from the products catalogue of kaliti metal factory. Taking a maximum wind surface load of 0.09718 KN/m^2 and purling spacing of 1.19469, the load carrying capacity of EGA 500 having thickness of 30mm is 2.62 KN/m^2 which is greater than the maximum wind surface load on the roof. Therefore EGA 500 corrugated sheet is chosen as a roof cover for the building.

Height of Truss = 1.25m

Spacing of Purlin = 1.19469m, on both wing.

Type of EGA-Sheet selected EGA-500, t=40mm

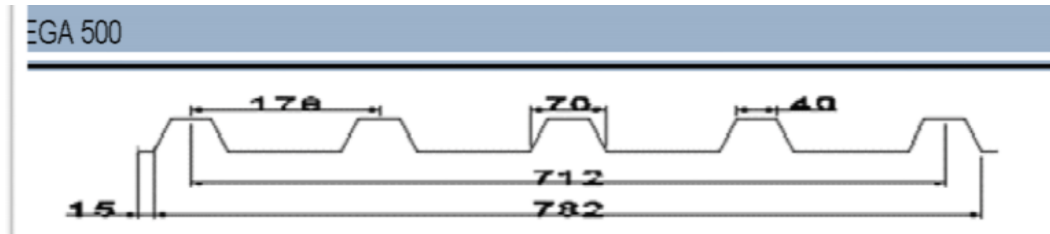


Figure 2. 5 EGA-500 Cross section

Cross section properties of EGA-500 are 0.4mm Thickness, 400mm² Area, 3.14Kg/m Weight, 74700 mm⁴ moment of inertia, 3756mm³ Section modulus, 160Mpa Allowable stress and 210000Mpa Modulus of Elasticity.

Dead Load

Weight of EGA=3.14Kg/m

Coating=0.70Kg/m

Total Load (TL) =weight of EGA + Coating

$$= 3.14+ 0.70=3.84 \text{ Kg/m}$$

Effective Width of EGA (beff) =806mm=0.712m

Load on EGA Kilo Newton per Meter Square (KN/m²)

$$=TL * g / (1000 * beff)$$

$$= 3.84 * 9.81 / (1000 * 0.712)$$

$$DL/m=0.053 \text{ KN/m}^2 * 1 \text{ m}$$

$$DL=0.053 \text{ KN/m}$$

Live Load

$$LLD=0.4 \text{ KN/m}^2 \dots \dots \dots \text{ES EN1991-2015 table 6.10}$$

$$LLC=1 \text{ KN} \dots \dots \dots \text{ES EN1991-2015 table 6.10}$$

$$LLD/m=0.4 * 1 = 0.4 \text{ KN/m}$$

$$WL/m = -0.9718 * 1 = -0.9718 \text{ KN/m (suction)}$$

$$WL/m = 0.182 * 1 = 0.182 \text{ KN/m (pressure)}$$

2.2.1 LOAD COMBINATION

For analysis and design use the following load combination which is taken from manual EURO CODE 1990 section A2.1B.

$$\text{Comb1: } 1.35\text{DL} + (-) 1.5\text{WL}$$

$$\text{Comb2: } 1.35\text{DL} + (-) 1.5\text{WL} + 0.9\text{LLD}$$

$$\text{Comb3: } 1.35\text{DL} + 1.5\text{LLD} + (-) 0.9\text{WL}$$

$$\text{Comb4: } 1.35\text{DL} + (-) 1.5\text{WL} + 0.9\text{LLC}$$

$$\text{Comb5: } 1.35\text{DL} + 1.5\text{LLC} + (-) 0.9\text{WLS}$$

There is no compression effect of wind so having only suction effect thus combo1, combo2, combo4 and combo 5 are used. Before combine, the loads arrange the load direction, dead load and live loads are the same in direction with gravity but wind load is perpendicular with cover, thus divide the dead load and live load by cosine value.

$$\text{Dead load} = 0.053\text{KN/m} / \cos(5.99)$$

$$= 0.0533\text{KN/m}$$

$$\text{Live load} = 0.4 / \cos 5.99$$

$$= 0.4022\text{KN/m}$$

$$\text{Live load} = 1 / \cos 5.99 = 1.005\text{KN/m}$$

$$\text{Combo1: } 1.35\text{DL} + (-) 1.5\text{WL}$$

$$= 1.35 * 0.0533 + 1.5 * 0.182 = 0.34495\text{KN/m}$$

$$\text{Combo2: } 1.35\text{DL} + (-) 1.5\text{WL} + 0.9\text{LLD}$$

$$= 1.35 * 0.0533 + 1.5 * -0.9718 = -1.3857\text{KN/m}$$

$$\text{Comb3: } 1.35\text{DL} + 1.5\text{LLD} + (-) 0.9\text{WL}$$

$$\text{combo1} + 0.9 * 0.4022 = 0.34495 + 0.9 * 0.4022$$

$$= 0.70693\text{KN/m}$$

$$\text{Comb4: } 1.35\text{DL} + (-) 1.5\text{WL} + 0.9\text{LLC}$$

$$\text{Combo2} + 0.9 * 0.4022 = -1.3857 + 0.9 * 0.4022$$

$$= -1.0237\text{KN/m}$$

$$\text{Combo5: } 1.35\text{DL} + 1.5\text{LLC} + (-) 0.9\text{WLS}$$

$$= 1.35 * 0.0533 + 1.5 * 0.4022 + 0.9 * 0.182 = 0.839\text{KN/m}$$

$$\text{Combo 6 } (1.35 * 0.0533 + 1.5 * 0.4022 - 0.9 * 0.9718) = -0.202\text{KN/m}$$

$$\text{Combo4: } \text{combo1} + 0.9 * 1.005$$

$$= 0.34495\text{KN/m}+0.9045\text{KN/m}$$

Combo5: Combo2+0.9*1.017

$$=-1.3857 \text{ KN/m}+0.9045\text{KN/m}$$

Combo6 $1.35*0.0533+1.5*1.005+0.9*-0.9718$

$$=-0.8026\text{KN/m}+1.5075\text{KN/m}$$

Combo7: $1.35*0.0533+1.5*1.005+0.9*0.182$

$$=0.2357\text{KN/m}+1.5075\text{KN/m}$$

2.2.2 CALCULATION OF MOMENT AND REACTION

Use RHS purlins as a support for the simplified construction first distribute the above combined load on the sheet and analyze by taking 1m sheet strip.

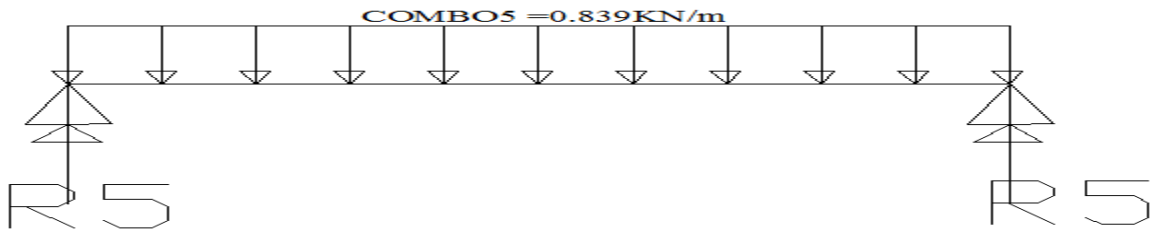


Figure 2. 6 analysis of EGA sheet for load combo5

Reaction R5 is the purlin and the load distributed on EGA sheet and to gate the maximum reaction apply the load at 0.75m

$$\text{Combo5} = 0.839\text{KN/m}$$

$$R5 * 1\text{m} = 1\text{m} * 0.839\text{KN/M} * 0.75\text{m} = 0.629\text{KN}$$

$$\text{Moment } M = \frac{wl^2}{8} = \frac{0.839 * 1 * 1}{8} = 0.1049\text{KNm}$$

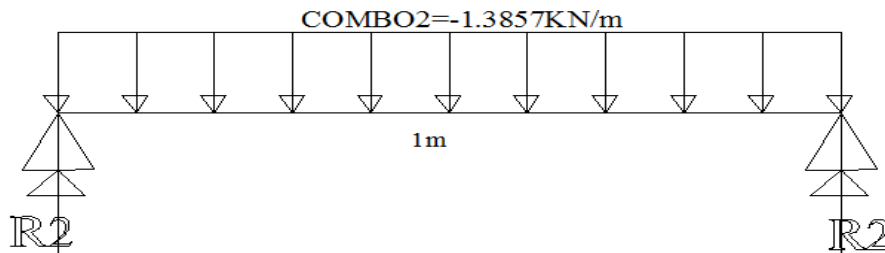


Figure 2. 7 analysis of EGA sheet for load combo 2

Reaction R2 is the purlin and the load distributed on EGA sheet.

$$\text{Combo2} = -1.3857 \text{ KN/m}$$

$$R2 = 1 \text{ m} * -1.3857 * 0.5 \text{ m} = -0.69285 \text{ KN/m}$$

$$\text{Moment } M = \frac{wl^2}{8} = \frac{-0.69285 * 1 * 1}{8} = -0.0866 \text{ KNm}$$

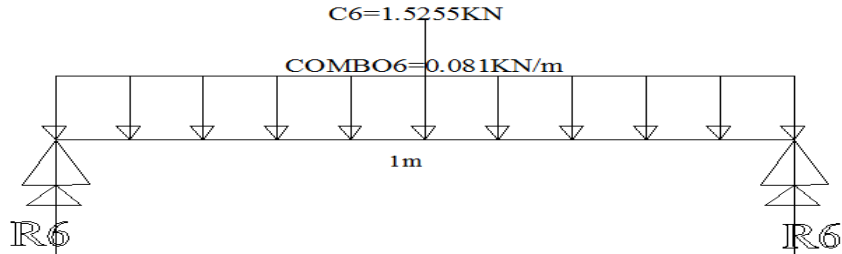


Figure 2. 8 analysis of EGA sheet for load combo 6

Reaction R6 is the purlin and the load is distributed on EGA sheet

$$\text{Combo6} = 0.081 \text{ KN/m} + 1.5255 \text{ KN}$$

$$R6 = 1 \text{ m} * 0.2357 \text{ KN/m} * 0.5 \text{ m} + 1.5075 \text{ KN/m} * 0.5 \text{ m} = 0.8716 \text{ KN/m sheet}$$

$$\text{Moment } M = \frac{wl^2}{8} + \frac{pl}{4} = \frac{0.2357 * 1 * 1}{8} + \frac{1.5075 * 1}{4} = 0.4063 \text{ KNm}$$

2.2.3 MOMENT FOR SERVICEABILITY

$$M = \frac{wl^2}{8} + \frac{pl}{4} = \frac{0.0533 \text{ KN/m} * 1 \text{ m} * 1 \text{ m}}{8} + \frac{1.005 \text{ KN} * 1 \text{ m}}{4} = 0.258 \text{ KNm}$$

$$\text{Reaction} = \text{shear} = 0.0533 \text{ KN/m} * 0.5 \text{ m} + 1.005 \text{ KN/m} * 0.5 \text{ m} = 0.529 \text{ KN}$$

Check the load capacity of cover weather resist or not

$f_y = 400 \text{ N/mm}^2$ for EGA sheet(kality EGA sheet company n.d.)

$$Z = M / f_y$$

$$Z = 0.4063 * 10^6 \text{ Nmm} / 400 \text{ N/mm}^2$$

$$Z = 1015.75 \text{ mm}^3 < 3756 \text{ mm}^3 \text{ok}$$

2.2.4 CHECK THE MOMENT CAPACITY

$$M_{plrd} = W_{pl} * \frac{f_y}{\gamma_{mo}}, f_y = 400 \text{ Mpa and } \gamma_{mo} = 1.0$$

$$= 3756 \text{ mm}^3 * 400 \text{ N/mm}^2 / 1.0$$

$$1.5024 \text{ KNm} > 0.4063 \text{ KNm} \text{ok}$$

2.2.5 CHECK FOR DEFLECTION

The capacity of cover for deflection

Allowable Deflection $\Delta_{all} = L/200$Roof general ((ENV 1993-1.1-1992 Table 4.1) n.d.)

Allowable deflection = $1195\text{mm}/200 = 5.975\text{mm}$

Deflection = $5wl^4 / (384EI) + PL^3 / 48EI$from basic structural analysis

$$= 5 * 0.0533 * 1195^4 / (384 * 210000 * 74700) + 1.005 * 1195^3 / (48 * 210000 * 74700)$$

$$= 0.0902\text{mm} + 0.23\text{mm}$$

$$= 0.32\text{mm} < 5.975\text{mm} \dots\dots\dots \text{ok}$$

Therefore, EGA 500 with thickness 0.4mm is provided for roof cover

2.3 DESIGN OF PURLIN

Purlins are beams used on trusses to support the sloping roof system between adjacent trusses. RHS, Channels, angle sections, and cold formed C- or Z-sections are widely used as purlins. They are placed in an inclined position over the main rafters of the trusses. To avoid bending in the top chords of roof trusses, it is theoretically desirable to place purlins only at panel points. From the above calculation the maximum load combination is combo6 and the result is $BM = 0.4063\text{KNm}$, $R = 0.8716\text{KN}$ then $\text{combo6} = 0.2357\text{KN/m} + 1.5075\text{KN/m}$.

Take purlin spacing = 1.19469m and truss spacing = 3.5m

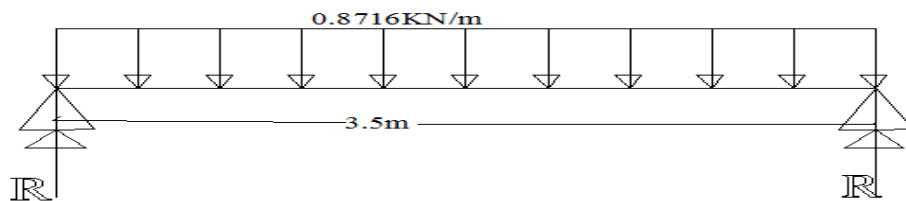


Figure 2. 9 analysis of purlin

$$\text{Moment, } MD = \frac{wl^2}{8} = \frac{0.8716 * 3.5 * 3.5}{8} = 1.3346\text{KNm}$$

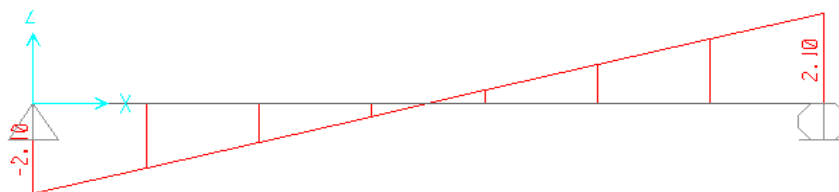


Figure 2. 10 SAP result for shear

$$V_d = 2.1\text{KN}$$

$$\text{Deflection} = 5WL^4/384EI + PL^3/48EI$$

Where: W=distributed un-factored load

P=concentrated un-factored load

L=length of the purlin

$$\begin{aligned} \text{Deflection} &= 5 * 0.2357 * 6^4 / (384 * 210000 * 43.55) + 1.5075 * 6^3 / (48 * 210000 * 43.55) \\ &= 7.41 \text{ mm} \end{aligned}$$

2.3.1 SELECTION OF SECTION

In order to select a section from kality manual first calculate the plastic modulus

$$M_{pl, rd} = W_{pl} * \frac{f_y}{\gamma_{mo}}, \text{ then } W_{ply} = \frac{M_{pl, rd}}{f_y}$$

$$W_{ply} > \frac{1.3346 * 10^6}{235} = 5679.148 \text{ mm}^3 = 5.679 \text{ cm}^3$$

Select the section which have the plastic modulus greater than 5.679cm³.

- ✓ Section index=RT53- 50x30x3mmRHS

Section properties are 3mm thickness, 3.3Kg/m weight per meter, 4.21cm² area, 12.83cm⁴ Moment of Inertia, 1.72cm Radius of gyration, 6.57cm³ Plastic modulus

2.3.2 CHECK THE CROSS SECTION

First check the class of the section ES EN 1993 2015 table 5.2

- ✓ c/t < 33ε.....class 1
- ✓ c/t < 38ε.....class 2

Take the yield strength of the section f_y = 235N/mm².....(ES EN 1993 n.d.)

$$\epsilon = \sqrt{235/f_y} \dots\dots\dots(\text{ES EN 1993 n.d.})$$

$$\epsilon = \sqrt{235/235} = 1$$

- ✓ 33ε=33
- ✓ C=b-3t(EURO CODE 1993 n.d.)

$$c = 50 \text{ mm} - 3 * 3 \text{ mm} = 41 \text{ mm}$$

$$c/t = 41 \text{ mm} / 3 \text{ mm} = 13.667$$

$$13.667 < 33 \dots\dots\dots \text{class 1}$$

2.3.3 DESIGN CAPACITY OF THE SECTION

1) Moment capacity

$$\frac{M_D}{M_{pl,Rd}} \leq 1 \dots\dots\dots (ES EN 1993 2015 n.d.)$$

$$M_{pl,Rd} = W_{pl} * f_y / \gamma_{mo} \dots\dots\dots (ES EN 1993 2015 n.d.)$$

$$M_{pl,Rd} = 6.357 * \frac{235}{1.0} = 1.543 \text{KNm}$$

$$M_D = 1.3346 \text{KNm}$$

$$1.543 \text{KNm} > 1.233 \text{KNm}$$

$$1.3346 / 1.543 = 0.865 < 1.0 \dots\dots\dots \text{ok!}$$

2) Shear capacity

$$\frac{V_d}{V_{pl,Rd}} \leq 1 \dots\dots\dots (ES EN 1993 2015 n.d.)$$

$$V_{pl,Rd} = A_v * f_y / (\sqrt{3} * \gamma_{mo}) \dots\dots\dots (ES EN 1993 2015 n.d.)$$

$$A_v = \frac{A_h}{(b+h)} \dots\dots\dots (ES EN 1993 2015 n.d.)$$

$$A_v = 4.21 \text{cm}^2 * \frac{50 \text{mm}}{(50 \text{mm} + 30 \text{mm})} = 2.63 \text{cm}^2$$

$$V_{pl,Rd} = \frac{2.63 \text{cm}^2 * 100 * 235 \text{N/mm}^2}{\sqrt{3} * 1.0} = 35.7 \text{KN}$$

$$V_d = 2.1 \text{KN}$$

$\frac{V_d}{V_{pl,Rd}} = \frac{2.1}{35.7} = 0.0288 < 1.0 \dots\dots\dots \text{OK, Since the purlin is act as a beam so the design is based on the bending or susceptible to bending instead of shear.}$

3) Check for deflection

$$\delta_{max} = L/200 \dots\dots\dots (\text{EURO CODE 1993 n.d.})$$

$$\delta_2 = L/250 \dots\dots\dots (\text{EURO CODE 1993 n.d.})$$

$$\delta_1 = 1 \text{mm} \dots\dots\dots \text{assumption}$$

$$\delta_0 = 0 \dots\dots\dots \text{initially (pre camber) in the unloaded state}$$

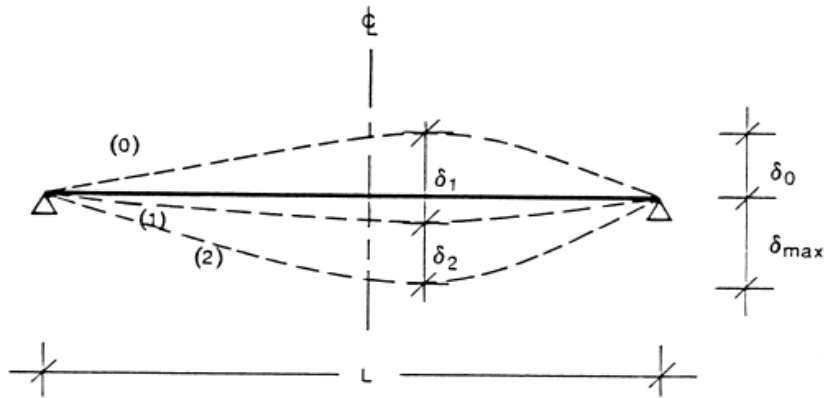


Figure 4.1 — Vertical deflections to be considered

Figure 2. 11 vertical deflection

$$\delta_{max} = \delta_2 + \delta_1 - \delta_0$$

$$L=3500\text{mm}$$

$$L/200=30\text{mm}$$

$$L/250=24\text{mm}$$

Actual deflection from the above calculation is 7.4175mm

7.41mm < 30mm.....OK! Safe.

2.4 TRUSS ANALYSIS AND DESIGN

Roof trusses are composed of tension and compression members joined together by welding or riveting. The loads supported on the roofing elements are transferred from purlin. The shape of roof trusses are largely determined largely by the area and space to be covered.

The load applied from the purlin to the truss is 2*one purlin support= 2*2.1KN=4.2KN.

The truss model

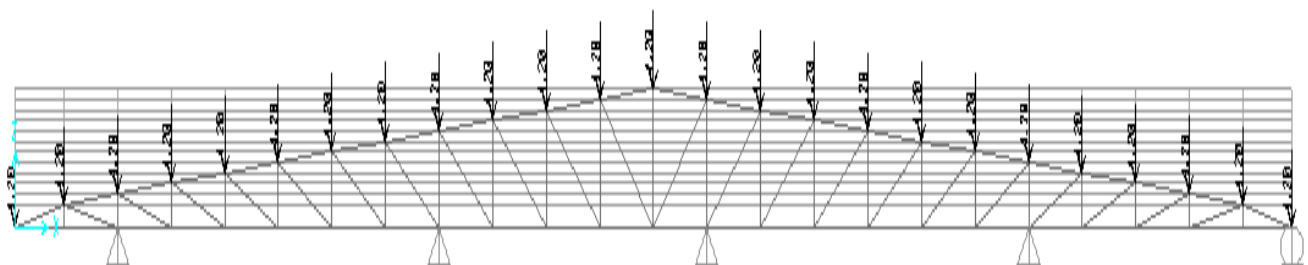


Figure 2. 12 truss layout

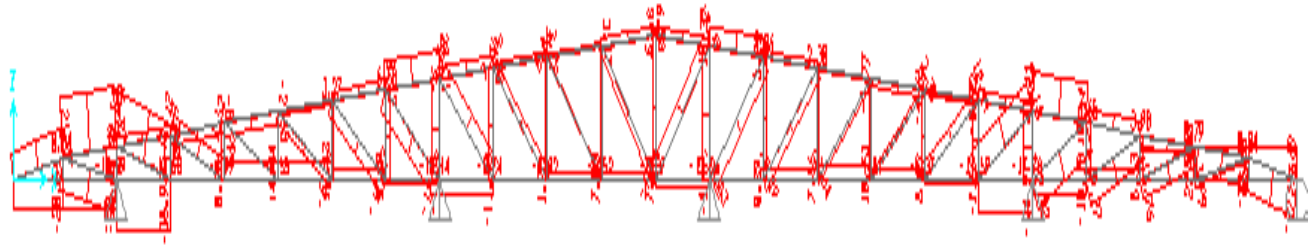


Figure 2. 13 SAP 2000 result for roof analysis

Table 2. 7 tension and compression effect on the truss

Members	Tension(KN)	Cooperation(KN)
Top cord	45.9642	27.249
Bottom cord	26.694	38.932
Diagonal	40.524	25.595
Vertical	6.671	18.086

Among the above results 45.964KN and 38.932KN are the maximum compression and tension forces respectively.

Selection of section from kality manual

For rafters

$$N_{pl}, R_d = A f_y / \gamma_{m0}$$

$$A = \frac{N_{pl}, R_d}{f_y} = \frac{45.964 * 1000}{235}$$

A=195.59mm², Then select the section which have the area greater than 1.956 cm².

Section index RT42 with Nominal size=b*h=20*40

Its properties are 3mm thickness, 2.36Kg/m Weight per meter, 3.01cm² Area, 5.21cm⁴ Moment of inertia, 1.320cm Radius of gyration and 3.5cm³ Plastic modulus.

For diagonal members

$$A=40.524*1000/235$$

$A=172.44\text{mm}^2$, Then select the section which have the area greater than 1.724cm^2 .

Section index RT32 with Nominal size $=b*h=20*30$

Its properties are 2.5mm Thickness, 1.64Kg/m Weight per meter, 2.09cm^2 Area, 2.21cm^4 Moment of inertia, 1.03cm Radius of gyration and 1.92cm^3 Plastic modulus.

2.4.1 CHECK SECTION CAPACITY IN TENSION FOR RAFTER

First identify the section class

$$\varepsilon = \sqrt{235/f_y} \dots\dots\dots \text{ES EN 1993 table 5.2}$$

$$\varepsilon = \sqrt{235/f_y} = 1$$

$$33\varepsilon = 33$$

$$C = b - 3t = 20\text{mm} - 3*3\text{mm} = 11\text{mm}$$

$$c/t = 11/3 = 3.667$$

$$3.667 < 33 \dots\dots\dots \text{class 1} \dots\dots\dots \text{ESEN1993 table 5.2}$$

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 1.0 \dots\dots\dots \text{ES EN 1993 2015 section 6.5}$$

$$N_{pl,Rd} = \frac{A_f y_f}{\gamma_{mo}} = \frac{3.01\text{cm}^2 * 100 * \frac{235\text{N}}{\text{mm}^2}}{1000 * 1.0}$$

$$= 70.7\text{KN} > 45.964\text{KN}$$

$$45.964/70.7 = 0.65$$

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 1.0 \dots\dots\dots \text{OK!}$$

2.4.2 CHECK SECTION CAPACITY IN COMPRESSION FOR RAFTER

First identify the section class

The buckling resistance of the compression member

$$N_{b,Rd} = X A_f y_f / \gamma_{m1} \dots\dots\dots (\text{ES EN 1993 2015 section 6.47 n.d.})$$

$$X = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}}, \text{ but } X < 1.0 \dots\dots\dots (\text{ES EN1993 section 6.49 n.d.})$$

$$\lambda = \frac{L_{cr} * 1}{i * \lambda_1}$$

$$L_{cr} = KL, \text{ For welding} = 0.65$$

$$L_{cr} = 0.65 * 1 = 0.65m = 65cm$$

$$i=r=1.32cm$$

$$\lambda_1=93.3\varepsilon=93.3$$

$$\lambda=65*1/ (1.32*93.3) =0.53$$

$$\phi = 0.5(1+\alpha (\lambda - 0.2) + \lambda^2)$$

Based on buckling curve, the curve is hot section and hot finished “a” curve see (EN 1993 table 6.2 n.d.) $\alpha=0.21$
see table 6.1

$$\phi = 0.5(1 + 0.21(0.53 - 0.2) + 0.53^2) = 0.675$$

$$X=1/(0.675+\sqrt{0.675 - 0.53^2})=0.77$$

$$X=0.77<1.....ok!$$

$$So, N_b, R_d = (1 * 3.5cm^2 * \frac{235N}{mm^2})/1=82.25KN>45.96KN$$

$N_b, R_d > NED.....the section is adequate!$

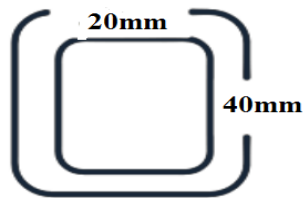


Figure 2. 14 Sectional view of RT42

2.4.3 CHECK SECTION CAPACITY IN TENSION FOR VERTICAL AND DIAGONAL MEMBERS

First identify the section design class

$$\varepsilon=\sqrt{235/f_y}ES EN 1993 table 5.2$$

$$\varepsilon =\sqrt{235/f_y}=1$$

$$33\varepsilon=33$$

$$C=b-3t =20mm-3*2.5mm=12.5mm$$

$$c/t=12.5/3=4.1667$$

$$4.1667<33.....class1.....ESEN1993 table 5.2$$

$$NED = N_{pl}, R_d \leq 1.0.....(ES EN 1993 2015 section 6.5 n.d.)$$

$$N_{pl,Rd} = \frac{A_f y}{\gamma_{mo}} = \frac{2.09 \text{cm}^2 * 100 * 235 \text{N/mm}^2}{1000 * 1.0} = 49.115 \text{KN}$$

$$49.115 \text{KN} > 40.524 \text{KN}$$

$$40.524 / 49.115 = 0.82$$

$$NED = N_{pl,Rd} \leq 1.0 \dots \dots \dots \text{OK!}$$

2.4.4 CHECK SECTION CAPACITY IN COMPRESSION FOR VERTICAL AND DIAGONAL MEMBERS

First identify the section class

The buckling resistance of the compression member

$$N_b, R_d = X A_f y / \gamma_{m1} \dots \dots \dots \text{ES EN 1993 2015 section 6.47}$$

$$X = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}}, \text{ but } X < 1.0 \dots \dots \dots \text{ES EN 1993 section 6.49}$$

$$\lambda = \frac{L_{cr} * 1}{i * \lambda_1}$$

$$L_{cr} = KL, \text{ For welding} = 0.65$$

$$L_{cr} = 0.65 * 1 = 0.65 \text{m} = 65 \text{cm}$$

$$i = r = 1.03 \text{cm}$$

$$\lambda_1 = 93.3 \epsilon = 93.3$$

$$\lambda = 65 * 1 / (1.03 * 93.3) = 0.676$$

$$\phi = 0.5(1 + \alpha(\lambda - 0.2) + \lambda^2)$$

Depend on the buckling curve, the curve is hot section and hot finished “a” curve

$$\alpha = 0.21 \text{ see table 6.1}$$

$$\phi = 0.5(1 + 0.21(0.676 - 0.2) + 0.676^2) = 0.7895$$

$$X = 1 / (0.7895 + \sqrt{0.7895^2 - 0.676^2}) = 0.73$$

$$X = 0.73 < 1 \dots \dots \text{ok!}$$

$$So, N_b, R_d = \frac{A_f y}{\gamma_{mo}} = \frac{2.09 \text{cm}^2 * 100 * 235 \text{N/mm}^2}{1000 * 1.0} = 49.115 \text{KN} > 40.524 \text{KN}$$

$N_b, R_d > NED \dots \dots \dots$ the section is adequate!

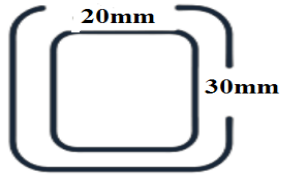


Figure 2. 15 Sectional view of RT32

3. SLAB DESIGN

Reinforced concrete slab is a broad flat plate usually horizontal with top and bottom Surfaces parallel or nearly so. It is used to provide flat surfaces mainly for roofs and floors of buildings, parking lots, air fields, roadways ...etc. Reinforced concrete beams, masonry or reinforced concrete walls, structural steel members may support it, directly by columns and continuously by the ground.

A slab is a member for which the minimum panel dimension is not less than 5 times the overall slab thickness. The design include Solid slab and ribbed slab.

3.1 SOLID SLAB ANALYSIS AND DESIGN

Beam supported slabs may be classified as: - One-way slabs with $L_y/L_x > 2$ and Two -way slabs $L_y/L_x < 2$. In the case of one way slab the main reinforcement are provided only in one direction, but in the case of two way slabs the main reinforcements are provided in both directions.

3.1.1 CLASSIFICATION OF PANELS

A slab subjected to dominantly uniformly distributed loads may be considered to be one-way spanning if either:

- ✓ It possesses two free (unsupported) and sensibly parallel edges, or
- ✓ It is the central part of a sensibly rectangular slab supported on four edges with a ratio of the longer to shorter span greater than 2.

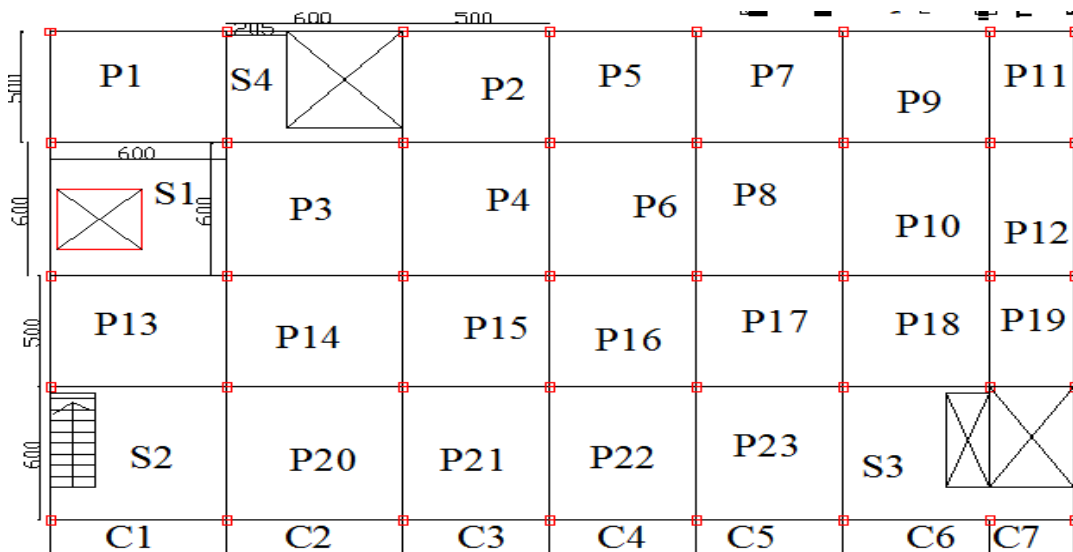


Figure 3. 1 Layout of floor plan

If $L_y/L_x > 2$ =one way

If $L_y/L_x < 2$ =two way

Table 3. 1 classification of panels

Panel	Lx	Ly	Ly/Lx	Panel type
P1	5	6	1.2	Two way
P2	5	6	1.2	Two way
P3	6	6	1.0	Two way
P4	5	6	1.2	Two way
P5	5	5	1.0	Two way
P6	5	6	1.2	Two way
P7	5	5	1.0	Two way
P8	5	6	1.2	Two way
P9	5	5	1.0	Two way
P10	5	6	1.2	Two way
P11	2.9	5	1.724	Two way
P12	2.9	6	2.069	One way
P13	5	6	1.2	Two way
P14	5	6	1.2	Two way
P15	5	5	1.0	Two way
P16	5	5	1.0	Two way
P17	5	5	1.0	Two way
P18	5	5	1.0	Two way
P19	2.9	5	1.724	Two way
P20	6	6	1.0	Two way
P21	5	6	1.2	Two way
P22	5	6	1.2	Two way
P23	5	6	1.2	Two way

C1	Cantilever	C5	Cantilever
C2	Cantilever	C6	Cantilever
C3	Cantilever	C7	Cantilever
C4	Cantilever		

3.1.2 DESIGN OF TWO WAY SLABS USING COEFFICIENT METHOD

Slabs with side ratio less than two are treated as two-way slabs and the analysis can be made by means of coefficients provide that the following conditions are satisfied.

- a. The slab is composed of rectangular panels, supported at all four edges by walls or beams and stiff enough to be treated as an unyielding.
- b. This method is intended for slab subjected to uniformly distributed load. If the Slab is subjected to concentrated load in addition to uniformly distributed load treated as equivalent uniform load using approximate rules, provide that the sum of non-uniform loads on a panel does not exceed 20 percent of the total load.

$$M_i = \alpha_i(gd + qd)Lx^2$$

$$\alpha_i = \begin{cases} \alpha_{fx} & \text{shorter direction} \\ \alpha_{fy} & \text{longer direction} \end{cases}$$

α_{fx} f – field (span)
 α_{fy} s – support

3.1.3 DEPTH DETERMINATION

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

$$D = \text{effective depth } (d) + \Phi/2 + \text{cover}$$

In order to determine the depth of the slab, first it is needed to find concrete cover and effective depth. Consider one meter strip width, $b=1000\text{mm}$.

1) Concrete Cover Determination

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

Nominal cover

The nominal concrete cover is the distance b/n the surface of reinforcement closest to the nearest concrete surface (including links and stirrups).

$$C_{nom} = C_{min} + \Delta C_{dev} \dots \dots \dots \text{(ESEN 1992: 2015 } \square\square\square \text{ 4.4.12 n.d.)}$$

Where: - C_{min} -minimum cover

ΔC_{dev} -allowance in design for deviation

- i. Minimum cover

C_{min} Shall provide in order to ensure

- ✓ The safe transmission of bond force
- ✓ The protection of the steel against corrosion (durability)
- ✓ An adequate fire resistance (see EN 1992-1-2)

The greater value for C_{min} satisfying the requirements for both bond and environmental conditions shall be used.

$$C_{min} = \max \left\{ \begin{array}{l} C_{min, b} \\ C_{min, dur} + \Delta C_{dur, \gamma} - \Delta C_{dur, st} - \Delta C_{dur, add} \\ 10mm \end{array} \right. \quad \text{Eqn(4.2)}$$

Where: - $C_{min, b}$ -minimum cover due to bond requirement.....ES EN Art. 4.4.1.2 (3)

$C_{min, dur}$ -minimum cover due to environmental conditions.....ES EN Art 4.4.1.2 (5)

$\Delta C_{dur, \gamma}$ -additive safety element.....ES EN Art 4.4.1.2 (6)

$\Delta C_{dur, st}$ -reduction of minimum cover for use of stainless steel.....ES EN Art 4.4.1.2 (7)

$\Delta C_{dur, add}$ -reduction of minimum cover for use of additional protection.....ES EN Art 4.4.1.2

But; the recommended value of $\Delta C_{dur, st}$, $\Delta C_{dur, \gamma}$ and $\Delta C_{dur, add}$ is zero.....Art. 4.4.1.2

- ✓ Cover Design for Bond

Assume Ø10 longitudinal bar and Ø20 nominal maximum aggregate size;

Therefore; $C_{min, b}=10mm$.

- ✓ Cover Design for Corrosion/Durability

The building is found on dry or permanently wet area so the condition of exposure is given to be XC1

Member with slab geometry and XC1...reduced by 1.....(ESEN1992 n.d.)

Here connect XC1 and member with slab geometry we get reduced class by 1. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths but based on the above table the exposure class is reduce by 1 and the structural class would be S3.

Therefore the value of minimum cover required for durability of reinforcement steel is determined using ES EN 1992:2015 table 4.4N.

Table 3. 2 structural class to find depth of the slab

environmental requirement					
exposure class according to table 4.1					
Structural class	X0	XC1	XC2/XC3	XC4	XD1/XS1
S1	10	10	10	15	20
S2	10	10	15	20	25
S3	10	10	20	25	30
S4	10	15	25	30	35

Here connect S3 and XC1 then get 10mm

So, $C_{min, dur} = 10mm$.

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

$C_{min} = 10mm$

ΔC_{dev} (Allowance in Design for Variation)

The value of ΔC_{dev} the recommended value is 10mm.

Therefore, $C_{nom} = C_{min} + \Delta C_{dev} = 10mm + 10mm = 20mm$ is the cover

✓ 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 from Table 5.8 Of EN 1992-1-2 REI60

$C_{fire} = 20mm$

$h_s = 80mm$

The cover is safe for the fire resistant.

✓ Effective Depth Determination: Serviceability requirement

From ES EN 1992:2015; section 7.4.2 and 7.4.3

$$\frac{l}{d} = K \left[11 + 1.5 \sqrt{f_{ck}} \frac{\rho_0}{\rho} + 3.2 \sqrt{f_{ck}} \left(\frac{\rho_0}{\rho} - 1 \right)^{3/2} \right] * F1 * F2 * F3 \dots \text{if } \rho < \rho_0 \text{ Art. 7.4.2 (7.16a)}$$

Where; l/d- is the limit span/depth

K is the factor to take into account the deference structural systems

ρ_0 - is the reference reinforcement ratio = $10^{-3/\sqrt{f_{ck}}}$

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

f_{ck} – in MPa

$$F1 = \frac{500}{\delta S} = \frac{500}{f_{yk} * \frac{A_{s,req}}{A_{s,prov}}}$$

$F2=0.8$, for flanged section where the ratio of the flange breath to the rib breadth exceeds 3.

Otherwise; $F2=1$ for other cases.

$F3 = 7/l_{eff}$ for beam and slabs, other than flat slab with a span exceeding 7m which support partitions liable to be damaged by excessive deflections (l_{eff} in meters, see Art. 5.3.2.2(1)). Or $F3 = 7/l_{eff}$ for flat slabs where the greater span exceeds 8.5m and which support partition liable to be damaged by excessive deflections otherwise; $F3=1$ for both cases.

Then in this case the lengths are below 7m so $F2=F3=1$

Let's assume $\rho = \rho_0$ and equation 7.16a as N

$A_{s, req} = A_{s, prov}$

$$\frac{l}{d} = K * N * F1 * F2 * F3$$

Where: $N = 11 + 1.5 \sqrt{f_{ck}} \frac{\rho_0}{\rho} + 3.2 \sqrt{f_{ck}} \left(\frac{\rho_0}{\rho} - 1 \right)^{3/2}$

$$F1 = \frac{500}{f_{yk} * A_{req}/A_{prov}}$$

$F1=500/400=1.25$since $A_{req} = A_{prov}$

$N=11+1.5*20^{0.5}=17.708$

$F2=1$ and $F3=1$ (because span of beam < 7m)

The value of K is 1.5 for interior span, 1.3 for end span and 0.4 for cantilever. Using table 7.4N of ESEN1992:2015.

Table 3. 3 depth determination

Panel	Support condition	Lx	Ly	fyk	fck	ρ_o	ρ	N	K	F1	F2	F3	D
P1	2	5	6	400	20	0.0045	0.0045	17.7	1.5	1.25	1	1	150.66
P2	2	5	6	400	20	0.0045	0.0045	17.7	1.3	1.25	1	1	173.84
P3	4	6	6	400	20	0.0045	0.0045	17.7	1.5	1.25	1	1	180.79

The reset of all panel depth are in the appendix part

Sample calculation

For panel1

$$\frac{l}{d} = K * N * F1 * F2 * F3$$

$$d = l/k*N*F1 = 5000/1.5*17.7*1.25$$

$$d = 150.66 \text{ mm}$$

For panel2

$$\frac{l}{d} = K * N * F1 * F2 * F3$$

$$d = 6000/17.17*1.5*1.25$$

$$d = 180.79 \text{ mm}$$

From the table maximum of all is 180.79mm, so d=180.79mm

The concrete cover =20mm

The diameter of the bar =10mm

Depth of the slab = d + Cc + 2* Ø/2.....since take effective depth between the two bars.

$$D = 180.79 + 20 + 10 = 210.79 \text{ mm}$$

Thus, Provide D=220mm

3.1.4 SLAB DEAD LOAD AND DESIGN LOAD CALCULATION

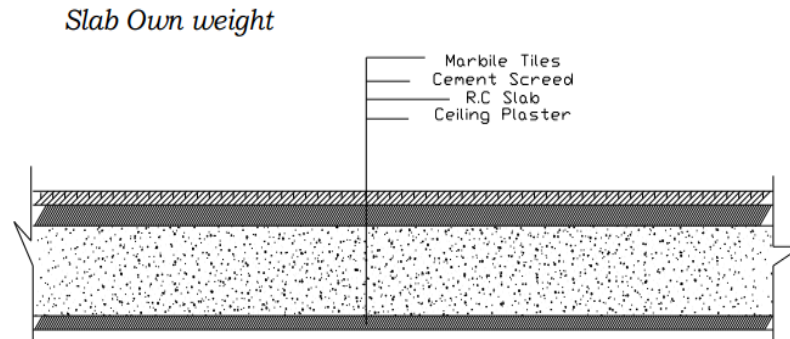


Figure 3. 2 slab sectional view

$$\text{GK of slab} = 0.22\text{m} \times 25\text{KN/m}^3 = 5.5\text{KN/m}^2$$

$$\text{GK of cement screed} = 23\text{KN/m}^3 \times 0.03\text{m} = 0.69\text{KN/m}^2$$

$$\text{GK of marble} = 0.03\text{m} \times 27\text{KN/m}^3 = 0.81\text{KN/m}^2$$

$$\text{GK of plastering} = 23\text{KN/m}^3 \times 0.02\text{m} = 0.46\text{KN/m}^2$$

$$\text{GK} = 5.5 + 0.69 + 0.46 + 0.81 = 7.46\text{KN/m}^2$$

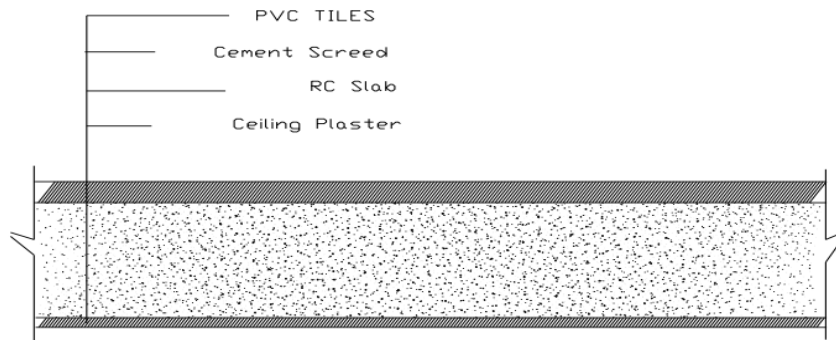


Figure 3. 3 slab section for PVC tiles floor finish

$$220 \text{ mm RC Slab} = 0.22 \times 25 = 5.5\text{KN/m}^2$$

$$20\text{mm Ceiling Plaster} = 0.02 \times 23 = 0.46\text{KN/m}^2$$

$$30\text{mm Cement Screed} = 0.03 \times 23 = 0.69\text{KN/m}^2$$

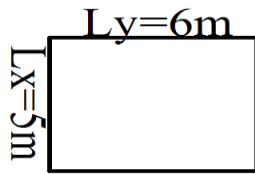
$$2\text{mm PVC Tiles} = 0.02 \times 16 = 0.32\text{KN/m}^2$$

$$\text{Total load GK} = 6.97\text{KN/m}^2$$

$$10\text{mm Ceramic finishing} = 0.01 \times 23 = 0.23\text{KN/m}^2$$

Sample calculation

Panel 1



Marble flooring lobby

Partitions load (i.e. from HCB and plastering)

Only around the edge of the cantilever panels

Live load of lobby is category C1 which is 2-3KN /m² for floors.....(ESEN 1 Table 6.1 n.d.)

Take the average value 2.5KN/m²

Design load $DL=1.35GK+1.5q_k$

$$DL=1.35*7.46+1.5*2.5=13.821\text{KN/m}^2$$

Panel2

Live load of shop is category D which is 4-5 KN /m² for floors.....(ESEN 1 Table 6.1 n.d.)

Take the average value 4.5KN/m²

Design load $DL=1.35GK+1.5q_k$

$$DL=1.35*7.46+1.5*4.5=16.821\text{KN/m}^2$$

For other panels the load is calculated by using excel software.

Table 3. 4 design load determination

Panel	Lx(m)	Ly(m)	Area(m ²)	Wt. of slab	Wt. of marble	Wt. of plaster	GK	Qk	DL
P1	5	6	30	5.5	0.81	1.15	7.461	2.5	13.821
P2	5	6	30	5.5	0.81	1.15	7.461	4.5	16.821

The rest results are in the appendix part

From the above weight of plastering includes both weight of ceiling plaster and weight of cement screed since they have the same unit weight=23 KN/m³.

3.1.5 DESIGN MOMENT

Coefficient method

Support condition for panel 1

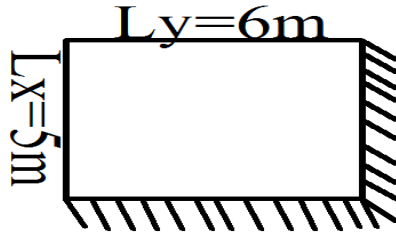


Figure 3. 4 shows the type of slab and its support condition

$$L_y/L_x = 5/6 = 1.2$$

Using the ratio read the following values from.....EBCS 2 1995

$$\alpha_{xs} = 0.063, \alpha_{xf} = 0.047, \alpha_{ys} = 0.047 \text{ and } \alpha_{yf} = 0.036$$

$$DL = 13.84 \text{KN/m}^2$$

$$M_{xs} = \alpha_{xs} * DL * L_x^2 = 0.063 * 13.84 * 5^2 = 21.798 \text{KNm/m}$$

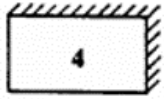
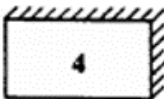
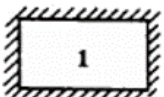
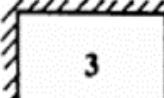
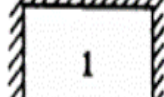
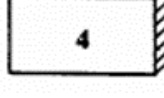
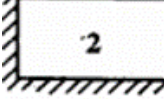
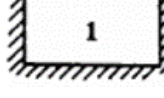
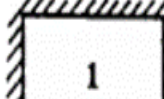
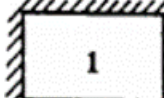
$$M_{xf} = \alpha_{xf} * DL * L_x^2 = 0.047 * 13.84 * 5^2 = 16.262 \text{KNm/m}$$

$$M_{ys} = \alpha_{ys} * DL * L_x^2 = 0.047 * 13.84 * 5^2 = 16.262 \text{KNm/m}$$

$$M_{yf} = \alpha_{yf} * DL * L_x^2 = 0.036 * 13.84 * 5^2 = 12.456 \text{KNm/m}$$

The rest moment is calculated by using excel software.

Table 3. 5 Moment result

panel	support condtion	Lx	Ly	Lx2	Ly/Lx	DL	coficients		Md= α_i *DL*Lx2	
P1		5	6	25	1.2	13.821	α_{xs}	0.063	Mxs	21.7681
		5	6	25	1.2	13.821	α_{xf}	0.047	Mxf	16.2397
		5	6	25	1.2	13.821	α_{ys}	0.047	Mys	16.2397
		5	6	25	1.2	13.821	α_{yf}	0.036	Myf	12.4389
P2		5	6	25	1.2	16.821	α_{xs}	0.063	Mxs	26.4931
		5	6	25	1.2	16.821	α_{xf}	0.047	Mxf	19.7647
		5	6	25	1.2	16.821	α_{ys}	0.047	Mys	19.7647
		5	6	25	1.2	16.821	α_{yf}	0.036	Myf	15.1389
P3		6	6	36	1	14.946	α_{xs}	0.032	Mxs	17.2178
		6	6	36	1	14.946	α_{xf}	0.024	Mxf	12.9133
		6	6	36	1	14.946	α_{ys}	0.032	Mys	17.2178
		6	6	36	1	14.946	α_{yf}	0.024	Myf	12.9133
P5		5	5	25	1	16.821	α_{xs}	0.039	Mxs	16.4005
P7		5	5	25	1	16.821	α_{xf}	0.03	Mxf	12.6158
P9		5	5	25	1	16.821	α_{ys}	0.039	Mys	16.4005
		5	5	25	1	16.821	α_{yf}	0.03	Myf	12.6158
P4,P6		5	6	30	1.2	16.821	α_{xs}	0.042	Mxs	21.1945
P8		5	6	30	1.2	16.821	α_{xf}	0.032	Mxf	16.1482
P10		5	6	30	1.2	16.821	α_{ys}	0.032	Mys	16.1482
		5	6	30	1.2	16.821	α_{yf}	0.024	Myf	12.1111
P11		2.9	5	15	1.72	16.038	α_{xs}	0.087	Mxs	20.2319
		2.9	5	15	1.72	16.038	α_{xf}	0.065	Mxf	15.1158
		2.9	5	15	1.72	16.038	α_{ys}	0.047	Mys	10.9299
P22		2.9	5	15	1.72	16.038	α_{yf}	0.036	Myf	8.37184
P13		5	6	30	1.2	13.038	α_{xs}	0.048	Mxs	18.7747
		5	6	30	1.2	13.038	α_{xf}	0.036	Mxf	14.081
		5	6	30	1.2	13.038	α_{ys}	0.039	Mys	15.2545
		5	6	30	1.2	13.038	α_{yf}	0.029	Myf	11.3431
P14		5	6	30	1.2	14.659	α_{xs}	0.042	Mxs	18.4703
P21		5	6	30	1.2	14.659	α_{xf}	0.032	Mxf	14.0726
P22		5	6	30	1.2	14.659	α_{ys}	0.032	Mys	14.0726
P23		5	6	30	1.2	14.659	α_{yf}	0.024	Myf	10.5545
P15		5	5	25	1	14.659	α_{xs}	0.032	Mxs	11.7272
P16		5	5	25	1	14.659	α_{xf}	0.024	Mxf	8.7954
P17		5	5	25	1	14.659	α_{ys}	0.032	Mys	11.7272
P18										
P19		5	5	25	1	14.66	α_{yf}	0.024	Myf	8.7957
P20		6	6	36	1	14.695	α_{xs}	0.032	Mxs	16.9286
		6	6	36	1	14.659	α_{xf}	0.024	Mxf	12.6654
		6	6	36	1	14.659	α_{ys}	0.032	Mys	16.8872
		6	6	36	1	14.659	α_{yf}	0.024	Myf	12.6654

The moment for cantilever slab is calculated as follows by using the formula, $Md = WL^2/2$

For panel C1

$$Md = 12.321 * 1.5^2/2 = 13.861\text{KNm/m}$$

It is similar for all cantilevers

One way slab Panel 12

For analysis let's assume that the slab is a 1m wide beam spanning in the major axis given in the figure.

$$Pd = 21.321\text{KN/m}^2 * 1\text{m} = 21.321\text{KN/m}$$

a. To determine maximum span (positive) moment

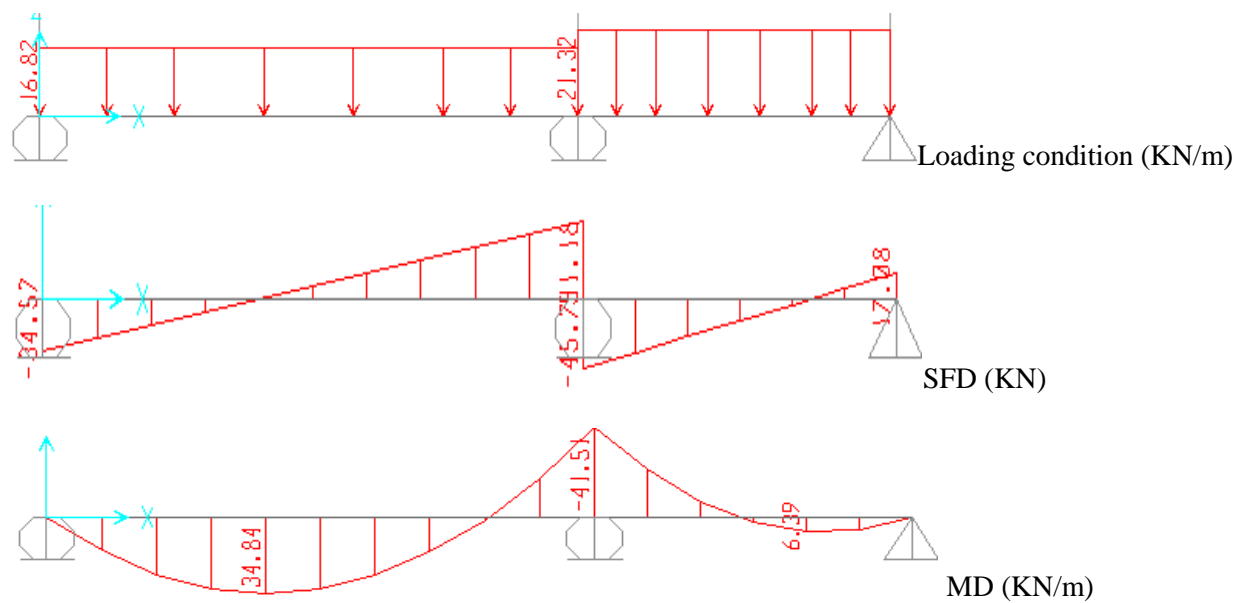
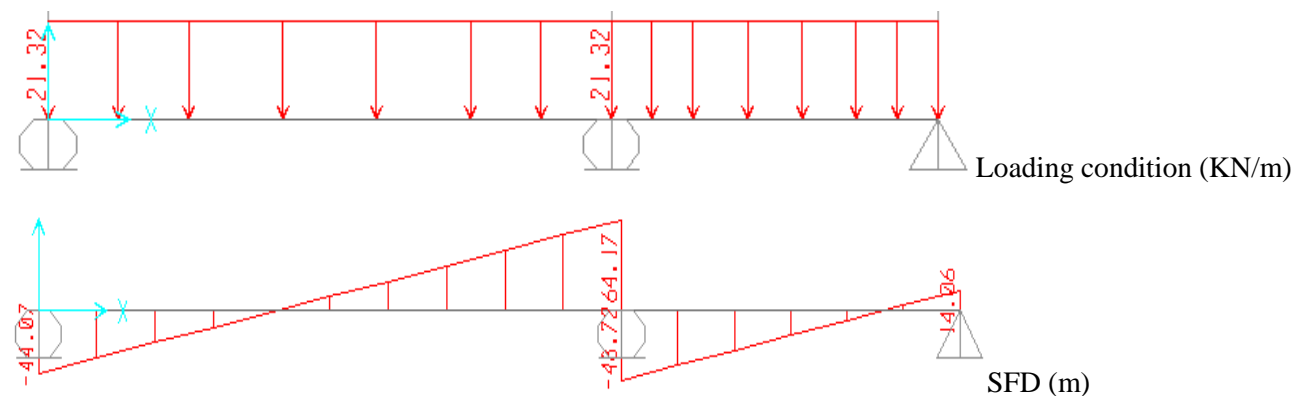


Figure 3. 5 Loading condition for maximum span moment

b. To determine maximum support (negative) moment and shear at interior support



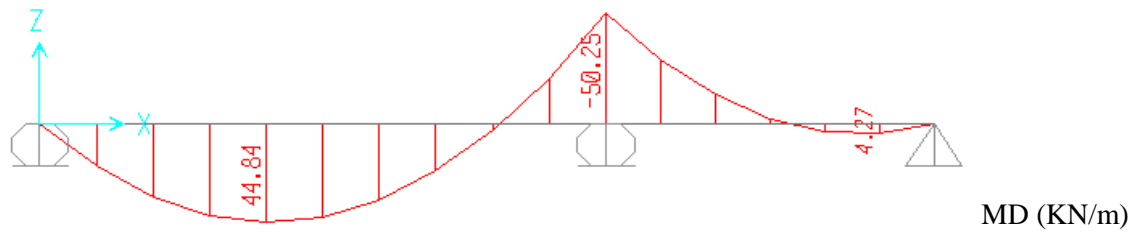


Figure 3. 6 Loading condition for maximum support moment and maximum shear

From analysis it was determined that:

Maximum span moment = 6.39kN-m/m

Maximum support moment = 50.26kNm/m

Maximum shear force at the interior support = 17.08kN/m

Maximum shear force at the exterior support = 48.72kN/m

3.1.6 SLAB DESIGN WITH STRIP METHOD

Slab with corner opening, for panel S2, S3, S4

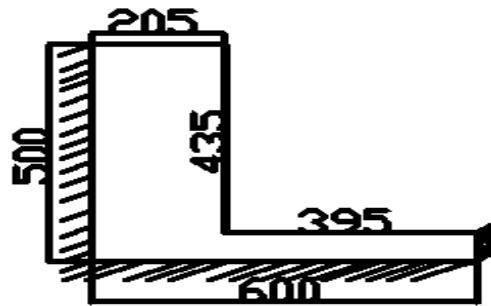


Figure 3. 7 Panel layout for S-2

Moment without opening [basic case]

Design Load = 16.821KN/m²

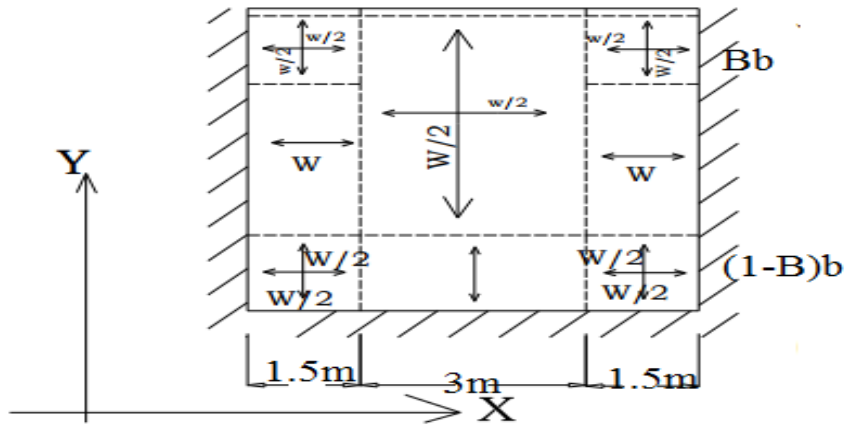
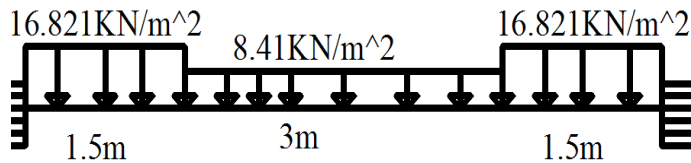


Figure 3. 8 load distribution on strip

Since the slab is rectangular $\beta=0.26$, thus the width of strong band along the free edge $\beta b/2 = 0.65\text{m}$

In the main slab portion $\frac{(1-0.26)*5}{2}=1.85\text{m}$

In the X-direction



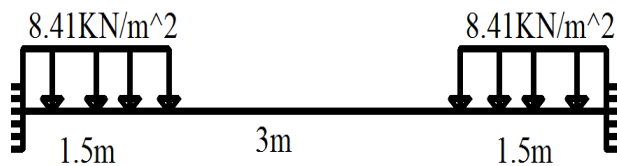
Cantilever moment

$$M_x = \frac{5 \cdot w \cdot a^2}{64} = \frac{5 \cdot 16.82 \cdot 6^2}{64} = 47.309 \text{ kN-m/m}$$

Field and support moments will be calculated using the ratio of two

$$M_{xf} = \frac{1}{3} \cdot 47.309 = 15.769 \text{ kN-m/m}$$

$$M_{xs} = \frac{2}{3} \cdot 47.309 = 31.529 \text{ kN-m/m}$$

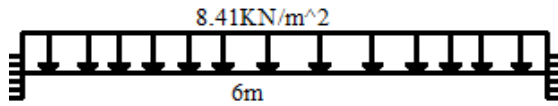


Cantilever moment

$$M_x = \frac{w \cdot a^2}{64} = \frac{8.41 \cdot 6^2}{64} = 4.73 \text{ kN-m/m}$$

$$M_{xf} = \frac{1}{3} \cdot 4.73 = 1.5767 \text{ kN-m/m}$$

$$M_{xs} = 2/3 * 4.73 = 3.153 \text{KN-m/m}$$

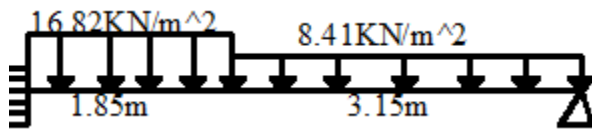


$$M_{xf} = \frac{w \cdot a^2}{24} = \frac{8.41 \cdot 6^2}{24} = 12.615 \text{KN-m/m}$$

$$M_{xs} = \frac{w \cdot a^2}{12} = \frac{8.41 \cdot 6^2}{12} = 25.23 \text{KN-m/m}$$

In the y-direction

Middle strip



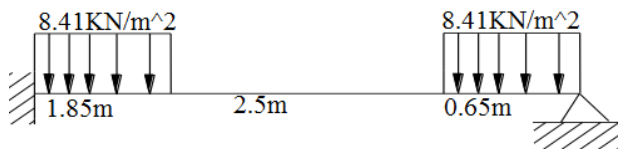
Cantilever moment

$$M_y = 16.82 \cdot 1.85^2 + 8.41 \cdot (3.15 + 0.65/2) \cdot 0.65 = 76.56 \text{KN-m/m}$$

$$M_{yf} = 1/3 \cdot 76.56 = 25.52 \text{KN-m/m}$$

$$M_{ys} = 2/3 \cdot 76.56 = 51.04 \text{KN-m/m}$$

Edge strip



Cantilever moment

$$M_y = 8.41 \cdot \frac{2.5^2}{2} = 26.28 \text{KN-m/m}$$

$$M_{ys} = 1/3 \cdot 26.28 = 8.76 \text{KN-m/m}$$

$$M_{xs} = 2/3 \cdot 26.28 = 17.52 \text{KN-m/m}$$

In order to support the slab strip cut by the hole an arrangement of strong band provided as shown in the sketch

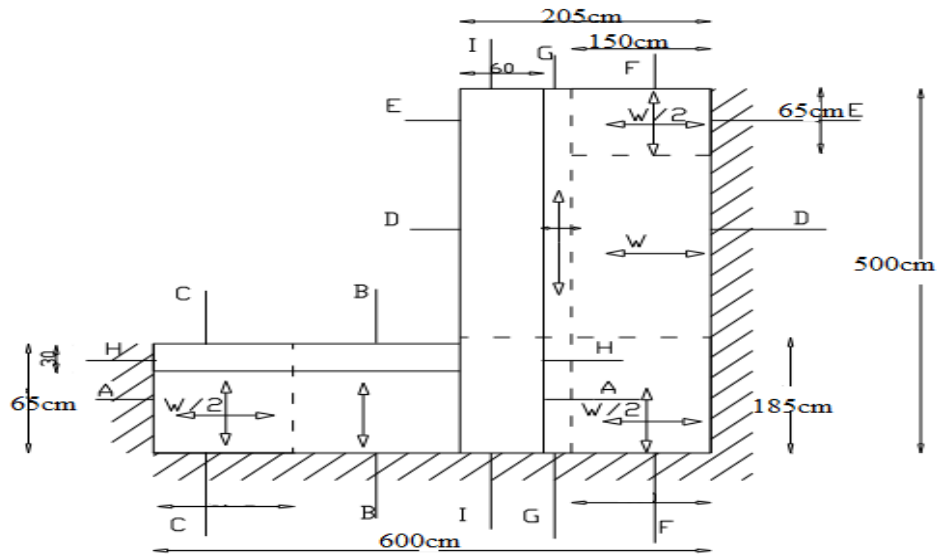
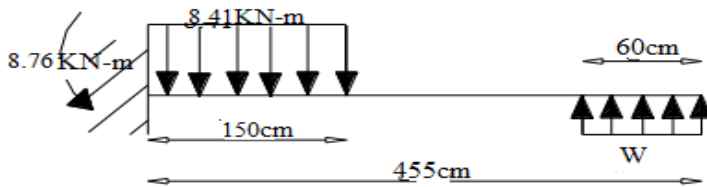


Figure 3. 9 layout for strong band

Strip A-A



$$8.76 - 8.41 \cdot 1.5 \cdot 0.5 + w \cdot 0.6 \cdot 4.25 = 0, w = 0.96 \text{ kN-m}$$

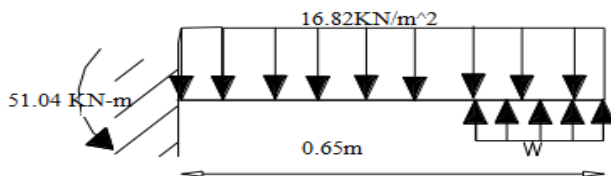
Cantilever moment

$$M_x = \frac{wl^2}{2} = \frac{8.41 \cdot 1.5^2}{2} = 9.46 \text{ kN-m}$$

$$M_{xf} = \frac{1}{3} \cdot 9.46 = 3.15 \text{ kN-m}$$

$$M_{xs} = \frac{2}{3} \cdot 9.46 = 6.3 \text{ kN-m}$$

Strip B-B



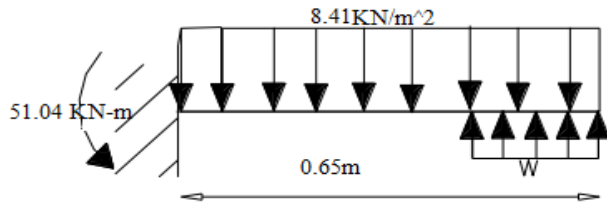
$$51.04 - 16.82 \cdot 0.65^2 \cdot 0.5 + w \cdot 0.3 \cdot 0.5, w = -315.58 \text{ kN-m}$$

$$\text{Cantilever moment} = \frac{16.82 \cdot 0.65^2}{2} = 3.55 \text{ KN-m}$$

$$M_{ys} = 2/3 \cdot 3.55 = 2.369 \text{ KN-m}$$

$$M_{yf} = 1/3 \cdot 3.55 = 1.183 \text{ KN-m}$$

Strip C-C



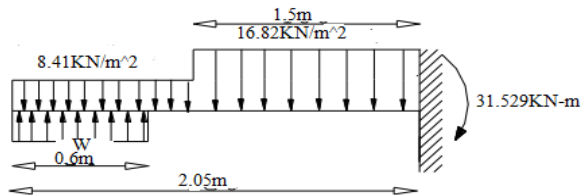
$$51.04 - 8.41 \cdot 0.65^2 \cdot 0.5 + w \cdot 0.3 \cdot 0.5, w = -328.42 \text{ KN-m}$$

$$\text{Cantilever moment} = \frac{8.41 \cdot 0.65^2}{2} = 1.77 \text{ KN-m}$$

$$M_{ys} = 2/3 \cdot 1.77 = 1.184 \text{ KN-m}$$

$$M_{yf} = 1/3 \cdot 1.77 = 0.59 \text{ KN-m}$$

Strip D-D



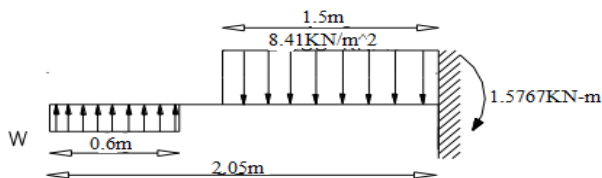
$$31.529 - 16.82 \cdot 1.5^2 \cdot 0.5 - 8.41 \cdot 0.55 \cdot 1.775 + w \cdot 0.6 \cdot 1.75 = 0, w = 13.83 \text{ KN-m}$$

$$\text{Cantilever moment} = 16.82 \cdot 1.5^2 \cdot 0.5 = 18.92 \text{ KN-m}$$

$$M_{xs} = 2/3 \cdot 18.92 = 12.61 \text{ KN-m}$$

$$M_{xf} = 1/3 \cdot 18.92 = 6.306 \text{ KN-m}$$

Strip E-E



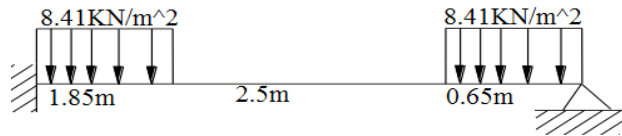
$$1.5767 - 8.41 \cdot 1.5^2 \cdot 0.5 + w \cdot 0.6 \cdot 1.75 = 0, w = 7.509 \text{ KN-m}$$

Cantilever moment=8.41*1.5²*0.5=9.46KN-m

Mxs=2/3*9.46=3.15KN-m

Mxf=1/3*9.46=6.307KN-m

Strip F-F

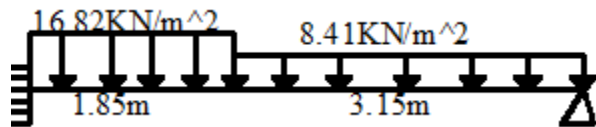


Cantilever moment $m_y=8.41 \cdot \frac{2.5^2}{2}=26.28\text{KN-m/m}$

Mys=1/3*26.28=8.76KN-m/m

Myf=2/3*26.28=17.52KN-m/m

Strip G-G



Cantilever moment

$M_y=16.82 \cdot 1.85^2+8.41 \cdot (3.15+0.65/2) \cdot 0.65=76.56\text{KN-m/m}$

Myf=1/3*76.56=25.52KN-m/m

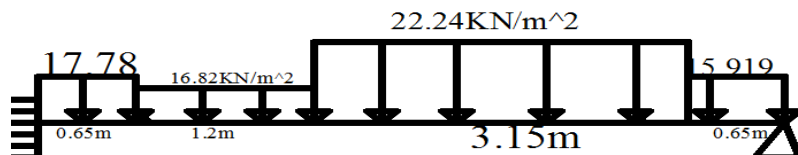
Mys=2/3*76.56=51.04KN-m/m

Strip H-H

Mxf=1/3*9.46=3.15KN-m

Mxs=2/3*9.46=6.3KN-m

Strip I-I



Cantilever Moment

$M=17.78 \cdot \frac{0.65^2}{2}+16.82 \cdot 1.2 \cdot 1.25+22.24 \cdot 3.15 \cdot 3.425+15.919 \cdot 0.65 \cdot 4.675-31.529=285.77\text{KN-m}$

$$M_{xf} = 1/3 * 285.77 = 95.257 \text{KN-m}$$

$$M_{xs} = 2/3 * 285.77 = 190.5 \text{KN-m}$$

3.1.7 MOMENT ADJUSTMENT

Redistribution of support and span moment obtained from a linear analysis may be reduced if $\frac{\Delta M}{M_{max}} \geq 20\%$ in order to maintain equilibrium. Usually it is the maximum support moment reduced, so economizing in reinforcing steel and reduced congestion else take average of the two-support moment.

As per EBCS 2, 1995, A. 3.3.3 if the support moment is decreased, the span moments M_{xf} and M_{yf} are then increased to allow the changes of support moment. This increment is calculated as being equal to the change of the support moment multiplied by the factors given in table A.2. If the support moment is increased, no adjustment shall be made to the span moment.

3.1.8 SUPPORT MOMENT ADJUSTMENT

If $\frac{\Delta M}{M_{max}} \leq 20\%$ take the average of the two support moment

If $\frac{\Delta M}{M_{max}} \geq 20\%$ use nominal moment adjustment

Table 3. 6-sample support moment adjustment

axis	panel	unadjusted moment					adjusted moment
			ΔM	$\Delta M/M_{max}$	M_{avg}	df	M_{ys}
on axis 2	B/n p1 & s4	16.24	3.63	0.2235222		0.5	14.425
		12.61				0.5	14.425
	B/n S1 & p3	17.21	1.217	0.0660444	17.8185		17.82
		18.427					
	B/n p13 & p14	15.25	1.18	0.077377	14.66		14.66
		14.07					
	B/n s2 & p20	16.24	0.64	0.0379147	16.56		16.56
		16.88					
	B/n C1 & C2	13.86	0	0	13.86		13.86

The reset are in the appendix part

Sample calculation

Between panel p1 and s4 $\frac{\Delta M}{M_{max}} = 0.22 \geq 0.2$use nominal moment adjustment

Therefore, calculate the distribution factor (DF) and stiffness factor (K)

$K=1/L$interior slab and $K=0.75/L$end span

$K1=1/6$ and $K2=1/6$

$$DF1 = \frac{K1}{K1+K2}, K1 = \frac{bh^3}{12} * 11 \text{ and } k2 = \frac{bh^3}{12} * 12$$

$Df = 0.5$ for both And $\Delta M=3.63$

$$M_{adjusted} = FEM \pm Df * \Delta M$$

$$M_{adjusted} = \begin{cases} 16.24 - 0.5 * 3.63 = 14.425 \\ 12.61 + 0.5 * 3.63 = 14.425 \end{cases}$$

$$\frac{\Delta M}{M_{max}} = \frac{1.217}{18.427} = 0.066 \leq 20\% \dots\dots\dots \text{take the average of the two support moment}$$

$$M_{avg} = \frac{17.21+18.427}{2} = 17.82 \dots\dots\dots M \text{ adjusted.}$$

3.1.9 FILED MOMENT ADJUSTMENT

Span moment adjustment is calculated for panels whose support moment are lowered during support moment adjustment, because as M_{sup} decreases M_{span} increases. According to EBCS-2-1995:

$$\Delta M_{xf} = C_x \Delta M_{xs} + C_x'' \Delta M_{ys}$$

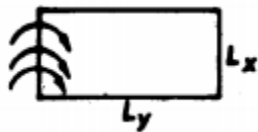
$$\Delta M_{yf} = C_y \Delta M_{ys} + C_y'' \Delta M_{xs}$$

Table 3. 7 adjusted filed moment

panle	$M_{max,sup}$	$M_{adj,sup}$	L_x	L_y	L_y/L_x	C_x	C_y	ΔM	M_{xf}	M_{yf}	$C_x \Delta M$	$C_y \Delta M$	$M_{xf,adj}$	$M_{yf,adj}$
p1	21.76	15.49	5	6	1.2	0.344	0.364	6.27	16.2	12.43	2.15688	2.28228	18.35688	14.71228
p2	19.76	13.76	5	5	1	0.38	0.28	6	19.76	15.14	2.28	1.68	22.04	16.82

The reset are in the appendix part

Sample calculation for panel 1:- Case1, $\frac{L_y}{L_x} = \frac{6}{5} = 1.2$

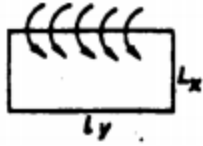


$$C_x=0.338, C_y=0.172 \text{ and } \Delta M = 16.23 - 14.42=1.58$$

$$M_{xf,adj} = 16.23 + 1.58 * 0.338 = 16.76 \text{KN-m}$$

$$M_{yf,adj} = 12.43 + 1.58 * 0.172 = 12.7 \text{KN-m}$$

Case2, $C_x=0.344$ and $C_y =0.364$



$$\Delta M = 21.76 - 15.49 = 6.27$$

$$M_{xf, adj} = 16.23 + 6.27 * 0.344 = 18.35 \text{ KN-m}$$

$$M_{yf, adj} = 12.43 + 6.27 * 0.364 = 14.71 \text{ KN-m}$$

Hence take the maximum of two cases.

3.1.10 REINFORCEMENT

The total depth for two-way panels and cantilevers is 220mm. From this the effective depth of slabs are measured from compression face of the slab to center of reinforcement on the tension side and can be calculated as follows.

$$\text{For shorter, } d_x = D - \text{cover} - \frac{\phi}{2} = 220 \text{ mm} - 20 \text{ mm} - 5 \text{ mm} = 195 \text{ mm}$$

$$\text{For longer, } d_y = D - \text{cover} - \phi - \frac{\phi}{2} = 220 \text{ mm} - 20 \text{ mm} - 10 \text{ mm} - 5 \text{ mm} = 185 \text{ mm}$$

Where: D-total depth of the slab,

d_x -effective depth of main reinforcement,

d_y -effective depth of transverse reinforcement, and

ϕ is bar diameter.

$$f_{ctm} (\text{Mpa}) = 2.2 \dots \dots \dots \text{ES EN 1992 table 3 for C20/25}$$

Sample calculation panel 1

Main reinforcement flexural design

$$M_{xsd} = 15.49 \text{ KNm, } d_x = 195 \text{ mm and } b = 1 \text{ m}$$

$$\mu_{sd} = \frac{M_{sd}}{f_{cd} * b * d^2} = \frac{15.49}{11.33 * 1000 * 195^2} = 0.0359$$

$$\mu_{sd} = 0.0359 < \mu_{us}^* = 0.295 (\text{For } 0\% \text{ moment redistribution}) \text{ design as singly reinforced section}$$

Using $\mu_{sd} = 0.0359$ read K_z from the general design chart No.1a. (EBCS 2).

$$K_z = 0.976$$

$$z = K_z * d = 0.976 * 195 = 190.32 \text{ mm}$$

$$A_{st, calc} = \frac{M_{ED}}{f_{yd} * z} = \frac{15.49}{347.83 * 190.32} = 233.99 \text{ mm}^2$$

The area of reinforcement should not be taken less than the minimum area of reinforcement given by ES EN 1992-1-1:2015, Expression 9.1N in order to control shrinkage, and cracking. And should not be taken greater than the maximum area of reinforcement provided by the code because if it is greater than the maximum value the ductility will be altered, and there will be congestion of reinforcement bar.

$$A_{st, \min} = \max \left\{ \begin{array}{l} 0.26 * \frac{f_{ctm} * b * t * d}{f_{yk}} = 0.26 * \frac{2.2 * 1000 * 195}{400} = 277.42 \text{mm}^2 \\ 0.0013 * b * t * d = 0.0013 * 1000 * 195 = 253.5 \text{mm}^2 \end{array} \right. \dots \text{ES EN 1992: 2015 Art. 9.2.1.1. (1).}$$

$$A_{st, \min} = 277.42 \text{mm}^2 < A_{st, \text{calc}} = 233.99 \text{mm}^2, \text{ provided } A_{st} = 277.42 \text{mm}^2$$

$$A_{st, \max} = 0.04 A_c = 0.04 * 1000 * 220 = 8800 \text{mm}^2 \dots \dots \text{ES EN 1992-1-1:2015 section 9.2.1.1(3)}$$

$$A_{st, \text{prov}} = 277.42 \text{mm}^2 < A_{st, \max} = 8800 \text{mm}^2 \dots \dots \dots \text{ok}$$

$$\text{Spacing of the bar, } S = \frac{b * t * a_{st}}{A_{st, \text{prov}}}$$

$$S = \frac{1000 * 78.5}{277.42} = 282.96 \text{mm}, \text{ provided } S = 270 \text{mm}$$

The spacing of the reinforcement bar should not exceed the maximum spacing (S_{ax}) given ES EN 1992-1-1:2015 section 9.3.1

$$S_{\max} = \min \left\{ \begin{array}{l} 3h = 3 * 220 = 660 \text{mm} \\ 400 \text{mm} \end{array} \right., \text{ take } S = 400 \text{mm}$$

$$S_{\text{prov}} = 270 \text{mm} < S_{\max} = 400 \text{mm} \dots \dots \dots \text{ok}$$

Provide principal reinforcement bar Ø10mm c/c 270mm

For the rest panels are designed by using excel

Table 3. 8 slab filed reinforcement

panel	location	MED	fed	fyd	b	d	μ_{sd}	Kz	Ascalc	Aspro	Scalc	Sprovidded
p1	short span	18.35	11.33	347.8	1000	195	0.0426	0.9776	276.741	276.741	284	Ø10C/C280mm
	long span	14.71	11.33	347.8	1000	185	0.0379	0.978	233.741	264.55	297	Ø10C/C290mm
p2	short span	22.04	11.33	347.8	1000	195	0.0512	0.9589	338.873	338.873	232	Ø10C/C230mm
	long span	16.82	11.33	347.8	1000	185	0.0434	0.976	267.817	267.817	293	Ø10C/C290mm
p3	short span	12.91	11.33	347.8	1000	195	0.03	0.9797	194.282	277.42	283	Ø10C/C280mm
	long span	12.91	11.33	347.8	1000	185	0.0333	0.9785	205.034	264.55	297	Ø10C/C290mm
p4	short span	17.87	11.33	347.8	1000	195	0.0415	0.9689	271.922	277.42	283	Ø10C/C280mm
	long span	12.99	11.33	347.8	1000	185	0.0335	0.9784	206.326	264.55	297	Ø10C/C290mm
p5	short span	12.61	11.33	347.8	1000	195	0.0293	0.9799	189.728	277.42	283	Ø10C/C280mm
	long span	12.61	11.33	347.8	1000	185	0.0325	0.9781	200.352	264.55	297	Ø10C/C290mm
p6	short span	17.87	11.33	347.8	1000	195	0.0415	0.9689	271.922	277.42	283	Ø10C/C280mm
	long span	12.99	11.33	347.8	1000	185	0.0335	0.9784	206.326	264.55	297	Ø10C/C290mm
p7	short span	12.61	11.33	347.8	1000	195	0.0293	0.9799	189.728	277.42	283	Ø10C/C280mm
	long span	12.61	11.33	347.8	1000	185	0.0325	0.9781	200.352	264.55	297	Ø10C/C290mm
p8	short span	17.87	11.33	347.8	1000	195	0.0415	0.9689	271.922	277.42	283	Ø10C/C280mm
	long span	12.99	11.33	347.8	1000	185	0.0335	0.9781	206.389	264.55	297	Ø10C/C290mm
p9	short span	13.92	11.33	347.8	1000	195	0.0323	0.9784	209.759	277.42	283	Ø10C/C280mm
	long span	13.57	11.33	347.8	1000	185	0.035	0.9768	215.891	264.55	297	Ø10C/C290mm
p10	short span	16.14	11.33	347.8	1000	195	0.0375	0.9754	243.96	277.42	283	Ø10C/C280mm
	long span	12.11	11.33	347.8	1000	185	0.0312	0.9786	192.309	264.55	297	Ø10C/C290mm

p11	short span	17.28	11.33	347.8	1000	195	0.0401	0.9699	262.673	277.42	283	Ø10C/C280mm
	long span	8.86	11.33	347.8	1000	185	0.0228	0.9828	140.097	264.55	297	Ø10C/C290mm
p12	short span	6.39	11.33	347.8	1000	195	0.0148	0.9889	95.268	277.42	283	Ø10C/C280mm
p13	short span	15.38	11.33	347.8	1000	195	0.0357	0.9765	232.211	277.42	283	Ø10C/C280mm
	long span	12.73	11.33	347.8	1000	185	0.0328	0.9783	202.217	264.55	297	Ø10C/C290mm
p14	short span	14.46	11.33	347.8	1000	195	0.0336	0.978	217.986	277.42	283	Ø10C/C280mm
	long span	10.75	11.33	347.8	1000	185	0.0277	0.9787	170.695	264.55	297	Ø10C/C290mm
p15	short span	8.79	11.33	347.8	1000	195	0.0204	0.9876	131.222	277.42	283	Ø10C/C280mm
	long span	8.79	11.33	347.8	1000	185	0.0227	0.9829	138.976	264.55	297	Ø10C/C290mm
p16	short span	8.79	11.33	347.8	1000	195	0.0204	0.9876	131.222	277.42	283	Ø10C/C280mm
	long span	8.79	11.33	347.8	1000	185	0.0227	0.9829	138.976	264.55	297	Ø10C/C290mm
p17	short span	8.79	11.33	347.8	1000	195	0.0204	0.9876	131.222	277.42	283	Ø10C/C280mm
	long span	8.79	11.33	347.8	1000	185	0.0227	0.9829	138.976	264.55	297	Ø10C/C290mm
p18	short span	10.68	11.33	347.8	1000	195	0.0248	0.9819	160.362	277.42	283	Ø10C/C280mm
	long span	11.86	11.33	347.8	1000	185	0.0306	0.9789	188.281	264.55	297	Ø10C/C290mm
p19	short span	15.11	11.33	347.8	1000	195	0.0351	0.9767	228.088	277.42	283	Ø10C/C280mm
	long span	8.37	11.33	347.8	1000	185	0.0216	0.9865	131.853	264.55	297	Ø10C/C290mm
p20	short span	13.19	11.33	347.8	1000	195	0.0306	0.9789	198.657	277.42	283	Ø10C/C280mm
	long span	13.05	11.33	347.8	1000	185	0.0337	0.977	207.576	264.55	297	Ø10C/C290mm
p21	short span	15.55	11.33	347.8	1000	195	0.0361	0.9749	235.163	277.42	283	Ø10C/C280mm
	long span	14.82	11.33	347.8	1000	185	0.0382	0.9719	236.967	264.55	297	Ø10C/C290mm
P22	short span	15.55	11.33	347.8	1000	195	0.0361	0.9749	235.163	277.42	283	Ø10C/C280mm
	long span	14.82	11.33	347.8	1000	185	0.0382	0.9719	236.967	264.55	297	Ø10C/C290mm
P23	short span	15.55	11.33	347.8	1000	195	0.0361	0.9749	235.163	277.42	283	Ø10C/C280mm
	long span	14.82	11.33	347.8	1000	185	0.0382	0.9719	236.967	264.55	297	Ø10C/C290mm

Support reinforcement in the appendix part

Slab design for 3rd floor

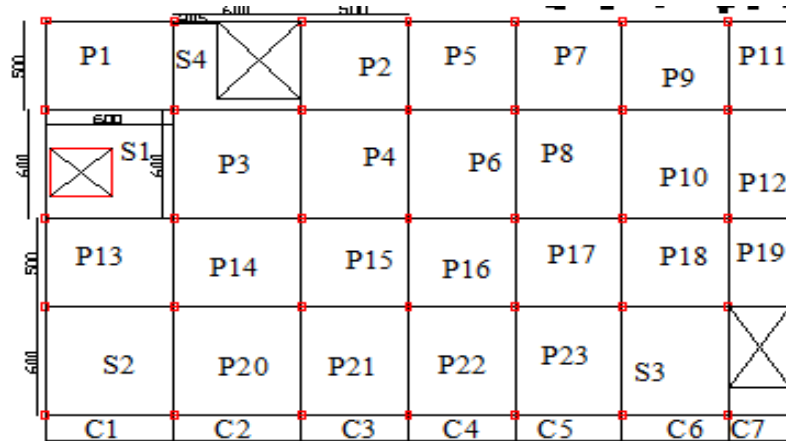


Figure 3.10 slab layout from 3rd floor

Sample calculation

For panel1

$$\frac{l}{d} = K * N * F1 * F2 * F3$$

$$d = l / (K * N * F1) = 5000 / (1.3 * 17.7 * 1.25)$$

$$d = 173.83 \text{ mm}$$

For panel2

$$\frac{l}{d} = K * N * F1 * F2 * F3$$

$$d = 5000 / (17.17 * 1.5 * 1.25)$$

$$d = 150.659 \text{ mm}$$

From the table maximum of all is 173.83mm, so $d = 173.83 \text{ mm}$

Concrete cover = 20mm and diameter of the bar = 10mm

Depth of the slab = $d + Cc + 2 * \varnothing / 2$... since take effective depth between the two bars.

$$D = 173.83 + 20 + 10 = 203.83 \text{ mm. Thus, Provide } D = 210 \text{ mm}$$

Load calculation

$$2 \text{ cm thick PVC tile} = 0.02 \text{ m} * 16 \text{ KN/m}^3 = 0.32 \text{ KN/m}^2, \text{ Plastering} = 0.02 \text{ m} * 23 \text{ KN/m} = 0.46 \text{ KN/m}^2$$

$$3 \text{ cm cement screed} = 0.03 \text{ m} * 23 \text{ KN/m}^3 = 0.69 \text{ KN/m}^2, \text{ GK of slab} = 0.21 \text{ m} * 25 \text{ KN/m}^3 = 5.25 \text{ KN/m}^2$$

$$\text{GK of marble} = 0.03\text{m} * 27\text{KN/m}^3 = 0.81\text{KN/m}^2$$

$$\text{GKslab} = 0.32 + 0.69 + 0.46 + 5.25 = 6.72\text{KN/m}^2$$

$$\text{GK} = 5.25 + 0.69 + 0.46 + 0.81 = 7.21\text{KN/m}^2$$

Sample calculation

For panel 1 - Design load $DL = 1.35GK + 1.5qk$

$$DL = 1.35 * 7.21 + 1.5 * 2.5 = 13.48\text{KN/m}^2$$

$$M_{xs} = \alpha_{xs} * DL * L_x^2 = 0.063 * 13.48 * 5^2 = 21.231\text{KNm/m}$$

$$M_{xf} = \alpha_{xf} * DL * L_x^2 = 0.047 * 13.48 * 5^2 = 15.839\text{KNm/m}$$

$$M_{ys} = \alpha_{ys} * DL * L_x^2 = 0.047 * 13.48 * 5^2 = 15.839\text{KNm/m}$$

$$M_{yf} = \alpha_{yf} * DL * L_x^2 = 0.036 * 13.48 * 5^2 = 12.132\text{KNm/m}$$

For Panel 6

Design load $DL = 1.35GK + 1.5qk$

$$DL = 1.35 * 6.72 + 1.5 * 3 = 12.9\text{KN/m}^2$$

$$M_{xs} = \alpha_{xs} * DL * L_x^2 = 0.063 * 12.9 * 5^2 = 20.317\text{KNm/m}$$

$$M_{xf} = \alpha_{xf} * DL * L_x^2 = 0.047 * 12.9 * 5^2 = 15.157\text{KNm/m}$$

$$M_{ys} = \alpha_{ys} * DL * L_x^2 = 0.047 * 12.9 * 5^2 = 15.157\text{KNm/m}$$

$$M_{yf} = \alpha_{yf} * DL * L_x^2 = 0.036 * 12.9 * 5^2 = 11.61\text{KNm/m}$$

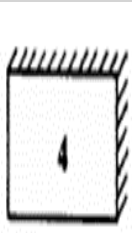

The moment for cantilever slab is calculated as follows by using the formula, $Md = WL^2/2$

$$\text{For panel C1, } Md = 14.6085 * 1.5^2/2 = 16.434\text{KNm/m}$$

It is similar for all cantilevers

The reset excel values are in the appendix part including support reinforcement

Table 3. 9 3rd floor slab moment

panel	support condtion	Lx	Ly	Lx2	Ly/Lx	DL	coficients		Md= α_i *DL*Lx2	
P1		5	6	25	1.2	13.4835	α_{xs}	0.063	Mxs	21.2365
		5	6	25	1.2	13.4835	α_{xf}	0.047	Mxf	15.8431
		5	6	25	1.2	13.4835	α_{ys}	0.047	Mys	15.8431
		5	6	25	1.2	13.4835	α_{yf}	0.036	Myf	12.1352
P2		5	6	25	1.2	12.358	α_{xs}	0.063	Mxs	19.4639
		5	6	25	1.2	12.358	α_{xf}	0.047	Mxf	14.5207
		5	6	25	1.2	12.358	α_{ys}	0.047	Mys	14.5207
		5	6	25	1.2	12.358	α_{yf}	0.036	Myf	11.1222

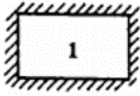
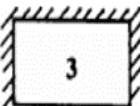
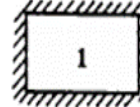
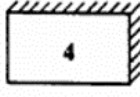
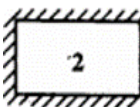
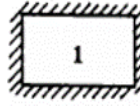
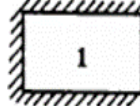
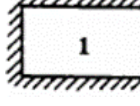
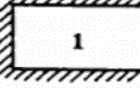
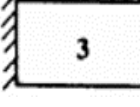
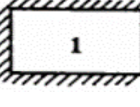
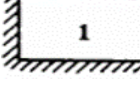
P3		6	6	36	1	13.4835	α_{xs}	0.032	Mxs	15.533
		6	6	36	1	13.4835	α_{xf}	0.024	Mxf	11.6497
		6	6	36	1	13.4835	α_{ys}	0.032	Mys	15.533
		6	6	36	1	13.4835	α_{yf}	0.024	Myf	11.6497
P5		5	5	25	1	12.358	α_{xs}	0.039	Mxs	12.0491
P7		5	5	25	1	12.358	α_{xf}	0.03	Mxf	9.2685
P9		5	5	25	1	12.358	α_{ys}	0.039	Mys	12.0491
		5	5	25	1	12.358	α_{yf}	0.03	Myf	9.2685
P4		5	6	25	1.2	12.358	α_{xs}	0.042	Mxs	12.9759
P8		5	6	25	1.2	12.358	α_{xf}	0.032	Mxf	9.8864
		5	6	25	1.2	12.358	α_{ys}	0.032	Mys	9.8864
		5	6	25	1.2	12.358	α_{yf}	0.024	Myf	7.4148
P11		2.9	5	8.41	1.7241	12.358	α_{xs}	0.087	Mxs	9.04198
		2.9	5	8.41	1.7241	12.358	α_{xf}	0.065	Mxf	6.7555
		2.9	5	8.41	1.7241	12.358	α_{ys}	0.047	Mys	4.88475
		2.9	5	8.41	1.7241	12.358	α_{yf}	0.036	Myf	3.74151
P13		5	6	25	1.2	12.822	α_{xs}	0.048	Mxs	15.3864
P14		5	6	25	1.2	12.822	α_{xf}	0.036	Mxf	11.5398
		5	6	25	1.2	12.822	α_{ys}	0.039	Mys	12.5015
		5	6	25	1.2	12.822	α_{yf}	0.029	Myf	9.29595
P21,P18		5	6	25	1.2	12.822	α_{xs}	0.042	Mxs	13.4631
P22		5	6	25	1.2	12.822	α_{xf}	0.032	Mxf	10.2576
P23		5	6	25	1.2	12.822	α_{ys}	0.032	Mys	10.2576
P15		5	6	25	1.2	12.822	α_{yf}	0.024	Myf	7.6932
P16		5	5	25	1	12.822	α_{xs}	0.032	Mxs	10.2576
P17		5	5	25	1	12.822	α_{xf}	0.024	Mxf	7.6932
P19		5	5	25	1	12.822	α_{ys}	0.032	Mys	10.2576
		5	5	25	1	12.822	α_{yf}	0.024	Myf	7.6932
P20		6	6	36	1	12.822	α_{xs}	0.032	Mxs	14.7709
		6	6	36	1	12.822	α_{xf}	0.024	Mxf	11.0782
		6	6	36	1	12.822	α_{ys}	0.032	Mys	14.7709
		6	6	36	1	12.822	α_{yf}	0.024	Myf	11.0782
s2		6	6	36	1	13.483	α_{xs}	0.032	Mxs	15.5324
		6	6	36	1	13.483	α_{xf}	0.024	Mxf	11.6493
		6	6	36	1	13.483	α_{ys}	0.032	Mys	15.5324
		6	6	36	1	13.483	α_{yf}	0.024	Myf	11.6493
s3		5	6	25	1.2	12.822	α_{xs}	0.056	Mxs	17.9508
		5	6	25	1.2	12.822	α_{xf}	0.042	Mxf	13.4631
		5	6	25	1.2	12.822	α_{ys}	0.039	Mys	12.5015
		5	6	25	1.2	12.822	α_{yf}	0.03	Myf	9.6165
P6		5	6	25	1.2	14.23	α_{xs}	0.042	Mxs	14.9415
		5	6	25	1.2	14.23	α_{xf}	0.032	Mxf	11.384
		5	6	25	1.2	14.23	α_{ys}	0.032	Mys	11.384
		5	6	25	1.2	14.23	α_{yf}	0.024	Myf	8.538
P10		5	6	25	1.2	16.48	α_{xs}	0.042	Mxs	17.304
		5	6	25	1.2	16.48	α_{xf}	0.032	Mxf	13.184
		5	6	25	1.2	16.48	α_{ys}	0.032	Mys	13.184
		5	6	25	1.2	16.48	α_{yf}	0.024	Myf	9.888

Table 3. 10 3rd floor slab reinforcement

panel	location	MED	fed	fyd	b	d	μ sd	Kz	Ascalc	Aspro	Scalc	Sprovided
p1	short span	17.902	11.33	347.83	1000	185	0.04617	0.9776	284.58	284.58	275.8	Ø10C/C270mm
	long span	14.313	11.33	347.83	1000	175	0.04125	0.978	240.43	255	307.8	Ø10C/C300mm
p2	short span	15.862	11.33	347.83	1000	185	0.04091	0.9589	257.07	260	301.9	Ø10C/C300mm
	long span	12.11	11.33	347.83	1000	175	0.0349	0.976	203.84	203.84	385.1	Ø10C/C380mm
p3	short span	11.649	11.33	347.83	1000	185	0.03004	0.9797	184.78	189.79	413.6	Ø10C/C400mm
	long span	11.649	11.33	347.83	1000	175	0.03357	0.9785	195.58	203.84	385.1	Ø10C/C380mm
p4	short span	8.574	11.33	347.83	1000	185	0.02211	0.9689	137.52	134.84	582.2	Ø10C/C400mm
	long span	6.746	11.33	347.83	1000	175	0.01944	0.9784	113.27	134.84	582.2	Ø10C/C400mm
p5	short span	12.049	11.33	347.83	1000	185	0.03107	0.9799	191.09	203.84	385.1	Ø10C/C380mm
	long span	12.049	11.33	347.83	1000	175	0.03473	0.9781	202.38	203.84	385.1	Ø10C/C380mm
p6	short span	11.833	11.33	347.83	1000	185	0.03052	0.9689	189.79	189.79	413.6	Ø10C/C400mm
	long span	8.766	11.33	347.83	1000	175	0.02526	0.9784	147.19	147.19	533.3	Ø10C/C400mm
p7	short span	12.049	11.33	347.83	1000	185	0.03107	0.9799	191.09	203.84	385.1	Ø10C/C380mm
	long span	12.049	11.33	347.83	1000	175	0.03473	0.9781	202.38	203.84	385.1	Ø10C/C380mm
p8	short span	10.043	11.33	347.83	1000	185	0.0259	0.9689	161.08	189.79	413.6	Ø10C/C400mm
	long span	7.494	11.33	347.83	1000	175	0.0216	0.9781	125.87	134.84	582.2	Ø10C/C400mm
p9	short span	12.049	11.33	347.83	1000	185	0.03107	0.9784	191.38	203.84	385.1	Ø10C/C380mm
	long span	12.049	11.33	347.83	1000	175	0.03473	0.9768	202.65	203.84	385.1	Ø10C/C380mm
p10	short span	13.9	11.33	347.83	1000	185	0.03585	0.9754	221.46	221.46	354.5	Ø10C/C350mm
	long span	10.244	11.33	347.83	1000	175	0.02952	0.9786	171.97	189.79	413.6	Ø10C/C400mm
p11	short span	7.17	11.33	347.83	1000	185	0.01849	0.9699	114.88	134.84	582.2	Ø10C/C400mm
	long span	3.848	11.33	347.83	1000	175	0.01109	0.9828	64.323	134.84	582.2	Ø10C/C400mm
p12	short span	6.39	11.33	347.83	1000	185	0.01648	0.9889	100.42	134.84	582.2	Ø10C/C400mm
p13	short span	12.302	11.33	347.83	1000	185	0.03173	0.9765	195.78	203.84	385.1	Ø10C/C380mm
	long span	7.519	11.33	347.83	1000	175	0.02167	0.9783	126.26	134.84	582.2	Ø10C/C400mm
p14	short span	9.915	11.33	347.83	1000	185	0.02557	0.978	157.55	189.79	413.6	Ø10C/C400mm
	long span	7.519	11.33	347.83	1000	175	0.02167	0.9787	126.21	134.84	582.2	Ø10C/C400mm
p15	short span	7.693	11.33	347.83	1000	185	0.01984	0.9876	121.05	134.84	582.2	Ø10C/C400mm
	long span	7.693	11.33	347.83	1000	175	0.02217	0.9829	128.58	134.84	582.2	Ø10C/C400mm
p16	short span	7.693	11.33	347.83	1000	185	0.01984	0.9876	121.05	134.84	582.2	Ø10C/C400mm
	long span	7.693	11.33	347.83	1000	175	0.02217	0.9829	128.58	134.84	582.2	Ø10C/C400mm
p17	short span	7.693	11.33	347.83	1000	185	0.01984	0.9876	121.05	134.84	582.2	Ø10C/C400mm
	long span	7.693	11.33	347.83	1000	175	0.02217	0.9829	128.58	134.84	582.2	Ø10C/C400mm
p18	short span	9.623	11.33	347.83	1000	185	0.02482	0.9819	152.3	189.79	413.6	Ø10C/C400mm
	long span	7.022	11.33	347.83	1000	175	0.02024	0.9789	117.85	134.84	582.2	Ø10C/C400mm
p19	short span	7.693	11.33	347.83	1000	185	0.01984	0.9767	122.4	134.84	582.2	Ø10C/C400mm
	long span	7.693	11.33	347.83	1000	175	0.02217	0.9865	128.11	134.84	582.2	Ø10C/C400mm
p20	short span	11.078	11.33	347.83	1000	185	0.02857	0.9789	175.87	189.79	413.6	Ø10C/C400mm
	long span	11.078	11.33	347.83	1000	175	0.03193	0.977	186.28	189.79	413.6	Ø10C/C400mm
p21	short span	10.078	11.33	347.83	1000	185	0.02599	0.9749	160.65	189.79	413.6	Ø10C/C400mm
	long span	7.977	11.33	347.83	1000	175	0.02299	0.9719	134.84	134.84	582.2	Ø10C/C400mm
P22	short span	10.078	11.33	347.83	1000	185	0.02599	0.9749	160.65	189.79	413.6	Ø10C/C400mm
	long span	7.977	11.33	347.83	1000	175	0.02299	0.9719	134.84	134.84	582.2	Ø10C/C400mm
P23	short span	10.073	11.33	347.83	1000	185	0.02598	0.9749	160.57	189.79	413.6	Ø10C/C400mm
	long span	7.599	11.33	347.83	1000	175	0.0219	0.9719	128.45	128.45	611.1	Ø10C/C400mm
s2	short span	15.532	11.33	347.83	1000	185	0.04005	0.9749	247.59	247.58	317.1	Ø10C/C310mm
	long span	15.532	11.33	347.83	1000	175	0.04476	0.9719	262.54	262.54	299	Ø10C/C290mm
s3	short span	13.872	11.33	347.83	1000	185	0.03577	0.9749	221.13	221.12	355	Ø10C/C350mm
	long span	9.825	11.33	347.83	1000	175	0.02832	0.9719	166.08	189.79	413.6	Ø10C/C400mm

3.2 RIBBED SLAB DESIGN

Ribbed floor is formed using temporary or permanent shuttering while the hollow block floor is generally constructed with blocks made of clay tile or with concrete containing a light weight aggregate. If the block is suitably manufactured and have adequate strength they can be considered to contribute to the strength of slab in the design calculation, but in many designs such allowance is not made.

The principal advantages of these floors is reduction in weight achieved by removing the concrete part below the neutral axis and in these case the hollow block floor replacing it with a lighter form of construction. Ribbed and hollow block floors are economical for buildings where there are long spans over about 5m and light or moderate live loads such as hospital, apartment buildings, hotel, schools, shopping etc.

According to ES EN 1992 Art 5.3.1(6), ribbed or waffle slab need not to be treated as discrete element for the purpose of analysis, provided that the flange or structural topping and transverse ribs have sufficient torsional stiffness. This may be assumed that;

1. The ribs spacing does not exceeding 1500mm.
2. The depth of the ribs below the flange does not exceed 4 times its width.
3. The depth of the flange is at least 1/10 of the clear distance between ribs or 50mm which is greater.
4. Transverse ribs are provided at clear spacing not exceeding 10 times the overall depth of the slab.

The minimum flange thickness of 50mm may be reduced to 40mm where permanent blocks are incorporated between the ribs.

Take the design cover from the above calculation in solid slab design which is Cover=20mm

Verify the general requirement for the ribbed slab

- ✓ Take ribs spacing = 400mm < 1.5m.....ok!
- ✓ Ribs width = 80mm > 70mm.....ok!
- ✓ Depth of the ribs = 200mm < (4*80) = 320mm
- ✓ Depth of flange (topping thickness) = $\frac{1}{10} * 320\text{mm} = 32\text{mm} < 50\text{mm}$, take 60mm
- ✓ Depth = 200 + 60 + 30 = 290mm

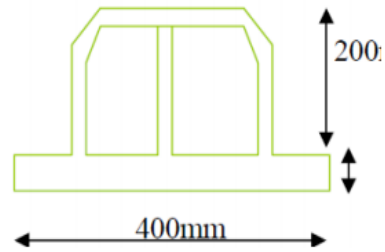


Figure 3.11 layout of HCB for ribbed slab

The bottom dimension may vary 40mm/30mm provided depth $D_p = 290\text{mm}$, the required depth for serviceability $D_c = 155\text{mm}$. safe!

3.2.1 DETERMINATION OF DESIGN LOAD

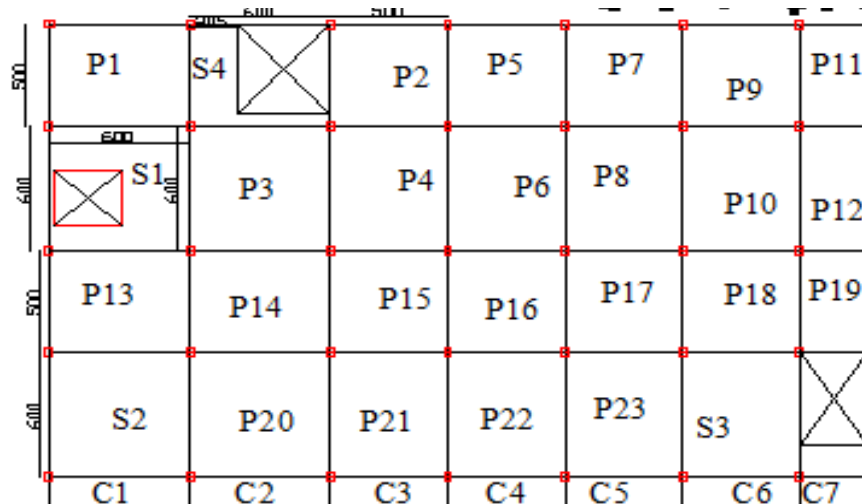


Figure 3.12 slab layout from 4th -5th floor

Load calculation

- ✓ 2cm thick PVC tile = $0.02\text{m} * 0.4\text{m} * 16\text{KN/m}^3 = 0.128 \text{ KN/m}$
- ✓ 3cm cement screed = $0.03\text{m} * 0.4\text{m} * 23\text{KN/m}^3 = 0.276\text{KN/m}$
- ✓ Flange or topping = $0.06\text{m} * 0.4\text{m} * 25\text{KN/m}^3 = 0.6\text{KN/m}$
- ✓ Ribs = $0.15\text{m} * 0.08\text{m} * 25\text{KN/m}^3 = 0.3\text{KN/m}$
- ✓ Plastering = $0.02\text{m} * 0.4\text{m} * 23\text{KN/m} = 0.184\text{KN/m}$
- ✓ HCB = $(0.03\text{m} * 0.4\text{m} + 0.02\text{m} * 3 * 0.19\text{m} + 0.32\text{m} * 0.02\text{m})14\text{KN/m}^3 = 0.43 \text{ KN/m}$

Dead load = $0.128 + 0.276 + 0.6 + 0.3 + 0.184 + 0.43 = 1.918\text{KN/m}$

For all panels without partition load $DL = 1.918\text{KN/m}$

This is common for all panel, the difference is the presence of the partition load.

Partition load for cantilevers Panel C1

$$L=6\text{m}, h=3.36\text{m}, t=6\text{mm}$$

$$A=0.006*3.36=0.02016\text{m}^2$$

$$\text{Dead load} = A * \text{Unit weight} = 0.02016 * 22\text{KN/m}^3 = 0.4422\text{KN/m}$$

$$\text{Load} = 0.4422\text{KN/m}$$

$$\text{The slab area} = 6\text{m} * 2.15\text{m} = 12.9\text{m}^2$$

Distributed load = DL * spacing of rib * length / A slab

$$\text{Load} = 0.4422 * 0.4 * 6 / 12.9 = 0.0825\text{KN/m}$$

Total dead load

$$\text{For panel C1-8 DL} = 0.0825 + 1.918 = 2\text{KN/m}$$

Live load

- ✓ Live loads for floors of cafeteria (under category C1) = 2.5KN/m^2 ...ES EN 1991 table 6.1, so distributing the live loads over the rib length $2.5\text{KN/m}^2 * 0.4\text{m} = 1\text{KN/m}$
- ✓ Live load for Rooms in residential buildings and houses; bedrooms, kitchens and toilets. = 1.75KN/m^2 , distributing the live loads over the rib length $1.75\text{KN/m}^2 * 0.4\text{m} = 0.7\text{KN/m}$
- ✓ Live load for corridor = 4KN/m^2 , Live load of corridor over the rib length $4 * 0.4 = 1.6\text{KN/m}$

$$\text{Design load PD} = 1.35\text{DDL} + 1.5\text{LL}$$

$$\text{Pd for cafeteria} = 1.35 * 1.918 + 1.5 * 1 = 4.0893\text{KN/m}$$

$$\text{Pd for corridor} = 1.35 * 1.918 + 1.5 * 1.6 = 4.989\text{KN/m}$$

$$\text{Pd for bedrooms} = 1.35 * 1.918 + 1.5 * 0.7 = 3.64\text{KN/m}$$

$$\text{Pd for cantilever} = 1.35 * 2 + 1.5 * 1 = 4.2\text{KN/m}$$

Therefore design the span and field moment by taking section.

Table 3. 11 sample design load for ribbed slab

panel	DL	LL	Pd
P1	1.918	1.6	4.9893
P2	1.918	0.7	3.6393
P3	1.918	1.6	4.9893
P4	1.918	1.6	4.9893
P5	1.918	0.7	3.6393

The rest are in the appendix part

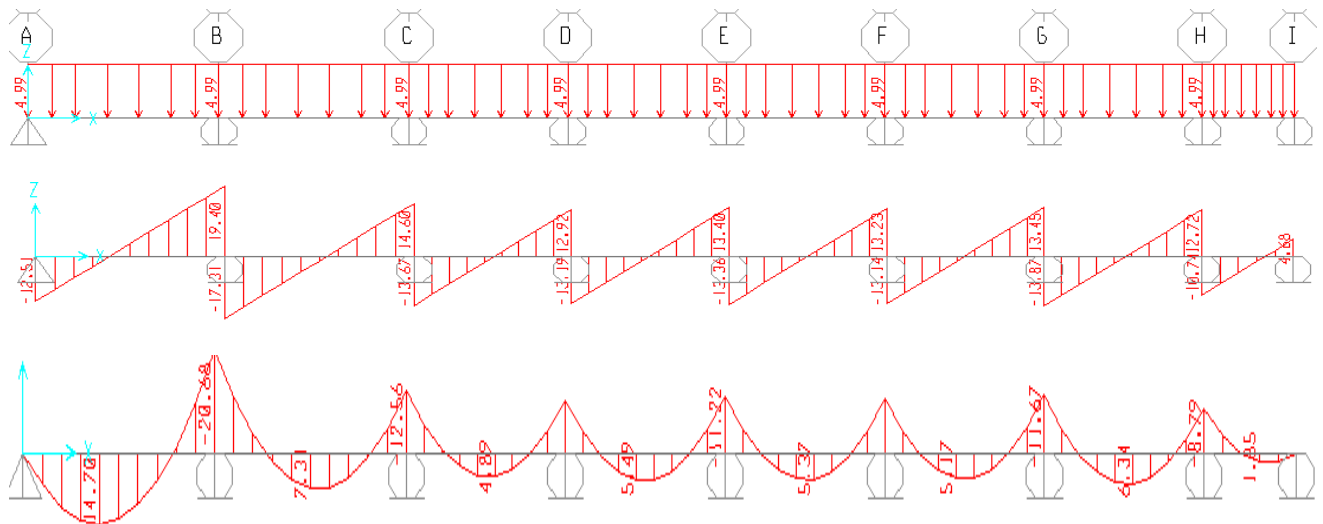
The arrangements of the ribs shown above is selected in the following reasons

- 1) The ribs have shorter span length which will reduce the span and support moment
- 2) Wall loads will be acting transversely to the ribs which will reduce wall load effect on a single rib. Analysis of the ribs is performed by SAP. Live load was alternatively loaded on the spans in order to obtain maximum spans and support moments.

3.2.2 ANALYSIS AND DESIGN OF RIBS (JOISTS)

Calculate the maximum moment and shear force for rib reinforcement design

Case 1 Full design load

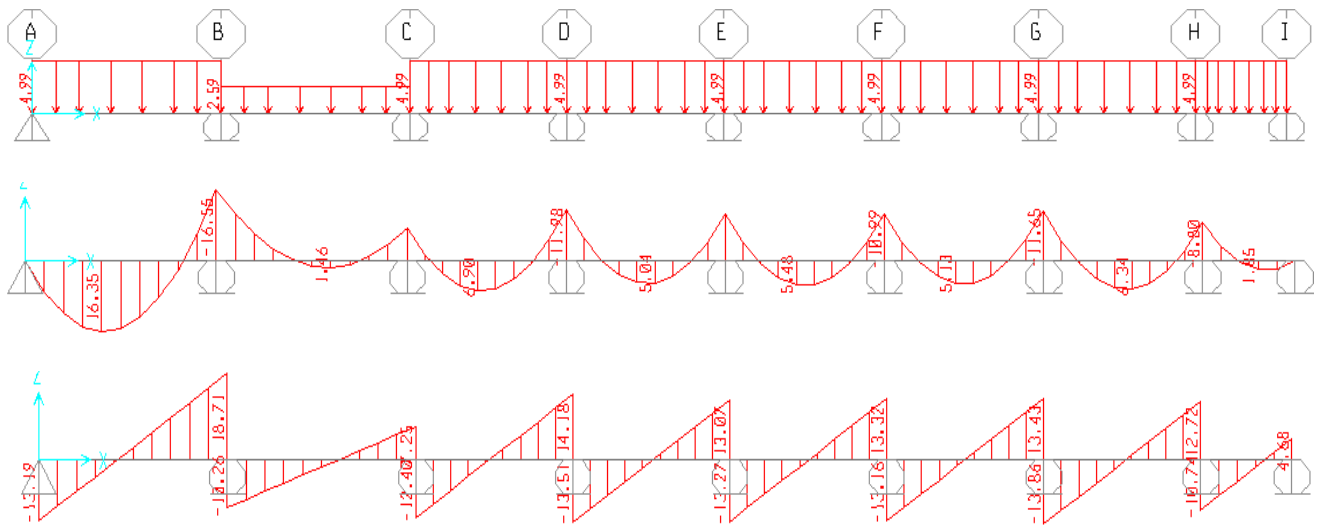


Support moment=20.68KNm

Field moment=14.7KNm

Maximum shear =19.4KN

Case2



Support moment=16.55kNm

Field moment=16.37kNm

Maximum shear =18.71

Case 3

Support moment=21.38kNm

Field moment=14.48kNm

Maximum shear =17.04kN

Case4

Support moment=20.5kNm

Field moment=14.72kNm

Maximum shear =19.37kN

Case5

Support moment=20.96kNm

Field moment=14.56kNm

The maximum shear=19.4kN

Case6 Only dead load acting

Support moment=11.38kNm

Field moment=8.06KNm

The maximum shear=10.68KN

Among the above analysis the maximum design loads are 21.38KNm for the support moment, 16.37KNm for filed moment and 19.4KN for shear. The reset diagrams are in the appendix part.

3.2.3 REINFORCEMENT DESIGN

Material properties

$f_{cd} = 11.33\text{Mpa}$, $f_{yd} = 347.83\text{Mpa}$

$b_e = 400\text{mm}$ and $b_w = 80\text{mm}$

Rib dimension

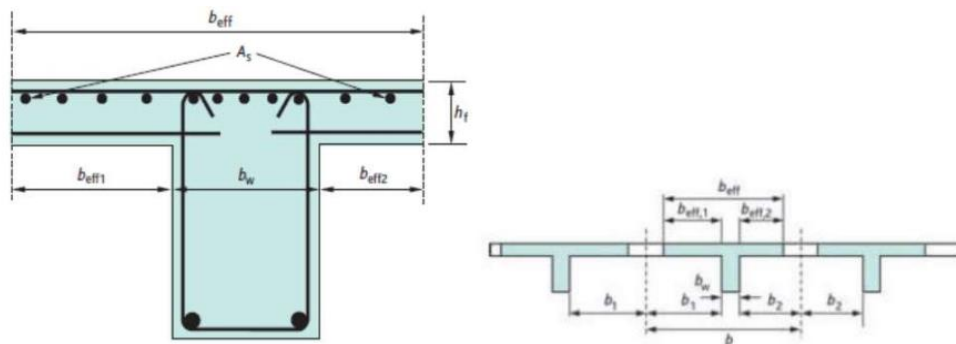


Figure 3. 13 flanged section of the ribbed slab

Cross section at span

- ✓ $h_f = 60\text{mm}$
- ✓ $b_w = 80\text{mm}$
- ✓ $D = 290\text{mm}$
- ✓ Concrete cover = 20mm

Take $\text{Ø}6$ bar for stirrup and $\text{Ø}12\text{mm}$ bar for longitudinal reinforcement

Note:-Mid span section is designed as a T-beam

Assuming the NA fall in the flange

Effective depth of ribs, $d_{eff} = D - C_c - \text{Østirrup} - \frac{\text{Ø}}{2}$

$$d_{eff} = 290\text{mm} - 20\text{mm} - 6\text{mm} - \frac{12\text{mm}}{2} = 258\text{mm}$$

Effective flange width computation

1. P in T beams the effective flange width, over which uniform conditions of stress can be assumed based 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 should be based on the distance l_0 between points of zero moment

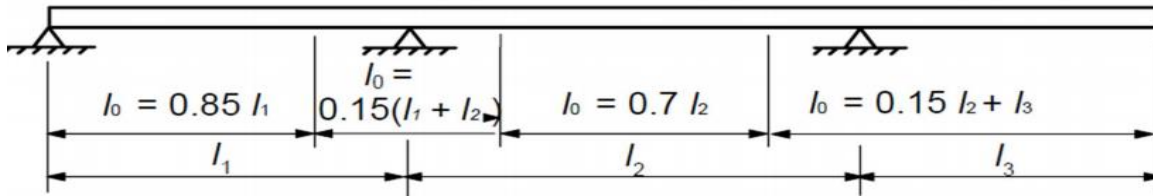


Figure 3. 14 Definition of l_0 , for calculation of effective flange width

According to (ES EN 1992:2015 Article 5.3.2.1 n.d.) the effective flange width b_{eff} for a T or L beam is determined as;

$$b_{eff} = \sum b_{eff,i} + b_w < b$$

Where: $b_{eff,i} = 0.2b_i + 0.1l_0 \leq 0.2l_0$ and $b_{eff,i} < b_i$

A, for end span (sagging moment)

$$l_0 = 0.85l_1 = 0.85 * 6000 = 5100\text{mm}$$

$$b_1 = b_2 = \frac{400 - 80}{2} = 160\text{mm}$$

$$b_{eff,1} = b_{eff,2} = 0.2b_2 + 0.1l_0 < 0.2l_0 = 0.2 * 160 + 0.1 * 5100 < 0.2 * 5100$$

$$= 542\text{mm} < 1020\text{mm} \dots \dots \dots \text{ok}$$

$$b_{eff} = b_{eff,1} + b_w + b_{eff,2} < b$$

$$b_{eff} = 542 + 80 + 1020 = 1642\text{mm} < 400\text{mm} \dots \dots \dots \text{not ok, thus } b_{eff} = 400\text{mm}$$

B, for interior sagging moment

$$l_0 = 0.7l_2 = 0.7 * 6000 = 4200\text{mm}$$

$$b_1 = b_2 = \frac{400 - 80}{2} = 160\text{mm}$$

$$b_{eff,1} = b_{eff,2} = 0.2b_2 + 0.1l_0 < 0.2l_0 = 0.2 * 160 + 0.1 * 4200 < 0.2 * 4200$$

$$= 452\text{mm} < 840\text{mm} \dots \dots \dots \text{ok}$$

$$b_{eff} = b_{eff,1} + b_w + b_{eff,2} < b$$

$$b_{eff} = 452 + 80 + 840 = 1372\text{mm} < 400\text{mm} \dots \dots \dots \text{not ok, thus } b_{eff} = 400\text{mm}$$

Assume the neutral axis depth is within the flange. Then adopting rectangular stress relationship for concrete in compression will simplify the procedure. It is possible to determine reinforcement ratio using the formula derived from rectangular stress block.

$$\rho = 0.5 \left(C1 - \sqrt{C1^2 - \frac{4Md}{C2bd^2}} \right) \text{ or } \rho = \frac{fcd}{fyd} \left(1 - \sqrt{1 - \frac{2Md}{fcd \cdot beff \cdot d^2}} \right)$$

$$m = \frac{fyd}{0.8fcd} = \frac{347.833}{0.8 \cdot 11.33} = 38.37, \quad b1 = \frac{\text{clear length}}{2} = b2 = 160 \text{ mm}$$

$$\rho = \frac{11.33}{347.83} \left(1 - \sqrt{1 - \frac{2 \cdot 21.38 \cdot 10^6}{11.33 \cdot 400 \cdot 258^2}} \right)$$

$$\rho = 0.00239671$$

Design reinforcement for section between axis 1 and C

$$Kx = \rho \cdot m = 0.00239671 \cdot 38.37 = 0.09196$$

$$X = Kx \cdot d = m \cdot \rho \cdot d = 0.09196 \cdot 258 = 23.726 \text{ mm}$$

Neutral axis depth = $x = 23.726 \text{ mm} < hf = 60 \text{ mm}$ok

Hence the assumption is correct the neutral axis is under the flange.

$$As, \text{ min} = 0.26 \cdot \frac{fctm}{fyk} \cdot bt \cdot d \text{ but not less than } 0.0013 \cdot bt \cdot d \dots \text{ES EN 1992: 2015 Art.9.2.1.1 (1)}$$

$f_{tm} = 2.2 \text{ Kpa}$ from table 3.1 for C20 concrete grade

$$bt = bw = 80 \text{ mm and } d = 258 \text{ mm}$$

$$As, \text{ min} = 0.26 \cdot \frac{2.2}{347.83} \cdot 80 \cdot 258 = 33.942 \text{ mm}^2, \quad As = 0.0013 \cdot 80 \cdot 258 = 26.832 \text{ mm}^2$$

$$\text{Use } As, \text{ min} = 33.942 \text{ mm}^2$$

$$As, \text{ calcu} = \rho \cdot b \cdot d = 0.00239671 \cdot 400 \cdot 258 = 247.34 \text{ mm}^2$$

$$As, \text{ max} = 0.04 \cdot Ac, \quad Ac = Acb + Acf = bw \cdot (deff - hf) + beff \cdot hf$$

$$Ac = 80 \cdot 198 + 400 \cdot 60 = 39,840 \text{ mm}^2$$

$$As, \text{ max} = 0.04 \cdot 39840 = 1593.6 \text{ mm}^2$$

$$As, \text{ min} < Ast, \text{ calcu} < As, \text{ max}$$

$$Ast, \text{ max} = 0.04Ac \dots\dots$$

But $Ast, \text{ min} < Ast, \text{ calcu} < Ast, \text{ max}$take the minimum reinforcement $Ast, \text{ calcu} = 247.34 \text{ mm}^2$

Now Determine No. of bar using Ø12mm reinforcement

$$n = \frac{A_{s,prov}}{a_s} = \frac{247.34\text{mm}^2}{\pi \frac{d^2}{4}} = \frac{247.34\text{mm}^2}{\pi \frac{12^2}{4}}$$

$n=2.18$ use 3Ø12

$A_s=3*113.04=339.12\text{mm}^2 < A_{s, max}$ ok

$$\rho_{prov} = \frac{A_s}{b} * d = \frac{339.12}{400 * 258} = 0.00328 = 0.328\%$$

But for simplified construction use 2Ø14mm

Transverse reinforcement at topping

Secondary reinforcement is required for temperature and shrinkage.

$$A_{s2}=0.12\% A_{topping}=\frac{0.12}{100}*1000*60=72\text{mm}^2$$

$$\text{Spacing}(S) = \frac{b*a_s}{A_s} = \frac{1000*3.14*3^2}{72} = 392.5\text{mm}$$

$S_{max} = \frac{1}{2} * \text{rib spacing} = 200\text{mm}$, Take $S=200\text{mm}$

Provide Ø6 c/c200mm

Spacing of bars according to EN 1992-1-1-2004

The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should be the maximum of the following

- ✓ $K1 * \text{Diameter of longitudinal bar}$
- ✓ $d_g + k_2$
- ✓ 20mm

Where: d_g is the maximum aggregate size

The recommended values for k_1 and K_2 are 1 and 5 mm respectively. Assume 20mm aggregate size.

$$S = \max \left\{ \begin{array}{l} 1 * 14\text{mm} = 14\text{mm} \\ 20 + 5 = 25\text{mm} \\ 20\text{mm} \end{array} \right.$$

Therefore; clear spacing between bars is 25mm for maximum aggregate size

Reinforcement for the span moment is 2Ø14mm c/c25mm.

Check Depth for Deflection: Serviceability Requirement

$$\rho = 0.328\%$$

$$\rho_0 = \frac{f_{ck}^{0.5}}{1000} = \frac{20^{0.5}}{1000} = 0.447\% \quad \rho \leq \rho_0$$

$$\frac{l}{d} = k \left[11 + 1.5\sqrt{f_{ck}} \frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}} \left(\frac{\rho_0}{\rho} - 1 \right)^{3/2} \right] * F1 * F2 * F3$$

$$\frac{l}{d} = k \left[11 + 1.5\sqrt{20} \frac{0.447}{0.328} + 3.2\sqrt{20} \left(\frac{0.447}{0.328} - 1 \right)^{3/2} \right]$$

$$F1 = \frac{500}{400 * 247.34 / 339.12} = 1.7138$$

$$\frac{l}{d} = 11.3 * 20.22 * 1 * 1.7168 * 0.8 = 31.384$$

$$d = 6000 / 31.384 = 191.18 \text{ mm} < 258 \text{ mm} \dots \dots \dots \text{ok}$$

Shear design

The design value of the shear resistance is

$$VR_{d,c} = [CR_{d,c} (100 * \rho_1 * f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w * d \geq [V_{min} + k_1 \sigma_{cp}] b_w * d$$

Where:- $f_{ck} = 20 \text{ Mpa}$

$$CR_{d,c} = 0.18 / \gamma_c = 0.18 / 1.5 = 0.12$$

$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{258}} = 1.88 < 2 \dots \dots \dots \text{ok (Take } k = 1.88)$$

$$\rho_1 = \frac{A_{s1}}{b_w * d} \leq 0.02$$

$$\rho_1 = \frac{247.34}{80 * 258} = 0.01198 \leq 0.02 \dots \dots \dots \text{ok}$$

$$V_{min} = 0.035 k^{3/2} * f_{ck}^{1/2}$$

$$V_{min} = 0.035 * 1.88^{3/2} * 20^{1/2} = 0.04$$

$$\sigma_{cp} = \frac{NED}{A_c} = 0$$

A_{st} - is the area of the tensile reinforcement

NED is the axial force in the cross-section due to loading

A_c is the area of concrete cross-section [mm^2]

$$VR_{d,c} = \left[0.12 * 1.87 (100 * 0.02 * 20)^{\frac{1}{3}} + 0 \right] * 400 * 258 = 79.199 \text{ KN}$$

$$VR_{d,c}(\text{min}) = [V_{min} + k_1 \sigma_{cp}] b_w * d$$

$$= (0.04+0)*400*258=4.128\text{KN}$$

$$V_d = 19.4\text{KN} > V_{Rd,c} = 79.199\text{KN}$$

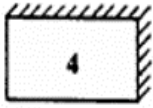
The concrete section can resist the coming shear alone, thus the section does not require design for shear reinforcement.

3.3 LOAD TRANSFER

3.3.1 FOR SOLID SLAB

Load transfer calculation for ground floor

$$\text{For slab 1: } \frac{L_y}{L_x} = \frac{6}{5} = 1.2$$



Type 4 based on support condition

Based on the L_y/L_x ratio read the value of β_{vxc} and β_{vyd} from table A-3 EBCS-2-1995

Table 3. 12 load transfer coefficient

L_y/L_x	β_{vxc}	β_{vxd}	β_{vyc}	β_{vyd}
1.2	0.47	0.31	0.4	0.26

Dead load and live load from previous load calculation

$$\text{Design load} = LL + DL = 13.821\text{KN/m}^2$$

$$V_{xc} = \beta_{vxc} * p_d * l_x \dots\dots\dots \text{for x direction continuous slab}$$

$$V_{xd} = \beta_{vxd} * p_d * l_x \dots\dots\dots \text{for x direction dis-continuous slab}$$

$$V_{yc} = \beta_{vyc} * p_d * l_x \dots\dots\dots \text{for y direction continuous slab}$$

$$V_{yd} = \beta_{vyd} * p_d * l_x \dots\dots\dots \text{for y direction dis-continuous slab}$$

For x direction

$$V_{xc} = \beta_{vxc} * p_d * l_x = 0.47 * 13.821 * 5 = 32.479\text{KN/m}$$

$$V_{yd} = \beta_{vyd} * p_d * l_x = 0.31 * 13.821 * 5 = 21.422\text{KN/m}$$

For y direction

$$V_{yc} = \beta_{vyc} * p_d * l_x = 0.4 * 13.821 * 5 = 27.64\text{KN/m}$$

$$V_{yd} = \beta_{vyd} * p_d * l_x = 0.26 * 13.821 * 5 = 17.96 \text{KN/m}$$

The rest are calculated by excel software

Table 3. 13 load transfer from slab to beam by using coefficient method

panel	type	ly/lx	load transfer					DL	V _{xc}	V _{xd}	V _{yc}	V _{yd}
			β_{xc}	β_{xd}	β_{yc}	β_{yd}	L _x					
P1	4	1.2	0.47	0.31	0.4	0.26	5	13.821	32.5	21.42	27.6	18
P2	4	1.2	0.47	0.31	0.4	0.26	5	16.821	39.5	26.07	33.6	21.9
P3	1	1	0.33		0.3		6	14.946	29.6	0	29.6	0
P4	1	1.2	0.39		0.3		5	16.821	32.8	0	27.8	0
P5	3	1	0.36	0.24	0.4		5	16.821	30.3	20.19	30.3	0
P6	1	1.2	0.39		0.3		5	16.821	32.8	0	27.8	0
P7	3	1	0.36	0.24	0.4		5	16.821	30.3	20.19	30.3	0
P8	1	1.2	0.39		0.3		5	16.821	32.8	0	27.8	0
P9	3	1	0.36	0.24	0.4		5	16.821	30.3	20.19	30.3	0
P10	1	1.2	0.39		0.3		5	16.821	32.8	0	27.8	0
P11	4	1.724	0.57	0.38	0.4	0.26	3	16.821	27.8	18.54	19.5	12.7
P13	2	1.2	0.42		0.4	0.24	5	13.038	27.4	0	23.5	15.6
P14	1	1.2	0.39		0.3		5	14.659	28.6	0	24.2	0
P15	1	1	0.33		0.3		5	14.659	24.2	0	24.2	0
P16	1	1	0.33		0.3		5	14.659	24.2	0	24.2	0
P17	1	1	0.33		0.3		5	14.659	24.2	0	24.2	0
P18	1	1	0.33		0.3		5	14.659	24.2	0	24.2	0
P19	1	1.24	0.48		0.3		3	13.038	18.1	0	12.5	0
P20	1	1	0.33		0.3		6	15.321	30.3	0	30.3	0
P21	1	1.2	0.39		0.3		5	14.659	28.6	0	24.2	0
P22	4	1.2	0.47	0.31	0.4	0.26	5	14.659	34.4	22.72	29.3	19.1
P23	1	1.2	0.39		0.3		5	14.659	28.6	0	24.2	0

Load calculation for cantilever

For cantilever1

$$V_{xc} = p_d * l_x, L_x=1.5m \quad p_d=12.321 \text{KN/m}^2$$

$$V_{xc} = p_d * l_x = 12.321 * 1.5 = 18.48 \text{KN/m} \quad \text{Similar for all cantilevers}$$

Table 3. 14 Load transfer for 3rd floor solid slab

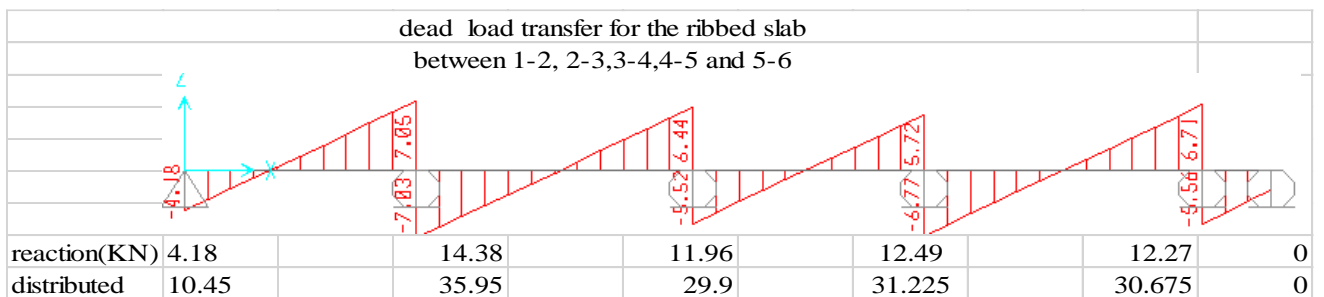
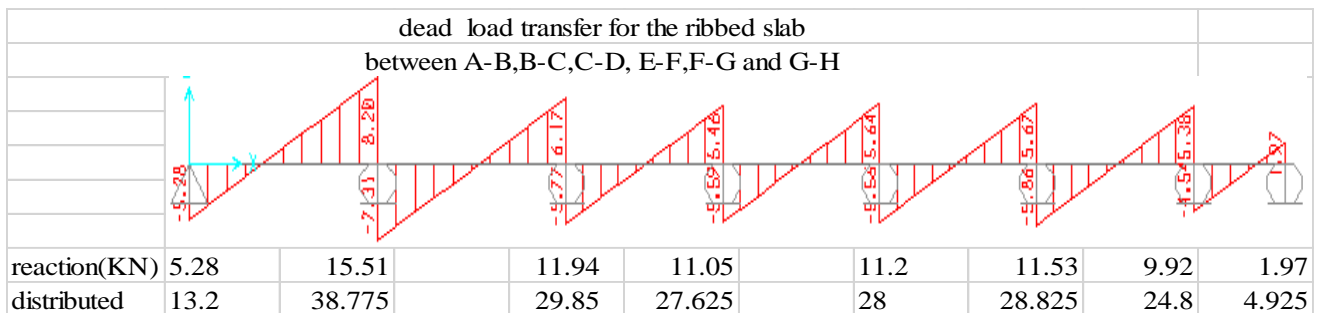
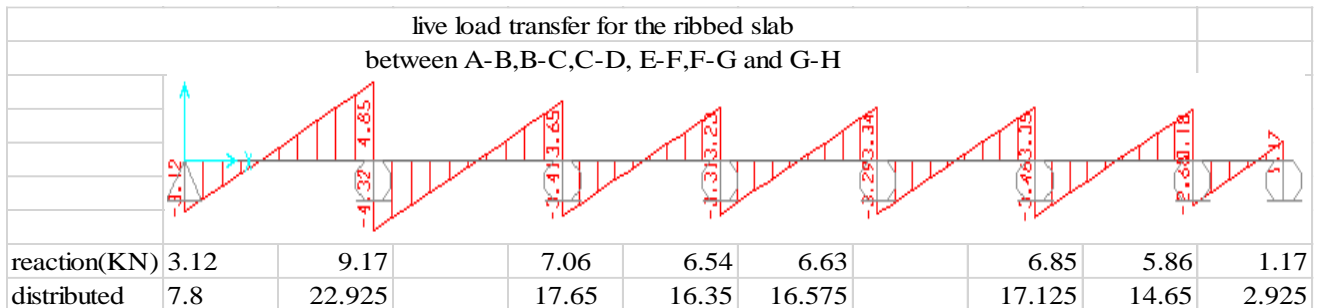
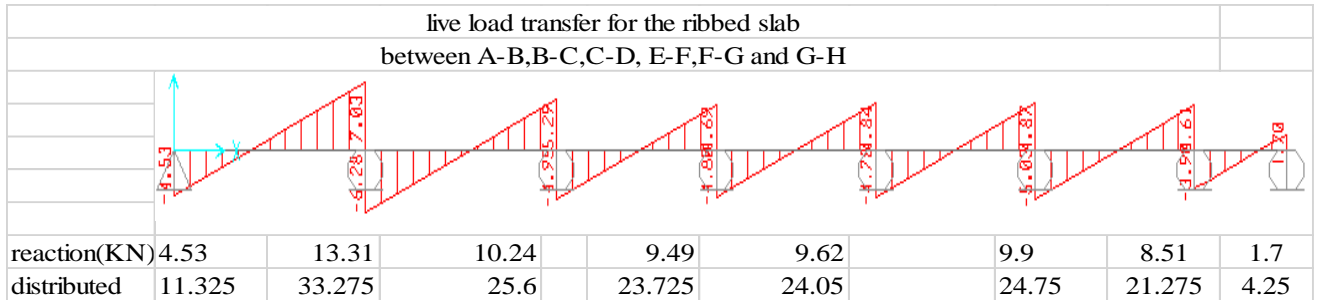
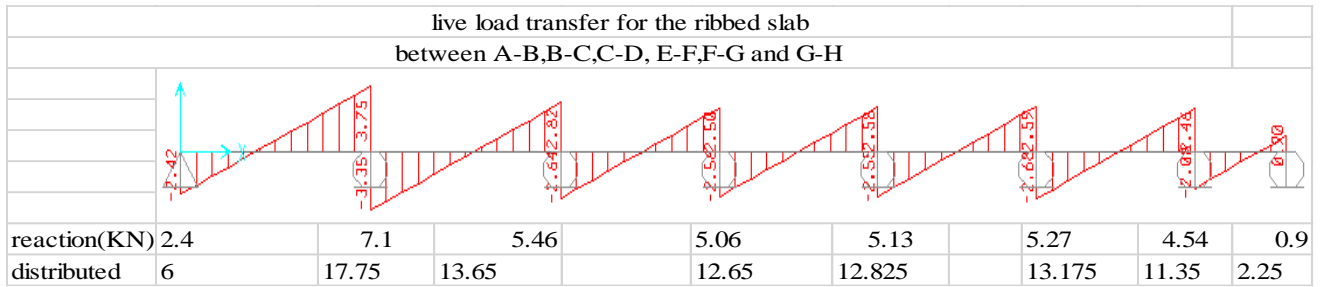
load transfer							
panel	Lx	DL	Vxc	panel	Lx	DL	Vxc
C1	1.5	14.6085	21.9128	C4	1.5	14.6085	21.9128
C2	1.5	14.6085	21.9128	C5	1.5	14.6085	21.9128
C3	1.5	14.6085	21.9128	C6	1.5	14.6085	21.9128
C7	1.5	14.6085	21.9128				

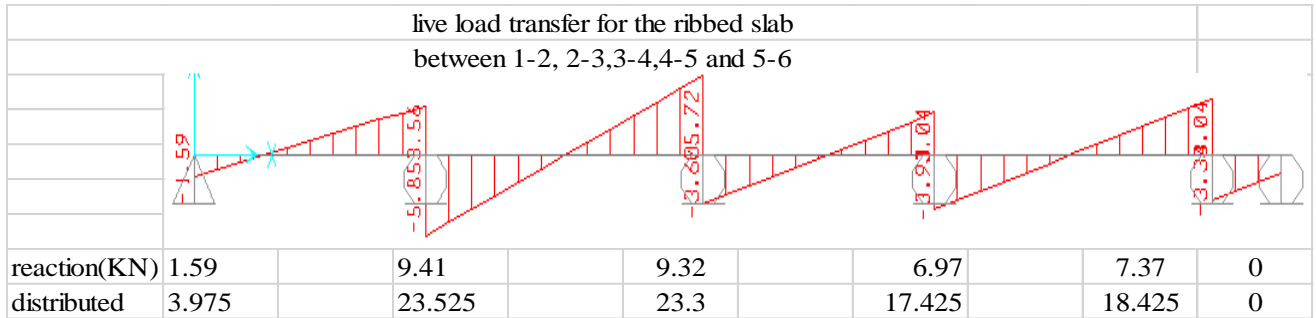
panel	type	ly/k	β_{xc}	β_{xd}	load transfer				DL	Vxc	Vxd	Vyc	Vyd
					β_{yc}	β_{yd}	Lx						
P1	4	1.2	0.47	0.31	0.4	0.26	5	13.4835	31.6862	20.8994	26.967	17.5286	
P2	4	1.2	0.47	0.31	0.4	0.26	5	12.3585	29.0425	19.1557	24.717	16.0661	
P3	1	1	0.33		0.33		6	13.4835	26.6973	0	26.6973	0	
P4	1	1.2	0.39		0.33		5	12.3585	24.0991	0	20.3915	0	
P5	3	1	0.36	0.24	0.36		5	12.3585	22.2453	14.8302	22.2453	0	
P6	1	1.2	0.39		0.33		5	14.2335	27.7553	0	23.4853	0	
P7	3	1	0.36	0.24	0.36		5	12.3585	22.2453	14.8302	22.2453	0	
P8	1	1.2	0.39		0.33		5	12.3585	24.0991	0	20.3915	0	
P9	3	1	0.36	0.24	0.36		5	12.3585	22.2453	14.8302	22.2453	0	
P10	1	1.2	0.39		0.33		5	16.4835	32.1428	0	27.1978	0	
P11	4	1.724	0.57	0.38	0.4	0.26	2.9	12.3585	20.4286	13.6191	14.3359	9.31831	
P13	2	1.2	0.42		0.36	0.24	5	12.822	26.9262	0	23.0796	15.3864	
P14	1	1.2	0.39		0.33		5	12.822	25.0029	0	21.1563	0	
P15	1	1	0.33		0.33		5	12.822	21.1563	0	21.1563	0	
P16	1	1	0.33		0.33		5	12.822	21.1563	0	21.1563	0	
P17	1	1	0.33		0.33		5	12.822	21.1563	0	21.1563	0	
P18	1	1	0.33		0.33		5	12.822	21.1563	0	21.1563	0	
P19	1	1.724	0.48		0.33		2.9	16.4835	22.945	0	15.7747	0	
P20	1	1	0.33		0.33		6	12.822	25.3876	0	25.3876	0	
P21	1	1.2	0.39		0.33		5	12.822	25.0029	0	21.1563	0	
P22	4	1.2	0.47	0.31	0.4	0.26	5	12.822	30.1317	19.8741	25.644	16.6686	
P23	1	1.2	0.39		0.33		5	12.822	25.0029	0	21.1563	0	
s2	2	1	0.33		0.33		6	13.4835	26.6973	0	26.6973	0	
s3	2	1.2	0.42		0.36	0.24	5	12.822	26.9262	0	23.0796		

3.3.2 Load transfer for ribbed slab to the Main beam

The joist transfer the load to the main beam in the form of uniformly distributed load within the effective joist width, from SAP analysis the reaction force for un-factored dead and live load is computed in the form of concentrated load so this load should be distributed through effective width in order to do this the reaction force should be divided by effective width ($b_{eff}=0.4m$).

Table 3. 15 load transfer for ribbed slab





3.3 STAIR CASE DESIGN AND ANALYSIS

Functionally, the staircase is an important component of a building. It consists of a flight of steps, usually with one or more intermediate landings (The horizontal top portion of a step where the foot rests) is termed tread and the vertical projection of the step (i.e. The vertical distance between two neighboring steps) is called riser. The width of the stair is generally around 1.1–1.6m, and in any case, should normally not be less than 850mm; large stair widths are encountered in entrances to public buildings. The horizontal projection (plan) of an inclined flight of steps between the first and last risers are termed going.

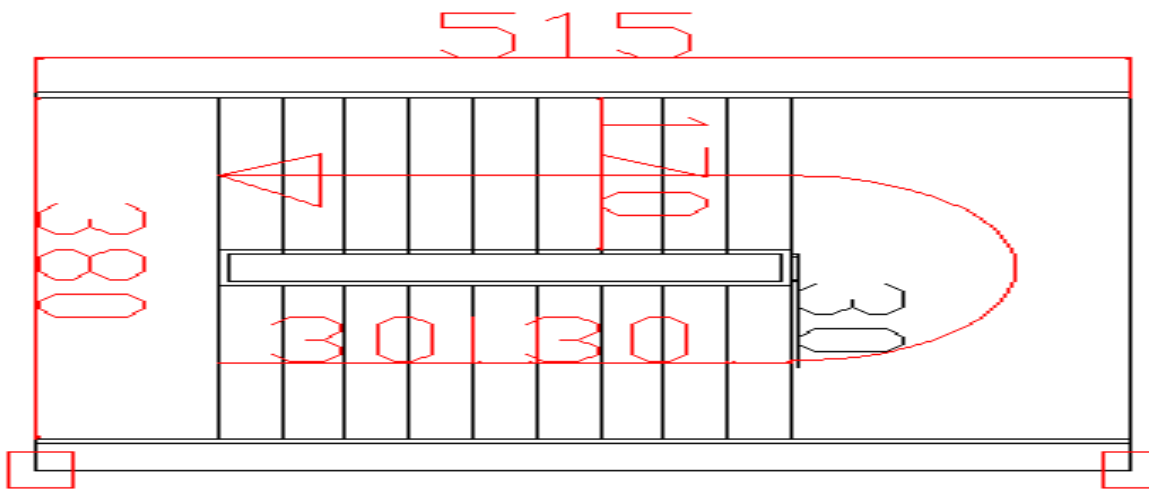


Figure 3. 15 plane view of stair one

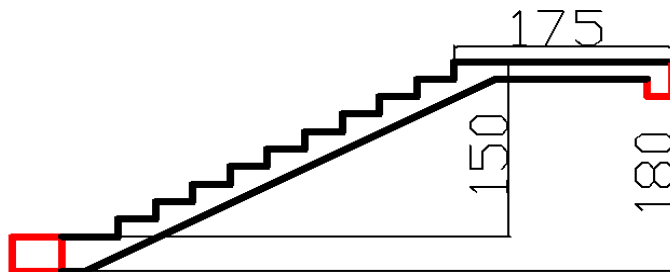


Figure 3. 16 Sectional view of stair one

3.3.1 GEOMETRICAL CONFIGURATIONS

A wide variety of staircases is met with in practice. But in this design the stair type is half turn type

Stair geometric data

Height = 3.20m

Height of flight one=1.50m

Height of flight two=1.70m

Number of going from plan view =19with 30cm width

Number of riser = number of going+2 =21(Total) with 15cm height

$$\text{Height of riser} = \frac{\text{foor height}}{\text{number of riser}} = \frac{320\text{cm}}{21} = 15\text{cm or } 150\text{mm}$$

3.3.2 DEPTH FOR DEFLECTION

Let us take one meter strip width, b=1000mm

$$D = \text{effective depth } (d) + \frac{\emptyset}{2} + \text{cover}$$

In order to determine the stair depth, first calculate the concrete cover and effective depth.

A, Concrete cover determination

Nominal concrete cover is the distance b/n the surface of reinforcement closest to the nearest concrete surface (including links and stirrups)

$$C_{nom} = C_{min} + \Delta C_{dev} \dots \dots \dots \text{(ESEN 1992: 2015 } \square \square \square \text{ 4.4.12 n.d.)}$$

$$C_{min} = \max\{C_{min, b}; C_{min, dur} + \Delta C_{dur, y} - C_{dur, st} - C_{dur, add}; 10\text{mm}\}$$

Assume $\Phi 12$ longitudinal bar, $\Phi 20$ nominal bar and maximum aggregate size;

Therefore; $C_{min, b} = 12\text{mm}$

Cover design for corrosion or durability

The condition of exposure is given to be XC1, Environmental requirement for $C_{min, dur} = XC1$ vs S3

$C_{min, dur} = 10\text{mm}$

$$C_{min} = \max \begin{cases} C_{min, b} = 12\text{mm} \\ C_{min, dur} = 10\text{mm} \\ 10\text{mm} \end{cases} \text{ Thus, } C_{min} = 12\text{mm}$$

ΔC_{dev} -allowance in Design for Variation

Note: The recommended value of ΔC_{dev} for use from National Annex is 10 mm.

$$C_{nom} = C_{min} + \Delta C_{dev} = 12\text{mm} + 10\text{mm} = 22\text{mm}$$

$$C_{cover} = 22\text{mm}$$

B, Effective depth determination for 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

$$\frac{l}{d} = k \left[11 + 1.5\sqrt{f_{ck}} \frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}} \left(\frac{\rho_0}{\rho} - 1 \right)^{3/2} \right] * F_1 * F_2 * F_3 \dots \dots \dots \text{if } \rho \leq \rho_0 \text{ Art. 7.4.2 (7.16a)}$$

Initially take $\rho_0 = \rho$ and $A_{sreq} = A_{sprov}$

$$\frac{l}{d} = K * N * F_1 * F_2 * F_3 \text{ where: } N=17.7082, K=1.5, F_1=1.25 \text{ and } F_2=F_3=1.$$

$$\frac{l}{d} = 1.5 * 17.7082 * 1.25 * 1 * 1 = 33.203$$

$$\text{For flight 1 } d_1 = 3580 / 33.203 = 107.82\text{mm} \sim 110\text{mm}$$

For flight 2 $d_2 = 3920 / 33.203 = 118.06\text{mm} \sim 120\text{mm}$, take the maximum d value for design.

$$D = d + \text{cover} + \frac{\phi_{longitudinal}}{2} = 120\text{mm} + 22\text{mm} + 6\text{mm} = 146.06\text{mm}, \text{ Use } D = 150\text{mm}$$

$$d = 150 - 28 = 122\text{mm}$$

3.3.3 DETERMINATION OF LOADS

Material data unit weight based on (ES EN1991-1-:2001 (table A2) n.d.)

- ✓ Unit weight of marble = 27 kN/m³
- ✓ Unit weight of cement screed = 23 kN/m³
- ✓ Unit weight of concrete = 25 kN/m³
- ✓ Unit weight of plastering = 23 kN/m³

Live load

Live load (LL) = 2 kN/m² for category A from.....(ES-EN-1991-2015 table 6.2 n.d.)

$$\text{One meter strip live load} = 2\text{ kN/m}^2 * 1\text{m} = 2\text{ kN/m}$$

Use 2cm plaster, 3cm marble and 3cm cement screed for finishing

Calculation of dead load using 1m width for Flight 2

Dead load of landing

$$\text{DL for concrete slab} = t_{\text{slab}} * \gamma_c * L_w = 0.150 * 25 * 1 = 3.75 \text{KN/m}$$

$$\text{DL for plastering} = t_p * \gamma_p * L_w = 0.02 * 23 * 1 = 0.46 \text{KN/m}$$

$$\text{DL for cement screed} = t_{\text{cs}} * \gamma_{\text{cs}} * L_w = 0.03 * 23 * 1 = 0.69 \text{KN/m}$$

$$\text{DL for marble} = t_m * \gamma_m * L_w = 0.03 * 27 * 1 = 0.81 \text{KN/m}$$

$$\text{Total dead load} = 3.75 + 0.46 + 0.69 + 0.81 = 5.71 \text{KN/m}$$

Stair from B to G floor

Flight dead load

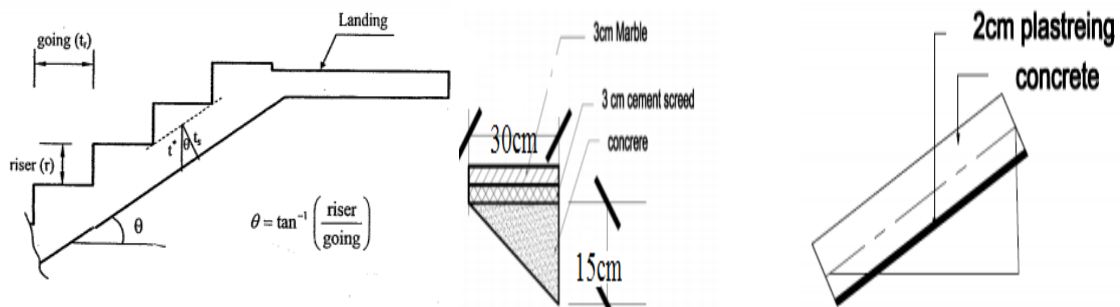


Figure 3. 17 flight and thread details

$$\tan \theta = 1.7/3.15 = 0.5396, \text{ since the projected length} = 3.15 \text{m}$$

$$\theta = 28.35^\circ$$

Step dead load

$$\text{DL for concrete slab} = 1/2 * h * L_w * \gamma_c = 0.5 * 0.15 * 1 * 25 = 1.875 \text{KN/m}$$

$$\text{DL for cement screed} = t_{\text{cs}} * \gamma_c * L_w = 0.03 * 23 * 1 = 0.69 \text{KN/m}$$

$$\text{DL for marble} = t_m * \gamma_m * L_w = 0.03 * 27 * 1 = 0.81 \text{KN/m}$$

$$\text{Total dead load} = 1.875 + 0.69 + 0.81 = 3.375 \text{KN/m}$$

Riser dead load

$$\text{DL for cement screed} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * l_w = \frac{11(0.15*0.03*23)}{3.15} * 1 = 0.361 \text{KN/m}$$

$$\text{DL for marble} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * l_w = \frac{11(0.15*0.03*27)}{3.15} * 1 = 0.424 \text{KN/m}$$

$$\text{Total dead load} = 0.361 + 0.4243 = 0.785 \text{KN/m}$$

Waist dead load

$$\text{DL for concrete} = \frac{ts \cdot L_{\text{inclined}} \cdot \gamma_c}{\text{projected length}} * l_w = \frac{0.15 * 3.58 * 25}{3.15} * 1 = 4.262 \text{KN/m}$$

$$\text{DL for plastering} = \frac{tp \cdot L_{\text{inclined}} \cdot \gamma_p}{\text{projected length}} * l_w = \frac{0.02 * 3.58 * 23}{3.15} * 1 = 0.523 \text{KN/m}$$

$$\text{Total dead load} = 4.262 + 0.523 = 4.785 \text{KN/m}$$

$$\text{DL}_{\text{flight}} = 4.785 + 0.785 + 3.375 = 8.945 \text{KN/m}$$

Design load for flight 2

$$P_d = 1.35 \text{DL} + 1.5 \text{LL} = 1.35 * 8.945 + 1.5 * 2 = 15.075 \text{KN/m}$$

Design load for landing

$$P_d = 1.35 \text{DL} + 1.5 \text{LL} = 1.35 * 5.71 + 1.5 * 2 = 10.71 \text{KN/m}$$

Calculation of dead load for flight one

Riser dead load

$$\text{DL for cement screed} = \frac{\text{no of riser} (h * t * \gamma)}{\text{projected length}} * l_w = \frac{10(0.15 * 0.03 * 23)}{2.9} = 0.36 \text{KN/m}$$

$$\text{DL for marble} = \frac{\text{no of riser} (h * t * \gamma)}{\text{projected length}} * l_w = \frac{10(0.15 * 0.03 * 27)}{2.9} = 0.42 \text{KN/m}$$

$$\text{Total dead load} = 0.36 + 0.42 = 0.78 \text{KN/m}$$

Waist dead load

$$\text{DL for concrete} = \frac{ts \cdot L_{\text{inclined}} \cdot \gamma_c}{\text{projected length}} * l_w = \frac{0.15 * 3.24 * 25}{2.9} = 4.189 \text{KN/m}$$

$$\text{DL for plastering} = \frac{tp \cdot L_{\text{inclined}} \cdot \gamma_p}{\text{projected length}} * l_w = \frac{0.02 * 3.24 * 23}{2.9} = 0.514 \text{KN/m}$$

$$\text{Total dead load} = 4.189 + 0.514 = 4.7 \text{KN/m}$$

$$\text{DL}_{\text{flight}} = 4.7 + 0.78 + 3.375 = 8.855 \text{KN/m}$$

Design load for flight 1

$$P_d = 1.35 \text{DL} + 1.5 \text{LL} = 1.35 * 8.855 + 1.5 * 2 = 14.95 \text{KN/m}$$

Stair from G to 2nd floor

Design load for landing

$$P_d = 1.35 \text{DL} + 1.5 \text{LL} = 1.35 * 5.71 + 1.5 * 2 = 10.1 \text{KN/m}$$

For flight

Riser dead load

$$\text{DL for cement screed} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * lw = \frac{10(0.15*0.03*23)}{3.3} * 1.7 = 0.313 \text{KN/m}$$

$$\text{DL for marble} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * lw = \frac{10(0.15*0.03*27)}{3.3} * 1.7 = 0.368 \text{KN/m}$$

$$\text{Total dead load} = 0.313 + 0.368 = 0.68 \text{KN/m}$$

Waist dead load

$$\text{DL for concrete} = \frac{ts * L_{\text{inclined}} * \gamma_c}{\text{projected length}} * lw = \frac{0.15 * 3.69 * 25}{3.3} * 1 = 4.163 \text{KN/m}$$

$$\text{DL for plastering} = \frac{tp * L_{\text{inclined}} * \gamma_p}{\text{projected length}} * lw = \frac{0.02 * 3.69 * 23}{3.3} * 1 = 0.514 \text{KN/m}$$

$$\text{Total dead load} = 4.163 + 0.514 = 4.677 \text{KN/m}$$

$$\text{DL}_{\text{flight}} = 4.677 + 0.68 + 3.375 = 8.732 \text{KN/m}$$

$$\text{Pd} = 1.35 \text{DL} + 1.5 \text{LL} = 1.35 * 8.732 + 1.5 * 2 = 14.788 \text{KN/m}$$

Stair from 2nd to 5th floor

H=0.16m, projected length=3.5m and inclined length=3.92m

Step dead load

$$\text{DL for concrete slab} = \frac{1}{2} * h * L_w * \gamma_c = 0.5 * 0.16 * 1 * 25 = 2 \text{KN/m}$$

$$\text{DL for cement screed} = tcs * \gamma_c * L_w = 0.03 * 23 * 1 = 0.69 \text{KN/m}$$

$$\text{DL for marble} = tm * \gamma_m * L_w = 0.03 * 27 * 1 = 0.81 \text{KN/m}$$

$$\text{Total dead load} = 2 + 0.69 + 0.81 = 3.5 \text{KN/m}$$

Riser dead load

$$\text{DL for cement screed} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * lw = \frac{11(0.16*0.03*23)}{3.5} * 1 = 0.347 \text{KN/m}$$

$$\text{DL for marble} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * lw = \frac{11(0.16*0.03*27)}{3.5} * 1 = 0.407 \text{KN/m}$$

$$\text{Total dead load} = 0.347 + 0.407 = 0.754 \text{KN/m}$$

Waist dead load

$$\text{DL for concrete} = \frac{ts * L_{\text{inclined}} * \gamma_c}{\text{projected length}} * lw = \frac{0.15 * 3.92 * 25}{3.5} * 1 = 4.2 \text{KN/m}$$

$$DL \text{ for plastering} = \frac{tp * Linclined * yp}{\text{projected length}} * lw = \frac{0.02 * 3.92 * 23}{3.5} * 1 = 0.515 \text{ KN/m}$$

$$\text{Total dead load} = 4.2 + 0.515 = 4.715 \text{ KN/m}$$

$$DL_{\text{flight}} = 4.715 + 0.754 + 3.5 = 8.969 \text{ KN/m}$$

Design load for flight 2

$$Pd = 1.35DL + 1.5LL = 1.35 * 8.969 + 1.5 * 2 = 15.11 \text{ KN/m}$$

3.3.4 DETERMINATION OF MOMENT

Based on end connection the stair in this building have two cases and use the maximum design load Pd = 15.11KN/m for flight and 10.1KN/m.

Case 1) between the basement and intermediate landing

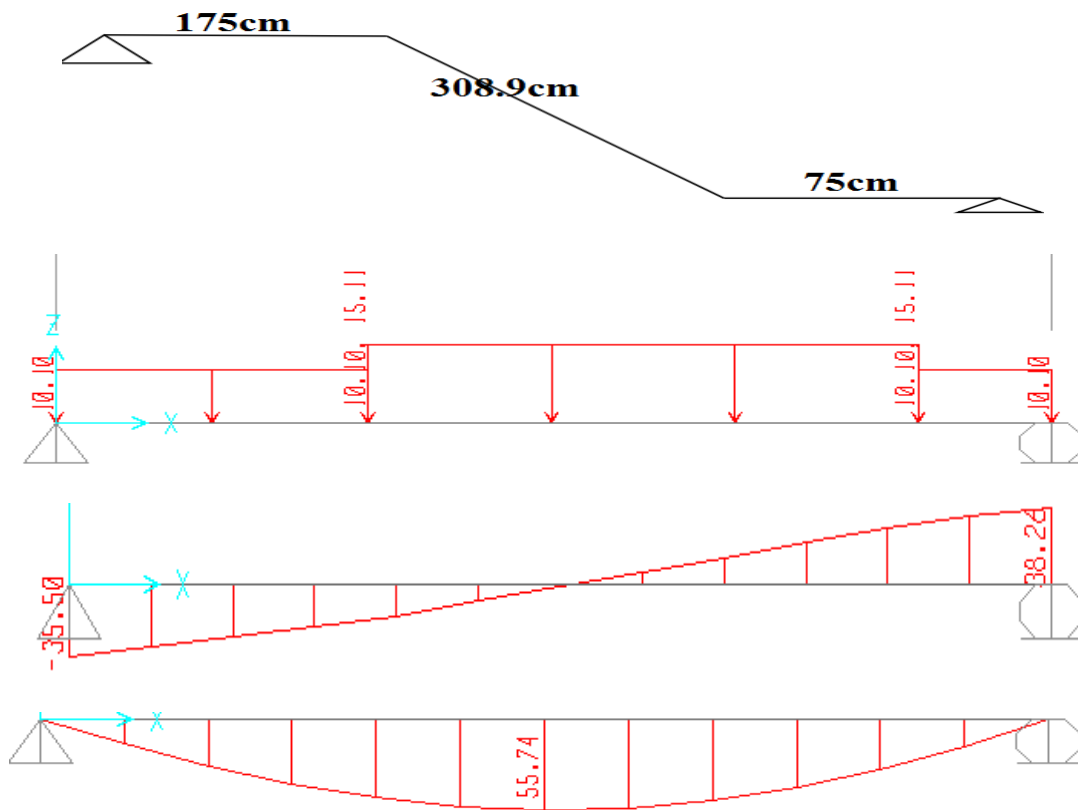


Figure 3. 18 Moment and shear for flight 1 for B to G

Case 2) between 1st, 2nd, 3rd, 4th, 5th, to intermediate landing

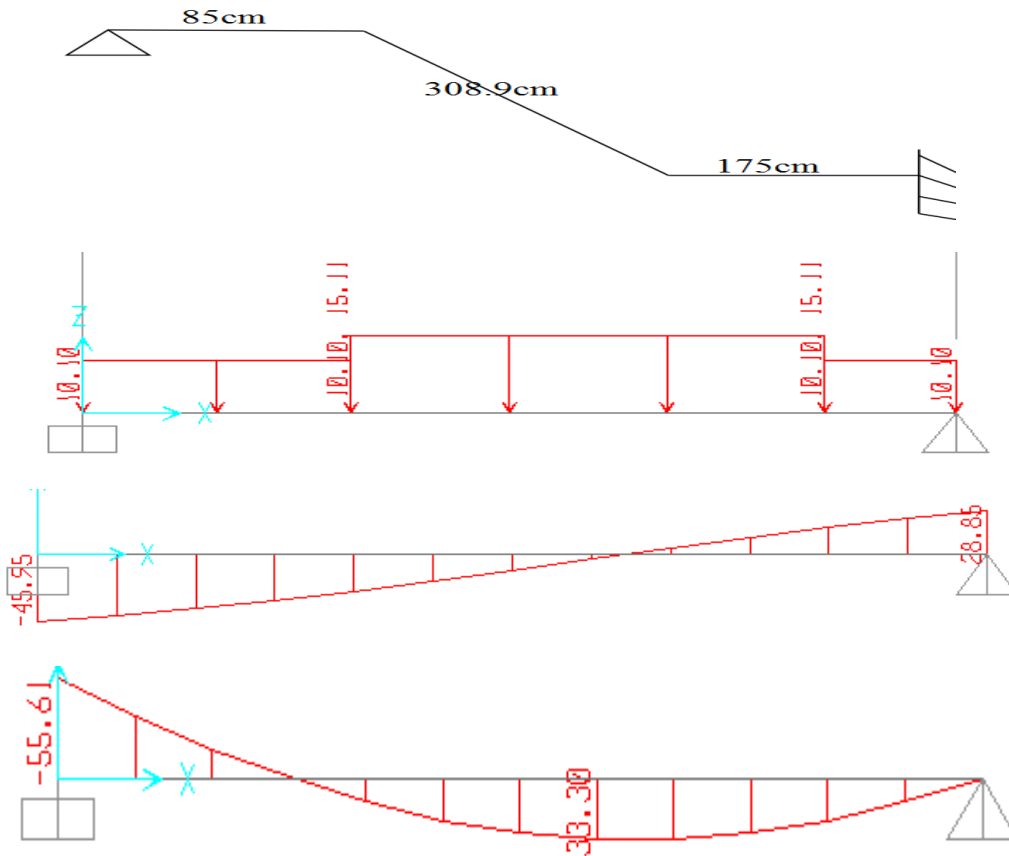


Figure 3. 19 Moment and shear for flight 2 for B to G

3.3.5 DESIGN OF STAIRCASE FOR FLEXURE

Using reinforcement $\varnothing 12$ then $as = \frac{\pi \cdot 12^2}{4} = 113.04 \text{ mm}^2$

Flight 1, Design for main reinforcement bar (principal reinforcement)

MD=55.74KN-m

$$\rho = \frac{f_{cd}}{f_{yd}} \left(1 - \sqrt{1 - \frac{2MD}{f_{cd} * b * d^2}} \right) \quad \text{or} \quad \rho = \frac{1}{2} \left(C1^2 - \sqrt{C1^2 - \frac{4MD}{C2 * b * d^2}} \right)$$

$$\rho = \frac{11.33}{347.83} \left(1 - \sqrt{1 - \frac{2 * 55.74 * 10^6 \text{ Nmm}}{11.33 \frac{10^6 \text{ N}}{\text{mm}^2} * 1000 \text{ mm} * (122 \text{ mm})^2}} \right)$$

$\rho = 0.0136$

$A_{st, cal} = \rho b d = 0.0136 * 1000 \text{ mm} * 122 \text{ mm} = 1659.2 \text{ mm}^2$

Check for minimum and maximum reinforcement area

Minimum requirement

$$A_{st, \min} = \max\left\{0.26 * \frac{f_{ctm} * b * d}{f_{yk}}, 0.0013 * b * d\right\} \dots \dots \dots \text{ES EN 1992:2015 art 9.2.1.1 (1)}$$

$$A_{st, \min} = \max\left\{0.26 * \frac{2.2 \text{Mpa} * 1000 \text{mm} * 122 \text{mm}}{400 \text{Mpa}}, 0.0013 * 1000 \text{mm} * 122 \text{mm}\right\}$$

$$A_{st, \min} = \max\{174.46 \text{mm}^2, 158.6 \text{mm}^2\}$$

$$A_{st, \min} = 174.46 \text{mm}^2$$

Maximum requirement

$$A_{st, \max} = 0.04 A_c = 0.4 * b * d$$

$$A_{st, \max} = 0.04 * 1000 \text{mm} * 122 \text{mm} = 4880 \text{mm}^2$$

$A_{st, \min} < A_{st, \text{cal}} < A_{st, \max}$ok! Because According to ES EN 1992-1-1:2015 section 9.2 Sections containing less reinforcement than $A_{st, \min}$ should be considered as unreinforced and the cross-sectional area of tension or compression reinforcement should not exceed $A_{st, \max}$ outside lap locations.

$$\text{Therefore provided } A_{st, \text{provided}} = 1659.2 \text{mm}^2$$

Flight 1, Secondary transvers reinforcement

According to ES EN 1992-1-1:2015 section 9.3.1.1 Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slabs. In areas near supports transverse reinforcement to principal top bars is not necessary where there is no transverse bending moment. Staircase is treated as one-way slab. Therefore, secondary transverse reinforcement for staircase is $A_{st} = 20\% A_{st, \text{provided}}$.

Where, A_{st} is area of Secondary transverse reinforcement in mm^2 and

$$A_{st, \text{pro}} \text{ -is area of principal reinforcement provided in } \text{mm}^2$$

$$A_{st} = 0.2 * 1659.2 \text{mm}^2 = 331.84 \text{mm}^2$$

Check for minimum and maximum reinforcement area

Minimum requirement

$A_{st} = 331.84 \text{mm}^2 > A_{st, \min} = 174.46 \text{mm}^2$ ok. Therefore, secondary reinforcement should be provided.

Calculation of spacing

Spacing for principal reinforcement

$$S = \frac{bt \cdot a_s}{A_{st,provided}} = \frac{1000\text{mm} \cdot 113.04\text{mm}^2}{1659.2\text{mm}^2} = 68.12\text{mm}, \text{ Use } \emptyset 12\text{c/c}60\text{mm}$$

Spacing for secondary reinforcement

$$S = \frac{bt \cdot a_s}{A_{st,provided}} = \frac{1000\text{mm} \cdot 113.04\text{mm}^2}{331.84\text{mm}^2} = 340.6\text{mm}, \text{ Use } \emptyset 12\text{c/c}335\text{mm}$$

Check for maximum spacing

According to ES EN 1992-1-1:2015 section 9.3.1.1 the spacing of bars should not exceed S_{max} , slab.

For principal reinforcement

$$S_{max, slab} = \min \begin{cases} 3h \text{ or } 3t_s \\ 400\text{mm} \end{cases}$$

$$S_{max, slab} = \min \begin{cases} 3 * 150 = 450\text{mm} \\ 400\text{mm} \end{cases}, \text{ take } 400\text{mm}$$

$$S_{provided} = 60\text{mm} < S_{max} = 400\text{mm} \dots \dots \dots \text{ok}$$

For secondary reinforcement

$$S_{max, slab} = \min \begin{cases} 3.5h \text{ or } 33.5t_s \\ 450\text{mm} \end{cases}$$

$$S_{max, slab} = \min \begin{cases} 3.5 * 150 = 525\text{mm} \\ 450\text{mm} \end{cases}, \text{ take } 450\text{mm}$$

$$S_{provided} = 335\text{mm} < S_{max} = 450\text{mm} \dots \dots \dots \text{ok}$$

Therefore, provide $\emptyset 12$ c/c 60mm for principal reinforcement bar and provide $\emptyset 12$ c/c 335mm for secondary reinforcement bar.

Flight 2, Design for main reinforcement bar.

$$MD=33.3\text{KN/m}$$

$$\rho = \frac{f_{cd}}{f_{yd}} \left(1 - \sqrt{1 - \frac{2MD}{f_{cd} * b * d^2}} \right) \quad \text{or} \quad \rho = \frac{1}{2} \left(C1^2 - \sqrt{C1^2 - \frac{4MD}{C2 * b * d^2}} \right)$$

$$\rho = \frac{11.33}{347.83} \left(1 - \sqrt{1 - \frac{2 * 33.3 * 10^6\text{Nmm}}{11.33 \frac{10^6\text{N}}{\text{mm}^2} * 1000\text{mm} * (122\text{mm})^2}} \right) = 0.00723$$

$$A_{st, cal} = \rho b d = 0.00723 * 1000\text{mm} * 122\text{mm} = 882.77\text{mm}^2$$

Check for minimum and maximum reinforcement area

Minimum requirement

$$A_{st, \min} = \max\left\{0.26 * \frac{f_{ctm} * b * d}{f_{yk}}, 0.0013 * b * d\right\}, \dots \dots \dots \text{ES EN 1992:0215 art 9.2.1.1 (1)}$$

$$A_{st, \min} = \max\left\{0.26 * \frac{2.2 \text{Mpa} * 1000 \text{mm} * 122 \text{mm}}{400 \text{Mpa}}, 0.0013 * 1000 \text{mm} * 122 \text{mm}\right\}$$

$$A_{st, \min} = \max\{174.46 \text{mm}^2, 158.6 \text{mm}^2\}$$

$$A_{st, \min} = 174.46 \text{mm}^2$$

Maximum requirement

$$A_{st, \max} = 0.04 A_c = 0.4 * b * d$$

$$A_{st, \max} = 0.04 * 1000 \text{mm} * 122 \text{mm} = 4880 \text{mm}^2$$

$$A_{st, \min} < A_{st, \text{cal}} < A_{st, \max} \dots \dots \dots \text{ok! Therefore, provided } A_{st, \text{provided}} = 882.77 \text{mm}^2$$

Flight 1, Secondary transvers reinforcement

$$A_{st} = 20\% A_{st, \text{provided}} = 0.2 * 882.77 = 176.55 \text{mm}^2$$

Check for minimum and maximum reinforcement area

Minimum requirement

$A_{st} = 176.55 \text{mm}^2 > A_{st, \min} = 174.46 \text{mm}^2 \dots \dots \dots \text{ok}$. Therefore, secondary reinforcement should be provided.

Spacing for principal reinforcement

$$S = \frac{b t a_s}{A_{st, \text{provided}}} = \frac{1000 \text{mm} * 113.04 \text{mm}^2}{882.77 \text{mm}^2} = 128.05 \text{mm}, \text{ Use } \emptyset 12 \text{c/c} 125 \text{mm}$$

Spacing for secondary reinforcement

$$S = \frac{b t a_s}{A_{st, \text{provided}}} = \frac{1000 \text{mm} * 113.04 \text{mm}^2}{176.55 \text{mm}^2} = 640.25 \text{mm}, \text{ Use } \emptyset 12 \text{c/c} 630 \text{mm}$$

Check for maximum spacing

For principal reinforcement

$$S_{\text{provided}} = 125 \text{mm} < S_{\max} = 400 \text{mm} \dots \dots \dots \text{ok}$$

For secondary reinforcement

$$S_{\text{provided}} = 630 \text{mm} < S_{\max} = 450 \text{mm} \dots \dots \dots \text{not ok, take the limiting value } 450 \text{mm}.$$

Therefore, provide $\emptyset 12$ c/c 125mm for principal reinforcement bar and provide $\emptyset 12$ c/c 450mm for secondary reinforcement bar.

3.3.6 DESIGN OF STAIRCASE FOR SHEAR

According to ES EN 1992-1-1:2015 section 6.2.1 for member's subject to predominantly uniformly distributed loading the design shear force need not to be checked at a distance less than d from the face of the support. Any shear reinforcement required should continue to the support. In addition, it should be verified that shear at the support does not exceed $V_{Rd, max}$.

Check if the $V_{Rd, max}$ greater than V_d at the support

$$V_{Rd, max} = \frac{\alpha_{cw} * b_w * z * v_1 * f_{cd}}{(\cot\theta + \sin\theta)}$$

Where: α_{cw} - is a coefficient taking account of the state of the stress in the compression chord

b_w - width of the slab to be analysis, in mm

v_1 - is a strength reduction factor for concrete cracked in shear

θ - is the angle between the concrete compression strut and the beam axis perpendicular to shear force

z - is the inner lever arm for a member with constant depth corresponding to the bending moment in the element under consideration. In shear analysis of reinforced concrete without axial force in mm.

$\alpha_{cw} = 1$, for non-pre-stressed structures

$$z = 0.9d = 0.9 * 122\text{mm} = 109.8\text{mm}$$

$$v_1 = 0.6 * \left(1 - \frac{f_{ck}}{250}\right) = 0.6 * \left(1 - \frac{20}{250}\right) = 0.552$$

$$b_w = 1000\text{mm} \text{ and } \theta = 28.35^\circ$$

At support $V_d = 38.26\text{KN}$ for flight1 and $V_d = 45.95\text{KN}$ for flight2.

$$V_{Rd, max} = \frac{1 * 1000\text{mm} * 109.8\text{mm} * 0.552 * 11.33\text{N/mm}^2}{(\cot 28.35^\circ + \sin 28.35^\circ)} = 294.95\text{KN}$$

$$V_{Rd, max} = 294.95\text{KN} > V_d = 38.26\text{KN} \dots \dots \dots \text{ok}$$

$$V_{Rd, max} = 294.95\text{KN} > V_d = 45.95\text{KN} \dots \dots \dots \text{ok}$$

According to ES EN 1992-1-1:2015 in regions of the member where $V_{Rd, c} \geq V_{Ed}$ no calculated shear reinforcement is necessary.

Check $V_{Rd, c}$ is greater than V_{Ed} at d distance from the face of support

$$VR_{d,c} = \max \left\{ \begin{array}{l} CRd * k(100 * \rho_1 * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp} \\ (V_{min} + k_1 * \sigma_{cp}) * b_w * d \end{array} \right\} \dots\dots ES EN 1992-1-1:2015 section 6.2.2$$

equation (6.2.a and 6.2.b)

$$CRd = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$$

$$K = 1 + \sqrt{\frac{200}{122}} = 2.28 \geq 2.0, \text{ take } K=2$$

$$\rho_1 = \frac{A_s}{b_w * d} \leq 0.02$$

$$\text{For flight1, } \rho_1 = \frac{A_s}{b_w * d} = \frac{1501.145}{1000 * 122} = 0.0123 \leq 0.02$$

$$\text{For flight2, } \rho_1 = \frac{A_s}{b_w * d} = \frac{882.77}{1000 * 122} = 0.00723 \leq 0.02$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} < 0.2f_{cd} = 0 \dots\dots\dots \text{because } N_{Ed}=0$$

$$v_{min} = 0.035 * k^{\frac{3}{2}} * f_{ck}^{0.5} = 0.035 * 2^{\frac{3}{2}} * 20^{0.5} = 0.443$$

$$VR_{d,c} = \max \left\{ \begin{array}{l} [0.12 * 2(100 * 0.0123 * 20)^{\frac{1}{3}} + 2 * 0] * 1000 * 122 = 85.156KN \\ (0.443 + 2 * 0) * 1000 * 122 = 54.046KN \end{array} \right.$$

$$VR_{d,c} = 85.156KN$$

For flight1 $VR_{d,c} = 85.156KN > V_d = 33.26KN \dots\dots\dots ok$

$$VR_{d,c} = \max \left\{ \begin{array}{l} [0.12 * 2(100 * 0.00723 * 20)^{\frac{1}{3}} + 2 * 0] * 1000 * 122 = 71.33KN \\ (0.443 + 2 * 0) * 1000 * 122 = 54.046KN \end{array} \right.$$

For flight2 $VR_{d,c} = 71.33KN > V_d = 45.95KN \dots\dots\dots ok$

Therefore, no need of shear reinforcement and minimum reinforcement is required for the staircase according to.....(ES EN 1992-1-1:2015 section 6.2.1. n.d.)

3.3.7 DESIGN OF STAIR 2

Use $D=150mm, d=150-28=122mm$

Design load with similar step

Step dead load= $3.375KN/m$

Riser dead load = 0.785 kN/m

Waist dead load

$$\text{DL for concrete} = \frac{ts \cdot L_{\text{inclined}} \cdot \gamma_c}{\text{projected length}} \cdot lw = \frac{0.15 \cdot 3.58 \cdot 25}{3.15} \cdot 1 = 4.262 \text{ kN/m}$$

$$\text{DL for plastering} = \frac{tp \cdot L_{\text{inclined}} \cdot \gamma_p}{\text{projected length}} \cdot lw = \frac{0.02 \cdot 3.58 \cdot 23}{3.15} \cdot 1 = 0.5228 \text{ kN/m}$$

$$\text{Total dead load} = 4.262 + 0.5228 = 4.783 \text{ kN/m}$$

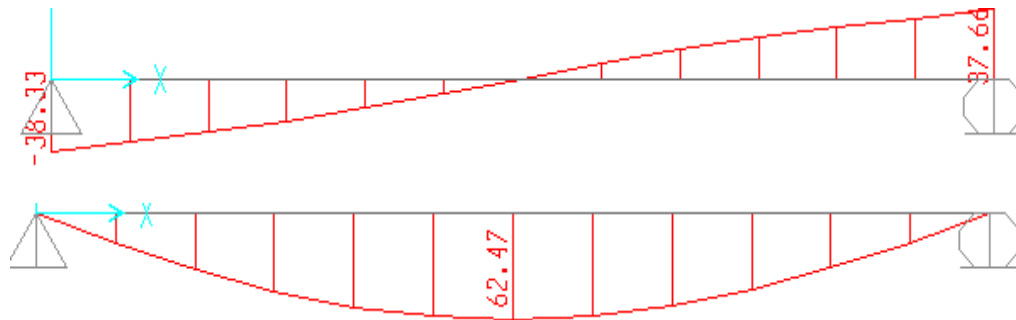
$$\text{DL}_{\text{flight}} = 4.783 + 0.785 + 3.375 = 8.943 \text{ kN/m}$$

Design load for landing, $p_d = 10.1 \text{ kN/m}$

Design load for flight

$$p_d = 1.35 \text{DL} + 1.5 \text{LL} = 1.35 \cdot 8.943 + 1.5 \cdot 2 = 15.07 \text{ kN/m}$$

✓ Determination of moment



$$M_D = 62.47 \text{ kN/m}$$

$$\rho = \frac{11.33}{347.83} \left(1 - \sqrt{1 - \frac{2 \cdot 62.47 \cdot 10^6 \text{ Nmm}}{11.33 \frac{10^6 \text{ N}}{\text{mm}^2} \cdot 1000 \text{ mm} \cdot (122 \text{ mm})^2}} \right) = 0.01599$$

$$A_{st, \text{cal}} = \rho b d = 0.01599 \cdot 1000 \text{ mm} \cdot 122 \text{ mm} = 1951.08 \text{ mm}^2$$

Check for minimum and maximum reinforcement area

$$A_{st, \text{min}} = 174.46 \text{ mm}^2 \text{ and } A_{st, \text{max}} = 4880 \text{ mm}^2$$

$A_{st, \text{min}} < A_{st, \text{cal}} < A_{st, \text{max}}$ok! Therefore, provided $A_{st, \text{provided}} = 1951.08 \text{ mm}^2$

Secondary transvers reinforcement

$$A_{st} = 20\% A_{st, \text{provid}} = 0.2 \cdot 1951.08 = 390.22 \text{ mm}^2$$

$A_{st} = 390.22\text{mm}^2 > A_{st, \min} = 174.46\text{mm}^2 \dots \dots \dots \text{ok}$. Therefore, secondary reinforcement should be provided.

Spacing for principal reinforcement

$$S = \frac{bt \cdot a_s}{A_{st, \text{provided}}} = \frac{1000\text{mm} \cdot 113.04\text{mm}^2}{1951.08\text{mm}^2} = 57.9\text{mm}, \text{ Use } \varnothing 12\text{c}/\text{c}55\text{mm}$$

Spacing for secondary reinforcement

$$S = \frac{bt \cdot a_s}{A_{st, \text{provided}}} = \frac{1000\text{mm} \cdot 113.04\text{mm}^2}{390.22\text{mm}^2} = 289.68\text{mm}, \text{ Use } \varnothing 12\text{c}/\text{c}280\text{mm}$$

Check for maximum spacing

For principal reinforcement

$$S_{\text{provided}} = 55\text{mm} < S_{\text{max}} = 400\text{mm} \dots \dots \dots \text{ok}$$

For secondary reinforcement

$$S_{\text{provided}} = 280\text{mm} < S_{\text{max}} = 450\text{mm} \dots \dots \dots \text{ok}$$

Shear design

Check if the $V_{Rd, \max}$ greater than V_d at the support

$$V_{Rd, \max} = 294.95\text{KN} > V_d = 38.33\text{KN} \dots \dots \dots \text{ok}$$

$$\rho_1 = \frac{1951.08}{1000 \cdot 122} = 0.01599$$

$$V_{Rd, c} = \max \left\{ \begin{aligned} & \left[0.12 \cdot 2(100 \cdot 0.01599 \cdot 20)^{\frac{1}{3}} + 2 \cdot 0 \right] \cdot 1000 \cdot 122 = 92.94\text{KN} \\ & (0.443 + 2 \cdot 0) \cdot 1000 \cdot 122 = 54.046\text{KN} \end{aligned} \right.$$

$V_{Rd, c} = 92.94\text{KN} > V_d = 38.33\text{KN} \dots \dots \dots \text{ok}$, Therefore, no need of shear reinforcement.

3.3.8 DESIGN STAIR 3

Which is one flight stair without landing from Ground floor to 1st floor

Calculation of dead load using 1.4m Flight width

Height of flight=3m

Number of going from plan view =14with 30cm width

Number of riser = number of going+2 =16(Total)

$$\text{height of riser} = \frac{300\text{cm}}{16} = 18.75 \sim 19\text{cm}$$

$$\tan\theta = \frac{3}{3.68}, \theta = 39.18^\circ \text{ then inclined length} = \frac{3.68}{\cos 39.18^\circ} = 4.747\text{m}$$

$$\frac{l}{d} = 33.203, d = \frac{4747}{33.203} = 142.969$$

$$D = d + \text{cover} + \frac{\phi_{\text{longitudinal}}}{2} = 142.969\text{mm} + 22\text{mm} + 6\text{mm} = 170.9\text{mm}, \text{ Use } D = 180\text{mm}, d = 180 - 28 = 152\text{mm}$$

Step dead load

$$\text{DL for concrete slab} = \frac{1}{2} * h * L_w * \gamma_c = 0.5 * 0.19 * 1 * 25 = 2.375\text{KN/m}$$

$$\text{DL for cement screed} = t_{cs} * \gamma_c * L_w = 0.03 * 23 * 1 = 0.69\text{KN/m}$$

$$\text{DL for marble} = t_m * \gamma_m * L_w = 0.03 * 27 * 1 = 0.81\text{KN/m}$$

$$\text{Total dead load} = 2.375 + 0.69 + 0.81 = 2.4487\text{KN/m}$$

Riser dead load

$$\text{DL for cement screed} = \frac{\text{no of riser}(h * t * \gamma)}{\text{projected length}} * l_w = \frac{16(0.19 * 0.03 * 23)}{3.68} * 1 = 0.57\text{KN/m}$$

$$\text{DL for marble} = \frac{\text{no of riser}(h * t * \gamma)}{\text{projected length}} * l_w = \frac{16(0.19 * 0.03 * 27)}{3.68} * 1 = 0.669\text{KN/m}$$

$$\text{Total dead load} = 0.669 + 0.57 = 1.239\text{KN/m}$$

Waist dead load

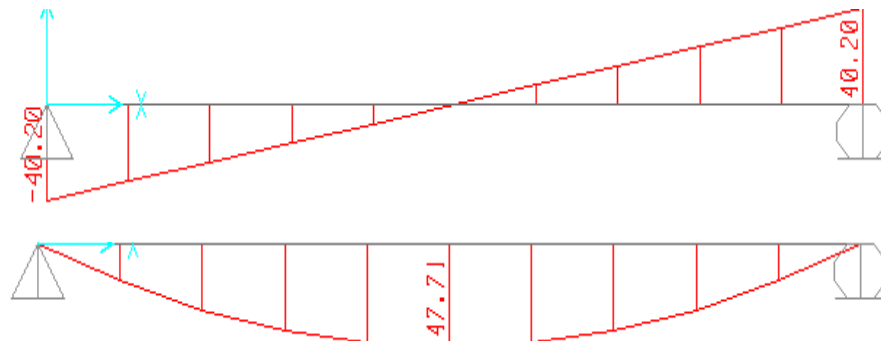
$$\text{DL for concrete} = \frac{t_s * L_{\text{inclined}} * \gamma_c}{\text{projected length}} * l_w = \frac{0.18 * 4.747 * 25}{3.68} * 1 = 5.805\text{KN/m}$$

$$\text{DL for plastering} = \frac{t_p * L_{\text{inclined}} * \gamma_p}{\text{projected length}} * l_w = \frac{0.02 * 4.747 * 23}{3.68} * 1 = 0.592\text{KN/m}$$

$$\text{Total dead load} = 5.805 + 0.592 = 6.398\text{KN/m}$$

$$\text{DL}_{\text{flight}} = 6.398 + 1.239 + 2.4487 = 10.08\text{KN/m}$$

$$P_d = 1.35\text{DL} + 1.5\text{LL} = 1.35 * 10.08 + 1.5 * 2 = 16.61\text{KN/m}$$



MD=47.71KNm

$$\rho = \frac{11.33}{347.83} \left(1 - \sqrt{1 - \frac{2 * 47.71 * 10^6 \text{Nmm}}{11.33 \frac{10^6 \text{N}}{\text{mm}^2} * 1000\text{mm} * (152\text{mm})^2}} \right) = 0.00661$$

$$A_{st, cal} = \rho b d = 0.00661 * 1000\text{mm} * 152\text{mm} = 1004.87\text{mm}^2$$

Check for minimum and maximum reinforcement area

$$A_{st, min} = \max\left\{0.26 * \frac{f_{ctm} * b * d}{f_{yk}}, 0.0013 * b * d\right\} \dots \dots \dots \text{ES EN 1992:0215 art 9.2.1.1 (1)}$$

$$A_{st, min} = \max\left\{0.26 * \frac{2.2\text{Mpa} * 1000\text{mm} * 152\text{mm}}{400\text{Mpa}}, 0.0013 * 1000\text{mm} * 152\text{mm}\right\}$$

$$A_{st, min} = \max\{217.36\text{mm}^2, 197.6\text{mm}^2\}$$

$$A_{st, min} = 197.6\text{mm}^2$$

$$A_{st, max} = 0.04A_c = 0.4 * b * d = 0.4 * 1000 * 152 = 6080\text{mm}^2$$

$A_{st, min} < A_{st, cal} < A_{st, max}$ok! Therefore, provided $A_{st, provided} = 1004.87\text{mm}^2$

Secondary transvers reinforcement

$$A_{st} = 20\%A_{st, provided} = 0.2 * 1004.87 = 200.974\text{mm}^2$$

$A_{st} = 200.974\text{mm}^2 > A_{st, min} = 197.6\text{mm}^2$ ok. Therefore, secondary reinforcement should be provided.

Spacing for principal reinforcement

$$S = \frac{b t a_s}{A_{st, provided}} = \frac{1000\text{mm} * 113.04\text{mm}^2}{1004.87\text{mm}^2} = 112.49\text{mm}, \text{ Use } \emptyset 12\text{c/c} 110\text{mm}$$

Spacing for secondary reinforcement

$$S = \frac{b t a_s}{A_{st, provided}} = \frac{1000\text{mm} * 113.04\text{mm}^2}{200.974\text{mm}^2} = 562.46\text{mm}, \text{ Use } \emptyset 12\text{c/c} 560\text{mm}$$

Check for maximum spacing

For principal reinforcement

$$S_{max, slab} = \min\left\{3h \text{ or } 3t_s, 400\text{mm}\right\} = \min\left\{3 * 180 = 540\text{mm}, 400\text{mm}\right\}, \text{ take } 400\text{mm}$$

$S_{provided} = 110\text{mm} < S_{max} = 400\text{mm}$ ok

For secondary reinforcement

$$S_{\max, \text{slab}} = \min \left\{ \begin{array}{l} 3.5h \text{ or } 3.5t_s \\ 450\text{mm} \end{array} \right\} = \min \left\{ \begin{array}{l} 3.5 * 180 = 630\text{mm} \\ 450\text{mm} \end{array} \right\}, \text{ take } 450\text{mm}$$

S_{provided} = 560mm < S_{max} = 450mm not ok, use Ø12c/c450mm

Shear design

Check if the V_{Rd, max} greater than V_d at the support

$$V_{Rd, \max} = 294.95\text{KN} > V_d = 40.2\text{KN} \dots \dots \text{ok}$$

$$\rho_1 = \frac{1004.87}{1000 * 152} = 0.00661$$

$$V_{Rd, c} = \max \left\{ \begin{array}{l} \left[0.12 * 2(100 * 0.00661 * 20)^{\frac{1}{3}} + 2 * 0 \right] * 1000 * 152 = 86.262\text{KN} \\ (0.443 + 2 * 0) * 1000 * 152 = 67.336\text{KN} \end{array} \right.$$

V_{Rd, c} = 92.94KN > V_d = 40.2KN ok, Therefore, no need of shear reinforcement.

3.3.9 STAIR DESIGN FOR BASEMENT

This is another straight stair with intermediate landing and designed with 5.5m width.

Projected length L₁=0.9m and L₂=1.8m, height of stair h_{f1} =0.83m and h_{f2}=1.2m with 5.5m width.

Number of going =10, height of riser = $\frac{203\text{cm}}{12} = 16.9 \sim 17\text{cm}$

$$\theta_1 = \tan^{-1} \left(\frac{0.83}{0.9} \right) = 42.68^\circ, \theta_2 = \tan^{-1} \left(\frac{1.2}{1.8} \right) = 33.69^\circ$$

$$\text{then inclined length, } l_1 = \frac{0.9}{\cos 42.68^\circ} = 1.22\text{m and } l_2 = \frac{1.8}{\cos 33.69^\circ} = 2.16\text{m}$$

$$\frac{l}{d} = 33.203, d = \frac{2160}{33.203} = 65.05\text{mm}$$

$$D = d + \text{cover} + \frac{\phi_{\text{longitudinal}}}{2} = 65.05\text{mm} + 22\text{mm} + 6\text{mm} = 93.05\text{mm}, \text{ Use } D = 130\text{mm}, d = 130 - 28 = 102\text{mm}$$

Dead load of landing

$$\text{DL for concrete slab} = t_{\text{slab}} * \gamma_c * L_w = 0.130 * 25 * 5.5 = 17.875\text{KN/m}$$

$$\text{DL for plastering} = t_p * \gamma_p * L_w = 0.02 * 23 * 5.5 = 2.53\text{KN/m}$$

$$\text{DL for cement screed} = t_{cs} * \gamma_{cs} * L_w = 0.03 * 23 * 5.5 = 3.795\text{KN/m}$$

$$\text{DL for marble} = t_m * \gamma_m * L_w = 0.03 * 27 * 5.5 = 4.455\text{KN/m}$$

$$\text{Total dead load} = 17.875 + 2.53 + 3.795 + 4.455 = 28.655\text{KN/m}$$

$$P_d = 1.35 * 28.655 + 1.5 * 11 = 55.18\text{KN/m}$$

Step dead load

$$\text{DL for concrete slab} = \frac{1}{2} * h * L_w * \gamma_c = 0.5 * 0.17 * 5.5 * 25 = 11.687 \text{KN/m}$$

$$\text{DL for cement screed} = t_{cs} * \gamma_c * L_w = 0.03 * 23 * 5.5 = 3.795 \text{KN/m}$$

$$\text{DL for marble} = t_m * \gamma_m * L_w = 0.03 * 27 * 5.5 = 4.455 \text{KN/m}$$

$$\text{Total dead load} = 11.687 + 3.795 + 4.455 = 19.937 \text{KN/m}$$

Riser dead load for flight1

$$\text{DL for cement screed} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * l_w = \frac{4(0.17*0.03*23)}{0.9} * 5.5 = 2.867 \text{KN/m}$$

$$\text{DL for marble} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * l_w = \frac{4(0.15*0.03*27)}{0.9} * 5.5 = 2.97 \text{KN/m}$$

$$\text{Total dead load} = 2.867 + 2.97 = 5.837 \text{KN/m}$$

Waist dead load

$$\text{DL for concrete} = \frac{t_s * l_{\text{inclined}} * \gamma_c}{\text{projected length}} * l_w = \frac{0.13 * 1.22 * 25}{0.9} * 5.5 = 24.23 \text{KN/m}$$

$$\text{DL for plastering} = \frac{t_p * l_{\text{inclined}} * \gamma_p}{\text{projected length}} * l_w = \frac{0.02 * 1.22 * 23}{0.9} * 5.5 = 3.43 \text{KN/m}$$

$$\text{Total dead load} = 24.23 + 3.43 = 27.66 \text{KN/m}$$

$$\text{DL}_{\text{flight1}} = 19.937 + 5.837 + 27.66 = 53.434 \text{KN/m}$$

$$\text{Pd for f1} = 1.35 \text{DL} + 1.5 \text{LL} = 1.35 * 53.434 + 1.5(2 * 5.5) = 88.63 \text{KN/m}$$

Riser dead load for flight2

$$\text{DL for cement screed} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * l_w = \frac{8(0.17*0.03*23)}{1.8} * 5.5 = 2.867 \text{KN/m}$$

$$\text{DL for marble} = \frac{\text{no of riser}(h*t*\gamma)}{\text{projected length}} * l_w = \frac{8(0.15*0.03*27)}{1.8} * 5.5 = 2.97 \text{KN/m}$$

$$\text{Total dead load} = 2.867 + 2.97 = 5.837 \text{KN/m}$$

Waist dead load

$$\text{DL for concrete} = \frac{t_s * l_{\text{inclined}} * \gamma_c}{\text{projected length}} * l_w = \frac{0.13 * 2.16 * 25}{1.8} * 5.5 = 21.45 \text{KN/m}$$

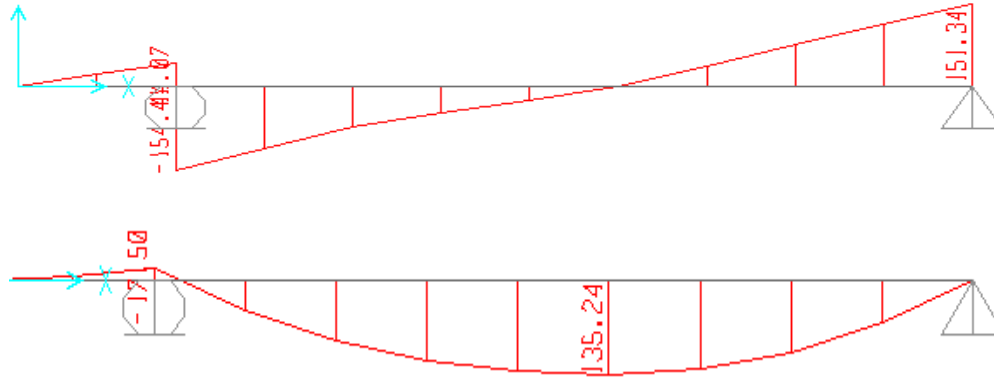
$$\text{DL for plastering} = \frac{t_p * l_{\text{inclined}} * \gamma_p}{\text{projected length}} * l_w = \frac{0.02 * 2.16 * 23}{1.8} * 5.5 = 3.036 \text{KN/m}$$

$$\text{Total dead load} = 21.45 + 3.036 = 24.486 \text{KN/m}$$

DL for flight2=24.486+5.837+19.937=50.26KN/m

Pd=1.35*50.26+1.5*11=84.35KN/m

Moment Analysis by using SAP 2000



MD=135.24KNm

$$\rho = \frac{11.33}{347.83} \left(1 - \sqrt{1 - \frac{2 * 135.24 * 10^6 \text{Nmm}}{11.33 \frac{10^6 \text{N}}{\text{mm}^2} * 5500 \text{mm} * (102 \text{mm})^2}} \right) = 0.007706$$

Ast, cal = ρbd=0.007706*5500mm*102mm=4323.28mm²

Check for minimum and maximum reinforcement area

Ast, min =max{0.26 * $\frac{f_{ctm} * b * d}{f_{yk}}$, 0.0013*b*d}.....ES EN 1992:0215 art 9.2.1.1 (1)

$$A_{st, min} = \max\left\{0.26 * \frac{2.2 \text{Mpa} * 5500 \text{mm} * 102 \text{mm}}{400 \text{Mpa}}, 0.0013 * 5500 \text{mm} * 102 \text{mm}\right\}$$

$$A_{st, min} = \max\{802.23 \text{mm}^2, 729.3 \text{mm}^2\}$$

$$A_{st, min} = 802.23 \text{mm}^2$$

$$A_{st, max} = 0.04A_c = 0.4 * b * d = 0.4 * 5500 * 102 = 22440 \text{mm}^2$$

Ast, min < Ast, cal < Ast, max.....ok! Therefore, provided Ast, provided = 4323.28mm²

Secondary transvers reinforcement

$$A_{st} = 20\%A_{st, provid} = 0.2 * 4323.28 = 864.656 \text{mm}^2$$

Ast = 864.656mm² > Ast, min = 802.23mm²ok. Therefore, secondary reinforcement should be provided.

Spacing for principal reinforcement

$$S = \frac{bt \cdot a_s}{A_{st,provided}} = \frac{5500mm \cdot 113.04mm^2}{4323.28mm^2} = 143.8mm, \text{ Use } \emptyset 12c/c140mm$$

Spacing for secondary reinforcement

$$S = \frac{bt \cdot a_s}{A_{st,provided}} = \frac{5500mm \cdot 113.04mm^2}{864.656mm^2} = 719.03mm, \text{ Use } \emptyset 12c/c710mm$$

Check for maximum spacing

For principal reinforcement

$$S_{max, slab} = \min \left\{ \begin{array}{l} 3h \text{ or } 3t_s \\ 400mm \end{array} \right. = \min \left\{ \begin{array}{l} 3 * 130 = 390mm \\ 400mm \end{array} \right. , \text{ take } 390mm$$

$$S_{provided} = 140mm < S_{max} = 390mm \dots \dots \dots \text{ok}$$

For secondary reinforcement

$$S_{max, slab} = \min \left\{ \begin{array}{l} 3.5h \text{ or } 3.5t_s \\ 450mm \end{array} \right. = \min \left\{ \begin{array}{l} 3.5 * 130 = 455mm \\ 450mm \end{array} \right. , \text{ take } 450mm$$

$$S_{provided} = 710mm < S_{max} = 450mm \dots \dots \dots \text{not ok, use } \emptyset 12c/c450mm$$

Shear design

$$\rho_1 = \frac{4323.28}{5500 * 102} = 0.0077$$

$$V_{Rd,c} = \max \left\{ \begin{array}{l} \left[0.12 * 2(100 * 0.0077 * 20)^{\frac{1}{3}} + 2 * 0 \right] * 55000 * 102 = 335.07KN \\ (0.443 + 2 * 0) * 5500 * 102 = 248.5KN \end{array} \right.$$

$V_{Rd,c} = 335.07KN > V_d = 154.4KN \dots \dots \dots \text{ok, Therefore, no need of shear reinforcement.}$

4.0 DESIGN FOR EARTHQUAKE RESISTANCE

If resources were unlimited, seismic protection would be achieved by simply providing as much earthquake resistance as possible to structures. In practice, it is not feasible to reduce seismic vulnerability to an absolute minimum because the costs would be prohibitive and certainly not justified since they would be for protection against a loading case that may not even occur during the useful life of the structure. Therefore Seismic design seeks to balance the investment in provision of seismic resistance against the level of damage, loss or disruption that earthquake loading could impose. For this reason, quantitative assessment and characterization of the expected levels of ground shaking constitute an indispensable first step of seismic design, and this process is seismic hazard analysis.

4.1 EARTH QUAKE ANALYSIS

There are three methods of earthquake analysis. (ES EN 1998-1:2015 section 4.3.3 n.d.)

Selecting an appropriate type of analysis method for this building

A. Check to use lateral force method of analysis

Requirements: $T_1 = \begin{cases} 4T_c \\ 2\text{sec} \end{cases}$ Where T_1 is fundamental period of vibration

Then according to (ES EN 1998-1:2015 section 4.3.3.2.2 n.d.) Expression 4.6 for buildings with heights of up to 40 m the value of T_1 (s) may be approximated by the following expression

$$T_1 = C_t H^{3/4}$$

Where; C_t is 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures; H is the height of the building in m, from the foundation or from the top of a rigid basement.

Then, $C_t=0.075$ for moment resistant space concrete frames and $H=23.93\text{m}$

$$T_1 = 0.075 * (23.93)^{3/4} = 0.81\text{sec}$$

There is no soil data thus assume the site to be ground type A and type 1 spectrum

According toES EN 1998-1:2015 section 3.2.2.2 table 3.2 $T_c = 0.25$

$$T_1 = 0.81\text{sec} \leq \begin{cases} 4 * 0.25 \\ 2\text{sec} \end{cases} = \begin{cases} 1\text{sec} \\ 2\text{sec} \end{cases} \dots\dots\dots\text{ok}$$

B. Check the criteria for regularity in elevation

According to ES EN 1998-1:2015 section 4.2.3.3 the criteria for regularity in elevation: in this building no setbacks are present because the building satisfy all above criteria. It is also regular in elevation.

4.1.1 DESIGN SPECTRUM FOR ELASTIC ANALYSIS

According to.....ES EN 1998-1:2015 section 3.2.2.5

The design spectrum for elastic analysis of the seismic action $S_d(T)$ is defined by the following expressions

$$0 \leq T \leq T_B: S_d(T) \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \leq T \leq T_C: S_d(T) = ag \cdot S \cdot \eta \cdot \frac{2.5}{q}$$

$$T_B \leq T \leq T_C: S_d(T) = \begin{cases} = ag \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \geq \beta \cdot ag \end{cases}$$

$$T_D \leq T: S_d = \begin{cases} ag \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C \cdot T_D}{T^2} \right] \\ \geq \beta \cdot ag \end{cases}$$

Where: T is the vibration period of a linear single-degree-of-freedom system

T_C is the upper limit of the period of the constant spectral acceleration branch

T_D is the value defining the beginning of constant displacement response range of the spectrum

T_B is the lower limit of the period of the constant spectral acceleration branch

ag is the design ground acceleration on type A ground ($ag = \gamma_1 \cdot agR$)

β is the lower bound factor for the horizontal design spectrum and S is the soil factor.

Values of the periods T_B , T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type, taken from ES EN 1998-1:2015 table 3.2.

Table 4. 1 Type 1 elastic response spectra for ground type A

S	TB	TC	TD
1	0.05	0.25	1.2

For $T_C \leq T_1 \leq T_D$

$$S_d(T_1) = \max \begin{cases} ag \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C}{T_1} \right] \\ \beta \cdot ag \end{cases}$$

The design ground acceleration, $ag = \gamma_1 agR$ But, $agR = a_0$ from national annex of Ethiopia

From zonation map Ethiopia for Addis Ababa (zone 3).

$$a_0 = \frac{a_g}{g} = 0.1 \rightarrow a_0 = 0.1g = 0.1 * \frac{9.81m}{s^2} = 0.981m/s^2$$

γ_1 is importance factor and According to ES EN 1998-1:2015 section 4.2.5 table 4.3 for importance class II and Ordinary buildings, not belonging in the other categories shall be by definition equal to 1.0

$$a_g = 1 * 0.981 = 0.981m/s^2$$

According to ES EN 1998-1:2015 section 5.3.3, the behavior factor q of 1.5 may be used in deriving the seismic action for DCM regardless of the structural system and the regularity in elevation.

$$S_d(T) = \max \left\{ \begin{array}{l} \frac{0.981m}{s^2} * 1 * \frac{2.5}{1.5} * \left[\frac{0.25sec}{1sec} \right] = 0.41m/s^2 \\ 0.2 * \frac{0.981m}{s^2} = 0.196m/s^2 \end{array} \right. , \text{ then } S_d(T) = 0.41m/s^2$$

The correction factor λ

$$\lambda = 0.85 \text{ if } T_1 \leq 2T_c \text{ otherwise } \lambda = 1, \text{ thus } T_1 = 0.81 \text{ sec and } 2T_c = 0.5sec$$

$$0.81sec \leq 0.5sec \dots \dots \dots \text{ not ok so, } \lambda = 1$$

4.1.2 DETERMINATION OF CENTER OF MASS

Centre of mass is a point on a floor level where the whole mass and its inertial effects can be replaced using lumped equivalent mass. Total weight of the building, that acts in the direction of gravity.

$$X_m = \frac{\sum w_i x_i}{\sum w_i}, Y_m = \frac{\sum w_i y_i}{\sum w_i}, \text{ Center of mass from ETABS}$$

Table 4. 2 Center of mass of the Building

Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM
ROOF	D1	139.147	139.147	16.257	10.873	139.1469	139.1469	16.257	10.873
5TH	D1	6112.84	6112.84	18.08	12.055	6251.9896	6251.99	18.039	12.029
4TH	D1	6137.27	6137.27	18.075	12.04	12389.26	12389.26	18.057	12.034
3RD	D1	1673.16	1673.16	17.948	11.74	14062.418	14062.42	18.044	11.999
2ND	D1	1580.68	1580.68	17.928	11.732	15643.099	15643.1	18.032	11.972
1ST	D1	1625.56	1625.56	18.105	11.392	17268.656	17268.66	18.039	11.917
GROUND	D1	1657.35	1657.35	18.173	11.385	18926.011	18926.01	18.051	11.871
BASEMEN	D1	530.437	530.437	17.657	10.963	19456.447	19456.45	18.04	11.846

4.1.3 STORY SHEAR DETERMINATION

The base shear force shall be distributed over the height of a structure concentrated at each floor level as $F_i =$

$$f_b * \frac{z_i m_i}{\sum z_j m_j} \dots \dots \dots \text{ES-EN 1998-1-2015}$$

Table 4. 3 Story Shear

Story	Loc	P	MX	MY
ROOF	Top	966.68	10826.8	-16868.601
ROOF	Bottom	1543.32	16688.2	-24778
5TH	Top	18643.2	219648	-334691.47
5TH	Bottom	19539.5	229527	-350064.24
4TH	Top	36639.3	432487	-659977.8
4TH	Bottom	37801.2	443101	-677888.34
3RD	Top	54346.5	640271	-979438.4
3RD	Bottom	55363.6	650496	-995572.57
2ND	Top	71966.8	843833	-1300450.6
2ND	Bottom	73032.3	855969	-1319666.1
1ST	Top	90280.3	1050162	-1640078.3
1ST	Bottom	91640.7	1065800	-1665346.2
GROUNI	Top	109106	1263579	-1990822.2
GROUNI	Bottom	110299	1275399	-2009771.8
BASEME	Top	114770	1324982	-2090703.6
BASEME	Bottom	115662	1334690	-2106086.8

5.0 FRAME ANALYSIS

5.1 ANALYSIS OF 3D FRAME

Modeling for 3D frame analysis using ETABS V 2017

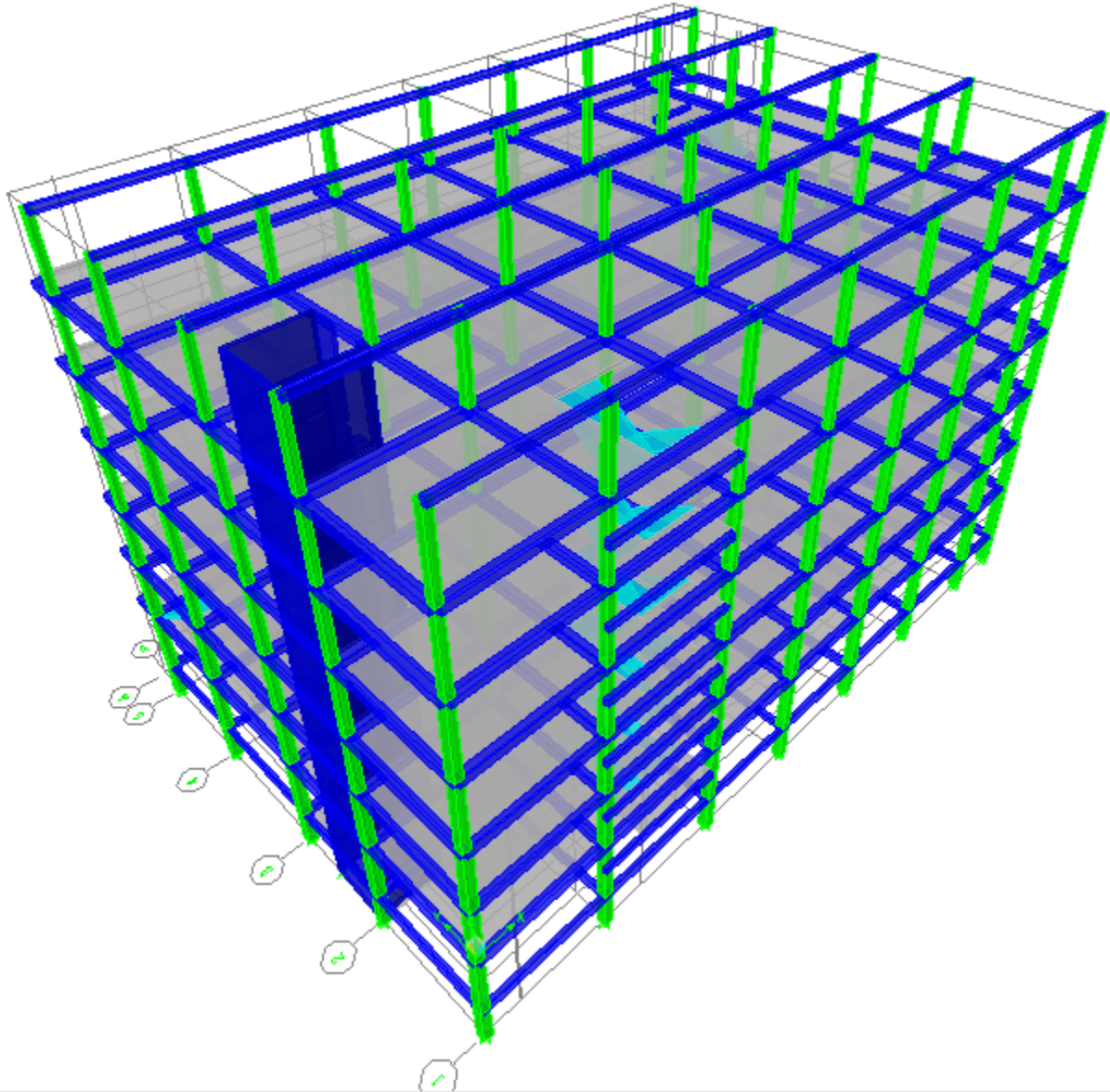


Figure 5. 1 building 3d model

General procedure of Modeling of frame

Step 1: plot grid coordinates

Plot grid coordinates that represent the span structural design

Step 2: Define Material

Concrete C-25 is defined

Material type =concrete

Symmetry type=Isotropic

Modulus of Elasticity=30000MPa

Poisson's ratio =0.2

Shear Modulus 11.67MPa

Coefficient of Thermal Expansion =0.0000099/c

Unit Weight =20KN/m³

Step 3 Define Frame section

For this project three types of frame sections are define

Beam grid beam 200mmx300mm Solid slab beam 300mmx400mm

Ribbed slab beam 400mmx300mm top tie beam 200mmx300mm

Column C 500X500 for basement and ground floor central columns

C400x400 for 1st, 2nd and 3rd floor central columns and G-1st edge columns

C350x350 for 1st, 2nd and 3rd floor edge columns

C300x300 for 4th and 5th floor

Slab section as shell element with thickness of 220mm

Slab section as shell element with thickness of 210mm

Slab section as shell element with thickness of 290mm

For roof structure GEA500 with 0.4mm thickness

For truss -50mmx30mmx3mm RHS purlin - 20mmx30mmx2.5mmRHS diagonal members

-20mmx40mmx3mm RHS top and bottom members

150mm shear wall depth and 150mm for stair landing

Diaphragm for each floor level

Step 4) Draw the different structural members

Using the grid system draw the structural member with their defined frame, section properties it includes assignment of the restraints (Fixed -joint)

Step 5) Assignment of loads

The loads which comes from the slab as factored live, dead load and the wall which rest on the beam should be assigned as a dead load

Earth quick load is assigned on the User loads with the respected direction

The Load combination

$$\text{Combo 1} = 1.35(\text{SW} + \text{FF} + \text{WL} + \text{PL}) + 1.5\text{LL} = 1.35\text{DL} + 1.5\text{LL}$$

$$\text{Combo 2} = 0.75 * \text{Combo 1} + \text{QXP}$$

$$\text{Combo 3} = 0.75 * \text{Combo 1} - \text{QXP}$$

$$\text{Combo 4} = 0.75 * \text{Combo 1} + \text{QXN}$$

$$\text{Combo 5} = 0.75 * \text{Combo 1} - \text{QXN}$$

$$\text{Combo 6} = 0.75 * \text{Combo 1} + \text{QYP}$$

$$\text{Combo 7} = 0.75 * \text{Combo 1} - \text{QYP}$$

$$\text{Combo 8} = 0.75 * \text{Combo 1} + \text{QYN}$$

$$\text{Combo 9} = 0.75 * \text{Combo 1} - \text{QYN}$$

Step 6) Analysis

After checking for errors the analysis result output is generated

5.2 DESIGN AND ANALYSIS OF BEAM

A beam is a member for which the span is not less than 3 times the overall section depth otherwise it should be considered as a deep beam (ES-EN-1992-2015-Art 5.3.31.2)

It is a structural member that supports applied loads which comes from the slab, partition wall and for design use the maximum bending moment and shear force. In this project, different section of the beam is considered by different elevation of the building, based on the functional use of the building and based on the slab types.

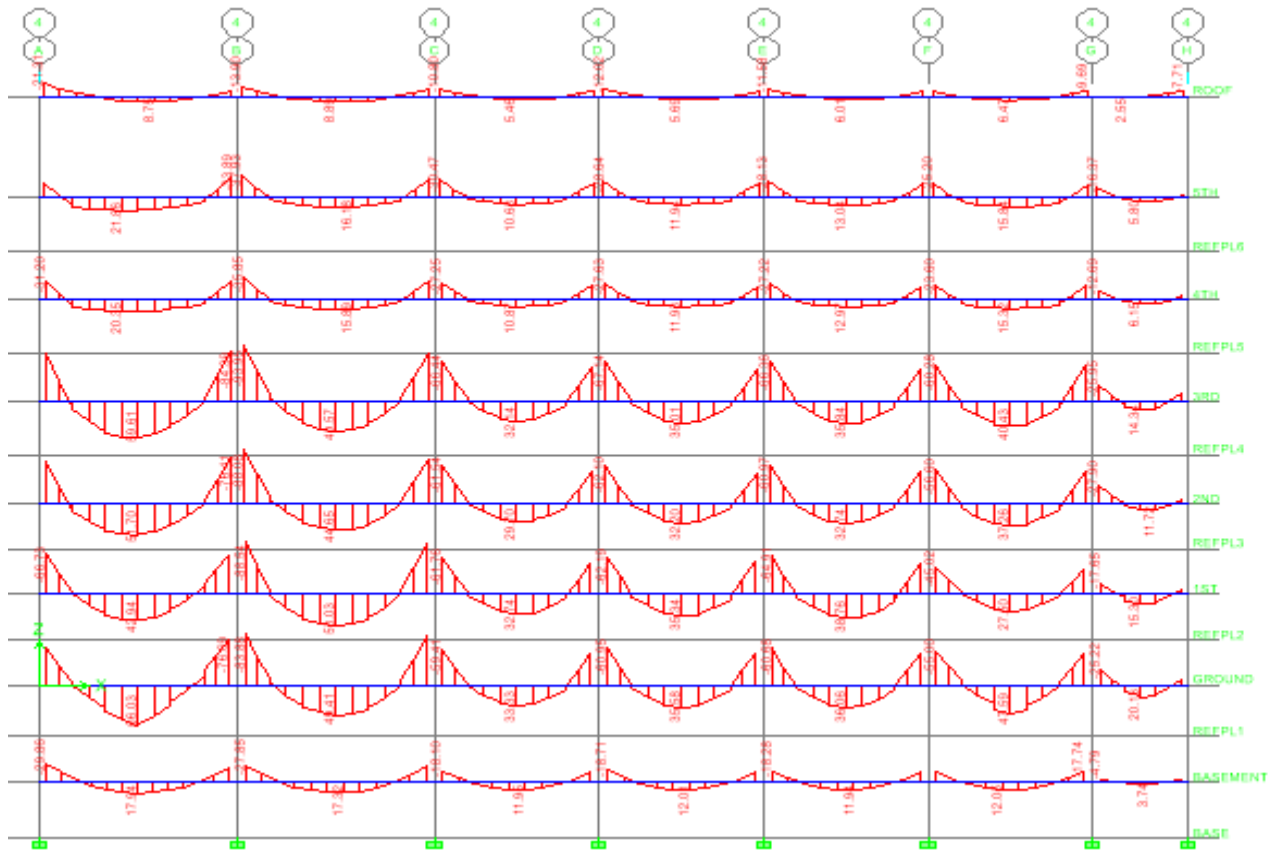


Figure 5. 2 beam result

Determine material design values (specifications)

1. Concrete cover determination

The nominal concrete cover is the distance between the surfaces of reinforcement closest to the nearest concrete surface.

$$C_{nom} = C_{min} + \Delta C_{dev} \dots \dots \dots \text{ES EN 1992:2015 Art 4.4.12(1)}$$

Where; C_{min} –minimum cover and ΔC_{dev} is allowance in design for deviation

Minimum cover, shall provide in order to ensure

- ✓ Safe transmission of bond force
- ✓ Corrosion resistance/ durability
- ✓ Fire resistance

$$C_{min} = \max \left\{ \begin{array}{l} C_{min, b} \\ C_{min, dur} + \Delta C_{dur, \gamma} - \Delta C_{dur, st} - \Delta C_{dur, add} \dots \dots \dots \text{Eqn(4.2)} \\ 10\text{mm} \end{array} \right.$$

Table 4.2: minimum cover, $C_{min, b}$ requirements with regard to bond.

Assume $\Phi 20$ longitudinal bar and $\Phi 20$ nominal maximum aggregate size;

Therefore; $C_{min, b} = 20\text{mm}$

Cover design for corrosion/durability

- ✓ Dry or permanent wet condition = xc1
- ✓ Exposure class to determine $C_{min, dur}$
- ✓ Member with slab reduced by 1
- ✓ Strength class C25
- ✓ The recommended structural class (design working life of 50 years) is S4 but reduced by 1 so S3.
- ✓ Environmental requirement for $C_{min, dur}$ (mm)

$$C_{min} = \max \begin{cases} C_{min, b} = 20\text{mm} \\ C_{min, dur} = 10\text{mm} \\ 10\text{mm} \end{cases}$$

Therefore; $C_{min} = 20\text{mm}$

ΔC_{dev} (Allowance in design for variation)

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

Then; $C_{nom} = C_{min} + \Delta C_{dev} = 20\text{mm} + 10\text{mm} = 30\text{mm}$

2. 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

$$F1 = \frac{500}{\delta S} = \frac{500}{f_{yk} \cdot \frac{A_{s, req}}{A_{s, prov}}}$$

$F2 = 0.8$, for flanged section where the ratio of the flange breadth to the rib breadth exceeds 3.

Otherwise; $F2 = 1$ for other cases.

$F3 = 7/l_{eff}$ For beam, $F3 = 1$ for both cases.

Then in this case the lengths are below 7m so $F2 = F3 = 1$

Let's assume $\rho = \rho_0$ and equation 7.16a as N

$A_{s, req} = A_{s, prov}$

$$\frac{l}{d} = K * N * F1 * F2 * F3$$

$$\text{Where: } N = 11 + 1.5 \sqrt{fck} \frac{\rho_o}{\rho} + 3.2 \sqrt{fck} \left(\frac{\rho_o}{\rho} - 1 \right)^{3/2}$$

$$F1 = \frac{500}{f_{yk} * A_{req} / A_{prov}}$$

$$F1 = 500 / 400 = 1.25 \dots \text{since } A_{req} = A_{prov}$$

$$N = 11 + 1.5 * 20^{0.5} = 17.708$$

$$F2 = 1 \text{ and } F3 = 1 \text{ (because span of beam } < 7\text{m)}$$

The value of K is 1.5 for interior span, 1.3 for end span and 0.4 for cantilever. Using table 7.4N of ESEN1992:2015.

$$\frac{6000}{d} = 1.3 * 17.708 * 1.25 * 1 * 1 = 28.7755, \text{ where } L = 6\text{m} = 6000\text{mm}$$

$$d = \frac{6000\text{mm}}{28.7755} = 208.51\text{mm}$$

$$D = d + cc + \frac{\emptyset}{2} = 208.51\text{mm} + 30\text{mm} + 8\text{mm} + \frac{20\text{mm}}{2} = 256.51\text{mm} \dots \text{take } D = 300\text{mm}$$

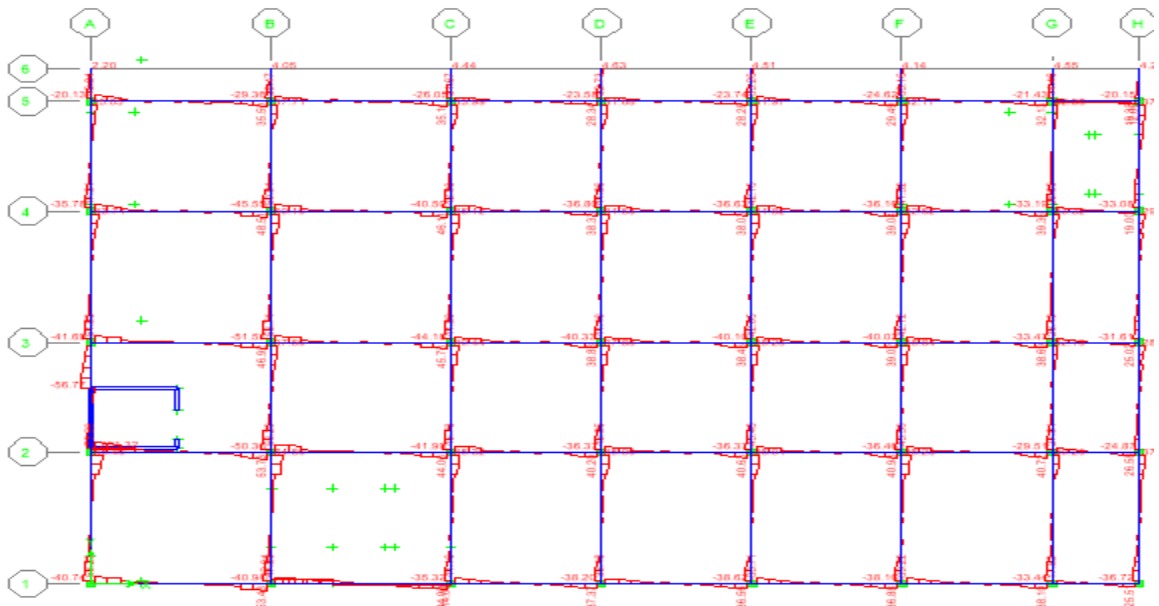


Figure 5.3 5th floor beam shear force diagram

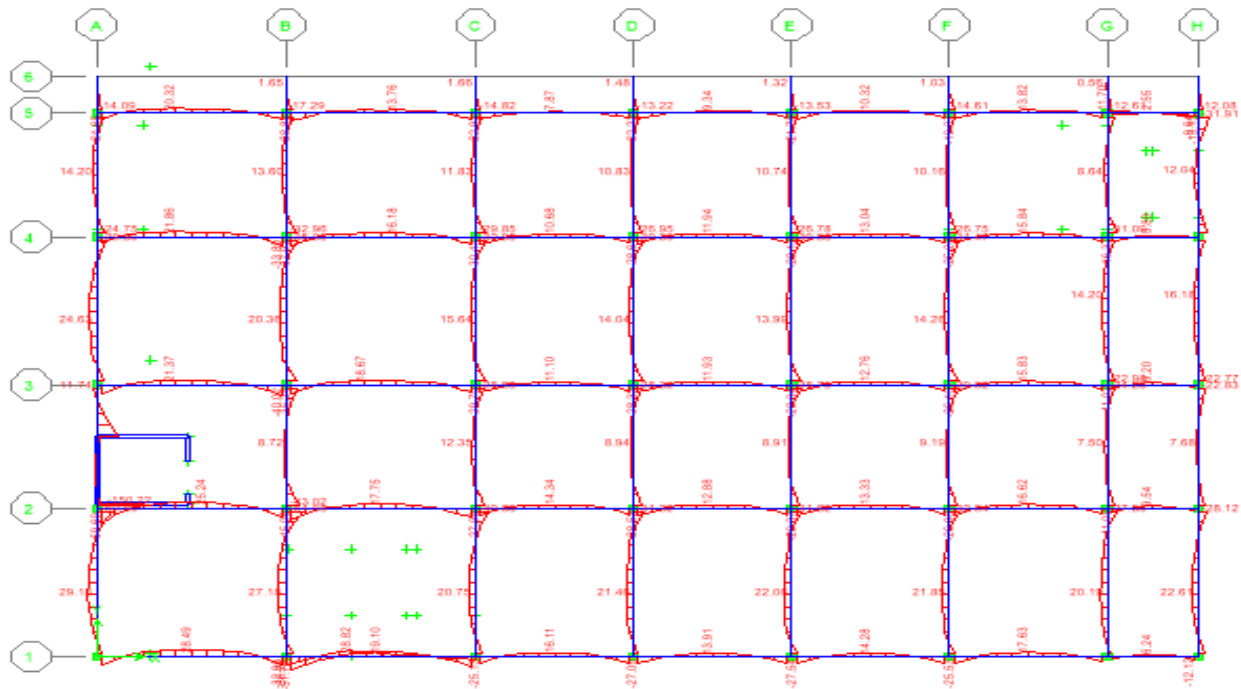


Figure 5. 4 5th floor beam moment diagram

5.2.1 FLEXURAL DESIGN OF THE BEAM

Let's take one axis 2 for the beam design

Step 1) Determine material design value/specification

For concrete C25 Grade of concrete for the beam

For steel

Let's Use 400MPa steel Grade which is specified in the code in the range of 400-600 according to ES-EN-1992-2015-Art-3.2.2.3

Step 2) Computing the effective depth with cc =30mm for beam

$$d = D - \left(\frac{\phi l}{2} + \phi s + cc \right)$$

$$d = 300 - \left(\frac{20}{2} + 8 + 30 \right) = 252\text{mm}$$

step3) Check the depth of flexure

$$M_{rd} = 0.8kx(1 - 0.4Kx)f_{cd} * b * d^2$$

$$= 0.8 * 0.448(1 - 0.4 * 0.448)11.33 * 400 * 252^2$$

$$= 84.66\text{KNm} > 25.34\text{KNm} \dots \text{ok for span}$$

Step 4) Designing the beam as double reinforcement beam using design charts

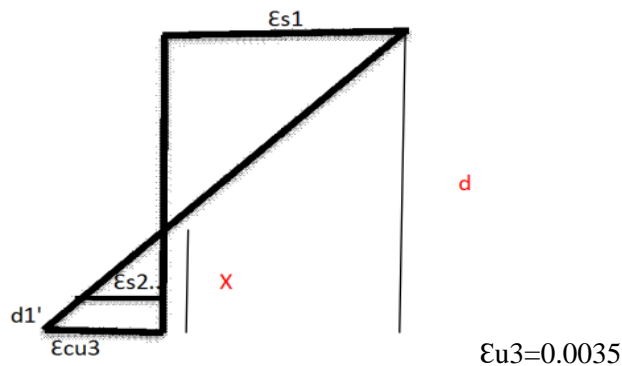
$$\mu_{sd,s} = \frac{M_{sd,s}}{f_{cd} * b * d^2} = \frac{25.34 * 10^6}{11.33 * 400 * 252^2} = 0.088$$

$\mu_{sd} = 0.088 < \mu_{us} = 0.295$ (for zero moment redistribution), so it should be design as double reinforcement

$$\text{Using } M_{sd,s} = \mu_{sd,s} * f_{cd} * b * d^2 = 0.295 * 11.33 * 400 * 252^2 = 84.9 \text{KNm}$$

$$K_z = 0.8 \rightarrow Z = K_z * d = 0.8 * 252 = 201.6 \text{mm}$$

$$K_x = 0.448 \rightarrow X = K_x * d = 0.448 * 252 = 112.896 \text{mm}$$



Check whether the top reinforcement is yield

For triangle similarity the strain in the top reinforcement become

$$\begin{aligned} \epsilon_{s1} &= \epsilon_{u3} \left(\frac{d}{X} - 1 \right) \\ &= 0.0035 (252 / 112.896 - 1) \\ &= 0.0043 \end{aligned}$$

$$\epsilon_{yd} = \frac{f_{yd}}{E} = \frac{347.83}{300000} = 0.00174$$

Hence $\epsilon_{s1} > \epsilon_{yd}$ ($0.0043 > 0.00174$) the top reinforcement is yielded

So A_{s1} = area of steel on the top which is equivalent to the moment resist by the concrete only

$$A_{s1} = \frac{M_{sd,s}}{f_{yd} * Z} = \frac{84.9 * 10^6}{347.83 * 201.6} = 1210.73 \text{mm}^2$$

Check the bottom reinforcement, using triangular similarity

$$\epsilon_{s2} = \frac{\epsilon_{u3}}{X * (X - d_2)} = \frac{0.0035}{112.896} (112.89 - 68) = 0.00139$$

$\epsilon_{s2} < \epsilon_{yd}$ ($0.00139 < 0.00174$) the bottom reinforcement is not yielded

So A_{s2} = Area of steel on the top which is equivalent to the bottom compression reinforcement

$$A_{s2} = \frac{M_{sd} - M_{sd,s}}{f_{yd} * (d - d_2)}$$

$$A_{s2} = \frac{(84.9 - 25.4) * 10^6}{347.83(252 - 68)} = 929.67 \text{ mm}^2$$

$$A_{st, \min} = \max \left\{ \begin{array}{l} 0.26 * \frac{f_{ctm} * b_t * d}{f_{yk}} = 0.26 * \frac{2.2 * 400 * 252}{400} = 144.14 \text{ mm}^2 \\ 0.0013 * b_t * d = 0.0013 * 400 * 252 = 131.04 \text{ mm}^2 \end{array} \right. \dots \text{ES EN 1992: 2015 Art. 9.2.1.1.}$$

(1).

$$A_{st, \max} = 0.04 A_c = 0.04 * 400 * 252 = 4032 \text{ mm}^2 \dots \dots \text{ES EN 1992-1-1:2015 section 9.2.1.1(3)}$$

$$A_{st, \min} = 144.14 \text{ mm}^2 < A_{st, \text{calc}} = 1210.73 \text{ mm}^2 < A_{st, \max} = 4032 \text{ mm}^2 \dots \dots \text{ok}$$

Now determine No. of bar using $\varnothing 20$ mm tension reinforcement.

So the total area of reinforcement at the bottom (mid span) be come

$$A_{ST} = A_{s1} + A_{s2} = 1210.73 + 929.67 = 2140.4 \text{ mm}^2$$

$$\# \text{ of bar} = \frac{A_{st, \text{cal}}}{a_s} = \frac{2140.4 \text{ mm}^2}{314 \text{ mm}^2} = 6.8 \sim 7$$

Provide 7 bars @ tension zone

Now determine No. of bar using $\varnothing 20$ mm compression reinforcement.

$$\text{At the Top } A_{s2} = 929.67 \text{ mm}^2$$

$$\# \text{ of bar} = \frac{A_{st, \text{cal}}}{a_s} = \frac{929.67 \text{ mm}^2}{314 \text{ mm}^2} = 2.9 \sim 3$$

Provide 3 bars @ compression zone

$$\text{For tension } A_{st, \text{pro}} = 7 * 314 = 2198 \text{ mm}^2 < A_{s, \max}$$

$$\rho_{\text{pro}} = \frac{2198 \text{ mm}^2}{400 * 252} = 0.0218 = 2.18\%$$

Design for support Moment

✓ check the depth of flexure

Assuming the neutral axis depth within the flange

$$\delta = K_1 + k_2 x_u / d \dots \dots K_1 + k_2 k_{x, \max} \dots \dots \dots f_{ck} < 50 \text{ MPa}$$

$$K1=0.44 K2$$

$$k2=1.25(0.6+0.0014/\epsilon_{cu2})$$

δ =is the ratio of redistributed moment to the elastic bending moment

$\delta=1$ for zero moment redistribution

$$\begin{aligned} M_{rd} &= 0.8k_x(1 - 0.4K_x)f_{cd} * b * d^2 \\ &= 0.8 * 0.448(1 - 0.4 * 0.448)11.33 * 400 * 252^2 \\ &=84.66\text{KNm}>59.69\text{KNm} \end{aligned}$$

✓ Check the neutral axis depth

$$\rho = \frac{f_{cd}}{f_{yd}} \left(1 - \sqrt{1 - \frac{2MED}{f_{cd} * b * d^2}}\right)$$

$$\rho = \frac{11.33}{347.83} \left(1 - \sqrt{1 - \frac{2 * 84.66}{11.33 * 400 * 252^2}}\right)$$

$$\rho = 0.0116$$

Computing the area of steel required

$$A_{s,cal} = \rho * b * d = 0.0116 * 400 * 252 = 1176.7 \text{mm}^2$$

$$\# \text{ of bar} = \frac{A_{s,cal}}{a_s} = \frac{1176.7 \text{mm}^2}{314 \text{mm}^2} = 3.7 \sim 4$$

Provide use \emptyset 4bars

5.2.2 CHECK THE DEPTH FOR DEFLECTION (SERVICEABILITY REQUIREMENT)

According to ES-EN-1992-2015 the limit state of the deformation may be checked by either by limiting the span/depth ratio Art 7.4.2

By comparing a calculated deflection

$$\frac{l}{d} = K \left[11 + 1.5 \sqrt{f_{ck}} \frac{\rho_o}{\rho} + 3.2 \sqrt{f_{ck}} \left(\frac{\rho_o}{\rho} - 1 \right)^{3/2} \right] * F1 * F2 * F3 \dots \dots \dots \text{if } \rho < \rho_o \text{ Art. 7.4.2 (7.16a)}$$

$$\frac{l}{d} = K \left[11 + 1.5 \sqrt{f_{ck}} \frac{\rho_o}{\rho - \rho'} + \frac{1}{12} \sqrt{f_{ck}} \frac{\rho'}{\rho_o} \right] * F1 * F2 * F3 \dots \dots \dots \text{if } \rho > \rho_o \text{ Art. 7.4.2 (7.16b)}$$

L/d =is the limit span /depth

K =is the factor to take into account the different structural system

ρ_0 is the reference ratio = $10^{-3} \sqrt{f_{ck}}$

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

ρ' is the required compression reinforcement ratio at the mid span to resist the moment due to design loads at support for cantilever f_{ck} in MPa

F1 = the factor to account for the steel Grade other than $f_{yk} = 400$ MPa

F2 = to account the beam type whether it is rectangular and T-section beams

F2 = 0.8, for flanged sections where the ratio of the flange breadth to the rib breadth exceeds 3, otherwise F2 = 1

F3 = $7/l_{eff}$ for beams and slabs, other than flat slabs, with span exceeding 7m which support parts liable to be damaged by excessive deflection for illustration let's see one for $L = 6$ m

$$\rho = 0.0116$$

$$\rho_0 = 10^{-3} \sqrt{20} = 0.00447$$

$$\rho' = 0.00422$$

$$F1 = \frac{500}{f_{yk} * A_{req} / A_{prov}} = \frac{500}{400 * 2140.4 / 2198} = 1.283$$

K for End support of a continuous span from ES-EN-1992-2015 Table 7.4N K = 1.3

Hence $\rho > \rho_0$

$$\frac{l}{d} = K \left[11 + 1.5 \sqrt{f_{ck}} \frac{\rho_0}{\rho - \rho'} + \frac{1}{12} \sqrt{f_{ck}} \frac{\rho'}{\rho_0} \right] * F1 * F2 * F3 \dots \dots \dots \text{if } \rho > \rho_0$$

$$\frac{6000}{d} = 1.3 \left[11 + 1.5 \sqrt{20} \frac{0.00447}{0.0116 - 0.00422} + \frac{1}{12} \sqrt{20} \frac{0.00422}{0.00447} \right] * 1.283 * 1 * 1$$

$$\frac{6000}{d} = 25.62 \rightarrow d = 234.19 \text{ mm} < 252 \text{ mm} \dots \dots \dots \text{ok safe.}$$

5.2.3 SHEAR DESIGN FOR THE BEAM

V_{sd} for the design of shear, the shear force which found d distance from the face of the columns is used, but for this design use the shear force at the support hence it is greater than that of the shear at d distance from the face of the columns. $V_{sd} = 72.29$ kN

The design value of the shear resistance is given ES-EN-1992-2015 section 6.2.2 as follows

$$V_{Rd,c} = [C_{Rd,c} k (100 * \rho_1 * f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w * d \geq [V_{min} + k_1 \sigma_{cp}] b_w * d$$

Where:- $f_{ck} = 20\text{Mpa}$

$$CRd, c = 0.18/\gamma_c = 0.18/1.5 = 0.12$$

$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{252}} = 1.89 < 2 \dots \dots \dots \text{ok (Take } k=1.89)$$

$$\rho_1 = \frac{A_{sl}}{b_w * d} \leq 0.02$$

$$\rho_1 = \frac{2140.4}{400 * 252} = 0.02 \leq 0.02 \dots \dots \dots \text{ok}$$

$$V_{min} = 0.035k^{3/2} * f_{ck}^{1/2}$$

$$V_{min} = 0.035 * 1.89^{3/2} * 20^{1/2} = 0.4$$

$$K_1 = 0.15$$

$$\sigma_{cp} = \frac{NED}{A_c} = 0$$

A_{st} - is the area of the tensile reinforcement

NED is the axial force in the cross-section due to loading

A_c is the area of concrete cross-section [mm²]

$$VRd, c = \left[0.12 * 1.89(100 * 0.02 * 20)^{\frac{1}{3}} + 0 \right] * 400 * 252 = 78.185\text{KN}$$

$$VRd, c(\text{min}) = [V_{min} + k_1 \sigma_{cp}] b_w * d$$

$$= (0.4 + 0) * 400 * 252 = 40.32\text{KN}$$

$$V_{sd} = 72.29\text{KN} > VRd, c = 79.199\text{KN}$$

The concrete section can resist the coming shear alone, thus the section does not require design for shear reinforcement.

Spacing of stirrup is the spacing of the shear reinforcement measured along the longitudinal axis of the member.

$$S_1, \text{max} = 0.75d(1 + \cot\alpha), \text{ considering } \alpha = 90^\circ$$

$$S_1, \text{max} = 0.75 * 252 = 189\text{mm}, B_w \text{ is the breadth of the web of the member}$$

$$\rho_w = 0.08 * \frac{\sqrt{20}}{f_{yk}} = 0.08 * \frac{\sqrt{20}}{400} = 0.00089$$

$$\text{Thus } S_{\text{max}} = \frac{A_{sw}}{\rho_w * b_w * \sin\alpha} = 2 * \frac{50.24}{0.00089 * 400 * \sin 90} = 280.8\text{mm}$$

$$S_{ca} = 189\text{mm} < S_{\text{max}} = 280\text{mm}$$

Use 2-leg stirrup Ø8-c/c-180mm

5.2.4 DESIGN FOR OTHER SECTION OF THE BEAM

Table 5. 1 beam reinforcement

story	beam on axis A to H					K _{usds}	K _z	As,min	As,max	As,cal.	As,prov.	Nreq.	Npro.
	beam ID	b(mm)	d(mm)	fcd	moment(K)								
5th	BASpan	400	252	11.33	29.18	0.10139	0.951	144.14	4032	350.06	350.056	1.11483	2Ø20mm
	supp	400	252	11.33	160.22	0.55671	0.946	144.14	4032	1932.2	1932.2	6.15359	7Ø20mm
	BB,span	400	252	11.33	20.76	0.07213	0.94	144.14	4032	251.96	251.96	0.80242	2Ø16mm
	supp	400	252	11.33	40.83	0.14187	0.961	144.14	4032	484.72	484.72	1.54369	2Ø20mm
	BC,span	400	252	11.33	20.76	0.07213	0.974	144.14	4032	243.16	243.16	0.77441	2Ø16mm
	supp	400	252	11.33	40.83	0.14187	0.949	144.14	4032	490.85	490.85	1.56321	2Ø20mm
	BD,span	400	252	11.33	21.46	0.07457	0.968	144.14	4032	252.92	252.92	0.80548	2Ø16mm
	supp	400	252	11.33	31.32	0.10883	0.9776	144.14	4032	365.5	365.5	1.16403	2Ø20mm
	BE,span	400	252	11.33	22.06	0.07665	0.978	144.14	4032	257.34	257.34	0.81954	2Ø16mm
	supp	400	252	11.33	31.82	0.11056	0.9589	144.14	4032	378.58	378.58	1.20567	2Ø20mm
	BF,span	400	252	11.33	21.86	0.07596	0.976	144.14	4032	255.52	255.52	0.81377	2Ø16mm
	supp	400	252	11.33	32.04	0.11133	0.9797	144.14	4032	373.11	373.11	1.18823	2Ø20mm
	BGspan	400	252	11.33	20.19	0.07015	0.9785	144.14	4032	235.4	235.4	0.74968	2Ø16mm
	supp	400	252	11.33	27.36	0.09507	0.9689	144.14	4032	322.16	322.16	1.02598	2Ø20mm
	BH,Span	400	252	11.33	22.61	0.07856	0.9784	144.14	4032	263.64	263.64	0.83963	2Ø16mm
	supp	400	252	11.33	28.12	0.09771	0.9799	144.14	4032	327.39	327.39	1.04264	2Ø20mm
4th	BASpan	400	252	11.33	26.96	0.09368	0.951	144.14	4032	323.42	323.42	1.03001	2Ø20mm
	supp	400	252	11.33	49.27	0.1712	0.946	144.14	4032	594.19	594.19	1.89232	2Ø20mm
	BB,span	400	252	11.33	24.72	0.08589	0.94	144.14	4032	300.02	300.02	0.95548	2Ø16mm
	supp	400	252	11.33	49.27	0.1712	0.961	144.14	4032	584.91	584.91	1.86278	2Ø20mm
	BC,span	400	252	11.33	21.54	0.07484	0.974	144.14	4032	252.3	252.3	0.80351	2Ø16mm
	supp	400	252	11.33	46.31	0.16091	0.949	144.14	4032	556.73	556.73	1.77301	2Ø20mm
	BD,span	400	252	11.33	19.85	0.06897	0.968	144.14	4032	233.95	233.95	0.74505	2Ø16mm
	supp	400	252	11.33	31.4	0.1091	0.9776	144.14	4032	366.44	366.44	1.167	2Ø20mm
	BE,span	400	252	11.33	20.29	0.0705	0.978	144.14	4032	236.69	236.69	0.75378	2Ø16mm
	supp	400	252	11.33	32.29	0.1122	0.9589	144.14	4032	384.17	384.17	1.22348	2Ø20mm
	BF,span	400	252	11.33	20.05	0.06967	0.976	144.14	4032	234.37	234.37	0.74639	2Ø16mm
	supp	400	252	11.33	32.3	0.11223	0.9797	144.14	4032	376.13	376.13	1.19788	2Ø20mm
	BGspan	400	252	11.33	18.22	0.06331	0.9785	144.14	4032	212.43	212.43	0.67654	2Ø16mm
	supp	400	252	11.33	37.74	0.13113	0.9689	144.14	4032	444.38	444.38	1.41523	2Ø20mm
	BH,Span	400	252	11.33	20.07	0.06974	0.9784	144.14	4032	234.03	234.03	0.7453	2Ø16mm
	supp	400	252	11.33	36.05	0.12526	0.9799	144.14	4032	419.72	419.72	1.33668	2Ø20mm

3rd	BASpan	300	252	11.33	57.44	0.26611	0.951	144.14	4032	689.07	689.07	2.19451	3Ø20mm
	supp	300	252	11.33	144.57	0.66977	0.568	144.14	4032	2903.8	2903.8	9.24768	10Ø20mm
	BB,span	300	252	11.33	52.92	0.24517	0.94	144.14	4032	642.28	642.28	2.04548	3Ø20mm
	supp	300	252	11.33	85.72	0.39713	0.876	144.14	4032	1116.4	1116.4	3.55534	4Ø20mm
	BC,span	300	252	11.33	59.45	0.27542	0.974	144.14	4032	696.35	696.35	2.21766	3Ø20mm
	supp	300	252	11.33	104.47	0.48399	0.879	144.14	4032	1355.9	1355.9	4.31823	5Ø20mm
	BD,span	300	252	11.33	54.82	0.25397	0.968	144.14	4032	646.09	646.09	2.05763	3Ø20mm
	supp	300	252	11.33	82.03	0.38003	0.896	144.14	4032	1044.5	1044.5	3.32635	4Ø20mm
	BE,span	300	252	11.33	56.54	0.26194	0.978	144.14	4032	659.55	659.55	2.10049	3Ø20mm
	supp	300	252	11.33	83.47	0.3867	0.896	144.14	4032	1062.8	1062.8	3.38474	4Ø20mm
	BF,span	300	252	11.33	55.13	0.25541	0.976	144.14	4032	644.42	644.42	2.0523	3Ø20mm
	supp	300	252	11.33	84.96	0.39361	0.895	144.14	4032	1083	1083	3.44901	4Ø20mm
	BGspan	300	252	11.33	48.03	0.22252	0.9785	144.14	4032	560	560	1.78342	2Ø20mm
	supp	300	252	11.33	73.81	0.34195	0.865	144.14	4032	973.49	973.49	3.10029	4Ø20mm
	BH,Span	300	252	11.33	41.47	0.19212	0.9784	144.14	4032	483.56	483.56	1.54	2Ø20mm
supp	300	252	11.33	60.04	0.27816	0.9799	144.14	4032	699.02	699.02	2.22619	3Ø20mm	
2nd	BASpan	300	252	11.33	48.95	0.22678	0.951	144.14	4032	587.23	587.23	1.87014	2Ø20mm
	supp	300	252	11.33	123.6	0.57262	0.656	144.14	4032	2149.5	2149.5	6.84569	7Ø20mm
	BB,span	300	252	11.33	45.95	0.21288	0.94	144.14	4032	557.69	557.69	1.77607	2Ø20mm
	supp	300	252	11.33	77.28	0.35803	0.776	144.14	4032	1136.2	1136.2	3.61833	4Ø20mm
	BC,span	300	252	11.33	53.53	0.248	0.974	144.14	4032	627	627	1.99683	2Ø20mm
	supp	300	252	11.33	98.43	0.45601	0.7658	144.14	4032	1466.4	1466.4	4.66998	5Ø20mm
	BD,span	300	252	11.33	58.26	0.26991	0.968	144.14	4032	686.64	686.64	2.18674	3Ø20mm
	supp	300	252	11.33	89.84	0.41621	0.789	144.14	4032	1299	1299	4.1371	5Ø20mm
	BE,span	300	252	11.33	58.3	0.27009	0.978	144.14	4032	680.08	680.08	2.16587	3Ø20mm
	supp	300	252	11.33	90.27	0.41821	0.776	144.14	4032	1327.1	1327.1	4.22654	5Ø20mm
	BF,span	300	252	11.33	58.29	0.27005	0.976	144.14	4032	681.36	681.36	2.16994	3Ø20mm
	supp	300	252	11.33	91.2	0.42252	0.768	144.14	4032	1354.8	1354.8	4.31456	5Ø20mm
	BGspan	300	252	11.33	50.99	0.23623	0.9785	144.14	4032	594.51	594.51	1.89333	2Ø20mm
	supp	300	252	11.33	78.73	0.36474	0.896	144.14	4032	1002.5	1002.5	3.19253	4Ø20mm
	BH,Span	300	252	11.33	43.73	0.20259	0.9784	144.14	4032	509.91	509.91	1.62392	2Ø20mm
supp	300	252	11.33	58.07	0.26903	0.9799	144.14	4032	676.09	676.09	2.15314	3Ø20mm	
1st	BASpan	300	252	11.33	52.5	0.24322	0.951	144.14	4032	629.81	629.81	2.00577	3Ø20mm
	supp	300	252	11.33	107.42	0.49766	0.786	144.14	4032	1559.2	1559.2	4.96553	5Ø20mm
	BB,span	300	252	11.33	49.25	0.22817	0.94	144.14	4032	597.74	597.74	1.90362	2Ø20mm
	supp	300	252	11.33	79.7	0.36924	0.961	144.14	4032	946.17	946.17	3.01327	4Ø20mm
	BC,span	300	252	11.33	56.67	0.26254	0.974	144.14	4032	663.78	663.78	2.11396	3Ø20mm
	supp	300	252	11.33	99.67	0.46176	0.7569	144.14	4032	1502.3	1502.3	4.78441	5Ø20mm
	BD,span	300	252	11.33	61.51	0.28497	0.968	144.14	4032	724.94	724.94	2.30873	3Ø20mm
	supp	300	252	11.33	96.9	0.44892	0.769	144.14	4032	1437.6	1437.6	4.57826	5Ø20mm
	BE,span	300	252	11.33	61.17	0.28339	0.978	144.14	4032	713.56	713.56	2.27249	3Ø20mm
	supp	300	252	11.33	96.76	0.44827	0.785	144.14	4032	1406.2	1406.2	4.47846	5Ø20mm
	BF,span	300	252	11.33	61.23	0.28367	0.976	144.14	4032	715.73	715.73	2.27938	3Ø20mm
	supp	300	252	11.33	97.39	0.45119	0.775	144.14	4032	1433.7	1433.7	4.56578	5Ø20mm
	BGspan	300	252	11.33	53.11	0.24605	0.9785	144.14	4032	619.22	619.22	1.97205	2Ø20mm
	supp	300	252	11.33	84.19	0.39004	0.9689	144.14	4032	991.32	991.32	3.15707	4Ø20mm
	BH,Span	300	252	11.33	43.44	0.20125	0.9784	144.14	4032	506.53	506.53	1.61316	2Ø20mm
supp	300	252	11.33	60.57	0.28061	0.9799	144.14	4032	705.19	705.19	2.24584	3Ø20mm	

Graound	BASpan	300	252	11.33	52.85	0.24485	0.951	144.14	4032	634.01	634.01	2.01914	3Ø20mm
	supp	300	252	11.33	95.84	0.44401	0.946	144.14	4032	1155.8	1155.8	3.68094	4Ø20mm
	BB,span	300	252	11.33	49.23	0.22807	0.94	144.14	4032	597.5	597.5	1.90285	2Ø20mm
	supp	300	252	11.33	78.9	0.36553	0.961	144.14	4032	936.67	936.67	2.98302	3Ø20mm
	BC,span	300	252	11.33	59.43	0.27533	0.974	144.14	4032	696.11	696.11	2.21692	3Ø20mm
	supp	300	252	11.33	112.09	0.5193	0.949	144.14	4032	1347.5	1347.5	4.29144	5Ø20mm
	BD,span	300	252	11.33	60.16	0.27871	0.968	144.14	4032	709.03	709.03	2.25806	3Ø20mm
	supp	300	252	11.33	95.35	0.44174	0.9776	144.14	4032	1112.7	1112.7	3.54374	4Ø20mm
	BE,span	300	252	11.33	60.69	0.28117	0.978	144.14	4032	707.96	707.96	2.25466	3Ø20mm
	supp	300	252	11.33	96.67	0.44786	0.7445	144.14	4032	1481.4	1481.4	4.71769	5Ø20mm
	BF,span	300	252	11.33	60.8	0.28168	0.976	144.14	4032	710.7	710.7	2.26338	3Ø20mm
	supp	300	252	11.33	97.18	0.45022	0.765	144.14	4032	1449.3	1449.3	4.61549	5Ø20mm
	BGspan	300	252	11.33	52.83	0.24475	0.9785	144.14	4032	615.96	615.96	1.96165	2Ø20mm
	supp	300	252	11.33	83.59	0.38726	0.856	144.14	4032	1114.1	1114.1	3.548	4Ø20mm
	BH,Span	300	252	11.33	42.84	0.19847	0.9784	144.14	4032	499.53	499.53	1.59087	2Ø20mm
supp	300	252	11.33	59.3	0.27473	0.9799	144.14	4032	690.41	690.14	2.19875	3Ø20mm	
greid beam	BASpan	200	252	11.33	23.24	0.1615	0.951	144.14	4032	278.8	278.8	0.88789	2Ø16mm
	supp	200	252	11.33	36.71	0.25511	0.946	144.14	4032	442.72	442.27	1.40993	2Ø20mm
	BB,span	200	252	11.33	17.83	0.12391	0.94	144.14	4032	216.4	216.4	0.68917	2Ø16mm
	supp	200	252	11.33	19.29	0.13405	0.961	144.14	4032	229	144.14	0.72931	2Ø16mm
	BC,span	200	252	11.33	39.49	0.27443	0.974	144.14	4032	462.55	462.55	1.4731	2Ø20mm
	supp	200	252	11.33	65.48	0.45504	0.765	144.14	4032	976.52	976.52	3.10993	4Ø20mm
	BD,span	200	252	11.33	17.66	0.12272	0.968	144.14	4032	208.14	208.14	0.66285	2Ø16mm
	supp	200	252	11.33	18.06	0.1255	0.9776	144.14	4032	210.76	210.76	0.67121	2Ø16mm
	BE,span	200	252	11.33	17.64	0.12259	0.978	144.14	4032	205.77	205.77	0.65533	2Ø16mm
	supp	200	252	11.33	18.14	0.12606	0.9589	144.14	4032	215.82	215.82	0.68733	2Ø16mm
	BF,span	200	252	11.33	17.66	0.12272	0.976	144.14	4032	206.43	206.43	0.65742	2Ø16mm
	supp	200	252	11.33	27.36	0.19013	0.9797	144.14	4032	318.61	318.61	1.01467	2Ø16mm
	BGspan	200	252	11.33	17.85	0.12404	0.9785	144.14	4032	208.12	208.12	0.6628	2Ø16mm
	supp	200	252	11.33	18.78	0.13051	0.9689	144.14	4032	221.13	221.13	0.70424	2Ø16mm
	BH,Span	200	252	11.33	23.04	0.16011	0.9784	144.14	4032	268.66	268.66	0.8556	2Ø16mm
supp	200	252	11.33	35	0.24322	0.9799	144.14	4032	407.49	407.49	1.29774	2Ø20mm	

beam on axis A to H												
beam ID	b(mm)	d(mm)	fed	moment(KNm)	usds	Kz	As,min	As,max	As,cal.	As,prov.	Nreq.	Npro.
B1,Span	400	252	11.33	20.32	0.1	0.951	144.14	4032	243.767	243.76	1.21301	2Ø20mm
supp	400	252	11.33	24.66	0.1	0.946	144.14	4032	297.395	297.395	1.47987	2Ø20mm
B2,span	400	252	11.33	28.88	0.1	0.94	144.14	4032	350.511	350.51	1.74418	2Ø20mm
supp	400	252	11.33	13.76	0	0.961	144.14	4032	163.353	163.35	0.81286	2Ø16mm
B3,span	400	252	11.33	7.8	0	0.974	144.14	4032	91.3625	91.36	0.45463	2Ø16mm
supp	400	252	11.33	22.22	0.1	0.949	144.14	4032	267.122	267.122	1.32923	2Ø20mm
B4,span	400	252	11.33	9.34	0	0.968	144.14	4032	110.079	144.14	0.54777	2Ø16mm
supp	400	252	11.33	21.78	0.1	0.978	144.14	4032	254.173	254.17	1.26479	2Ø20mm
B5,span	400	252	11.33	10.32	0	0.978	144.14	4032	120.385	144.14	0.59905	2Ø16mm
supp	400	252	11.33	19.37	0.1	0.959	144.14	4032	230.456	230.45	1.14678	2Ø20mm
B6,span	400	252	11.33	13.82	0	0.976	144.14	4032	161.544	161.54	0.80386	2Ø16mm
supp	400	252	11.33	17.78	0.1	0.98	144.14	4032	207.048	207.048	1.03029	2Ø20mm
B7span	400	252	11.33	2.55	0	0.979	144.14	4032	29.7312	144.14	0.14795	2Ø16mm

The reset are in the appendix part

5.3 ANALYSIS AND DESIGN OF COLUMN

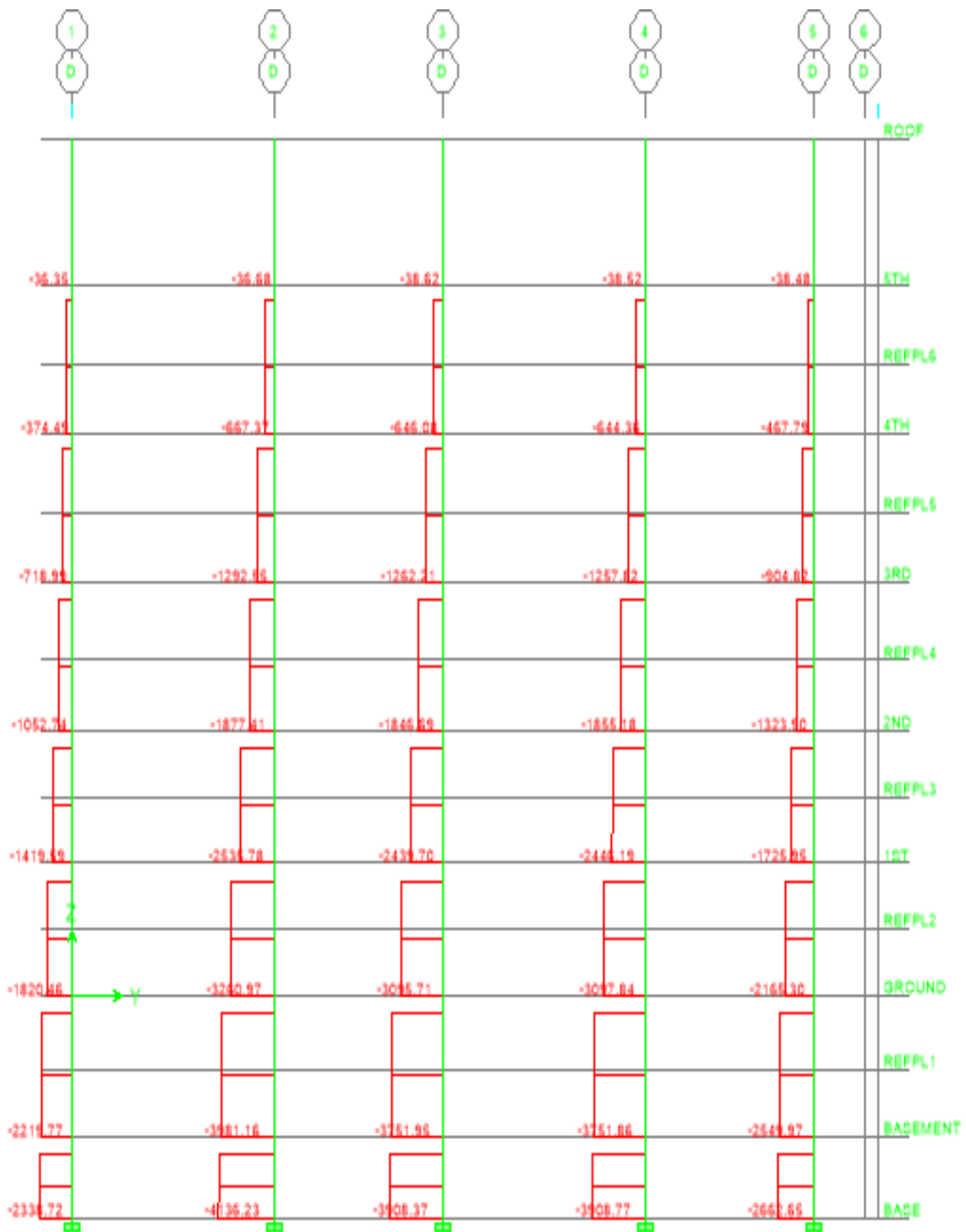


Figure 5. 5 axial force result

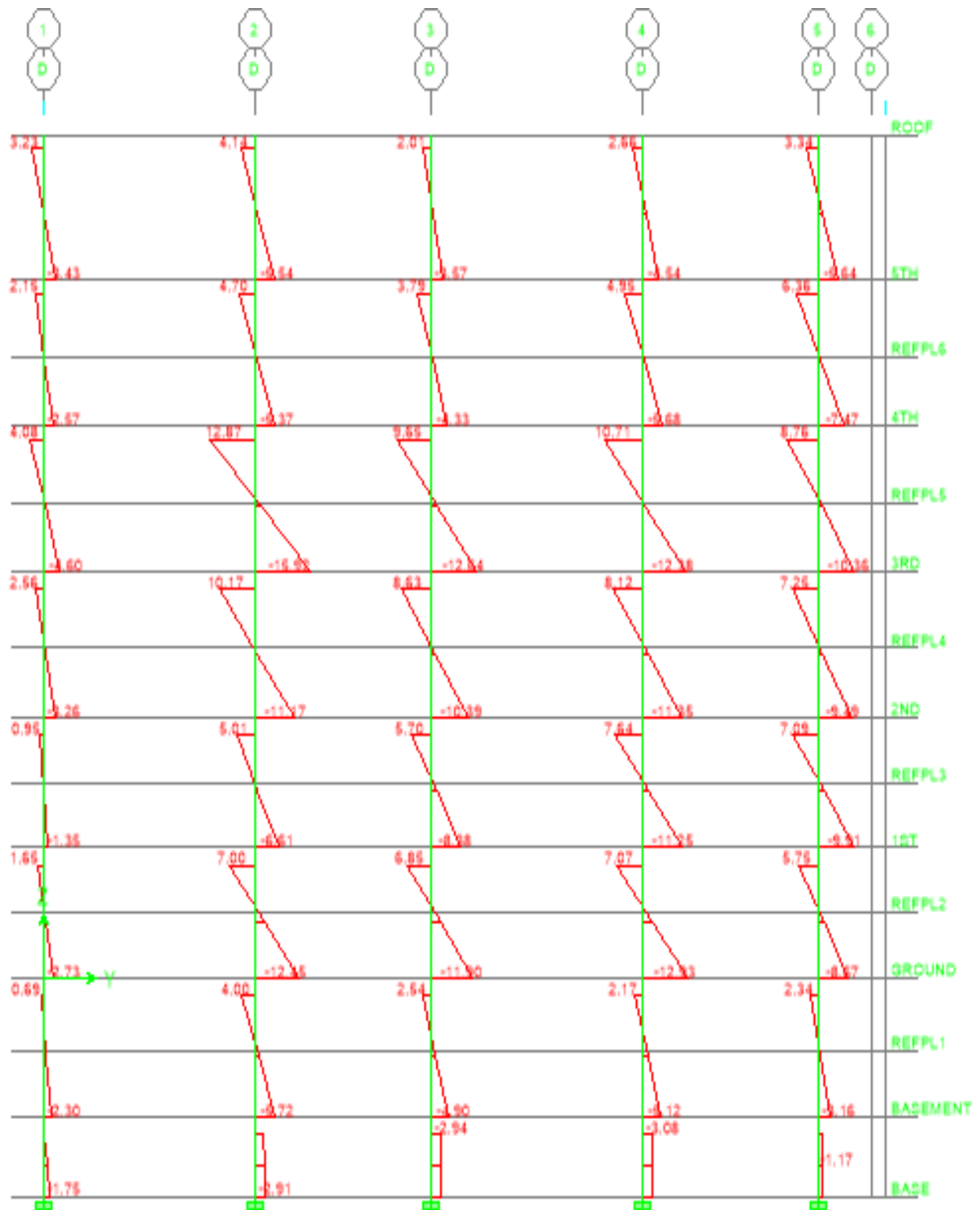


Figure 5. 6 moment result

Axis 3 and C

Analysis and design of column according to ES EN 1992 2015.

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 they may also have to resist bending forces due to some eccentricity or due to the continuity of the structure.

The strength of column depends on many factors including the following;

- ✓ Strength of the material
- ✓ Shape & size of the cross section
- ✓ Length
- ✓ The degree of positional & directional restraint at its end

Short/Slender Columns

Columns are broadly categorized into two as short and long (slender) columns based on the slenderness ratio of the column.

Short columns: are columns whose strength is governed by the strength of the materials and the geometry of the cross section. In short columns, second-order is very small and so negligible. In these cases, it is not necessary to consider slenderness effects and compression members can be designed based on forces determined from first-order analysis.

Long column (slender): are column whose cross-sectional dimension is small compared to its length. In reinforced concrete buildings concrete beams ,floors and columns are cast monolithically causing some moments in the columns due to end restraint ,Moreover, perfect vertical alignment of columns in a multistory building it not possible causing loads to be eccentric relative to the center of the columns. The eccentric loads will be cause moments in columns therefore a column subjected to pure axial loads does not exist in concrete buildings however it can be assumed that axially loaded columns are those with relatively small eccentricity.

For different combination of the load the governing load case pattern is not easily identified so the design of the column is done by all possible combination of the load cases in this case in this sample calculation We take the column between story1-story2 on axis 3D whose cross-sectional dimension 500mmx500mm .

Table 5. 2 Internal forces on the column

Story	Column	P	Myy	Mxx
GROUND	C2	-2894.55	-14.54	10.01
GROUND	C3	-2784.9	-16.473	-8.344
GROUND	C4	-2549.97	14.216	-3.162
GROUND	C4	-2534.85	-15.564	2.341
GROUND	C5	-2578.69	14.12	-2.842
GROUND	C5	-2571.13	-0.598	-0.668
GROUND	C5	-2563.57	-15.107	1.774
GROUND	C7	-2460.11	12.354	0.145
GROUND	C7	-2452.55	0.909	-2.465
GROUND	C7	-2444.99	-10.92	-4.098
GROUND	C10	-2146.59	8.54	-32.723
GROUND	C10	-2139.03	-1.333	7.22
GROUND	C11	-4330.26	10.514	-9.848
GROUND	C11	-4318.45	-3.773	1.817
GROUND	C11	-4306.63	-17.965	13.436
GROUND	C12	-4125.08	9.043	5.624

Column Design for Typical section

Step 1:- Determine material design value /specification/.

$f_{cd} = 11.33 \text{ Mpa}$, for steel $f_{yd} = 347.83 \text{ Mpa}$

Step2:-Concrete cover determination

The nominal concrete cover is the distance between the surfaces of reinforcement closest to the nearest concrete surface.

$$C_{nom} = C_{min} + \Delta C_{dev} \dots \dots \dots \text{ES EN 1992:2015 Art 4.4.12(1)}$$

Where; C_{min} –minimum cover and

ΔC_{dev} is allowance in design for deviation

Minimum cover, shall provide in order to ensure

- ✓ Safe transmission of bond force
- ✓ Corrosion resistance/ durability
- ✓ Fire resistance

$$C_{min} = \max \left\{ \begin{array}{l} C_{min, b} \\ C_{min, dur} + \Delta C_{dur, \gamma} - \Delta C_{dur, st} - \Delta C_{dur, add} \dots \dots \dots \text{Eqn(4.2)} \\ 10\text{mm} \end{array} \right.$$

But the recommended value of $\Delta C_{dur} \gamma = \Delta C_{dur, st} = \Delta C_{dur, add} = 0$

According to Art 4.4.1.2 (6, 7 and 8)

- ✓ In order to transmit bond force safely and to ensure adequate compaction of the concrete, the minimum cover should not be less than $C_{min, b}$
- ✓ According to table 4.2, for separated arrangement of bars, $C_{min, b} = \text{diameter of bar}$, Assume $\varnothing 20$ longitudinal bars and $C_{min, b} = 20\text{mm}$
- ✓ On class designation, Xc1, concrete inside building with low air humidity concrete permanently submerged in water
- ✓ On exposure class to XC1 is reduced by 1 according to table for exposure class that determines $C_{min, dur}$.
- ✓ The recommended structural class (design working life of 50yr) is S4 for indicative concrete strength given in Annex E and the recommended modifications to the structural class is given table 4.3N but based on above table the exposure class is reduced by 1 and the structural class would be S3.
- ✓ For structural class S3, and exposure class Xc1, $C_{min, dur}(\text{mm}) = 10$

Then, $d_{ur}(\text{mm}) = 10$

$$C_{min} = \max \begin{cases} C_{min, b} = 20\text{mm} \\ C_{min, dur} = 10\text{mm} \\ 10\text{mm} \end{cases}$$

Therefore; $C_{min} = 20\text{mm}$

ΔC_{dev} (Allowance in design for variation)

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

Then; $C_{nom} = C_{min} + \Delta C_{dev} = 20\text{mm} + 10\text{mm} = 30\text{mm}$

Interior column design, Consider story 1 column 3D, Column size 500x500

Analyze one column by using 9 combination and to get the maximum value by using Combo 1 and

Table 5. 3 the geometry of the columns and the beam

X-sectional area data			
element	depth(mm)	width(mm)	length(mm)
column	500	500	3000
top beam			
B1(X-X)	400	300	5000
B2(X-X)	400	300	5000
B3(Y-Y)	400	300	6000
B4(Y-Y)	400	300	5000
bott.beam			
B1(X-X)	400	300	5000
B2(X-X)	400	300	5000
B3(Y-Y)	400	300	6000
B4(Y-Y)	400	300	5000

The moment and axial force from the modeling and analysis are the following

design action		reinforcement data		concrete data	
Ned(KN)		fyk(Mpa)	400	fck(Mpa)	25
M(X-X)KN-m	5.7	γms	1.15	γmc	1.5
M(X-X)KN-m	-8.38	Es(Gpa)	200	Ecm(Gpa)	31
M(Y-Y)KN-m	6.85	main bar(mm)	20	Cover(mm)	30
M(Y-Y)KN-m	-11.9	TieØ(mm)	8	fcd(Mpa)	11.33
		fyd(Mpa)	347.3		

Design constants

5.3.1 DETERMINATION OF EFFECTIVE DEPTH

$L_o = f * L$, Where:

- ✓ L_o is the effective length of a column
- ✓ f is a factor accounts for supporting conditions (simply supported or fixed support

$$f = 0.5 \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right) + \left(1 + \frac{k_2}{0.45 + k_2}\right)}$$

- ✓ K_1 & K_2 are the relative flexibilities of rotational restraints at both ends 1 and 2

$$k_i = \frac{\text{column stiffness}}{\text{beam stiffness}}$$

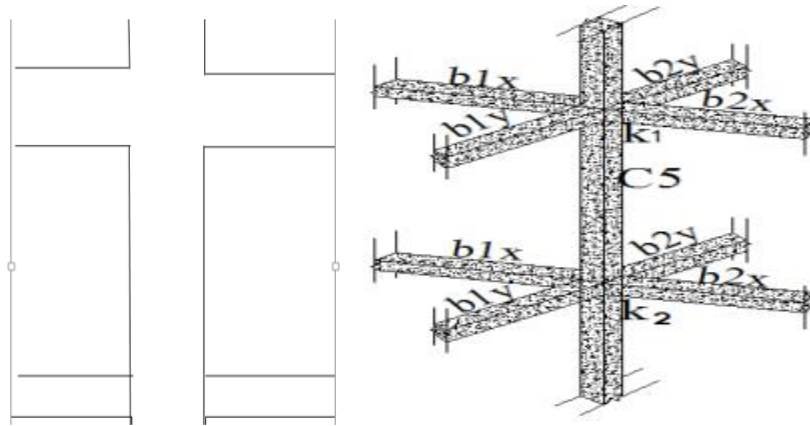


Figure 5. 7 2D and 3D column View

$$\text{Axis x-x, } I_c = \frac{bh^3}{12} = 500 * \frac{500^3}{12} = 5.208 * 10^9 \text{mm}^4 \quad I_b = \frac{bh^3}{12} = 300 * \frac{400^3}{12} = 1.6 * 10^9 \text{mm}^4$$

$$\text{Axis y-y, } I_c = \frac{bh^3}{12} = 500 * \frac{500^3}{12} = 5.208 * 10^9 \text{mm}^4 \quad I_b = \frac{bh^3}{12} = 400 * \frac{300^3}{12} = 9 * 10^8 \text{mm}^4$$

$$k_1(k_2)x = \frac{\frac{5.208 * 10^9 * 31}{300}}{2 * 31 \sum \left(\frac{1.6 * 10^9}{5000} + \frac{1.6 * 10^9}{5000} \right)} = 1.356, \quad k_2(k_1)y = \frac{\frac{5.208 * 10^9 * 31}{300}}{2 * 31 \sum \left(\frac{9 * 10^8}{6000} + \frac{9 * 10^8}{5000} \right)} = 0.263$$

$$f = 0.5 \sqrt{\left(1 + \frac{1.356}{0.45 + 1.356} \right) + \left(1 + \frac{0.263}{0.45 + 0.263} \right)} = 0.883$$

There for calculate the effective length by substituting the values

$$L_{ox} = f_x * L = 0.883 * 3000 \text{mm} = 2649.38 \text{mm}, \quad L_{oy} = f_y * L = 0.883 * 3000 \text{mm} = 2649.38 \text{mm}$$

Minimum reinforcement provided

According to ES EN 1992, article 9.5.2.2

The minimum reinforcement provided should be

$$A_{st, \min} = \max \left\{ \begin{array}{l} \frac{0.1 * N_{Ed}}{f_{yd}} = \frac{0.1 * 2439.70}{347.83} = 701.4 \text{mm}^2 \\ 0.002 * A_c, A_c = 500 * 500, = 0.002 * 500 * 500 = 500 \text{mm}^2 \end{array} \right.$$

Maximum reinforcement = 0.04 A_c = 0.04 * 500 * 500 = 10000 mm² ES EN 1992, article 9.5.2

$$d' = 30 + 10 + 8 = 48 \text{mm}$$

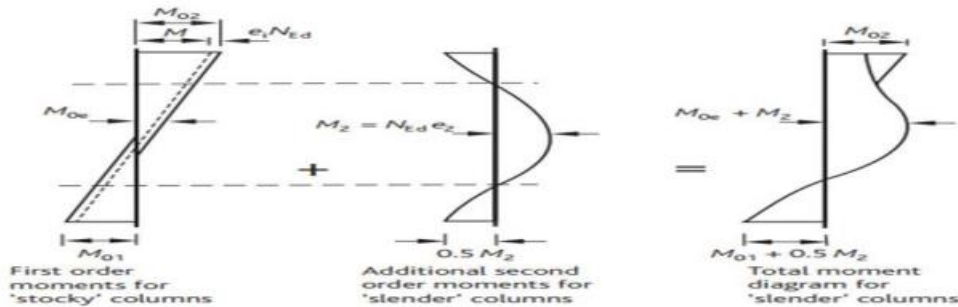
$$d = 500 - 48 = 452 \text{mm}$$

5.3.2 DESIGN EFFECTS

The effect of imperfection may be taken as according to ES EN 1992 article 5.2.

$$e_i = \max \left\{ \begin{array}{l} \frac{l_0}{400} = \frac{2649.38}{400} = 6.62 \\ \frac{h}{30} = \frac{400}{30} = 13.33 \\ 20\text{mm} \end{array} \right. , \text{ take } 20\text{mm}$$

Action effects calculated without consideration of the effect of structural deformation but including geometric imperfection.



First order moment along x-x

$$M_{o1} = \min \left\{ \begin{array}{l} M_{x - x_{top}} \\ M_{x - x_{bottom}} \end{array} \right. + N_{ed} * e_{ix}$$

$$M_{o1} = \min \left\{ \begin{array}{l} 5.7 + 2439.7 * 0.02 = 48.794\text{KNm} \\ 8.38 + 2439.7 * 0.02 = 57.174\text{KNm} \end{array} \right. , \text{ take } 48.79\text{KNm}$$

$$M_{o2} = \max \left\{ \begin{array}{l} 5.7 + 2439.7 * 0.02 = 48.794\text{KNm} \\ 8.38 + 2439.7 * 0.02 = 57.174\text{KNm} \end{array} \right. , \text{ take } 57.17\text{KNm}$$

$$M_{oe} = \max \left\{ \begin{array}{l} 0.4M_{o1} + M_{o2} \\ 0.4M_{o2} \end{array} \right. = 0.4 * 48.79 + 0.6 * 57.17 = 53.82\text{KNm}$$

First order long y-y

$$M_{o1} = \min \left\{ \begin{array}{l} M_{y - y_{top}} \\ M_{y - y_{bottom}} \end{array} \right. + N_{ed} * e_{iy}$$

$$M_{o1} = \min \left\{ \begin{array}{l} 6.85 + 2439.7 * 0.02 = 55.644\text{KNm} \\ 11.9 + 2439.7 * 0.02 = 60.694\text{KNm} \end{array} \right. , \text{ take } 55.64\text{KNm}$$

$$M_{o2} = \min \left\{ \begin{array}{l} 6.85 + 2439.7 * 0.02 = 55.644\text{KNm} \\ 11.9 + 2439.7 * 0.02 = 60.694\text{KNm} \end{array} \right. , \text{ take } 60.694\text{KNm}$$

$$M_{oe} = \max \left\{ \begin{array}{l} 0.4M_{o1} + M_{o2} \\ 0.4M_{o2} \end{array} \right. = 0.4 * 55.64 + 0.6 * 60.694 = 58.67\text{KNm}$$

5.3.3 CHECKING FOR SLENDERNESS

The slenderness effect is neglected if the slenderness λ ($\lambda = l_e/i$)....ES EN 1992 article 5.8.3.1

$$\lambda_{lim} = \frac{20ABC}{\sqrt{n}}$$

Where:- $A = \frac{1}{(1+0.2\phi_{eff})}$, $B = \sqrt{1 + 2w}$, $C = 1.7 - rm$ and $n = \frac{NEd}{Ac*fc_d}$

$rm = \frac{Mo_1}{Mo_2} = \frac{55.64}{60.69} = 0.916$ and $rm = \frac{Mo_1}{Mo_2} = \frac{48.79}{57.17} = 0.85 \rightarrow C = 1.7 - 0.85 = 0.784$, $A=0.7$, since ϕ_{eff} is not known

$B=1.1$, since w is not known, $Asfy_d/(Acfc_d)$ and $n = \frac{2439.7}{500*500*11.33} = 0.861$

$\lambda_{limx} = \frac{20*0.7*1.1*0.85}{\sqrt{0.861}} = 14.1$, $\lambda_{limy} = \frac{20*0.7*1.1*0.916}{\sqrt{0.861}} = 15.215$

$\lambda = \frac{l_o}{i}$, $i = \sqrt{\frac{I}{A}} = \sqrt{\frac{500^3*500}{12*500*500}} = 144.33 \rightarrow \lambda = \frac{2649.38}{144.33} = 18.356$

$\lambda_{limx} = 14.1, \lambda_{limy} = 15.215 < \lambda = 18.356$ Since the column will be categorized under slender column.

Second order eccentricity, e_2 computation

Determine Kr

$kr = (n_u - n) / (n_u - n_{bal}) \leq 1.0$, where $n = NED / (Ac * fc_d)$, $n_u = 1 + \omega$

$n_{bal} = 0.4$, Ac = the area of concrete

So $n = NED / (Ac * fc_d) = 2439.7 * 10^3 / (500^2 * 11.33) = 0.86$

$\frac{h'}{h} = \frac{48}{500} = 0.096$,

$\mu_x = \frac{MEdx}{b * h^2 * fc_d} = \frac{57.17}{500 * 500^2 * 11.33} = 0.0403$

$\mu_y = \frac{MEdy}{b * h^2 * fc_d} = \frac{60.694}{500 * 500^2 * 11.33} = 0.0428$

$Vsd = \frac{NEd}{fc_d * b * h} = \frac{2439.7 * 10^3}{11.33 * 500 * 500} = 0.86$, From column chart $\omega = 0.15$

$Kr = (1.15 - 0.86) / (1.15 - 0.4) = 0.386 < 1 \dots \dots \text{ok!}$

For members with constant symmetrical cross sections (incl. reinforcement), the following may be used $\frac{1}{r} =$

$\frac{kr * k\psi}{r_o}$

Where: kr is a correction factor depending on axial load

ψ is a factor for taking account of creep take $\psi = 0$

$k\psi = 1 + \beta\psi = 1$

$1/r_o = \xi y_d / (0.45d)$ d is the effective depth

$$\xi_{yd} = f_{yd}/ES = 347.83/200000 = 0.00174$$

$$1/r_o = 0.00174 / (0.45 * 452) = 8.554 * 10^{-6}$$

$$\frac{1}{r} = 0.386 * 8.554 * 10^{-6} = 3.3 * 10^{-6}$$

$$e2x = e2y = \frac{1}{r} * \frac{l_o^2}{c} = 3.3 * 10^{-6} * \frac{2649.38}{10} = 0.8747$$

$$M2 = N_{ed} * e2 = 0.8747 * 2439.7 = 2.134$$

$$ME_{dx} = \max \begin{cases} M_{o2} = 57.17 \\ M_{oe} + M2 = 53.82 + 2.134 = 55.954, \text{ take } 57.17 \\ M_{o1} + 0.5M2 = 48.79 + 0.5 * 2.134 = 49.857 \end{cases}$$

$$ME_{dy} = \max \begin{cases} M_{o2} = 60.694 \\ M_{oe} + M2 = 58.62 + 2.134 = 60.75, \text{ take } 60.75 \\ M_{o1} + 0.5M2 = 55.64 + 0.5 * 2.134 = 56.7 \end{cases}$$

Hence the eccentricity value is small so the change in the design moment is negligible so take $\omega = 0.16$

$$A_{stot} = \frac{\omega * A_c * f_{cd}}{f_{yd}} = 0.16 * 500 * 500 * \frac{11.33}{347.83} = 1302.935 \text{ mm}^2$$

$$A_{smin} = 701.4 \text{ mm}^2 < A_{stot} = 1302.935 \text{ mm}^2 < A_{smax} = 10000 \text{ mm}^2 \dots \dots \text{ok}$$

$$\text{Use } \phi 20, a_s = \pi * \frac{d^2}{4} = 314 \text{ mm thus, } n_o = \frac{1302.935}{314} = 4.14 \approx 5$$

Therefore provide 5 ϕ 20 bars

Table 5. 4 column design exl sample

	Axis x-x	Axis y-y		axis x-x	axis y-y	V _{ed}	U _{ed,x}	U _{ed,y}	ω	
λ=	16.220			128.207	288.707	0.65815	0.067993282	0.15311	0.45	
	16.204 42.93									
λ _{min} =	51.253			128.207	288.707	0.65815	0.067993282	0.15311	0.45	
Remark	Not slender	Not slender		128.207	288.707	0.65815	0.067993282	0.15311	0.45	
				Reinforcement calculation			Tie reinforcement			
			ω=	0.45			Ø[mm]=			

			As,tot=	4435.356352		15		Spacing=	8 400mm			
	Axis x-x	Axis y-y		axis x-x	axis y-y		Ved	$\varphi_{ed,x}$	$\varphi_{ed,y}$	ω		
λ =	16.220 16.204 45.57			121.22		289.82	0.63034	0.064287798	0.15370	0.4		
λ_{min} =	52.372			121.22		289.82	0.63034	0.064287798	0.15370	0.4		
Remark	Not slender	Not slender		121.22		289.82	0.63034	0.064287798	0.15370	0.4		
				Reinforcement calculation				Tie reinforcement				
			ω =	0.4 3942.53898		13		\varnothing [mm]=	8 400mm			
			As,tot=					Spacing=				
	Axis x-x	Axis y-y		axis x-x	axis y-y		Ved	$\varphi_{ed,x}$	$\varphi_{ed,y}$	ω		
λ =	18.424 18.401 20.83			128.058		175.408	0.69191	0.090393882	0.12382	0.4		
λ_{min} =	49.987			128.058		175.408	0.69191	0.090393882	0.12382	0.4		
Remark	Not slender	Not slender		128.058		175.408	0.69191	0.090393882	0.12382	0.4		
				Reinforcement calculation				Tie reinforcement				
			ω =	0.4 3258.296677		11		\varnothing [mm]=	8 400mm			
			As,tot=					Spacing=				

6. FOUNDATION ANALYSES AND DESIGN

6.1 INTRODUCTION

A building is generally, composed of a super structure above the ground and sub structure which forms the foundation below ground .among the substructure elements footing is one of them reinforced concrete footings are structural members used to support columns and walls and transmit and distribute their load to the soil .the design is based on the assumption that the footing is rigid ,so that the variation of the soil pressure under the footing is rigid so that the variation of the soil pressure under the footing is linear uniform soil pressure is achieved when the column load coincides with the centric of the footing .Although this assumption is acceptable for rigid ,such an assumption is becomes relatively more flexibility the proper design of footings requires that

- ✓ The load capacity of the soil is not exceeded
- ✓ Excessive settlement, differential settlement or rotations are avoided
- ✓ Adequate safety against sliding and /or over turning is maintained

the most common types of footings used in buildings are the single footing and wall footing when a column loads is transmitted to the soil by the footing .the soil becomes compressed .the amount of settlement depends on many factor such as the type of footing .if different footing of the same structure have different settlement new stress develop in the structure .excessive differential settlement may lead to the damage of nonstructural members in the building or even failure of the affected parts.

Foundation footing for structure is selected depending on the following criteria

- ✓ Function of the structure
- ✓ Amount of load it carry
- ✓ Subsurface condition
- ✓ Cost of foundation in comparison with cost of the superstructure.

For our purpose we select a square isolated footing for simplicity of analysis if the bearing failure mode of the two square footing overlaps. Design axial loads and bending moments are obtained from 3D frame analysis using ETABS 2013.

6.2 DESIGN OF ISOLATED FOOTING

An isolated footing is a footing that carries a single column. The function of an isolated footing is to spread the column load laterally to the soil so that the stress intensity is reduced to a value that the soil can safely carry.

The approximate contact pressure under a given symmetrical foundation can be obtained from the flexural formula, provided that the considered load lies within the kern of the footing. Analysis output from ETABS 2013

Design of isolated footing which have axial force

Table 6. 1 axial load and moments

support reaction at the base					
Story	Point	FZ	MX	MY	
BASE	1	1728	6	8	
BASE	2	3037	4	0	
BASE	3	2920	3	-1	
BASE	4	2663	3	1	
BASE	5	2691	3	1	
BASE	7	2573	4	2	
BASE	8	2021	5	-3	
BASE	9	1276	8	0	
BASE	10	2274	1	4	
BASE	11	4498	-2	1	
BASE	12	4288	-1	2	
BASE	13	3909	-1	3	
BASE	14	3928	-1	3	
Axial load	4333.9KN		ex =my/p		0.04
MX	175.44KNm		ey =mx/p		0.05

MY	162.06KNm	Qult=kpa	400
----	-----------	----------	-----

The reset value are tabulated in the appendix part

Use square isolated footing footing A = LxL or BxB

Using the general formula for rigid footing stress distribution; the stress due vertical loading on the soil Use ultimate bearing capacity qult=400 Kpa

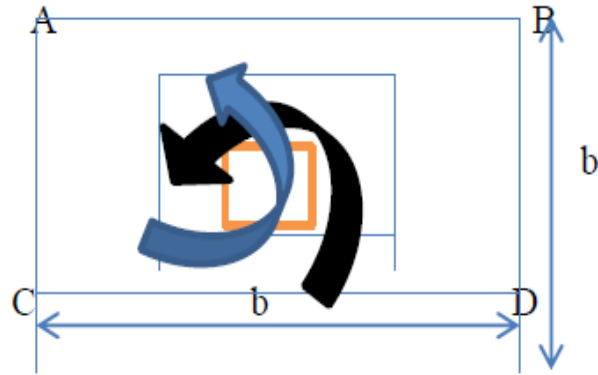
$$\sigma = \frac{p}{a} \left(1 \pm \frac{6ex}{B} \pm \frac{6ey}{B} \right)$$

$$\sigma_{\max} = \frac{p}{a} \left(1 + \frac{6ex}{B} + \frac{6ey}{B} \right) = \frac{4333.97}{B \times B} \left(1 + \frac{6 * 0.05}{B} + \frac{6 * 0.04}{B} \right) < 400$$

$$\sigma_{\min} = \frac{p}{a} \left(1 - \frac{6ex}{B} - \frac{6ey}{B} \right) = \frac{4333.97}{B \times B} \left(1 - \frac{6 * 0.05}{B} - \frac{6 * 0.04}{B} \right) > 0$$

Table 6. 2 footing check

B	3.85	3.6
Footing dimension	3.6x3.6m	
Area of footing	12.96	
Qmax	384.5667<400OK
Qmin = P/A(1-(6ex/B)-6ey/B)	284.245>0OK



$$\sigma_A = \frac{p}{a} \left(1 - \frac{6ex}{B} + \frac{6ey}{B} \right) = \frac{4333.97}{3.6 \times 3.6} \left(1 - \frac{6 * 0.04}{3.6} + \frac{6 * 0.05}{3.6} \right) = 339.98 \text{KN/m}^2$$

$$\sigma_B = \frac{p}{a} \left(1 + \frac{6ex}{B} + \frac{6ey}{B} \right) = \frac{4333.97}{3.6 \times 3.6} \left(1 + \frac{6 * 0.04}{3.6} + \frac{6 * 0.05}{3.6} \right) = 384.57 \text{KN/m}^2$$

$$\sigma_C = \frac{p}{a} \left(1 - \frac{6ex}{B} - \frac{6ey}{B} \right) = \frac{4333.97}{3.6 \times 3.6} \left(1 - \frac{6 * 0.04}{3.6} - \frac{6 * 0.05}{3.6} \right) = 284.25 \text{KN/m}^2$$

$$\sigma_D = \frac{p}{a} \left(1 - \frac{6ex}{B} - \frac{6ey}{B} \right) = \frac{4333.97}{3.6 \times 3.6} \left(1 - \frac{6 * 0.04}{3.6} - \frac{6 * 0.05}{3.6} \right) = 228.83 \text{KN/m}^2$$

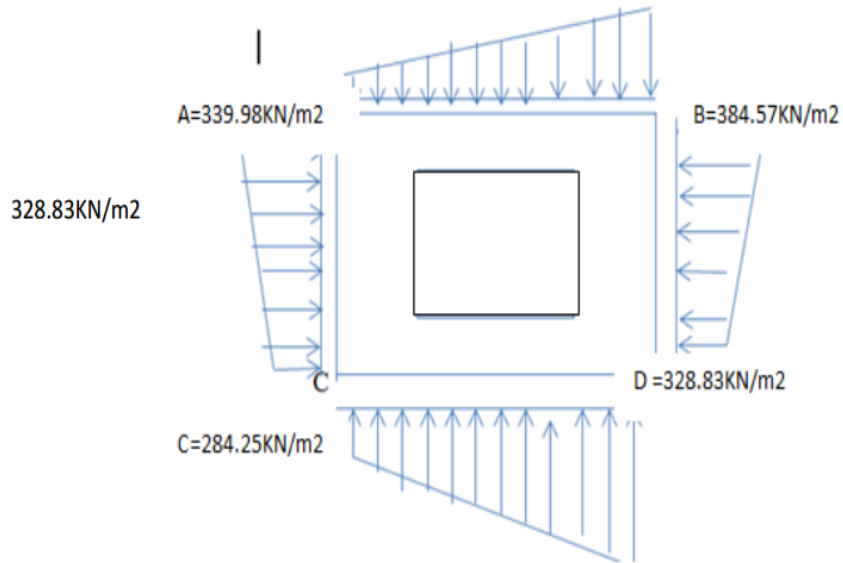


Figure 6. 1 stress distribution for isolated footing

$$\sigma_{avg} = \frac{339.97 + 384.57 + 284.25 + 328.84}{4} = 334.41 \text{ kN/m}^2$$

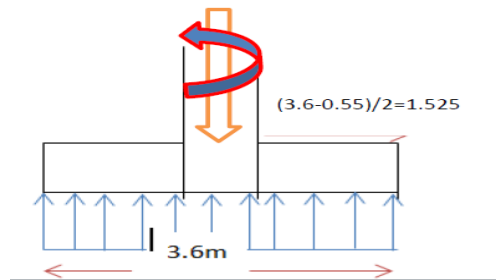


Figure 6. 2 average stress distribution for isolated footing

$$\sigma_{avg} = 334.41 \text{ kN/m}^2 = 1203.88 \text{ kN/m}$$

$$\text{Moment at face of column} = 1203.88 * 1.525^2 / 2 = 1398.86 \text{ kNm}$$

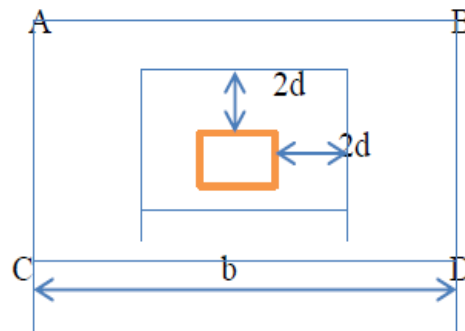
6.2.1 DEPTH DETERMINATION

The requirements for the determination of footing depth are:

- Punching shear resistance requirement
- Wide beam shear resistance requirement
- Bending moment resistance

Check for punching shear

The critical section for punching shear is located at a distance 2d from the face of the column



The code recommend the punching as follows

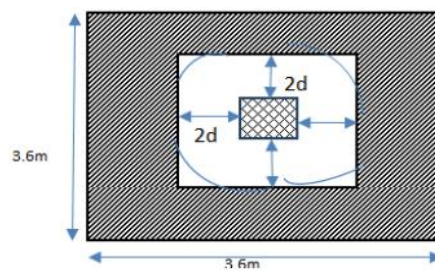


Figure 6. 3 punching shear resistance layout

$$\text{Perimeter} = 4 * \text{col.dim} + 4 * 2 \frac{\pi r}{4} = 4 * (600 + \pi d) = 4 * (600 + \pi * 650) = 10569.2 \text{mm}$$

$$\begin{aligned} A_{\text{effective}} &= (3.6 * 3.6) - ((4d + c) * c + (4d * c) + \pi(2d)^2) \\ &= 3.6 * 3.6 - (4 * 0.650 + 0.600) * 0.6 + (4 * 0.65 * 0.6) + \pi(2 * 0.65)^2 = 3.61 \text{m}^2 \end{aligned}$$

$$V_{Rd} = (C R_{d,ck} (100 \rho_1 * f_{ck})^{\frac{1}{3}} + k_1 \sigma_{cp}) u d > V_{\text{min}} \dots \text{mpa}$$

$$V_{RD,C} = 0.12, d = 0.65 \text{m}$$

$$k_1 = 1 + (\sqrt{200/d}) < 2 = 1.63$$

$$d_{\text{bott}} = 0.65 \text{m}, d_{\text{top}} = 0.64 \text{m}$$

$$M_{ED} = 1354.36 \text{KNm}$$

$$M_{sd} = \frac{1354.36 * 10^6}{11.33 * 3600 * 650^2} = 0.283$$

$$K_z = \text{from the table} = 0.83 \text{ or } K_z = 0.999 \exp(-0.66 * 0.133) = 0.8287 = 0.83$$

$$Z = d * k_z = 0.83 * 650 = 539.5 \text{mm}$$

$$A_S = \frac{M_{Ed}}{f_{ydz}} = \frac{1398.86}{347.83 * 539.5} = 7454.45 \text{mm}^2$$

$$\rho_{x1} = \frac{A_{st}}{d \times b_w} \rho_{x1} = \frac{A_{st}}{d \times b_w} = \frac{7454.45}{650 \times 3600} = 0.0048$$

$$\rho_{y1} = \frac{A_{st}}{d \times b_w} = \rho_{x1} = \frac{8325.57}{640 \times 3600} = 0.00485$$

$$\rho = \sqrt{\rho_{x1} \times \rho_{y1}} = \sqrt{0.0048 \times 0.0085} = 0.004 \dots < 0.002 \dots \text{OK}$$

$$\sigma_{CP} = \frac{N_{ED}}{A_C}, \sigma_{CP} = \frac{4333.9}{12.96} = 334.41, \text{KN/m}^2 = 0.334 \text{Mpa}$$

$$V_{Rd,C} = C R_{d,C} K (100 \rho_1 f_{ck})^{\frac{1}{3}} \geq V_{min}$$

$$V_{min} = 0.035 \times K^{\frac{3}{2}} \times f_{ck}^{\frac{1}{2}} = 0.035 \times 0.15^{\frac{3}{2}} \times 20^{\frac{1}{2}} = 0.33 \text{Mpa}$$

$$V_{Rd,C} = 0.12 \times 1.63 (100 \times 0.0048 \times 20)^{\frac{1}{3}} + 1.63 \times 0.334 = 0.96 \text{Mpa}$$

Punching check

$$V_{ED}, (P\text{-contact pressure}) = \frac{4333.9}{(ud)} - 334.33 = \frac{p}{ud} = \frac{4333.9}{10.56 \times 0.5} = 998.13 \text{KN/m}^2$$

$$\text{Applied shear} = 998.13 - 334.33 = 663.8 \text{KN/m}^2 = 0.664 \text{Mpa} < 0.96 \text{Mpa} \dots \dots \text{ok!}$$

Wide beam shear

Wide beam shear is must be resisted d distance from face of column. See the following.

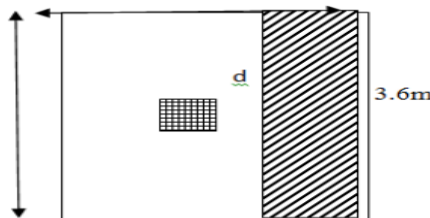


Figure 6. 4 wide beam shear resistance layout

$$V_{Rd,C} = [C R_{d,C} K (100 \rho_1 f_{ck})^{\frac{1}{3}} + k_1 \sigma_{cp}] b_w \times d < [V_{min} + k_1 \sigma_{cp}] b_w \times d$$

$$V_{Rd,C} = (C R_{d,C} k (100 \rho_1 f_{ck})^{\frac{1}{3}} + \sigma_{cp} \times k_1) b_w \times d > V_{min} \times b_w \times d$$

$$V_{Rd,c} = 0.96 \text{mpa} \times 3600 \text{ mm} \times 500 \text{mm} = 1728 \text{KN}$$

$$V_{min} = 0.035 \times K^{\frac{3}{2}} \times f_{ck}^{\frac{1}{2}} \times 0.5 \times 3600 \times 500 = 0.035 \times 0.15^{\frac{3}{2}} \times 20^{\frac{1}{2}} \times 0.5 \times 3600 \times 500 = 0.47 \times 5500 \times 230 \text{mpa} = 586.32 \text{KN}$$

$$V_{Ed} = 334.33 \times (3600 \times (3600 - 650)) = 3731.122 \text{KN}$$

$$V_{acting} = 4333.9 - 3731.122 = 602.78 \text{KN}$$

Vacting is less than the resistance of the concreteOK!

Therefore the dimensions that we applied are enough

$$D=650+50+10=710\text{mm}$$

The dimension is $LxBxD=3.6x3.6x0.71\text{m}$

6.2.2 REINFORCEMENT DESIGN

$$M_{ED}=1353.36\text{KNm}$$

$$f_{cd}=11.33\text{Mpa}$$

$$f_{yd}=347.83\text{mpa}$$

$$\mu_{sd} = \frac{M_{ED}}{f_{cd} b d^2} = \frac{1354.36}{11.33 \times 1000 \times 650^2}$$

$$K_z=0.83 \rightarrow Z=K_z \cdot d=0.83 \cdot 650=539.5\text{mm}$$

$$A_s = \frac{M_{ED}}{z f_{yd}} = \frac{1354.36}{539.5 \times 347.83} = 8325.57\text{mm}^2$$

$$A_{smin} = \max \left\{ \begin{array}{l} \frac{0.26 f_{ct} b d}{f_{yk}} = 4111.2\text{mm}^2 \\ 0.0013 b d \end{array} \right.$$

$$A_s \text{ provided} = 8325.57\text{mm}^2$$

$$\text{Use } 24\text{mm dia, } a_s = 452.45\text{mm}^2$$

$$\text{Spacing} = \frac{a_s}{A_s} = \frac{1000 \times 452.45}{8325.57} = 50\text{mm}$$

Provide $18\phi 24$ c/c 50mm

Since our footing is square footing the transverse bar is also the same with this

Provide both sides $18\phi 24$ c/c 50mm.

$$\text{Development length} = l_d = \frac{\sigma_s}{4 f_{bd}} \cdot \phi, f_{yd}=347.83, f_{bd}=2 \cdot f_{ctd} = 2.33$$

$$l_d = 474.31\text{mm}$$

$$\text{Available length} = 3650\text{mm} > 475\text{mm}$$

Design of isolated footing which have axial force 3000-4000KN

Table 6. 3 wide beam and punching shear check for isolated footing

load=	3641.94KN	
Mx	137.76KNm	
My	161.02KNM	
ex=my/p	0.05	
ey=mx/p	0.04	
qult=kpa	400Kpa	
qmax=p/A(1+(6e/B)	B^3-9.1B-2.4	
B	3.14	3.3m
ANSWER	-0.014856=0	B=3.3m
footing dimation =3.3x3.3m		
area of footing =	10.89m2	
qmax=	389.1546206<400OK!
qmin=p/A(1-(6ex/B)-6ey/B)	279.7048835>0OK!
CRD,ck	0.12	
D	550mm	
Fck	20Mpa	
k1	1.603022689	
d1	550mm	
Bar dia.	20	
d2	540	

Hc	600
Bc	600
B	1000
Fyd	347.83
qa(stress in one corner)	328.3492111
qb(stress)	389.1546206
qc(strees)	279.7048835
qd(stress)	340.510293
Q avrage(stress)	334.4297521
contact pressure	1103.618182
dist.at face of column	1.35
MED	1005.672068
μ sd	0.293427693
Kz	0.821664314
Z	451.9153729
AS	6397.823226
Fctm	2.2
Asmin	3478.710865
Asprov	6397.823226
ρ x l	0.011632406

ρ_{y2}	0.011847821
ρ_1	0.011739619
A_c	10.89
σ_{cp}	334.4297521
punching check	
Perimeter	9312.4
A effective	4088180
shear resistance at 2d	1367.209024
app. Shear	2274.730976
shear resistance	2748621.158
VRD,c,VED	...OK!
wide beam shear	
d distance from face of column	
dis.from face	0.8
VRD,c	1415299.989
shear app	882.8945455
VED	2759.045455
VRD,c,VED	..ok
A effective	2640000

$$M_{ED}=80\text{KNm}$$

Using excel

Use $\text{Ø}24\text{c/c } 60\text{mm}$for longitudinal

Transverse bar $d=526\text{mm}$

Provide $\text{Ø}24\text{c/c } 50\text{mm}$

Design of footing (Axial force between 2000KN-3000KN)

$$P=2520.38\text{KN}$$

$$M_x=167.6\text{KNm}$$

$$M_y=173.6\text{KNm}$$

Dimension of footing= $3\times 3\text{m}$

Contact pressure= 840.13KN/m

$$M_{ED}=840.18\text{KNm}$$

$$F_{cd}=11.33\text{Mpa}$$

$$F_{yd}=347.83\text{Mpa}$$

$$b=1\text{m}, d=550$$

Concrete cover = 50mm

Depth of footing= 610mm

$$\phi_{sd}=0.262$$

$$A_S=6775\text{mm}^2$$

Provide $\phi 20\text{ c/c } 70\text{mm}$.

For transverse side $d=526\text{mm}$

Provide $\text{Ø}20\text{ c/c } 50\text{mm}$.

Design of footing for axial force 1000KN-2000KN

$$P=1987.38\text{KN}$$

$$M_x=149.29\text{KNm}$$

$M_y = 159.81 \text{ kNm}$

Dimension of footing = $2.7 \times 2.7 \text{ m}$

Contact pressure = 737.1 kN/m

$M_{ED} = 405.76 \text{ kNm}$

$F_{cd} = 11.33 \text{ Mpa}$

$F_{yd} = 347.83 \text{ Mpa}$

$b = 1 \text{ m}$, $d = 400 \text{ mm}$

Concrete cover = 50 mm

Depth of footing = 460 mm

$\phi_{sd} = 0.222$

$A_S = 3896.3 \text{ mm}^2$

Provide $\phi 20 \text{ c/c } 80 \text{ mm}$

For transverse side $d = 376 \text{ mm}$

Provide $\phi 20 \text{ c/c } 70 \text{ mm}$

Foundation reinforcement detail

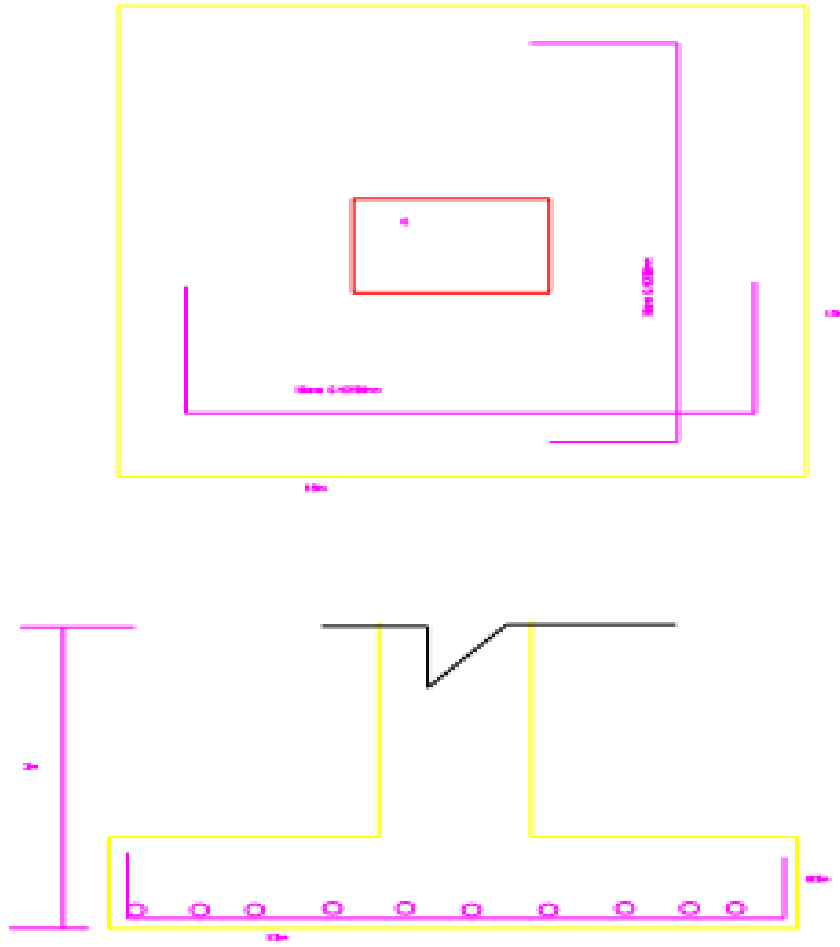


Figure 6. 5 reinforcement detail

7 SHEAR WALL

7.1 DESIGN OF SHEAR WALL

A reinforced concrete wall is a vertical load bearing member whose greatest lateral dimension is more than four times its least lateral dimension, and in which the reinforcement is taken in to account when considering its strength. A reinforced wall shall be considered as either short or slender and as either braced or un braced as follows; Short or Slender Walls. A wall may be considered short when the ratio of its effective height to its thickness does not exceed 7. It shall otherwise be considered slender. Braced or un-braced Walls: A wall may be considered as braced if, at right Angles to the plane of the wall, lateral stability to the structure as a whole is provided by walls or Other suitable bracing designed to resist all lateral forces in that direction.

It shall otherwise be considered as un-braced .The shear wall is subjected to both flexure and axial load, so we designed as column load bearing member resist lateral and axial load. From ETABS 2013 modeling analysis we get the maximum axial load and moments from Envelope.

Wall A

Axial load, $P=2980.77\text{KN}$

Shear force= 60.15KN

Given parameters

C25

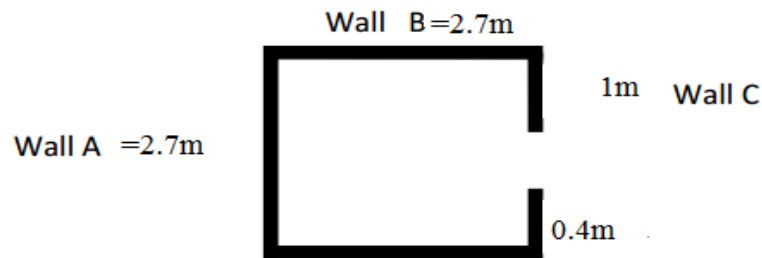
S-400 class B

Step 1) Determine material design Constant

$$f_{cd} = \frac{\alpha_{cc} * f_{ck}}{\gamma_c} = \frac{0.85 * 20}{1.5} = 11.33\text{Mpa}$$

$$f_{yd} = \frac{400}{1.15} = 347.8\text{Mpa}$$

Step 2) Determination of effective depth and length



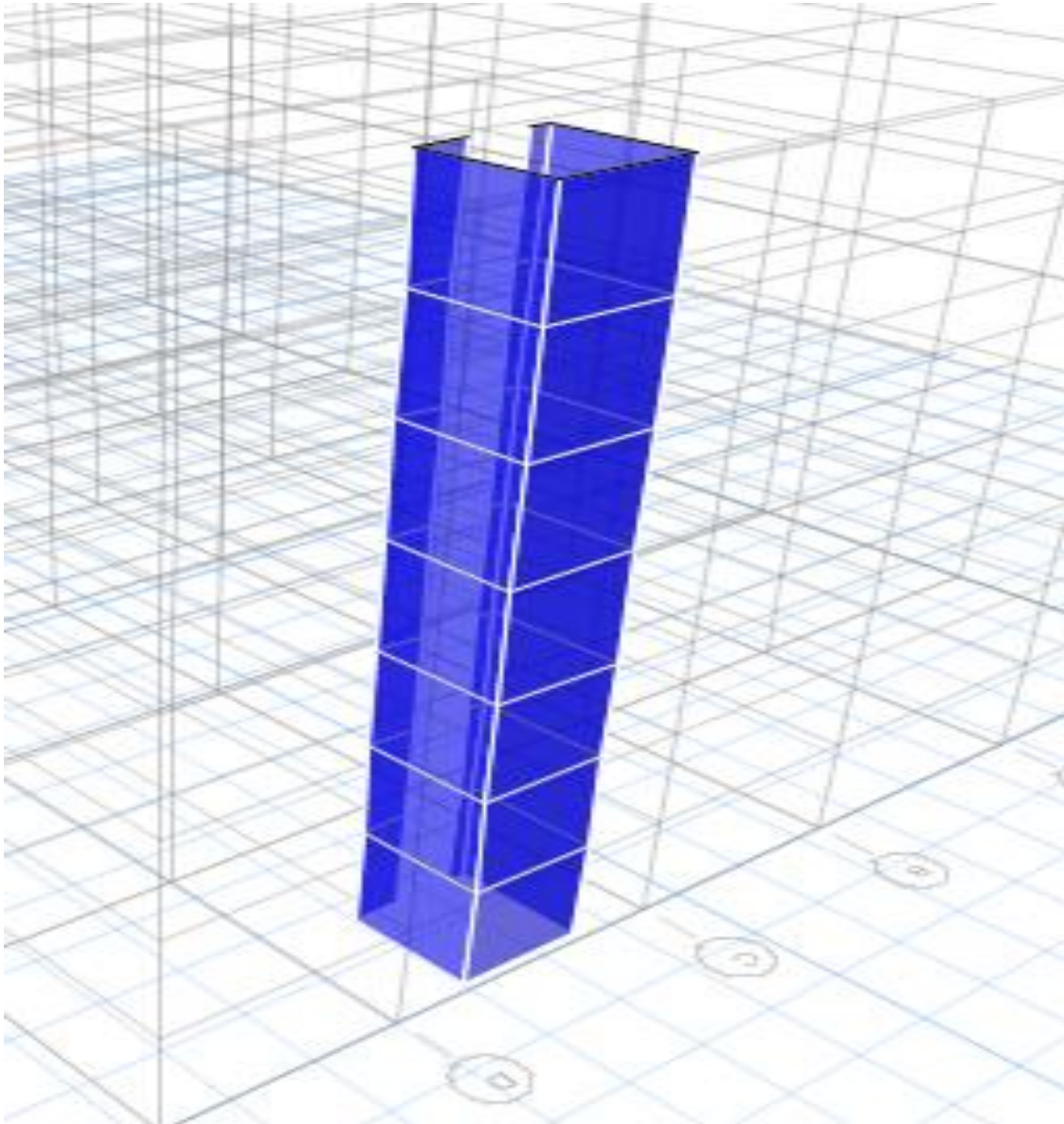


Fig 3D layout of shear wall

The total depth of the shear wall is given 150mm from the architectural drawing using $\varnothing 10$ mm bar and the clear cover 20mm

Effective depth of the wall become $d=t-\text{cover}-\varnothing/2 = 150-20-10/2 = 125\text{mm}$

The effective height L of reinforced concrete wall in the non-sway mode shall be determined as from

$L_e = \beta L$, where L -the store height of the wall

β -the coefficient defined below $\beta = 1.0$ Wall with two edge restrained

So $L_e = 1 * 3 = 3\text{m}$

Step 3 Design the Shear wall

Hence the wall loads are vary from floor to floor we consider the wall result which found on the ground to base and provided this for the rest of the floor. The effect of imperfection may be taken as according to ES EN 1992:Art5.2.7

$$e_i = \max \left\{ \begin{array}{l} \frac{L_0}{400} \\ \frac{h}{30} \\ 20\text{mm} \end{array} \right.$$

$$e_i = \max \left\{ \begin{array}{l} \frac{3000\text{mm}}{400} \\ \frac{150}{30} \\ 20\text{mm} \end{array} \right. = \max \left\{ \begin{array}{l} 7.5\text{mm} \\ 5\text{mm} \\ 20\text{mm} \end{array} \right. \quad \text{take } e_i = 20\text{mm}$$

Hence the shear wall is subjected to the pure axial load and so let's reduce the capacity by 20%
 $NED = 0.8 \times (f_{cd} \times A_c + A_s \times f_{yd})$

$$A_s = (NED / 0.8) - f_{cd} \times A_c$$

For wall A

$$A_s = 1.25 \times (2980.77 \times 10^6) - 11.33 \times 2700 \times 150$$

$$A_s = 3541.44 \text{mm}^2$$

$$A_{sr} \text{ in the rows become } A_s / 2 = 3541.44 / 2 \text{ mm}^2 = 1770.22 \text{mm}^2$$

Using Ø10 bars

Spacing becomes

$$S = a_s \times b / A_{sr}$$

$$= 78.5 \times 2700 / 1770.22 = 119.7 \text{mm}$$

Use Ø10 bar c/c 120mm on the two face and the reinforcement for the transverse direction becomes

Spacing = 2xspacing of the main bar

$$= 2 \times 120 = 240$$

Use Ø10 bar c/c 240mm on the two face of the wall

Step 4) Check for shear resistance of the wall

$$VRD,c = (CRD,c * K (100 * \rho * f_{ck})^{1/3} + K_1 * \delta_{cp}) b_w * d (V_{min} + K_1 * \delta_{cp}) b_w * d$$

where

$$CRD,c = 0.18 / \gamma_c = 0.18 / 1.5 = 0.12$$

$$K = (1 + \sqrt{200/d}) \cdot 2.0 \cdot d \text{ in mm}$$

$$= (1 + \sqrt{200/2175}) \leq 2.0 = 2.06 \leq 2.0$$

$$K1 = 0.15$$

$$g = A_{sl}/b_w d \leq 2.0$$

A_{sl} = is the area of the tensile reinforcement, which extends ($l_{bd} + d$) beyond the section considered

b_w = is the smallest width of the cross section in the tensile area (mm)

$$g = 3541.44 / (175 \times 2700) \leq 0.02$$

$$= 0.00749 \leq 0.02 \dots \dots \dots \text{Not Ok}$$

$$g = 0.00749$$

$$V_{min} = 0.035 \cdot K \cdot K^{3/2} \cdot f_{ck}^{1/2}$$

$$= 0.035 \cdot 2.0^{3/2} \cdot 20^{1/2}$$

$$= 0.443$$

$$\delta_{cp} = NED / AC \leq 0.2 \cdot f_{cd}$$

NED = the axial force in the cross section in the tensile area (mm)

AC = is the area of the concrete cross section (mm^2)

$$NED = 2980.77 \text{ so } \delta_{cp} = 2980.77 / (2700 \times 200) = 0.055$$

$$\text{so, } VRD,c = (CRD,c \cdot K (100 \cdot g \cdot f_{ck})^{1/3} + K1 \cdot \delta_{cp}) \cdot b_w \cdot d (V_{min} + K1 \cdot \delta_{cp}) \cdot b_w \cdot d$$

$$= 0.55 \cdot 2.0 \cdot (100 \cdot 0.00749 \cdot 20)^{1/3} + 0.15 \cdot 0.012 \cdot 2300 \cdot 175 (0.433 + 0.15 \cdot 0.012) \cdot 2300 \cdot 175$$

$$= 341.738 \text{ KN } 175.007 \text{ KN}$$

$$VRD,c = 341.738 \text{ KN} > 122.763 \text{ KN} \dots \dots \dots \text{OK!}$$

Therefore no need of shear Reinforcement

CONCLUSION

In this final year project which required a lot struggle to fruit out the final design of this B+G +5 mixed use building. And each important part of the building was carefully analyzed and designed for the appropriate loading conditions as much as possible to get the final design result.

In the design of this building, we have tried to satisfy the most basic requirements of design of a structure in accordance with revised Ethiopian building code standard derived from the European building code standard. The design of reinforced concrete members was made economical, safe and serviceable as much as possible.

RECOMMENDATION

In fact this project is the final qualification method so some qualifiers are not the part of this. Because they face deferent challenges like communications, coordination, economical and personal skills or try to pass with the shadow of other group members. If possible the institute should try to solve such problem for the future.

Since structural analysis and design procedures are bulky and complex to handle and having repetitive works throughout the entire project structures. So to solve such problems and finish the project according to the schedule structural design and analysis software's like ETABS and SAP and others are mandatory. This implies the institute/department should give short lectures about the software's and how to use them for student who are taking/learning structural analysis and design project.

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APPENDIX

Table A. 1 depth determination for G and 1st floor solid slab

panel	Support condition	Lx	Ly	Fyk	Fck	ρ_o	P	N	K	F1	F2	F3	d(mm)
P1	2	5	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P2	2	5	6	400	20	0.0045	0.0047	17.7	1.3	1.25	1	1	173.84
P3	4	6	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	180.79
P4	4	5	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P5	3	5	5	400	20	0.0045	0.0047	17.7	1.3	1.25	1	1	173.84
P6	4	5	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P7	3	5	5	400	20	0.0045	0.0047	17.7	1.3	1.25	1	1	173.84
P8	4	5	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P9	3	5	5	400	20	0.0045	0.0047	17.7	1.3	1.25	1	1	173.84
P10	4	5	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P11	2	2.9	5	400	20	0.0045	0.0047	17.7	1.3	1.25	1	1	100.82
P12	3	2.9	6	400	20	0.0045	0.0047	17.7	1.3	1.25	1	1	100.82
P13	3	5	6	400	20	0.0045	0.0047	17.7	1.3	1.25	1	1	173.84
P14	4	5	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P15	4	5	5	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P16	4	5	5	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P17	4	5	5	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P18	4	5	5	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66

P19	2	2.9	5	400	20	0.0045	0.0047	17.7	1.3	1.25	1	1	100.82
P20	4	6	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	180.79
P21	4	5	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P22	4	5	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
P23	4	5	6	400	20	0.0045	0.0047	17.7	1.5	1.25	1	1	150.66
C1	2	1.5	6	400	20	0.0045	0.0047	17.7	0.4	1.25	1	1	169.49
C2	3	1.5	6	400	20	0.0045	0.0047	17.7	0.4	1.25	1	1	169.49
C3	3	1.5	5	400	20	0.0045	0.0047	17.7	0.4	1.25	1	1	169.49
C4	3	1.5	5	400	20	0.0045	0.0047	17.7	0.4	1.25	1	1	169.49
C5	3	1.5	5	400	20	0.0045	0.0047	17.7	0.4	1.25	1	1	169.49
C6	3	1.5	5	400	20	0.0045	0.0047	17.7	0.4	1.25	1	1	169.49
C7	2	1.5	2.9	400	20	0.0045	0.0047	17.7	0.4	1.25	1	1	169.49

Table A. 2 slab design load

panel	Lx(m)	Ly(m)	Area(m ²)	wt.of slab	wt.of marble	wt.of plaster	GK	qk	DL
P1	5	6	30	5.5	0.81	1.15	7.46	2.5	13.821
P2	5	6	30	5.5	0.81	1.15	7.46	4.5	16.821
P3	6	6	36	5.5	0.81	1.15	7.46	3.25	14.946
P4	5	6	30	5.5	0.81	1.15	7.46	4.5	16.821
P5	5	5	25	5.5	0.81	1.15	7.46	4.5	16.821
P6	5	6	30	5.5	0.81	1.15	7.46	4.5	16.821
P7	5	5	25	5.5	0.81	1.15	7.46	4.5	16.821

P8	5	6	30	5.5	0.81	1.15	7.46	4.5	16.821
P9	5	5	25	5.5	0.81	1.15	7.46	4.5	16.821
P10	5	6	30	5.5	0.81	1.15	7.46	4.5	16.821
P11	2.9	5	14.5	5.5	0.23	1.15	6.88	4.5	16.038
P12	2.9	6	17.4	5.5	0.81	1.15	7.46	7.5	21.321
P13	5	6	30	5.5	0.23	1.15	6.88	2.5	13.038
P14	5	6	30	5.5	0.32	1.15	6.97	3.5	14.659
P15	5	5	25	5.5	0.32	1.15	6.97	3.5	14.659
P16	5	5	25	5.5	0.32	1.15	6.97	3.5	14.659
P17	5	5	25	5.5	0.32	1.15	6.97	3.5	14.659
P18	5	5	25	5.5	0.32	1.15	6.97	3.5	14.659
P19	2.9	5	14.5	5.5	0.23	1.15	6.88	2.5	13.038
P20	6	6	36	5.5	0.81	1.15	7.46	3.5	15.321
P21	5	6	30	5.5	0.32	1.15	6.97	3.5	14.659
P22	5	6	30	5.5	0.32	1.15	6.97	3.5	14.659
P23	5	6	30	5.5	0.32	1.15	6.97	3.5	14.659
C1	1.5	6	9	5.5	0.81	1.15	7.46	1.5	12.321
C2	1.5	6	9	5.5	0.81	1.15	7.46	1.5	12.321
C3	1.5	5	7.5	5.5	0.81	1.15	7.46	1.5	12.321
C4	1.5	5	7.5	5.5	0.81	1.15	7.46	1.5	12.321
C5	1.5	5	7.5	5.5	0.81	1.15	7.46	1.5	12.321

C6	1.5	5	7.5	5.5	0.81	1.15	7.46	1.5	12.321
C7	1.5	2.9	4.35	5.5	0.81	1.15	7.46	1.5	12.321

Table A. 3 adjusted support moment

axis	panel	unadjusted moment				df	adjusted moment
			ΔM	$\Delta M/M_{max}$	M_{avg}		
on axis 2		16.24				0.5	14.425
	B/n p1 & s4	12.61	3.63	0.22352		0.5	14.425
		17.21					
	B/n S1 & p3	18.427	1.217	0.06604	17.82		17.82
		15.25					
	B/n p13 & p14	14.07	1.18	0.07738	14.66		14.66
		16.24					
	B/n s2 & p20	16.88	0.64	0.03791	16.56		16.56
	B/n C1 & C2	13.86	0	0	13.86		13.86

on axis 3		6.3				0.5	13.5684
	B/n p2 & s4	19.76	13.46	0.68117		0.5	13.5684
		17.21					
	B/n p3 & P4	16.14	1.07	0.06217	16.68		
		11.72					
	B/n p14 & p15	14.07	2.35	0.16702	12.9		
		14.07					
	B/n p20 & p21	16.88	2.81	0.16647	15.48		
	B/n C2 & C3	13.86	0	0	13.86		13.86
on axis 4		19.76					
	B/n p2 & p5	16.4	3.36	0.17004	18.08		
on axis 6		6.3				0.5	10.185
	B/n p23 & s3	14.07	7.77	0.55224		0.5	10.185
on axis 7		16.4				0.6	12.9476
	B/n p9 & p11	10.92	5.48	0.33415		0.4	12.9476
		16.14				0.6	37.6356
	B/n p10 & p12	50.26	34.12	0.67887		0.4	37.6356
		11.72					
	B/n p18 & p19	10.92	0.8	0.06826	11.32		

axis	panel	unadjusted moment					adjusted moment	
			ΔM	$\Delta M/M_{max}$	M_{avg}	df	M_{xs}	
axis D		21.768				0.55	15.4936	
	B/n p1 & S1	10.36	11.408	0.524072		0.45	15.4936	
		51.04				0.54	32.7718	
	B/n S4 & p3	17.21	33.83	0.662813		0.46	32.7718	
		26.49				0.54	23.628	
	B/n p2 & P4	21.19	5.3	0.200076		0.46	23.628	
		16.4				0.54	18.9866	
	B/n p5 & p6	21.19	4.79	0.22		0.46	18.9866	
			4.79	0.22			18.98	
	B/n p7 & p8						18.98	
	B/n p9 & p10	with similar moment	4.79	0.22			18.98	
		20.23				0.55	12.618	
B/n p11 & p12		6.39	13.84	0.684132		0.45	12.618	
axis c		10.36				0.54	14.9014	
	B/n s1 & p13	18.77	8.41	0.448055		0.46	14.9014	
		17.21						
	B/n p3 & p14	18.47	1.26	0.068219	17.84			
		21.19				0.54	16.0762	
	B/n p4 & P15	11.72	9.47	0.446909		0.46	16.0762	
			9.47	0.44			16.0762	
	B/n p6 & p16		9.47	0.44			16.0762	
	B/n p8 & p17		9.47	0.44			16.0762	
	B/n p10 & p1	with similar moment	9.47	0.44			16.0762	
			41.51				0.54	30.0188
B/n p12 & p19		20.23	21.28	0.512648		0.46	30.0188	
axis B		18.77				0.46	33.5958	
	B/n p13 & S2	51	32.23	0.631961		0.54	33.5958	
		18.47						
	B/n p14 & p20	16.92	1.55	0.08392	17.695			
		11.77				0.46	14.852	
	B/n p15 & P21	18.47	6.7	0.36275		0.54	14.852	
			6.75	0.36			14.85	
	B/n p16 & p22		6.75	0.36			14.85	
	B/n p17 & p23		6.75	0.36			14.85	
			11.72				0.46	29.7888
	B/n p18 & s3	51	39.28	0.770196			0.54	29.7888
axis A		51.04				0.8	21.296	
	B/n s2&C1	13.86	37.18	0.728448		0.2	21.296	
		16.92					0.16	16.088
	B/n p20&C2	11.72	5.2	0.307329		0.84	16.088	
		18.47					0.16	17.7324
	B/n P21&C3	13.86	4.61	0.249594		0.84	17.7324	
			4.61	0.29			17.73	
	B/n P22&C4		4.61	0.29			17.73	
	B/n P23&C5		4.61	0.29			17.73	
			51.04				0.8	21.296
	B/n s3&C6		13.86	37.18	0.728448		0.2	21.296

Table A. 4 Adjusted filed moment

panle	Mmax,sup	Madj,sup	Lx	Ly	Ly/Lx	Cx	Cy	ΔM	Mxf	Myf	Cx ΔM	Cy ΔM	Mxf,adj	Myf,adj
p1	21.76	15.49	5	6	1.2	0.34	0.36	6.27	16.2	12.4	2.16	2.282	18.357	14.71
p2	19.76	13.76	5	5	1	0.38	0.28	6	19.8	15.1	2.28	1.68	22.04	16.82
p4	21.19	16.07	5	6	1.2	0.34	0.17	5.12	16.1	12.1	1.73	0.881	17.871	12.99
p6	21.19	16.07	5	6	1.2	0.34	0.17	5.12	16.1	12.1	1.73	0.881	17.871	12.99
p8	21.19	16.07	5	6	1.2	0.34	0.17	5.12	16.1	12.1	1.73	0.881	17.871	12.99
p9	16.4	12.94	5	5	1	0.38	0.28	3.46	12.6	12.6	1.31	0.969	13.925	13.58
p13	20.23	12.61	3	5	1.72	0.29	0.07	7.62	15.1	8.3	2.17	0.564	17.282	8.864
p13	18.75	14.9	5	6	1.2	0.33	0.36	3.85	14.1	11.3	1.29	1.401	15.386	12.73
p114	14.07	12.9	5	6	1.2	0.34	0.17	1.17	14.1	10.6	0.4	0.201	14.465	10.75
p18	11.72	11.33	5	6	1.2	0.34	0.36	0.39	10.6	11.7	0.13	0.142	10.684	11.86
p20	16.88	15.48	6	6	1	0.38	0.28	1.4	12.7	12.7	0.53	0.392	13.192	13.05
p21	18.47	14.07	5	6	1.2	0.34	0.17	4.4	14.1	14.1	1.49	0.757	15.557	14.83
p22	18.47	14.07	5	6	1.2	0.34	0.17	4.4	14.1	14.1	1.49	0.757	15.557	14.83
p23	18.47	14.07	5	6	1.2	0.34	0.17	4.4	14.1	14.1	1.49	0.757	15.557	14.83

Table A. 5 Reinforcement for G and 1st solid slab

panel	strip	MD field	fcd	fyd	b	d	μ_{sd}	Kz	Ascalc	Aspro	Scalc	Sprovided
S1	I-I	25.52	11.33	347.83	1000	185	0.065812	0.9565	414.626	414.62	189.33	Ø10C/C185mm
	G-G	95.25	11.33	347.83	1000	195	0.221088	0.87	1614.15	1614.15	48.6324	Ø10C/C45mm
		MD support										
	B-B	2.369	11.33	347.83	1000	185	0.006109	0.9989	36.8557	264.55	296.73	Ø10C/C295mm
	F-F	17.52	11.33	347.83	1000	195	0.040666	0.9698	266.349	277.42	282.964	Ø10C/C280mm
	G-G	51.04	11.33	347.83	1000	185	0.131625	0.9299	852.974	852.97	92.0314	Ø10C/C90mm
	I-I	190.5	11.33	347.83	1000	195	0.442177	0.6789	4137.02	4137.01	18.9751	Ø10C/C15mm
S2		MD filed										
	A-A	9.23	11.33	347.83	1000	185	0.023803	0.9789	146.529	264.55	296.73	Ø10C/C290mm
	B-B	18.01	11.33	347.83	1000	195	0.041804	0.9689	274.052	277.42	282.964	Ø10C/C280mm
		MD support										
	A-A	18.42	11.33	347.83	1000	185	0.047502	0.9659	296.359	296.359	264.881	Ø10C/C260mm
	strip1	10.15	11.33	347.83	1000	195	0.02356	0.9789	152.871	277.42	282.964	Ø10C/C280mm
	strip2	19.37	11.33	347.83	1000	195	0.04496	0.9679	295.051	295.05	266.057	Ø10C/C260mm
	strip5	119.2	11.33	347.83	1000	195	0.27668	0.8352	2104.19	2104.18	37.3067	Ø10C/C35mm
	strip6	46.78	11.33	347.83	1000	195	0.108583	0.9465	728.682	728.68	107.729	Ø10C/C105mm

on axis C	B/n s1 & p13	14.9	11.33	347.83	1000	195	0.03458	0.9772	224.8	277.42	282.964	Ø10C/C280mm
	B/n p3 & p14	17.84	11.33	347.83	1000	195	0.04141	0.9683	271.63	277.42	282.964	Ø10C/C280mm
	B/n p4 & P15	16.07	11.33	347.83	1000	195	0.0373	0.9758	242.8	277.42	282.964	Ø10C/C280mm
	B/n p6 & p16	16.07	11.33	347.83	1000	195	0.0373	0.9758	242.8	277.42	282.964	Ø10C/C280mm
	B/n p8 & p17	16.07	11.33	347.83	1000	195	0.0373	0.9758	242.8	277.42	282.964	Ø10C/C280mm
	B/n p10 & p18	16.07	11.33	347.83	1000	195	0.0373	0.9758	242.8	277.42	282.964	Ø10C/C280mm
	B/n p12 & p19	30.01	11.33	347.83	1000	195	0.06966	0.9515	465	465.003	168.816	Ø10C/C160mm
on axis D	B/n p1 & S1	15.49	11.33	347.83	1000	195	0.03595	0.9763	233.92	277.42	282.964	Ø10C/C280mm
	B/n S4 & p3	32.77	11.33	347.83	1000	195	0.07606	0.9488	509.21	509.214	154.159	Ø10C/C150mm
	B/n p2 & P4	23.62	11.33	347.83	1000	195	0.05483	0.9586	363.28	363.279	216.087	Ø10C/C210mm
	B/n p5 & p6	18.98	11.33	347.83	1000	195	0.04406	0.961	291.19	291.186	269.587	Ø10C/C265mm
	B/n p7 & p8	18.98	11.33	347.83	1000	195	0.04406	0.961	291.19	291.186	269.587	Ø10C/C265mm
	B/n p9 & p10	18.98	11.33	347.83	1000	195	0.04406	0.961	291.19	291.186	269.587	Ø10C/C265mm
	B/n p11 & p12	12.61	11.33	347.83	1000	195	0.02927	0.9796	189.79	277.42	282.964	Ø10C/C280mm

on axis C	B/n s1 & p13	14.9	11.33	347.83	1000	195	0.03458	0.9772	224.8	277.42	282.964	Ø10C/C280mm
	B/n p3 & p14	17.84	11.33	347.83	1000	195	0.04141	0.9683	271.63	277.42	282.964	Ø10C/C280mm
	B/n p4 & P15	16.07	11.33	347.83	1000	195	0.0373	0.9758	242.8	277.42	282.964	Ø10C/C280mm
	B/n p6 & p16	16.07	11.33	347.83	1000	195	0.0373	0.9758	242.8	277.42	282.964	Ø10C/C280mm
	B/n p8 & p17	16.07	11.33	347.83	1000	195	0.0373	0.9758	242.8	277.42	282.964	Ø10C/C280mm
	B/n p10 & p18	16.07	11.33	347.83	1000	195	0.0373	0.9758	242.8	277.42	282.964	Ø10C/C280mm
	B/n p12 & p19	30.01	11.33	347.83	1000	195	0.06966	0.9515	465	465.003	168.816	Ø10C/C160mm
on axis D	B/n p1 & S1	15.49	11.33	347.83	1000	195	0.03595	0.9763	233.92	277.42	282.964	Ø10C/C280mm
	B/n S4 & p3	32.77	11.33	347.83	1000	195	0.07606	0.9488	509.21	509.214	154.159	Ø10C/C150mm
	B/n p2 & P4	23.62	11.33	347.83	1000	195	0.05483	0.9586	363.28	363.279	216.087	Ø10C/C210mm
	B/n p5 & p6	18.98	11.33	347.83	1000	195	0.04406	0.961	291.19	291.186	269.587	Ø10C/C265mm
	B/n p7 & p8	18.98	11.33	347.83	1000	195	0.04406	0.961	291.19	291.186	269.587	Ø10C/C265mm
	B/n p9 & p10	18.98	11.33	347.83	1000	195	0.04406	0.961	291.19	291.186	269.587	Ø10C/C265mm
	B/n p11 & p12	12.61	11.33	347.83	1000	195	0.02927	0.9796	189.79	277.42	282.964	Ø10C/C280mm

axis	location	MED	fed	fyd	b	d	μ sd	Kz	Ascalc	Aspro	Scalc	Sprovidied
on axis 2	B/n p1 & s4	14.42	11.33	347.83	1000	185	0.03719	0.9699	231.05	264.55	296.73	Ø10C/C290mm
	B/n S1 & p3	17.87	11.33	347.83	1000	185	0.04608	0.9598	289.34	289.338	271.309	Ø10C/C270mm
	B/n p13 & p14	14.66	11.33	347.83	1000	185	0.03781	0.9697	234.94	264.55	296.73	Ø10C/C290mm
	B/n s2 & p20	16.56	11.33	347.83	1000	185	0.04271	0.9677	265.94	265.938	295.182	Ø10C/C290mm
	B/n C1 & C2	13.86	11.33	347.83	1000	185	0.03574	0.97	222.05	264.55	296.73	Ø10C/C290mm
on axis 3	B/n p2 & s4	13.56	11.33	347.83	1000	185	0.03497	0.9748	216.17	264.55	296.73	Ø10C/C290mm
	B/n p3& P4	16.68	11.33	347.83	1000	185	0.04302	0.9599	270.04	270.042	290.696	Ø10C/C290mm
	B/n p14 & p15	12.9	11.33	347.83	1000	185	0.03327	0.9785	204.88	264.55	296.73	Ø10C/C290mm
	B/n p20 & p21	15.48	11.33	347.83	1000	185	0.03992	0.9689	248.29	264.55	296.73	Ø10C/C290mm
	B/n C2 & C3	13.86	11.33	347.83	1000	185	0.03574	0.97	222.05	264.55	296.73	Ø10C/C290mm
on axis 4	B/n p2 & p5	18.08	11.33	347.83	1000	185	0.04663	0.9597	292.77	292.768	268.13	Ø10C/C260mm
	B/n p4 & p6	16.14	11.33	347.83	1000	185	0.04162	0.9688	258.9	264.55	296.73	Ø10C/C290mm
	B/n p15& p16	11.72	11.33	347.83	1000	185	0.03022	0.9789	186.06	264.55	296.73	Ø10C/C290mm
	B/n p21& p22	14.07	11.33	347.83	1000	185	0.03628	0.9759	224.05	264.55	296.73	Ø10C/C290mm
	B/n C3 & C4	13.86	11.33	347.83	1000	185	0.03574	0.9764	220.6	264.55	296.73	Ø10C/C290mm
on axis 6	B/n p7 & p9	16.4	11.33	347.83	1000	185	0.04229	0.9678	263.34	264.55	296.73	Ø10C/C290mm
	B/n p8 & p10	16.14	11.33	347.83	1000	185	0.04162	0.9685	258.98	264.55	296.73	Ø10C/C290mm
	B/n p17 & p18	11.72	11.33	347.83	1000	185	0.03022	0.9789	186.06	264.55	296.73	Ø10C/C290mm
	B/n p23& pS3	10.16	11.33	347.83	1000	185	0.0262	0.9798	161.15	264.55	296.73	Ø10C/C290mm
	B/n C5 & C6	13.86	11.33	347.83	1000	185	0.03574	0.9764	220.6	264.55	296.73	Ø10C/C290mm
on axis 7	B/n p9 & p11	12.94	11.33	347.83	1000	185	0.03337	0.9781	205.59	264.55	296.73	Ø10C/C290mm
	B/n p10& p12	37.63	11.33	347.83	1000	185	0.09704	0.9399	622.18	622.177	126.17	Ø10C/C120mm
	B/n p18& p19	11.32	11.33	347.83	1000	185	0.02919	0.9797	179.56	264.55	296.73	Ø10C/C290mm
	B/n C6 & C7	13.86	11.33	347.83	1000	185	0.03574	0.9764	220.6	264.55	296.73	Ø10C/C290mm
on axis 5	B/n p5 & p7	16.4	11.33	347.83	1000	185	0.04229	0.9678	263.34	264.55	296.73	Ø10C/C290mm
	B/n p6 & p8	16.14	11.33	347.83	1000	185	0.04162	0.9685	258.98	264.55	296.73	Ø10C/C290mm
	B/n p16 & p17	11.72	11.33	347.83	1000	185	0.03022	0.9789	186.06	264.55	296.73	Ø10C/C290mm
	B/n p22& p23	14.07	11.33	347.83	1000	185	0.03628	0.9759	224.05	264.55	296.73	Ø10C/C290mm
	B/n C4 & C5	13.6	11.33	347.83	1000	185	0.03507	0.9769	216.35	264.55	296.73	Ø10C/C290mm
on axis A	B/n s2&C1	21.29	11.33	347.83	1000	195	0.04942	0.9589	327.34	327.341	239.811	Ø10C/C235mm
	B/n p20&C2	16.08	11.33	347.83	1000	195	0.03732	0.9757	242.98	277.42	282.964	Ø10C/C280mm
	B/n P21&C3	17.73	11.33	347.83	1000	195	0.04115	0.9687	269.85	277.42	282.964	Ø10C/C280mm
	B/n P22&C4	17.73	11.33	347.83	1000	195	0.04115	0.9687	269.85	277.42	282.964	Ø10C/C280mm
	B/n P23&C5	17.73	11.33	347.83	1000	195	0.04115	0.9687	269.85	277.42	282.964	Ø10C/C280mm
	B/n s3&C6	21.29	11.33	347.83	1000	195	0.04942	0.9589	327.34	327.341	239.811	Ø10C/C235mm
on axis B	B/n p13 & S1	33.59	11.33	347.83	1000	195	0.07797	0.9499	521.35	521.351	150.57	Ø10C/C150mm
	B/n p14 & p20	17.69	11.33	347.83	1000	195	0.04106	0.9688	269.21	277.42	282.964	Ø10C/C280mm
	B/n p15 & P21	14.85	11.33	347.83	1000	195	0.03447	0.9771	224.07	277.42	282.964	Ø10C/C280mm
	B/n p16 & p22	14.85	11.33	347.83	1000	195	0.03447	0.9771	224.07	277.42	282.964	Ø10C/C280mm
	B/n p17 & p23	14.85	11.33	347.83	1000	195	0.03447	0.9771	224.07	277.42	282.964	Ø10C/C280mm
	B/n p18 & s3	29.78	11.33	347.83	1000	195	0.06912	0.9523	461.05	461.051	170.263	Ø10C/C170mm

Table A. 6 slab depth for 3rd floor

panel	support con.	lx	ly	fyk	fck	$\rho_o=p$	N	K	F1	F2	F3	D
P1	2	5	6	400	20	0.0045	17.7	1.3	1.25	1	1	173.837
P2	2	5	5	400	20	0.0045	17.7	1.3	1.25	1	1	173.837
P4	4	5	6	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P5	3	5	5	400	20	0.0045	17.7	1.3	1.25	1	1	173.837
P6	4	5	6	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P7	3	5	5	400	20	0.0045	17.7	1.3	1.25	1	1	173.837
P8	4	5	6	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P9	3	5	5	400	20	0.0045	17.7	1.3	1.25	1	1	173.837
P10	4	5	6	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P11	2	2.9	5	400	20	0.0045	17.7	1.3	1.25	1	1	100.826
P12	3	2.9	6	400	20	0.0045	17.7	1.3	1.25	1	1	100.826
P13	3	5	6	400	20	0.0045	17.7	1.3	1.25	1	1	173.837
P14	4	5	6	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P15	4	5	5	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P16	4	5	5	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P17	4	5	5	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P18	4	5	5	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P19	2	2.9	5	400	20	0.0045	17.7	1.3	1.25	1	1	100.826
P21	4	5	6	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P22	4	5	6	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
P23	4	5	6	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
S3	3	5	6	400	20	0.0045	17.7	1.5	1.25	1	1	150.659
C1	2	1.5	6	400	20	0.0045	17.7	1.3	1.25	1	1	52.1512
C2	3	1.5	6	400	20	0.0045	17.7	0.4	1.25	1	1	169.492
C3	3	1.5	5	400	20	0.0045	17.7	0.4	1.25	1	1	169.492
C4	3	1.5	5	400	20	0.0045	17.7	0.4	1.25	1	1	169.492
C5	3	1.5	5	400	20	0.0045	17.7	0.4	1.25	1	1	169.492
C6	3	1.5	5	400	20	0.0045	17.7	0.4	1.25	1	1	169.492
C7	2	1.5	2.9	400	20	0.0045	17.7	0.4	1.25	1	1	169.492

Table A. 7 design load for 3rd floor sab

panle	lx	ly	area	wt. of slab	wt. of pvc	wt.pla.t&part.	GK	QK	DL
P1	5	6	30	5.25	0.81	1.15	7.21	2.5	13.4835
P2	5	5	25	5.25	0.81	1.15	7.21	1.75	12.3585
P3	6	6	36	5.25	0.81	1.15	7.21	2.5	13.4835
P4	5	6	30	5.25	0.81	1.15	7.21	1.75	12.3585
P5	5	5	25	5.25	0.81	1.15	7.21	1.75	12.3585
P6	5	6	30	5.25	0.81	1.15	7.21	3	14.2335
P7	5	5	25	5.25	0.81	1.15	7.21	1.75	12.3585
P8	5	6	30	5.25	0.81	1.15	7.21	1.75	12.3585
P9	5	5	25	5.25	0.81	1.15	7.21	1.75	12.3585
P10	5	6	30	5.25	0.81	1.15	7.21	4.5	16.4835
P11	2.9	5	14.5	5.25	0.81	1.15	7.21	1.75	12.3585

P12	2.9	6	17.4	5.25	0.81	1.15	7.21	2.5	13.4835
P13	5	6	30	5.25	0.32	1.15	6.72	2.5	12.822
P14	5	6	30	5.25	0.32	1.15	6.72	2.5	12.822
P15	5	5	25	5.25	0.32	1.15	6.72	2.5	12.822
P16	5	5	25	5.25	0.32	1.15	6.72	2.5	12.822
P17	5	5	25	5.25	0.32	1.15	6.72	2.5	12.822
P18	5	5	25	5.25	0.32	1.15	6.72	2.5	12.822
P19	2.9	5	14.5	5.25	0.81	1.15	7.21	4.5	16.4835
P21	5	6	30	5.25	0.32	1.15	6.72	2.5	12.822
P22	5	6	30	5.25	0.32	1.15	6.72	2.5	12.822
P23	5	6	30	5.25	0.32	1.15	6.72	2.5	12.822
s2	6	6	36	5.25	0.81	1.15	7.21	2.5	13.4835
S3	5	6	30	5.25	0.32	1.15	6.72	2.5	12.822
C1	1.5	6	9	5.25	0.81	1.15	7.21	3.25	14.6085
C2	1.5	6	9	5.25	0.81	1.15	7.21	3.25	14.6085
C3	1.5	5	7.5	5.25	0.81	1.15	7.21	3.25	14.6085
C4	1.5	5	7.5	5.25	0.81	1.15	7.21	3.25	14.6085
C5	1.5	5	7.5	5.25	0.81	1.15	7.21	3.25	14.6085
C6	1.5	5	7.5	5.25	0.81	1.15	7.21	3.25	14.6085
C7	1.5	2.9	4.35	5.25	0.81	1.15	7.21	3.25	14.6085

Table A. 8 moment adjustment for 3rd floor slab

panle	Mmax,sup	Madj,sup	Lx	Ly	Ly/Lx	Cx	Cy	ΔM	Mxf	Myf	Cx ΔM	Cy ΔM	Mxf,adj	Myf,adj
p1	21.236	15.25	5	6	1.2	0.344	0.364	5.986	15.843	12.135	2.05918	2.1789	17.9022	14.3139
p2	19.463	15.93	5	5	1	0.38	0.28	3.533	14.52	11.122	1.34254	0.98924	15.8625	12.1112
p4	12.049	15.93	5	6	1.2	0.338	0.172	-3.881	9.886	7.414	-1.3118	-0.6675	8.57422	6.74647
p6	14.941	13.61	5	6	1.2	0.338	0.172	1.331	11.384	8.538	0.44988	0.22893	11.8339	8.76693
p8	12.976	12.51	5	6	1.2	0.338	0.172	0.466	9.886	7.414	0.15751	0.08015	10.0435	7.49415
p11	9.041	7.583	2.9	5	1.72414	0.285	0.074	1.458	6.755	3.741	0.41553	0.10789	7.17053	3.84889
p13	15.356	13.07	5	6	1.2	0.334	0.364	2.286	11.539	9.296	0.76352	0.8321	12.3025	10.1281
p14	13.386	14.397	5	6	1.2	0.338	0.172	-1.011	10.257	7.693	-0.3417	-0.1739	9.91528	7.51911
p18	13.386	15.228	5	6	1.2	0.344	0.364	-1.842	10.257	7.693	-0.6336	-0.6705	9.62335	7.02251
p21	13.386	11.73	5	6	1.2	0.338	0.172	1.656	10.257	7.693	0.55973	0.28483	10.8167	7.97783
p22	13.386	11.73	5	6	1.2	0.338	0.172	1.656	10.257	7.693	0.55973	0.28483	10.8167	7.97783
p23	13.386	13.93	5	6	1.2	0.338	0.172	-0.544	10.257	7.693	-0.1839	-0.0936	10.0731	7.59943
s3	17.95	16.73	5	6	1.2	0.338	0.172	1.22	13.46	9.616	0.41236	0.20984	13.8724	9.82584

axis	panel	adjusted moment			Mavg	df	adjusted moment
			ΔM	$\Delta M/M_{max}$			
on axis 2	B/n p1 & s4	15.84	3.23	0.203914		0.5	14.225
		12.61				0.5	14.225
	B/n S1 & p3	17.21	1.68	0.097618	16.37		16.37
		15.53					
	B/n p13 & p14	12.5	2.25	0.18	11.375		11.375
		10.25					

	B/n s2 & p20	15.53	0.76	0.051456	15.15		15.15
		14.77					
	B/n C1 & C2	16.434	0	0	16.434		16.434
on axis 3	B/n p3& P4	15.533	5.647	0.363549		0.5	12.7095
		9.886				0.5	12.7095
	B/n p20 & p21	14.77	4.513	0.439992		0.5	12.5135
		10.257				0.5	12.5135
	B/n C2 & C3	16.434	0	0	16.434		16.434
on axis 4	B/n p2 & p5	14.52	2.48	0.170799	13.28		13.28
		12.04					
on axis 5	B/n p6&p8	11.384	1.498	0.131588	10.635		10.635
		9.886					
on axis6	B/n p8 & p10	9.8864	3.2936	0.249894		0.5	11.5332
		13.18				0.5	11.5332
on axis 7	B/n p9 & p11	12.04	7.156	0.594352		0.63	7.53172
		4.884				0.37	7.53172
	B/n p10 & p12	13.8	36.46	0.725428		0.63	36.7698
		50.26				0.37	36.7698

axis	panel	unadjusted moment					adjusted moment
			ΔM	$\Delta M/M_{max}$	M_{avg}	df	M_{xs}
axis D	B/n p1 & S1	21.236	10.876	0.512149		0.55	15.2542
		10.36				0.45	15.2542
	B/n S4 & p3	12	3.33	0.2775		0.54	13.7982
		15.33				0.46	13.7982
	B/n p2 & P4	19.46	6.535	0.335817		0.54	15.9311
		12.925				0.46	15.9311
	B/n p5 & p6	12.049	2.892	0.22		0.54	13.61068
		14.941				0.46	13.61068
	B/n p7 & p8	12.049	0.926				
		12.975				0.071368	12.512
B/n p9 & p10	12.049	5.255	0.303687		0.54	14.8867	
	17.304				0.46	14.8867	
B/n p11 & p12	9.0419	2.6519	0.29329		0.55	7.583355	
	6.39				0.45	7.583355	
axis c	B/n s1 & p13	10.36	5.026	0.326661		0.54	13.07404
		15.386				0.46	13.07404
B/n p3 & p14	15.33	1.867	0.138676				
	13.463				14.3965		
B/n p4 & P15	12.975	2.718	0.20948		0.54	11.50728	
	10.257				0.46	11.50728	
B/n p6 & p16	14.941	4.684	0.3135		0.54	12.41164	
	10.257				0.46	12.41164	
B/n p8 & p17	12.975	2.718	0.20948		0.54	11.50728	
	10.257				0.46	11.50728	

		17.304	3.844	0.222145		0.54	15.22824
	B/n p10 & p18	13.46				0.46	15.22824
		15				0.54	12.43878
	B/n p12 & p19	10.257	4.743	0.3162		0.46	12.43878
axis B		15.386					
	B/n p13 & S2	15.5324	0.1464	0.009425	15.4592		
		13.463					
	B/n p14 & p20	14.77	1.307	0.097081	14.1165		
		10.257				0.46	11.73176
	B/n p15 & P21	13.463	3.206	0.238134		0.54	11.73176
	B/n p16 & p22	10.257	3.206	0.238134		0.46	11.73176
	simar p17-18	13.463				0.54	11.73176
		13.46				0.46	15.5254
	B/n p18 & s3	17.95	4.49	0.250139		0.54	15.5254
axis A		15.5324				0.8	16.25368
	B/n s2&C1	16.434	0.9016	0.058046		0.2	16.25368
		14.77				0.16	15.03624
	B/n p20&C2	16.434	1.664	0.112661		0.84	15.03624
		13.463				0.16	13.93836
	B/n P21&C3	16.434	2.971	0.220679		0.84	13.93836
	B/n P22&C4		2.971	0.2207			13.93
	B/n P23&C5		2.971	0.2207			13.93
		17.95				0.8	16.7372
	B/n s3&C6	16.434	1.516	0.084457		0.2	16.7372

Table A. 9 support reinforcement for 3rd floor slab

axis	location	MED	fcd	fyd	b	d	μ sd	Kz	Ascalk	Aspro	Scalc	Sprovided
on axis 2	B/n p1 & s4	14.225	11.33	347.83	1000	175	0.041	0.9699	240.946	244.707	320.792	Ø10C/C310mm
	B/n S1 & p3	16.37	11.33	347.83	1000	175	0.04718	0.9598	280.197	280.197	280.16	Ø10C/C270mm
	B/n p13 & p14	11.375	11.33	347.83	1000	175	0.03278	0.9697	192.712	197.655	397.157	Ø10C/C390mm
	B/n s2 & p20	15.15	11.33	347.83	1000	175	0.04366	0.9677	257.198	280.197	280.16	Ø10C/C270mm
	B/n C1 & C2	16.434	11.33	347.83	1000	175	0.04736	0.97	278.334	280.197	280.16	Ø10C/C270mm
on axis 3	B/n p2 & s4	14.52	11.33	347.83	1000	175	0.04185	0.9748	244.707	244.707	320.792	Ø10C/C310mm
	B/n p3& P4	12.709	11.33	347.83	1000	175	0.03663	0.9599	217.511	217.511	360.901	Ø10C/C350mm
	B/n p14 & p15	10.257	11.33	347.83	1000	175	0.02956	0.9785	172.208	197.655	397.157	Ø10C/C390mm
	B/n p20 & p21	12.513	11.33	347.83	1000	175	0.03606	0.9689	212.167	244.707	320.792	Ø10C/C310mm
	B/n C2 & C3	16.434	11.33	347.83	1000	175	0.04736	0.97	278.334	280.197	280.16	Ø10C/C270mm
on axis 4	B/n p2 & p5	13.434	11.33	347.83	1000	175	0.03872	0.9597	229.967	244.707	320.792	Ø10C/C310mm
	B/n p4 & p6	9.886	11.33	347.83	1000	175	0.02849	0.9688	167.641	197.655	397.157	Ø10C/C390mm
	B/n p15& p16	10.257	11.33	347.83	1000	175	0.02956	0.9789	172.138	197.655	397.157	Ø10C/C390mm
	B/n p21& p22	10.257	11.33	347.83	1000	175	0.02956	0.9759	172.667	197.655	397.157	Ø10C/C390mm
	B/n C3 & C4	16.434	11.33	347.83	1000	175	0.04736	0.9764	276.51	280.197	280.16	Ø10C/C270mm
on axis 6	B/n p7 & p9	12.049	11.33	347.83	1000	175	0.03473	0.9678	204.532	244.707	320.792	Ø10C/C310mm
	B/n p8 & p10	11.533	11.33	347.83	1000	175	0.03324	0.9685	195.631	197.655	397.157	Ø10C/C390mm
	B/n p17 & p18	10.257	11.33	347.83	1000	175	0.02956	0.9789	172.138	197.655	397.157	Ø10C/C390mm
	B/n p23& S3	10.257	11.33	347.83	1000	175	0.02956	0.9798	171.98	197.655	397.157	Ø10C/C390mm
	B/n C5 & C6	16.434	11.33	347.83	1000	175	0.04736	0.9764	276.51	280.197	280.16	Ø10C/C270mm
on axis 7	B/n p9 & p11	7.5317	11.33	347.83	1000	175	0.02171	0.9781	126.504	126.504	620.534	Ø10C/C400mm
	B/n p10& p12	36.769	11.33	347.83	1000	175	0.10597	0.9399	642.68	642.068	122.261	Ø10C/C120mm
	B/n p18& p19	10.257	11.33	347.83	1000	175	0.02956	0.9797	171.998	197.655	397.157	Ø10C/C390mm
	B/n C6 & C7	16.434	11.33	347.83	1000	175	0.04736	0.9764	276.51	280.197	280.16	Ø10C/C270mm

on axis 5	B/n p5 & p7	12.049	11.33	347.83	1000	175	0.03473	0.9678	204.532	244.707	320.792	Ø10C/C310mm
	B/n p6 & p8	10.635	11.33	347.83	1000	175	0.03065	0.9685	180.398	197.655	397.157	Ø10C/C390mm
	B/n p16 & p17	10.257	11.33	347.83	1000	175	0.02956	0.9789	172.138	197.655	397.157	Ø10C/C390mm
	B/n p22& p23	10.257	11.33	347.83	1000	175	0.02956	0.9759	172.667	197.655	397.157	Ø10C/C390mm
	B/n C4 & C5	16.434	11.33	347.83	1000	175	0.04736	0.9769	276.368	280.197	280.16	Ø10C/C270mm
on axis A	B/n s2&C1	16.253	11.33	347.83	1000	185	0.04191	0.9589	263.403	280.197	280.16	Ø10C/C270mm
	B/n p20&C2	15.036	11.33	347.83	1000	185	0.03878	0.9757	239.484	244.707	320.792	Ø10C/C310mm
	B/n P21&C3	13.938	11.33	347.83	1000	185	0.03594	0.9687	223.6	244.707	320.792	Ø10C/C310mm
	B/n P22&C4	13.93	11.33	347.83	1000	185	0.03592	0.9687	223.472	244.707	320.792	Ø10C/C310mm
	B/n P23&C5	13.93	11.33	347.83	1000	185	0.03592	0.9687	223.472	244.707	320.792	Ø10C/C310mm
	B/n s3&C6	16.737	11.33	347.83	1000	185	0.04316	0.9589	271.247	280.197	280.16	Ø10C/C270mm
on axis B	B/n p13 & S1	15.459	11.33	347.83	1000	185	0.03987	0.9499	252.909	280.197	280.16	Ø10C/C270mm
	B/n p14 & p20	14.117	11.33	347.83	1000	185	0.03641	0.9688	226.449	244.707	320.792	Ø10C/C310mm
	B/n p15 & P21	11.731	11.33	347.83	1000	185	0.03025	0.9771	186.577	197.655	397.157	Ø10C/C390mm
	B/n p16 & p22	11.731	11.33	347.83	1000	185	0.03025	0.9771	186.577	197.655	397.157	Ø10C/C390mm
	B/n p17 & p23	11.731	11.33	347.83	1000	185	0.03025	0.9771	186.577	197.655	397.157	Ø10C/C390mm
	B/n p18 & s3	15.525	11.33	347.83	1000	185	0.04004	0.9523	253.349	280.197	280.16	Ø10C/C270mm
on axis C	B/n s1 & p13	13.074	11.33	347.83	1000	185	0.03372	0.9772	207.915	244.707	320.792	Ø10C/C310mm
	B/n p3 & p14	14.397	11.33	347.83	1000	185	0.03713	0.9683	231.059	244.707	320.792	Ø10C/C310mm
	B/n p4 & P15	11.507	11.33	347.83	1000	185	0.02967	0.9758	183.258	197.655	397.157	Ø10C/C390mm
	B/n p6 & p16	12.411	11.33	347.83	1000	185	0.03201	0.9758	197.655	197.655	397.157	Ø10C/C390mm
	B/n p8 & p17	11.507	11.33	347.83	1000	185	0.02967	0.9758	183.258	197.655	397.157	Ø10C/C390mm
	B/n p10 & p18	15.228	11.33	347.83	1000	185	0.03927	0.9758	242.518	244.707	320.792	Ø10C/C310mm
	B/n p12 & p19	12.438	11.33	347.83	1000	185	0.03208	0.9515	203.144	244.707	320.792	Ø10C/C310mm
on axis D	B/n p1 & S1	15.254	11.33	347.83	1000	185	0.03934	0.9763	242.807	244.707	320.792	Ø10C/C310mm
	B/n S4 & p3	13.798	11.33	347.83	1000	185	0.03558	0.9488	225.997	244.707	320.792	Ø10C/C310mm
	B/n p2 & P4	15.931	11.33	347.83	1000	185	0.04108	0.9586	258.266	280.197	280.16	Ø10C/C270mm
	B/n p5 & p6	13.61	11.33	347.83	1000	185	0.0351	0.961	220.088	244.707	320.792	Ø10C/C310mm
	B/n p7 & p8	12.512	11.33	347.83	1000	185	0.03227	0.961	202.332	244.707	320.792	Ø10C/C310mm
	B/n p9 & p10	14.886	11.33	347.83	1000	185	0.03839	0.961	240.722	240.722	326.102	Ø10C/C320mm
	B/n p11 & p12	7.583	11.33	347.83	1000	185	0.01956	0.9796	120.297	120.297	652.552	Ø10C/C400mm

Table A. 10 Ribbed slab design load

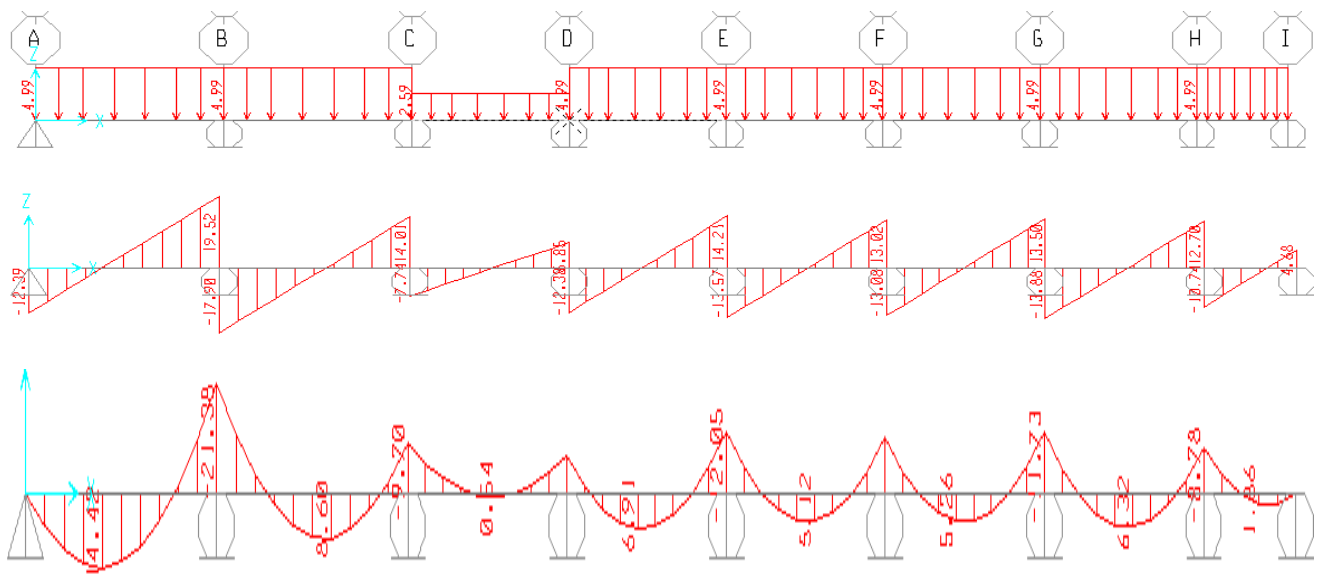
panel	DL	LL	Pd
P1	1.918	1.6	4.9893
P2	1.918	0.7	3.6393
P3	1.918	1.6	4.9893

P4	1.918	1.6	4.9893
P5	1.918	0.7	3.6393
P6	1.918	1.6	4.9893
P7	1.918	0.7	3.6393
P8	1.918	1.6	4.9893
P9	1.918	0.7	3.6393
P10	1.918	1.6	4.9893
P11	1.918	0.7	3.6393
P12	1.918	1.6	4.9893
P13	1.918	1	4.0893
P14	1.918	1	4.0893
P15	1.918	1	4.0893
P16	1.918	1	4.0893
P17	1.918	1	4.0893
P18	1.918	1	4.0893
P19	1.918	1	4.0893
P20	1.918	1	4.0893
P21	1.918	1	4.0893
P22	1.918	1	4.0893
P23	1.918	1	4.0893
C1	2	1	4.2

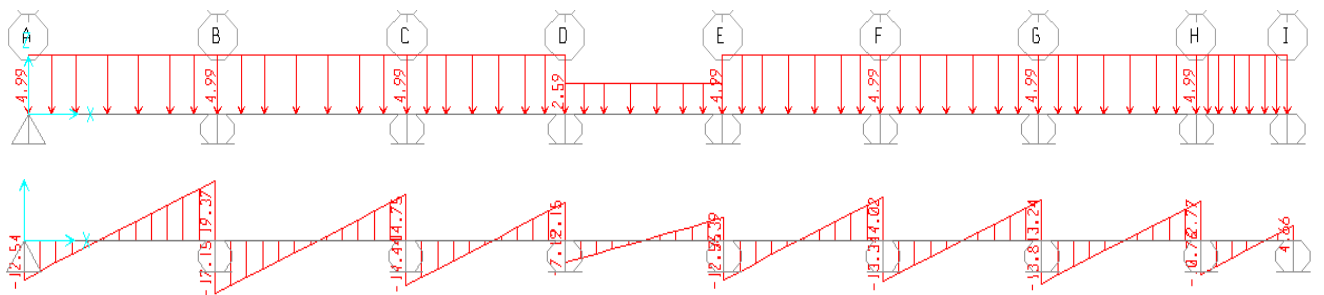
C2	2	1	4.2
C3	2	1	4.2
C4	2	1	4.2
C5	2	1	4.2
C6	2	1	4.2
C7	2	1	4.2

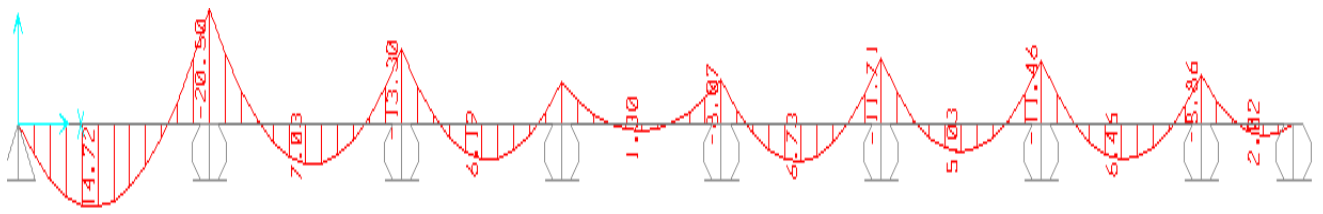
Ribbed slab analysis along x-axis

Case3

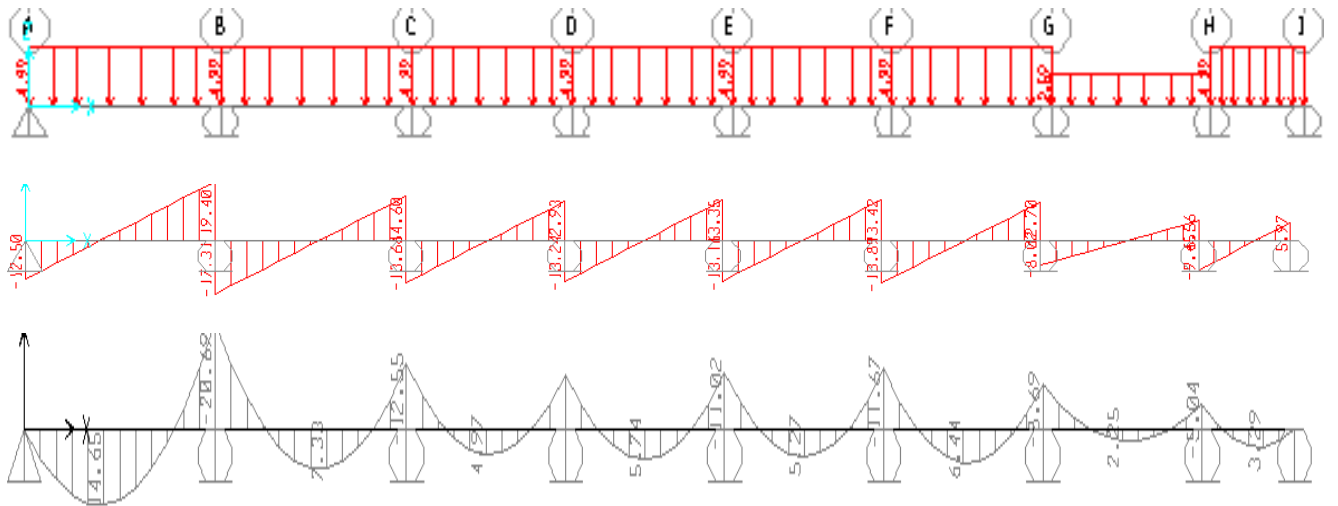


Case4





Case7



Case6

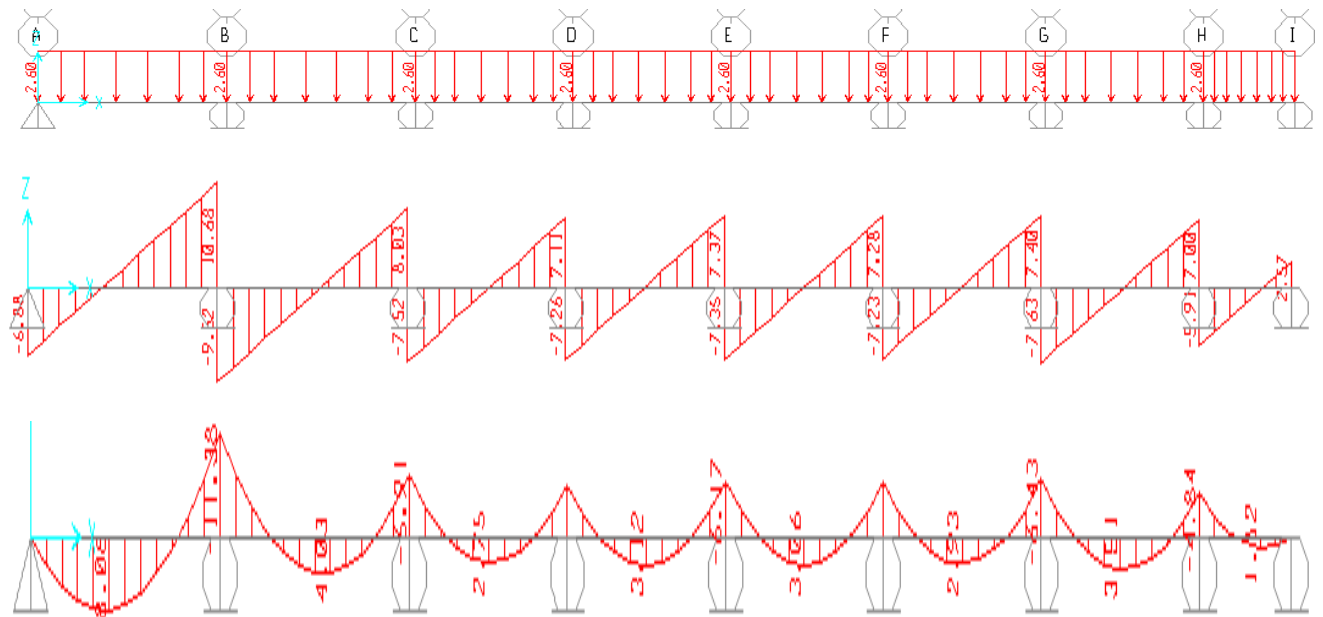


Table A. 11 mass calculation for each floor

		Weight calculation for roof									
description	un.wt	length	area	volume	weight	xi	yi	wixi	wiyi		
Truss	KN/m ³	m	m ²	m ³	KN	m	m	KNm	KNm		
1	77	23.81	0.00301	0.00000903	0.0006953	11.905	0.4	0.0082777	0.00027812		
2	77	23.81	0.00301	0.00000903	0.0006953	11.905	6.2	0.0082777	0.00431092		
3	77	23.81	0.00301	0.00000903	0.0006953	11.905	12	0.0082777	0.00834372		
4	77	23.81	0.00301	0.00000903	0.0006953	11.905	17.8	0.0082777	0.01237652		
5	77	23.81	0.00301	0.00000903	0.0006953	11.905	23.6	0.0082777	0.01640932		
6	77	23.81	0.00301	0.00000903	0.0006953	11.905	29.4	0.0082777	0.02044211		
7	77	23.81	0.00301	0.00000903	0.0006953	11.905	35.2	0.0082777	0.02447491		
				sum	0.0048672			0.0579437	0.08663563		
		purline									
p1	77	35	0.00421	0.00001263	0.0009725	0.45	17.5	0.0004376	0.01701893		
p2	77	35	0.00421	0.00001263	0.0009725	0.9095	17.5	0.0008845	0.01701893		
p3	77	35	0.00421	0.00001263	0.0009725	1.9095	17.5	0.001857	0.01701893		
p4	77	35	0.00421	0.00001263	0.0009725	2.9095	17.5	0.0028295	0.01701893		
p5	77	35	0.00421	0.00001263	0.0009725	3.9095	17.5	0.003802	0.01701893		
p6	77	35	0.00421	0.00001263	0.0009725	4.9095	17.5	0.0047745	0.01701893		
p7	77	35	0.00421	0.00001263	0.0009725	5.9095	17.5	0.005747	0.01701893		
p8	77	35	0.00421	0.00001263	0.0009725	6.9095	17.5	0.0067196	0.01701893		
p9	77	35	0.00421	0.00001263	0.0009725	7.9095	17.5	0.0076921	0.01701893		
p10	77	35	0.00421	0.00001263	0.0009725	8.9095	17.5	0.0086646	0.01701893		
p11	77	35	0.00421	0.00001263	0.0009725	9.9095	17.5	0.0096371	0.01701893		
p12	77	35	0.00421	0.00001263	0.0009725	10.9095	17.5	0.0106096	0.01701893		
p13	77	35	0.00421	0.00001263	0.0009725	11.9095	17.5	0.0115821	0.01701893		
				sum	0.0126426			0.0752373	0.22124603		
		EGA Sheet									
EGA	77	0	420.534	0.1682	12.9514	5.9735	17.6	77.365188	227.94464		
				sum	12.9514			77.36519	227.9446		
		Top tie Beam									
Beams	un.wt	length(m)	width(m)	height(m)	volume(m ³)	weight(KN)	Xi	Yi	wiXi	Wyi	
B1	25	35	0.4	0.3	4.2	105	2.361	17.5	247.905	1837.5	
B2	25	35	0.2	0.3	2.1	52.5	8.361	17.5	438.9525	918.75	
B3	25	35	0.2	0.3	2.1	52.5	13.361	17.5	701.4525	918.75	
B4	25	35	0.2	0.3	2.1	52.5	19.361	17.5	1016.4525	918.75	
B5	25	35	0.3	0.3	3.15	78.75	24.361	17.5	1918.42875	1378.125	
				sum		341.25			4323.19125	5971.875	
				overall weight		354.21891	overall moment		4400.68962	6200.12748	

mass calculation for 5th floor											
ribbed slab	length,ly	width,lx	depth(m)	volume	un.wt(KN/m)	weight(KN)	center of mass		moment		
							xi	yi	Wi*xi	Wi*yi	
p1	6	5	0.29	8.7	14	121.8	2.5	3	304.5	365.4	
p2	5	5	0.29	7.25	14	101.5	2.5	14.5	253.75	1471.75	
p3	6	6	0.29	10.44	14	146.16	8	9	1169.28	1315.44	
p4	5	6	0.29	8.7	14	121.8	8	14.5	974.4	1766.1	
p5	5	5	0.29	7.25	14	101.5	2.5	19.5	253.75	1979.25	
p6	5	6	0.29	8.7	14	121.8	8	19.5	974.4	2375.1	
p7	5	5	0.29	7.25	14	101.5	2.5	24.5	253.75	2486.75	
p8	5	6	0.29	8.7	14	121.8	8	24.5	974.4	2984.1	
p9	5	5	0.29	7.25	14	101.5	2.5	29.5	253.75	2994.25	
p10	5	6	0.29	8.7	14	121.8	8	29.5	974.4	3593.1	
p11	2.9	5	0.29	4.205	14	58.87	2.5	33.45	147.175	1969.2	
p12	2.9	6	0.29	5.046	14	70.644	8	33.45	565.152	2363.04	
p13	6	5	0.29	8.7	14	121.8	13.5	3	1644.3	365.4	
p14	6	5	0.29	8.7	14	121.8	13.5	9	1644.3	1096.2	
p15	5	5	0.29	7.25	14	101.5	13.5	14.5	1370.25	1471.75	
p16	5	5	0.29	7.25	14	101.5	13.5	19.5	1370.25	1979.25	
p17	5	5	0.29	7.25	14	101.5	13.5	24.5	1370.25	2486.75	
p18	5	5	0.29	7.25	14	101.5	13.5	29.5	1370.25	2994.25	
p19	2.9	5	0.29	4.205	14	58.87	13.5	33.45	794.745	1969.2	
p20	6	6	0.29	10.44	14	146.16	19	9	2777.04	1315.44	
p21	5	6	0.29	8.7	14	121.8	19	14.5	2314.2	1766.1	
p22	5	6	0.29	8.7	14	121.8	19	19.5	2314.2	2375.1	
p23	5	6	0.29	8.7	14	121.8	19	24.5	2314.2	2984.1	
S1	6	6	0.29	10.44	14	146.16	8	3	1169.28	438.48	
S2	6	6	0.29	10.44	14	146.16	19	3	2777.04	438.48	
S3	5	6	0.29	8.7	14	121.8	19	29.5	2314.2	3593.1	
S4	6	5	0.29	8.7	14	121.8	2.5	9	304.5	1096.2	
C1	6	1.5	0.29	2.61	14	36.54	22.75	3	831.285	109.62	
C2	6	1.5	0.29	2.61	14	36.54	22.75	9	831.285	328.86	
C3	5	1.5	0.29	2.175	14	30.45	22.75	14.5	692.738	441.525	
C4	5	1.5	0.29	2.175	14	30.45	22.75	19.5	692.738	593.775	
C5	5	1.5	0.29	2.175	14	30.45	22.75	24.5	692.738	746.025	
C6	5	1.5	0.29	2.175	14	30.45	22.75	29.5	692.738	898.275	
C7	2.9	1.5	0.29	1.2615	14	17.661	22.75	33.45	401.788	590.76	
					sum	3259.165			37783	55742.1	
					for beams						
BX1	24.35	0.4	0.3	2.922	25	73.05	12.175	0	889.384	0	
BX2	24.35	0.4	0.3	2.922	25	73.05	12.175	6	889.384	438.3	
BX3	24.35	0.4	0.3	2.922	25	73.05	12.175	12	889.384	876.6	
BX4	24.35	0.4	0.3	2.922	25	73.05	12.175	17	889.384	1241.85	
BX5	24.35	0.4	0.3	2.922	25	73.05	12.175	22	889.384	1607.1	
BX6	24.35	0.4	0.3	2.922	25	73.05	12.175	27	889.384	1972.35	
BX7	24.35	0.4	0.3	2.922	25	73.05	12.175	32	889.384	2337.6	

BX8	24.35	0.4	0.3	2.922	25	73.05	12.175	34.9	889.384	2549.45
BYD	35.25	0.4	0.3	4.23	25	105.75	5	17.625	528.75	1863.84
BYC	35.25	0.4	0.3	4.23	25	105.75	11	17.625	1163.25	1863.84
BYB	35.25	0.4	0.3	4.23	25	105.75	16	17.625	1692	1863.84
BYA	35.25	0.4	0.3	4.23	25	105.75	22	17.625	2326.5	1863.84
					sum	1007.4			12825.6	18478.6
				for columns						
C1	3.36	0.3	0.4	0.4032	25	10.08	0	0	0	0
C2	3.36	0.4	0.3	0.4032	25	10.08	0	6	0	60.48
C3	3.36	0.4	0.3	0.4032	25	10.08	0	12	0	120.96
C4	3.36	0.4	0.3	0.4032	25	10.08	0	17	0	171.36
C5	3.36	0.4	0.3	0.4032	25	10.08	0	22	0	221.76
C6	3.36	0.4	0.3	0.4032	25	10.08	0	27	0	272.16
C7	3.36	0.4	0.3	0.4032	25	10.08	0	32	0	322.56
C8	3.36	0.4	0.3	0.4032	25	10.08	0	34.9	0	351.792
C9	3.36	0.3	0.4	0.4032	25	10.08	5	0	50.4	0
C10	3.36	0.4	0.3	0.4032	25	10.08	5	6	50.4	60.48
C11	3.36	0.4	0.3	0.4032	25	10.08	5	12	50.4	120.96
C12	3.36	0.4	0.3	0.4032	25	10.08	5	17	50.4	171.36
C13	3.36	0.4	0.3	0.4032	25	10.08	5	22	50.4	221.76
C14	3.36	0.4	0.3	0.4032	25	10.08	5	27	50.4	272.16
C15	3.36	0.4	0.3	0.4032	25	10.08	5	32	50.4	322.56
C16	3.36	0.4	0.3	0.4032	25	10.08	5	34.9	50.4	351.792
C17	3.36	0.4	0.3	0.4032	25	10.08	11	0	110.88	0
C18	3.36	0.4	0.3	0.4032	25	10.08	11	6	110.88	60.48
C19	3.36	0.4	0.3	0.4032	25	10.08	11	12	110.88	120.96
C20	3.36	0.4	0.3	0.4032	25	10.08	11	17	110.88	171.36
C21	3.36	0.4	0.3	0.4032	25	10.08	11	22	110.88	221.76
C22	3.36	0.4	0.3	0.4032	25	10.08	11	27	110.88	272.16
C23	3.36	0.4	0.3	0.4032	25	10.08	11	32	110.88	322.56
C24	3.36	0.4	0.3	0.4032	25	10.08	11	34.9	110.88	351.792
C25	3.36	0.4	0.3	0.4032	25	10.08	17	0	171.36	0
C26	3.36	0.4	0.3	0.4032	25	10.08	17	6	171.36	60.48
C27	3.36	0.4	0.3	0.4032	25	10.08	17	12	171.36	120.96
C28	3.36	0.4	0.3	0.4032	25	10.08	17	17	171.36	171.36
C29	3.36	0.4	0.3	0.4032	25	10.08	17	22	171.36	221.76
C30	3.36	0.4	0.3	0.4032	25	10.08	17	27	171.36	272.16
C31	3.36	0.4	0.3	0.4032	25	10.08	17	32	171.36	322.56
C32	3.36	0.4	0.3	0.4032	25	10.08	17	34.9	171.36	351.792
C33	3.36	0.4	0.3	0.4032	25	10.08	23	0	231.84	0
C34	3.36	0.4	0.3	0.4032	25	10.08	23	6	231.84	60.48
C35	3.36	0.4	0.3	0.4032	25	10.08	23	12	231.84	120.96
C36	3.36	0.4	0.3	0.4032	25	10.08	23	17	231.84	171.36
C37	3.36	0.4	0.3	0.4032	25	10.08	23	22	231.84	221.76
C38	3.36	0.4	0.3	0.4032	25	10.08	23	27	231.84	272.16
C39	3.36	0.4	0.3	0.4032	25	10.08	23	32	231.84	322.56
C40	3.36	0.4	0.3	0.4032	25	10.08	23	34.9	231.84	351.792
					sum	403.2			4515.84	7605.36

				for stairs							
flight 1,1	3.089	1.7	0.15	0.787695	25	19.692375	3.2145	10.975	63.3011	216.124	
flight1,2	3.089	1.7	0.15	0.787695	25	19.692375	3.2145	8.975	63.3011	176.739	
landing 1,1	0.75	3.75	0.15	0.421875	25	10.546875	4.745	9.975	50.0449	105.205	
landing 1,2	1.83	3.75	0.15	1.029375	25	25.734375	0.835	9.975	21.4882	256.7	
flight 2,1	3.089	1.25	0.15	0.5791875	25	14.479688	19.3245	34.225	279.813	495.567	
flight2,2	3.089	1.25	0.15	0.5791875	25	14.479688	19.3245	32.625	279.813	472.4	
landing 2,1	1.78	3.05	0.15	0.81435	25	20.35875	16.89	33.525	343.859	682.527	
landing 2,2	1.72	3.05	0.15	0.7869	25	19.6725	21.38	33.525	420.598	659.521	
						sum	144.65663		1522.22	3064.78	
				for walls							
P1,PW1	2.05	0.2	3.07	1.2587	14	17.6218	0.575	-0.2	10.1325	-3.5244	
PW1	1.7	0.2	3.07	1.0438	14	14.6132	4.425	-0.2	64.6634	-2.9226	
S4,PW1	1.95	0.2	3.07	1.1973	14	16.7622	-0.2	6.975	-3.3524	116.916	
PW3	4.6	0.2	3.07	2.8244	14	39.5416	2.5	12	98.854	474.499	
PW5	4.6	0.15	3.07	2.1183	14	29.6562	2.5	22.075	74.1405	654.661	
PWD	8.5	0.15	3.07	3.91425	14	54.7995	5.125	415.6	280.847	22774.7	
P7,PW6	4.6	0.15	3.07	2.1183	14	29.6562	2.5	26.925	74.1405	798.493	
PWD	3.5	0.15	3.07	1.61175	14	22.5645	5.125	23.9	115.643	539.292	
P9,PW7	4.6	0.15	3.07	2.1183	14	29.6562	2.5	32.075	74.1405	951.223	
PWD	3.5	0.15	3.07	1.61175	14	22.5645	5.125	28.9	115.643	652.114	
P11,PW8	4.6	0.2	3.07	2.8244	14	39.5416	2.5	34.975	98.854	1382.97	
PW1	1.2	0.1	3.07	0.3684	14	5.1576	-0.1	33.45	-0.5158	172.522	
PW1	0.404	0.2	3.07	0.248056	14	3.472784	2.125	33.45	7.37967	116.165	
WX	5.5	0.1	3.07	1.6885	14	23.639	4.325	33.45	102.239	790.725	
WCDY	9.25	0.15	3.07	4.259625	14	59.63475	7.275	17.525	433.843	1045.1	
	5.509	0.15	3.07	2.5368945	14	35.516523	7.275	25.8045	258.383	916.486	
	5.385	0.15	3.07	2.4797925	14	34.717095	7.275	32.1575	252.567	1116.41	
WCDX1	3.45	0.15	3.07	1.588725	14	22.24215	9.104	22.075	202.493	490.995	
X2	3.45	0.15	3.07	1.588725	14	22.24215	9.104	28.484	202.493	633.545	
X2	3.6	0.2	3.07	2.2104	14	30.9456	9	34.15	278.51	1056.79	
CY1	5.7	0.15	3.07	2.62485	14	36.7479	10.875	3	399.633	110.244	
CY2	3.7	0.15	3.07	1.70385	14	23.8539	10.875	8	259.411	190.831	
CY3	4.7	0.15	3.07	2.16435	14	30.3009	10.875	14.35	329.522	434.818	
CY4	4.7	0.15	3.07	2.16435	14	30.3009	10.875	19.35	329.522	586.322	
CY5	4.7	0.15	3.07	2.16435	14	30.3009	10.875	24.35	329.522	737.827	
CY6	4.7	0.15	3.07	2.16435	14	30.3009	10.875	29.35	329.522	889.331	
CY7	2.6	0.15	3.07	1.1973	14	16.7622	10.875	33.3	182.289	558.181	
WBC1	4.6	0.2	3.07	2.8244	14	39.5416	13.5	0	533.812	0	
WBC2	3.699	0.15	3.07	1.7033895	14	23.847453	13.9505	6.075	332.684	144.873	
WBC4	4.6	0.15	3.07	2.1183	14	29.6562	13.5	16.925	400.359	501.931	
WBC5	4.6	0.15	3.07	2.1183	14	29.6562	13.5	22.075	400.359	654.661	
WBC6	4.6	0.15	3.07	2.1183	14	29.6562	13.5	27.075	400.359	802.942	
WBC7	4.6	0.2	3.07	2.8244	14	39.5416	13.5	32.075	533.812	1268.3	
WBC8	1.65	0.2	3.07	1.0131	14	14.1834	14.975	34.975	212.396	496.064	
WBA1	5.6	0.2	3.07	3.4384	14	48.1376	19	0	914.614	0	
WBA2	4.1	0.15	3.07	1.88805	14	26.4327	18.25	6.075	482.397	160.579	
WBA4	4.1	0.15	3.07	1.88805	14	26.4327	18.25	16.925	482.397	447.373	
WBA5	4.1	0.15	3.07	1.88805	14	26.4327	18.25	22.075	482.397	583.502	
WBA6	4.1	0.15	3.07	1.88805	14	26.4327	18.25	27.075	482.397	715.665	
WBA7	5.6	0.2	3.07	3.4384	14	48.1376	19	32.075	914.614	1544.01	

WBA8	5.6	0.2	3.07	3.4384	14	48.1376	19	34.975	914.614	1683.61
WB	3.55	0.15	3.07	1.634775	14	22.88685	15.875	7.975	363.329	182.523
W7-8	5.496	0.15	3.07	2.530908	14	35.432712	12.825	33.45	454.425	1185.22
					sum	1267.6586			13205.5	48555.9
			Imposed load							
pane	Ly(m)	Lx(m)	area(m2)	load	weight	center of mass		moment		
						Xi	Yi	wixi	wiyi	
p1	6	5	30	4	120	2.5	3	300	360	
p2	5	5	25	3.5	87.5	2.5	14.5	218.75	1268.75	
p3	6	6	36	4	144	8	9	1152	1296	
p4	5	6	30	4	120	8	14.5	960	1740	
p5	5	5	25	3.5	87.5	2.5	19.5	218.75	1706.25	
p6	5	6	30	4	120	8	19.5	960	2340	
p7	5	5	25	3.5	87.5	2.5	24.5	218.75	2143.75	
p8	5	6	30	4	120	8	24.5	960	2940	
p9	5	5	25	3.5	87.5	2.5	29.5	218.75	2581.25	
p10	5	6	30	4	120	8	29.5	960	3540	
p11	2.9	5	14.5	1.75	25.375	2.5	33.45	63.4375	848.794	
p12	2.9	6	17.4	7.5	130.5	8	33.45	1044	4365.23	
p13	6	5	30	3.5	105	13.5	3	1417.5	315	
p14	6	5	30	3.5	105	13.5	9	1417.5	945	
p15	5	5	25	3.5	87.5	13.5	14.5	1181.25	1268.75	
p16	5	5	25	3.5	87.5	13.5	19.5	1181.25	1706.25	
p17	5	5	25	3.5	87.5	13.5	24.5	1181.25	2143.75	
p18	5	5	25	3.5	87.5	13.5	29.5	1181.25	2581.25	
p19	2.9	5	14.5	1.75	25.375	13.5	33.45	342.563	848.794	
p20	6	6	36	3.5	126	19	9	2394	1134	
p21	5	6	30	3.5	105	19	14.5	1995	1522.5	
p22	5	6	30	3.5	105	19	19.5	1995	2047.5	
p23	5	6	30	3.5	105	19	24.5	1995	2572.5	
S1	6	6	36	4	144	8	3	1152	432	
S2	6	6	36	3.5	126	19	3	2394	378	
S3	5	6	30	3.5	105	19	29.5	1995	3097.5	
S4	6	5	30	4	120	2.5	9	300	1080	
C1	6	1.5	9	4	36	22.75	3	819	108	
C2	6	1.5	9	4	36	22.75	9	819	324	
C3	5	1.5	7.5	4	30	22.75	14.5	682.5	435	
C4	5	1.5	7.5	1.75	13.125	22.75	19.5	298.594	255.938	
C5	5	1.5	7.5	1.75	13.125	22.75	24.5	298.594	321.563	
C6	5	1.5	7.5	1.75	13.125	22.75	29.5	298.594	387.188	
C7	2.9	1.5	4.35	1.75	7.6125	22.75	33.45	173.184	254.638	
				sum	2920.2375			32786.5	49289.1	
			SHEAR WALL							
CDYW1	2.9	0.15	3.07	1.33545	25	33.38625	7.175	1.65	239.546	55.0873
CDYW2	2.9	0.15	3.07	1.33545	25	33.38625	9.725	1.65	324.681	55.0873
CDXW1	2.4	0.15	3.07	1.1052	25	27.63	8.45	0.275	233.474	7.59825
CDXW2	0.85	0.15	3.07	0.391425	25	9.785625	7.675	3.025	75.1047	29.6015
CDXW3	0.25	0.15	3.07	0.115125	25	2.878125	9.525	3.025	27.4141	8.70633
					sum	107.06625			900.22	156.081
					overall weight	9109.3839		overall moment	103539	182892

mass calculation for 3rd and 4th floor										
ribbed slab	weight			Wi*Xi	Wi*Yi					
sum	3259.17			37783	55742.1					
Columns	weight			Wi*Xi	Wi*Yi					
sum	403.2			4515.84	7605.36					
Beams	weight			Wi*Xi	Wi*Yi					
sum	1007.4			12825.6	18478.6					
stairs	weight			Wi*Xi	Wi*Yi					
sum	144.657			1522.22	3064.78					
			Imposed load							
panle	Ly(m)	Lx(m)	area(m2)	load	weight	center of mass		moment		
						Xi	Yi	wixi	wiyi	
p1	6	5	30	4	120	2.5	3	300	360	
p2	5	5	25	1.75	43.75	2.5	14.5	109.375	634.375	
p3	6	6	36	4	144	8	9	1152	1296	
p4	5	6	30	4	120	8	14.5	960	1740	
p5	5	5	25	1.75	43.75	2.5	19.5	109.375	853.125	
p6	5	6	30	4	120	8	19.5	960	2340	
p7	5	5	25	1.75	43.75	2.5	24.5	109.375	1071.88	
p8	5	6	30	4	120	8	24.5	960	2940	
p9	5	5	25	1.75	43.75	2.5	29.5	109.375	1290.63	
p10	5	6	30	4	120	8	29.5	960	3540	
p11	2.9	5	14.5	1.75	25.375	2.5	33.45	63.4375	848.794	
p12	2.9	6	17.4	4	69.6	8	33.45	556.8	2328.12	
p13	6	5	30	1.75	52.5	13.5	3	708.75	157.5	
p14	6	5	30	2.5	75	13.5	9	1012.5	675	
p15	5	5	25	2.5	62.5	13.5	14.5	843.75	906.25	
p16	5	5	25	2.5	62.5	13.5	19.5	843.75	1218.75	
p17	5	5	25	2.5	62.5	13.5	24.5	843.75	1531.25	
p18	5	5	25	2.5	62.5	13.5	29.5	843.75	1843.75	
p19	2.9	5	14.5	1.75	25.375	13.5	33.45	342.563	848.794	
p20	6	6	36	2.5	90	19	9	1710	810	
p21	5	6	30	2.5	75	19	14.5	1425	1087.5	
p22	5	6	30	2.5	75	19	19.5	1425	1462.5	
p23	5	6	30	2.5	75	19	24.5	1425	1837.5	
S2	6	6	36	1.75	63	19	3	1197	189	
S3	5	6	30	2.5	75	19	29.5	1425	2212.5	
C1	6	1.5	9	4	36	22.75	3	819	108	
C2	6	1.5	9	2.5	22.5	22.75	9	511.875	202.5	
C3	5	1.5	7.5	4	30	22.75	14.5	682.5	435	
C4	5	1.5	7.5	2.5	18.75	22.75	19.5	426.563	365.625	
C5	5	1.5	7.5	2.5	18.75	22.75	24.5	426.563	459.375	
C6	5	1.5	7.5	2.5	18.75	22.75	29.5	426.563	553.125	
C7	2.9	1.5	4.35	2.5	10.875	22.75	33.45	247.406	363.769	
				sum	2025.48			23936	36510.6	

walls	length,ly	width,lx	depth(m)	volume	un.wt(KN)	weight	xi	yi	Wi*xi	Wi*yi
Y1W1	1.025	0.2	3.07	0.62935	14	8.8109	0	0.7125	0	6.27777
X1W1	0.85	0.2	3.07	0.5219	14	7.3066	0.575	-0.1	4.2013	-0.7307
Y1W2	1.025	0.2	3.07	0.62935	14	8.8109	0	0.7125	0	6.27777
X1W2	0.85	0.2	3.07	0.5219	14	7.3066	4.275	-0.1	31.2357	-0.7307
X3W1	4.6	0.2	3.07	2.8244	14	39.5416	2.5	6.075	98.854	240.215
Y1W1	0.75	0.2	3.07	0.4605	14	6.447	-0.1	13.3275	-0.6447	85.9224
D1W1	0.7	0.15	3.07	0.32235	14	4.5129	3.775	12.5	17.0362	56.4113
3-4XW1	3.85	0.15	3.07	1.77293	14	24.821	1.925	13.625	47.7803	338.185
4W1	4.6	0.15	3.07	2.1183	14	29.6562	2.5	17.05	74.1405	505.638
DW3	3.8	0.15	3.07	1.7499	14	24.4986	5.125	14.05	125.555	344.205
DW4	3.8	0.15	3.07	1.7499	14	24.4986	5.125	19.95	125.555	488.747
DW5	3.8	0.15	3.07	1.7499	14	24.4986	5.125	24.05	125.555	589.191
DW6	3.8	0.15	3.07	1.7499	14	24.4986	5.125	29.95	125.555	733.733
5W1	4.6	0.15	3.07	2.1183	14	29.6562	2.5	22.05	74.1405	653.919
6W1	4.6	0.15	3.07	2.1183	14	29.6562	2.5	27.05	74.1405	802.2
7W1	4.6	0.15	3.07	2.1183	14	29.6562	2.5	32.075	74.1405	951.223
8W1	4.6	0.2	3.07	2.8244	14	39.5416	2.5	34.975	98.854	1382.97
4-5XW1	3.85	0.15	3.07	1.77293	14	24.821	1.925	20.375	47.7803	505.727
5-6XW1	3.85	0.15	3.07	1.77293	14	24.821	1.925	23.475	47.7803	582.672
6-7XW1	3.85	0.15	3.07	1.77293	14	24.821	1.925	30.425	47.7803	755.177
7-8XW1	4.25	0.15	3.07	1.95713	14	27.3998	2.125	33.375	58.2245	914.467
D2W2	0.7	0.15	3.07	0.32235	14	4.5129	3.775	21.65	17.0362	97.7043
D3W3	0.7	0.15	3.07	0.32235	14	4.5129	3.775	22.175	17.0362	100.074
D4W4	0.7	0.15	3.07	0.32235	14	4.5129	3.775	31.65	17.0362	142.833
Y1w2	0.65	0.2	3.07	0.3991	14	5.5874	-0.1	20.575	-0.5587	114.961
Y1W3	0.7	0.2	3.07	0.4298	14	6.0172	-0.1	23.3	-0.6017	140.201
Y1W4	0.6	0.2	3.07	0.3684	14	5.1576	-0.1	30.6	-0.5158	157.823
Y1W5	0.804	0.201	3.07	0.49612	14	6.94574	-0.1	33.45	-0.6946	232.335
1DW1	1.25	0.1	3.07	0.38375	14	5.3725	1.625	33.45	8.73031	179.71
1DW2	0.9	0.15	3.07	0.41445	14	5.8023	14.225	33.6	82.5377	194.957
1XW3	4.6	0.2	3.07	2.8244	14	39.5416	13.5	0	533.812	0
1XW4	5.6	0.2	3.07	3.4384	14	48.1376	19	0	914.614	0
2XW3	4.6	0.2	3.07	2.8244	14	39.5416	13.5	6	533.812	237.25
2XW4	3.563	0.2	3.07	2.18768	14	30.6275	17.9815	6	550.729	183.765
7XW3	4.6	0.2	3.07	2.8244	14	39.5416	13.5	32.075	533.812	1268.3
7XW4	5.6	0.2	3.07	3.4384	14	48.1376	19	32.075	914.614	1544.01
8XW4	5.6	0.2	3.07	3.4384	14	48.1376	19	34.975	914.614	1683.61
8XW3	1.65	0.2	3.07	1.0131	14	14.1834	14.975	34.975	212.396	496.064
BCXW1	3.55	0.15	3.07	1.63478	14	22.8869	12.775	33.45	292.38	765.565
BCYW1	0.846	0.15	3.07	0.38958	14	5.45416	12.625	33.423	68.8588	182.294
BCYW2	1.1	0.15	3.07	0.50655	14	7.0917	13.625	33.55	96.6244	237.927
CYW1	5.7	0.2	3.07	3.4998	14	48.9972	10.9	3.05	534.069	149.441
CYW3	4.7	0.2	3.07	2.8858	14	40.4012	10.9	14.55	440.373	587.837
CYW4	4.7	0.2	3.07	2.8858	14	40.4012	10.9	19.55	440.373	789.843
CYW5	4.7	0.2	3.07	2.8858	14	40.4012	10.9	24.55	440.373	991.849
CYW6	4.7	0.2	3.07	2.8858	14	40.4012	10.9	29.55	440.373	1193.86
CYW7	2.6	0.2	3.07	1.5964	14	22.3496	10.9	34.55	243.611	772.179

p13	6	5	0.22	6.6	25	165	13.5	3	2227.5	495
p14	6	5	0.22	6.6	25	165	13.5	9	2227.5	1485
p15	5	5	0.22	5.5	25	137.5	13.5	14.5	1856.25	1993.75
p16	5	5	0.22	5.5	25	137.5	13.5	19.5	1856.25	2681.25
p17	5	5	0.22	5.5	25	137.5	13.5	24.5	1856.25	3368.75
p18	5	5	0.22	5.5	25	137.5	13.5	29.5	1856.25	4056.25
p19	2.9	5	0.22	3.19	25	79.75	13.5	33.45	1076.63	2667.64
p20	6	6	0.22	7.92	25	198	19	9	3762	1782
p21	5	6	0.22	6.6	25	165	19	14.5	3135	2392.5
p22	5	6	0.22	6.6	25	165	19	19.5	3135	3217.5
p23	5	6	0.22	6.6	25	165	19	24.5	3135	4042.5
Sp1	1.95	6	0.22	2.574	25	64.35	5.975	3	384.491	193.05
sp2	2.9	2.9	0.22	1.8502	25	46.255	8.55	4.55	395.48	210.46
sp3	1.15	6	0.22	1.518	25	37.95	10.425	3	395.629	113.85
S2	6	6	0.22	7.92	25	198	19	3	3762	594
S3	5	6	0.22	6.6	25	165	19	29.5	3135	4867.5
S4	3.28	5	0.22	3.608	25	90.2	2.5	7.64	225.5	689.128
C1	6	1.5	0.22	1.98	25	49.5	22.75	3	1126.13	148.5
C2	6	1.5	0.22	1.98	25	49.5	22.75	9	1126.13	445.5
C3	5	1.5	0.22	1.65	25	41.25	22.75	14.5	938.438	598.125
C4	5	1.5	0.22	1.65	25	41.25	22.75	19.5	938.438	804.375
C5	5	1.5	0.22	1.65	25	41.25	22.75	24.5	938.438	1010.63
C6	5	1.5	0.22	1.65	25	41.25	22.75	29.5	938.438	1216.88
C7	2.9	1.5	0.22	0.957	25	23.925	22.75	33.45	544.294	800.291
					sum	4290.88			50588.5	74640.2
					for beams					
BX1	24.35	0.4	0.3	2.922	25	73.05	12.175	0	889.384	0
BX2	24.35	0.4	0.3	2.922	25	73.05	12.175	6	889.384	438.3
BX3	24.35	0.4	0.3	2.922	25	73.05	12.175	12	889.384	876.6
BX4	24.35	0.4	0.3	2.922	25	73.05	12.175	17	889.384	1241.85
BX5	24.35	0.4	0.3	2.922	25	73.05	12.175	22	889.384	1607.1
BX6	24.35	0.4	0.3	2.922	25	73.05	12.175	27	889.384	1972.35
BX7	24.35	0.4	0.3	2.922	25	73.05	12.175	32	889.384	2337.6
BX8	24.35	0.4	0.3	2.922	25	73.05	12.175	34.9	889.384	2549.45
BY1	35.25	0.2	0.3	2.115	25	52.875	0	17.625	0	931.922
BYD	35.25	0.4	0.3	4.23	25	105.75	5	17.625	528.75	1863.84
BYC	35.25	0.4	0.3	4.23	25	105.75	11	17.625	1163.25	1863.84
BYB	35.25	0.4	0.3	4.23	25	105.75	16	17.625	1692	1863.84
BYA	35.25	0.4	0.3	4.23	25	105.75	22	17.625	2326.5	1863.84
BY2	35.25	0.4	0.3	4.23	25	105.75	23.3	17.625	2463.98	1863.84
					sum	1166.03			15289.5	21274.4
walls	length,ly	width,lx	depth(m)	volume	un.wt(KN	weight	xi	yi	Wi*xi	Wi*yi
1XW1	0.85	0.2	3.14	0.5338	14	7.4732	0.575	-0.2	4.29709	-1.4946
1XW1,2	0.85	0.2	3.14	0.5338	14	7.4732	4.425	-0.2	33.0689	-1.4946
1XW3	4.6	0.2	3.14	2.8888	14	40.4432	13.5	0	545.983	0
1XW4	5.6	0.2	3.14	3.5168	14	49.2352	19	0	935.469	0

CYW2	5.7	0.2	3.14	3.5796	14	50.1144	10.9	8.95	546.247	448.524
CYW3	4.7	0.2	3.14	2.9516	14	41.3224	10.9	14.55	450.414	601.241
CYW4	4.7	0.2	3.14	2.9516	14	41.3224	10.9	19.55	450.414	807.853
CYW5	4.7	0.2	3.14	2.9516	14	41.3224	10.9	24.55	450.414	1014.46
CYW6	4.7	0.2	3.14	2.9516	14	41.3224	10.9	29.55	450.414	1221.08
CYW7	2.6	0.2	3.14	1.6328	14	22.8592	10.9	34.55	249.165	789.785
3XW1	4.6	0.2	3.14	2.8888	14	40.4432	2.5	12.075	101.108	488.352
3XW2	3.6	0.15	3.14	1.6956	14	23.7384	9	12.075	213.646	286.641
3XW3	4.6	0.15	3.14	2.1666	14	30.3324	13.5	12.075	409.487	366.264
3XW4	5.6	0.15	3.14	2.6376	14	36.9264	19	12.075	701.602	445.886
4XW1	4.6	0.15	3.14	2.1666	14	30.3324	2.5	17.05	75.831	517.167
4XW2	3.6	0.15	3.14	1.6956	14	23.7384	9	16.925	213.646	401.772
5XW1	4.6	0.15	3.14	2.1666	14	30.3324	2.5	22.05	75.831	668.829
5XW2	3.6	0.15	3.14	1.6956	14	23.7384	9	21.925	213.646	520.464
5XW3	4.6	0.15	3.14	2.1666	14	30.3324	13.5	22.075	409.487	669.588
5XW4	5.6	0.15	3.14	2.6376	14	36.9264	19	22.075	701.602	815.15
6XW1	4.6	0.15	3.14	2.1666	14	30.3324	2.5	27.05	75.831	820.491
6XW2	3.6	0.15	3.14	1.6956	14	23.7384	9	26.925	213.646	639.156
7XW1	4.6	0.15	3.14	2.1666	14	30.3324	2.5	32.075	75.831	972.912
7XW3	4.6	0.2	3.14	2.8888	14	40.4432	13.5	32.075	545.983	1297.22
7XW4	5.6	0.2	3.14	3.5168	14	49.2352	19	32.075	935.469	1579.22
8XW1	4.6	0.2	3.14	2.8888	14	40.4432	2.5	34.975	101.108	1414.5
8XW2,1	0.78	0.2	3.14	0.48984	14	6.85776	8.365	34.975	57.3652	239.85
8XW2,2	0.625	0.2	3.14	0.3925	14	5.495	10.4875	34.975	57.6288	192.188
8XW3	1.65	0.2	3.14	1.0362	14	14.5068	14.975	34.975	217.239	507.375
8XW4	5.6	0.2	3.14	3.5168	14	49.2352	19	34.975	935.469	1722
DYW3	3.8	0.15	3.14	1.7898	14	25.0572	5.125	14.95	128.418	374.605
DYW4	3.8	0.15	3.14	1.7898	14	25.0572	5.125	19.05	128.418	477.34
DYW5	3.8	0.15	3.14	1.7898	14	25.0572	5.125	24.95	128.418	625.177
DYW6	3.8	0.15	3.14	1.7898	14	25.0572	5.125	29.05	128.418	727.912
CD3	2.1	0.15	3.14	0.9891	14	13.8474	7.275	13.05	100.74	180.709
CD4	2.1	0.15	3.14	0.9891	14	13.8474	7.275	18.05	100.74	249.946
CD5	2.1	0.15	3.14	0.9891	14	13.8474	7.275	23.05	100.74	319.183
CD6	3.3	0.15	3.14	1.5543	14	21.7602	7.275	28.65	158.305	623.43
CD7	2.85	0.15	3.14	1.34235	14	18.7929	7.275	33.425	136.718	628.153
CDY3	0.9	0.15	3.14	0.4239	14	5.9346	9.425	16.4	55.9336	97.3274
CDY4	0.9	0.15	3.14	0.4239	14	5.9346	9.425	21.4	55.9336	127
CDY5	0.9	0.15	3.14	0.4239	14	5.9346	9.425	26.4	55.9336	156.673
CDY6	0.9	0.15	3.14	0.4239	14	5.9346	9.425	27.45	55.9336	162.905
DIY3	0.9	0.15	3.14	0.4239	14	5.9346	2.175	12.6	12.9078	74.776
DIY4	0.9	0.15	3.14	0.4239	14	5.9346	2.175	21.5	12.9078	127.594
DIY5	0.9	0.15	3.14	0.4239	14	5.9346	2.175	22.55	12.9078	133.825
DIY6	0.9	0.15	3.14	0.4239	14	5.9346	2.175	31.55	12.9078	187.237
DIY7,1	1.25	0.15	3.14	0.58875	14	8.2425	1.624	33.45	13.3858	275.712
DIY7,2	1.05	0.15	3.14	0.49455	14	6.9237	4.325	33.525	29.945	232.117
3-4XW1	2.25	0.15	3.14	1.05975	14	14.8365	1.125	13.825	16.6911	205.115
3-4XW2	2.15	0.15	3.14	1.01265	14	14.1771	8.425	15.85	119.442	224.707
4-5XW1	2.25	0.15	3.14	1.05975	14	14.8365	1.125	20.275	16.6911	300.81
4-5XW2	2.15	0.15	3.14	1.01265	14	14.1771	8.425	20.175	119.442	286.023

5-6XW1	2.25	0.15	3.14	1.05975	14	14.8365	1.125	23.775	16.6911	352.738		
5-6XW2	2.15	0.15	3.14	1.01265	14	14.1771	8.425	25.175	119.442	356.908		
6-7XW1	2.25	0.15	3.14	1.05975	14	14.8365	1.125	30.325	16.6911	449.917		
6-7XW2	2.15	0.15	3.14	1.01265	14	14.1771	8.425	28.825	119.442	408.655		
7-8XW1	4.25	0.1	3.14	1.3345	14	18.683	2.125	33.5	39.7014	625.881		
7-8XW2	3.55	0.15	3.14	1.67205	14	23.4087	12.775	33.45	299.046	783.021		
1YW1,1	1.025	0.2	3.14	0.6437	14	9.0118	0	0.7125	0	6.42091		
1YW1,2	1.025	0.2	3.14	0.6437	14	9.0118	0	5.3375	0	48.1005		
1YW2	1.95	0.2	3.14	1.2246	14	17.1444	-0.2	7.175	-3.4289	123.011		
1YW3	0.95	0.2	3.14	0.5966	14	8.3524	-0.2	13.425	-1.6705	112.131		
1YW4	0.85	0.2	3.14	0.5338	14	7.4732	-0.2	20.625	-1.4946	154.135		
1YW5	0.9	0.2	3.14	0.5652	14	7.9128	-0.2	23.4	-1.5826	185.16		
1YW6	0.8	0.2	3.14	0.5024	14	7.0336	-0.2	30.65	-1.4067	215.58		
1YW7	0.804	0.2	3.14	0.50491	14	7.06877	-0.2	33.45	-1.4138	236.45		
BCYW1	0.846	0.15	3.14	0.39847	14	5.57852	12.625	33.375	70.4289	186.183		
BCYW2	1.1	0.15	3.14	0.5181	14	7.2534	15.55	33.502	112.79	243.003		
					SUM	1429.3			12932.5	30730.5		
					SHEAR WALL							
CDYW1	2.9	0.15	3.07	1.33545	25	33.3863	7.175	1.65	239.546	55.0873		
CDYW2	2.9	0.15	3.07	1.33545	25	33.3863	9.725	1.65	324.681	55.0873		
CDXW1	2.4	0.15	3.07	1.1052	25	27.63	8.45	0.275	233.474	7.59825		
CDXW2	0.85	0.15	3.07	0.39143	25	9.78563	7.675	3.025	75.1047	29.6015		
CDXW3	0.25	0.15	3.07	0.11513	25	2.87813	9.525	3.025	27.4141	8.70633		
					sum	107.066			900.22	156.081		
					for stairs							
flight 1,1	3.089	1.7	0.15	0.7877	25	19.6924	3.2145	10.975	63.3011	216.124		
flight1,2	3.089	1.7	0.15	0.7877	25	19.6924	3.2145	8.975	63.3011	176.739		
landing 1,1	0.75	3.75	0.15	0.42188	25	10.5469	4.745	9.975	50.0449	105.205		
landing 1,2	1.83	3.75	0.15	1.02938	25	25.7344	0.835	9.975	21.4882	256.7		
flight 2,1	3.089	1.25	0.15	0.57919	25	14.4797	19.3245	34.225	279.813	495.567		
flight2,2	3.089	1.25	0.15	0.57919	25	14.4797	19.3245	32.625	279.813	472.4		
landing 2,1	1.78	3.05	0.15	0.81435	25	20.3588	16.89	33.525	343.859	682.527		
landing 2,2	1.72	3.05	0.15	0.7869	25	19.6725	21.38	33.525	420.598	659.521		
					sum	144.657			1522.22	3064.78		
					Imposed load							
panle	Ly(m)	Lx(m)	area(m2)	load	weight	center of mass		moment				
						Xi	Yi	wixi	wiyi			
p1	6	5	30	2.5	75	2.5	3	187.5	225			
p2	5	5	25	4.5	112.5	2.5	14.5	281.25	1631.25			
p3	6	6	36	3.25	117	8	9	936	1053			
p4	5	6	30	4.5	135	8	14.5	1080	1957.5			
p5	5	5	25	4.5	112.5	2.5	19.5	281.25	2193.75			
p6	5	6	30	4.5	135	8	19.5	1080	2632.5			
p7	5	5	25	4.5	112.5	2.5	24.5	281.25	2756.25			
p8	5	6	30	4.5	135	8	24.5	1080	3307.5			
p9	5	5	25	4.5	112.5	2.5	29.5	281.25	3318.75			
p10	5	6	30	4.5	135	8	29.5	1080	3982.5			
p11	2.9	5	14.5	4.5	65.25	2.5	33.45	163.125	2182.61			

C24	3.14	0.4	0.3	0.3768	25	9.42	11	34.9	103.62	328.758
C25	3.14	0.4	0.3	0.3768	25	9.42	17	0	160.14	0
C26	3.14	0.4	0.3	0.3768	25	9.42	17	6	160.14	56.52
C27	3.14	0.4	0.3	0.3768	25	9.42	17	12	160.14	113.04
C28	3.14	0.4	0.3	0.3768	25	9.42	17	17	160.14	160.14
C29	3.14	0.4	0.3	0.3768	25	9.42	17	22	160.14	207.24
C30	3.14	0.4	0.3	0.3768	25	9.42	17	27	160.14	254.34
C31	3.14	0.4	0.3	0.3768	25	9.42	17	32	160.14	301.44
C32	3.14	0.4	0.3	0.3768	25	9.42	17	34.9	160.14	328.758
C33	3.14	0.4	0.3	0.3768	25	9.42	23	0	216.66	0
C34	3.14	0.4	0.3	0.3768	25	9.42	23	6	216.66	56.52
C35	3.14	0.4	0.3	0.3768	25	9.42	23	12	216.66	113.04
C36	3.14	0.4	0.3	0.3768	25	9.42	23	17	216.66	160.14
C37	3.14	0.4	0.3	0.3768	25	9.42	23	22	216.66	207.24
C38	3.14	0.4	0.3	0.3768	25	9.42	23	27	216.66	254.34
C39	3.14	0.4	0.3	0.3768	25	9.42	23	32	216.66	301.44
C40	3.14	0.4	0.3	0.3768	25	9.42	23	34.9	216.66	328.758
					sum	376.8			4220.16	7107.39
					overall weight	10028.3	overall moment	110345	184745	

mass calculation for 1st floor											
solid slab	length,ly	width,lx	depth(m)	volume	un.wt(KN	weight(KN	center of mass		moment		
							xi	yi	Wi*xi	Wi*yi	
p1	6	5	0.22	6.6	25	165	2.5	3	412.5	495	
p2	5	5	0.22	5.5	25	137.5	2.5	14.5	343.75	1993.75	
p3	6	6	0.22	7.92	25	198	8	9	1584	1782	
p4	5	6	0.22	6.6	25	165	8	14.5	1320	2392.5	
p5	5	5	0.22	5.5	25	137.5	2.5	19.5	343.75	2681.25	
p6	5	6	0.22	6.6	25	165	8	19.5	1320	3217.5	
p7	5	5	0.22	5.5	25	137.5	2.5	24.5	343.75	3368.75	
p8	5	6	0.22	6.6	25	165	8	24.5	1320	4042.5	
p9	5	5	0.22	5.5	25	137.5	2.5	29.5	343.75	4056.25	
p10	5	6	0.22	6.6	25	165	8	29.5	1320	4867.5	
p11	2.9	5	0.22	3.19	25	79.75	2.5	33.45	199.375	2667.64	
p12	2.9	6	0.22	3.828	25	95.7	8	33.45	765.6	3201.17	
p13	6	5	0.22	6.6	25	165	13.5	3	2227.5	495	
p14	6	5	0.22	6.6	25	165	13.5	9	2227.5	1485	
p15	5	5	0.22	5.5	25	137.5	13.5	14.5	1856.25	1993.75	
p16	5	5	0.22	5.5	25	137.5	13.5	19.5	1856.25	2681.25	
p17	5	5	0.22	5.5	25	137.5	13.5	24.5	1856.25	3368.75	

p17	5	5	0.22	5.5	25	137.5	13.5	24.5	1856.25	3368.75
p18	5	5	0.22	5.5	25	137.5	13.5	29.5	1856.25	4056.25
p19	2.9	5	0.22	3.19	25	79.75	13.5	33.45	1076.63	2667.64
p20	6	6	0.22	7.92	25	198	19	9	3762	1782
p21	5	6	0.22	6.6	25	165	19	14.5	3135	2392.5
p22	5	6	0.22	6.6	25	165	19	19.5	3135	3217.5
p23	5	6	0.22	6.6	25	165	19	24.5	3135	4042.5
Sp1	1.95	6	0.22	2.574	25	64.35	5.975	3	384.491	193.05
sp2	2.9	2.9	0.22	1.8502	25	46.255	8.55	4.55	395.48	210.46
sp3	1.15	6	0.22	1.518	25	37.95	10.425	3	395.629	113.85
SP2,1	4.52	4.5	0.22	4.4748	25	111.87	18.26	3	2042.75	335.61
SP2,2	1.48	6	0.22	1.9536	25	48.84	21.26	3	1038.34	146.52
SP3,1	4.52	4.5	0.22	4.4748	25	111.87	18.26	3.675	2042.75	411.122
SP3,2	1.48	6	0.22	1.9536	25	48.84	21.26	3	1038.34	146.52
S4	3.28	5	0.22	3.608	25	90.2	2.5	29.325	225.5	2645.12
C1	6	1.5	0.22	1.98	25	49.5	22.75	3	1126.13	148.5
C7	2.9	1.5	0.22	0.957	25	23.925	22.75	33.45	544.294	800.291
					sum	4034.8			44973.8	68099
					for beams					
					sum	1166.03			15289.5	21274.4
walls	length,ly	width,lx	depth(m)	volume	un.wt(KN)	weight	xi	yi	Wi*xi	Wi*yi
1XW1	0.85	0.2	2.78	0.4726	14	6.6164	0.575	-0.2	3.80443	-1.3233
1XW1,2	0.85	0.2	2.78	0.4726	14	6.6164	4.425	-0.2	29.2776	-1.3233
1XW3	4.6	0.2	2.78	2.5576	14	35.8064	13.5	0	483.386	0
1XW4	5.6	0.2	2.78	3.1136	14	43.5904	19	0	828.218	0
CYW2	5.7	0.2	2.78	3.1692	14	44.3688	10.9	8.95	483.62	397.101
CYW3	4.7	0.2	2.78	2.6132	14	36.5848	10.9	14.55	398.774	532.309
CYW4	4.7	0.2	2.78	2.6132	14	36.5848	10.9	19.55	398.774	715.233
CYW5	4.7	0.2	2.78	2.6132	14	36.5848	10.9	24.55	398.774	898.157
CYW6	4.7	0.2	2.78	2.6132	14	36.5848	10.9	29.55	398.774	1081.08
CYW7	2.6	0.2	2.78	1.4456	14	20.2384	10.9	34.55	220.599	699.237
3XW1	4.6	0.2	2.78	2.5576	14	35.8064	2.5	12.075	89.516	432.362
3XW2,1	3.6	0.15	2.78	1.5012	14	21.0168	9	12.075	189.151	253.778
CYW3	5.7	0.2	2.78	3.1692	14	44.3688	4.9	3	217.407	133.106
CYW4	6.05	0.15	2.78	2.52285	14	35.3199	8.375	3.175	295.804	112.141
4XW1	4.6	0.15	2.78	1.9182	14	26.8548	2.5	17.05	67.137	457.874
4XW2	3.6	0.15	2.78	1.5012	14	21.0168	9	16.925	189.151	355.709
5XW1	4.6	0.15	2.78	1.9182	14	26.8548	2.5	22.05	67.137	592.148
5XW2	3.6	0.15	2.78	1.5012	14	21.0168	9	21.925	189.151	460.793
5XW3	4.6	0.15	2.78	1.9182	14	26.8548	13.5	22.075	362.54	592.82
5XW4	5.6	0.15	2.78	2.3352	14	32.6928	19	22.075	621.163	721.694
6XW1	4.6	0.15	2.78	1.9182	14	26.8548	2.5	27.05	67.137	726.422
6XW2	3.6	0.15	2.78	1.5012	14	21.0168	9	26.925	189.151	565.877
7XW1	4.6	0.15	2.78	1.9182	14	26.8548	2.5	32.075	67.137	861.368
7XW3	4.6	0.2	2.78	2.5576	14	35.8064	13.5	32.075	483.386	1148.49
7XW4	5.6	0.2	2.78	3.1136	14	43.5904	19	32.075	828.218	1398.16
8XW1	4.6	0.2	2.78	2.5576	14	35.8064	2.5	34.975	89.516	1252.33

8XW2,1	0.78	0.2	2.78	0.43368	14	6.07152	8.365	34.975	50.7883	212.351
8XW2,2	0.625	0.2	2.78	0.3475	14	4.865	10.4875	34.975	51.0217	170.153
8XW3	1.65	0.2	2.78	0.9174	14	12.8436	14.975	34.975	192.333	449.205
8XW4	5.6	0.2	2.78	3.1136	14	43.5904	19	34.975	828.218	1524.57
DYW3	3.8	0.15	2.78	1.5846	14	22.1844	5.125	14.95	113.695	331.657
DYW4	3.8	0.15	2.78	1.5846	14	22.1844	5.125	19.05	113.695	422.613
DYW5	3.8	0.15	2.78	1.5846	14	22.1844	5.125	24.95	113.695	553.501
DYW6	3.8	0.15	2.78	1.5846	14	22.1844	5.125	29.05	113.695	644.457
CD3	2.1	0.15	2.78	0.8757	14	12.2598	7.275	13.05	89.19	159.99
CD4	2.1	0.15	2.78	0.8757	14	12.2598	7.275	18.05	89.19	221.289
CD5	2.1	0.15	2.78	0.8757	14	12.2598	7.275	23.05	89.19	282.588
CD6	3.3	0.15	2.78	1.3761	14	19.2654	7.275	28.65	140.156	551.954
CD7	2.85	0.15	2.78	1.18845	14	16.6383	7.275	33.425	121.044	556.135
CDY3	0.9	0.15	2.78	0.3753	14	5.2542	9.425	16.4	49.5208	86.1689
CDY4	0.9	0.15	2.78	0.3753	14	5.2542	9.425	21.4	49.5208	112.44
CDY5	0.9	0.15	2.78	0.3753	14	5.2542	9.425	26.4	49.5208	138.711
CDY6	0.9	0.15	2.78	0.3753	14	5.2542	9.425	27.45	49.5208	144.228
DIY3	0.9	0.15	2.78	0.3753	14	5.2542	2.175	12.6	11.4279	66.2029
DIY4	0.9	0.15	2.78	0.3753	14	5.2542	2.175	21.5	11.4279	112.965
DIY5	0.9	0.15	2.78	0.3753	14	5.2542	2.175	22.55	11.4279	118.482
DIY6	0.9	0.15	2.78	0.3753	14	5.2542	2.175	31.55	11.4279	165.77
DIY7,1	1.25	0.15	2.78	0.52125	14	7.2975	1.624	33.45	11.8511	244.101
DIY7,2	1.05	0.15	2.78	0.43785	14	6.1299	4.325	33.525	26.5118	205.505
3-4XW1	2.25	0.15	2.78	0.93825	14	13.1355	1.125	13.825	14.7774	181.598
3-4XW2	2.15	0.15	2.78	0.89655	14	12.5517	8.425	15.85	105.748	198.944
4-5XW1	2.25	0.15	2.78	0.93825	14	13.1355	1.125	20.275	14.7774	266.322
4-5XW2	2.15	0.15	2.78	0.89655	14	12.5517	8.425	20.175	105.748	253.231
5-6XW1	2.25	0.15	2.78	0.93825	14	13.1355	1.125	23.775	14.7774	312.297
5-6XW2	2.15	0.15	2.78	0.89655	14	12.5517	8.425	25.175	105.748	315.989
6-7XW1	2.25	0.15	2.78	0.93825	14	13.1355	1.125	30.325	14.7774	398.334
6-7XW2	2.15	0.15	2.78	0.89655	14	12.5517	8.425	28.825	105.748	361.803
7-8XW1	4.25	0.1	2.78	1.1815	14	16.541	2.125	33.5	35.1496	554.124
7-8XW2	3.55	0.15	2.78	1.48035	14	20.7249	12.775	33.45	264.761	693.248
1YW1,1	1.025	0.2	2.78	0.5699	14	7.9786	0	0.7125	0	5.68475
1YW1,2	1.025	0.2	2.78	0.5699	14	7.9786	0	5.3375	0	42.5858
1YW2	1.95	0.2	2.78	1.0842	14	15.1788	-0.2	7.175	-3.0358	108.908
1YW3	0.95	0.2	2.78	0.5282	14	7.3948	-0.2	13.425	-1.479	99.2752
1YW4	0.85	0.2	2.78	0.4726	14	6.6164	-0.2	20.625	-1.3233	136.463
1YW5	0.9	0.2	2.78	0.5004	14	7.0056	-0.2	23.4	-1.4011	163.931
1YW6	0.8	0.2	2.78	0.4448	14	6.2272	-0.2	30.65	-1.2454	190.864
1YW7	0.804	0.2	2.78	0.44702	14	6.25834	-0.2	33.45	-1.2517	209.341
BCYW1	0.846	0.15	2.78	0.35278	14	4.93895	12.625	33.375	62.3542	164.837
BCYW2	1.1	0.15	2.78	0.4587	14	6.4218	15.55	33.502	99.859	215.143
2XW2	2.2	0.15	2.78	0.9174	14	12.8436	12.3	6.075	157.976	78.0249
1-2XW1	2.6	0.15	2.78	1.0842	14	15.1788	12.5	3.575	189.735	54.2642
1-2XW2	1.2	0.15	2.78	0.5004	14	7.0056	11.8	1.575	82.6661	11.0338

CBW1	1.4	0.15	2.78	0.5838	14	8.1732	12.075	0.85	98.6914	6.94722
CBW2	1.4	0.15	2.78	0.5838	14	8.1732	13.175	0.85	107.682	6.94722
					SUM	1336.95			11616	26890.7
			shear wall		sum	107.066			900.22	156.081
			stairs		sum	114.657			1522.22	3064.78
			Imposed load							
panle	Ly(m)	Lx(m)	area(m2)	load	weight	center of mass		moment		
						Xi	Yi	wixi	wiyi	
p1	6	5	30	2.5	75	2.5	3	187.5	225	
p2	5	5	25	4.5	112.5	2.5	14.5	281.25	1631.25	
p3	6	6	36	3.25	117	8	9	936	1053	
p4	5	6	30	4.5	135	8	14.5	1080	1957.5	
p5	5	5	25	4.5	112.5	2.5	19.5	281.25	2193.75	
p6	5	6	30	4.5	135	8	19.5	1080	2632.5	
p7	5	5	25	4.5	112.5	2.5	24.5	281.25	2756.25	
p8	5	6	30	4.5	135	8	24.5	1080	3307.5	
p9	5	5	25	4.5	112.5	2.5	29.5	281.25	3318.75	
p10	5	6	30	4.5	135	8	29.5	1080	3982.5	
p11	2.9	5	14.5	4.5	65.25	2.5	33.45	163.125	2182.61	
p12	2.9	6	17.4	7.5	130.5	8	33.45	1044	4365.23	
p13	6	5	30	2.5	75	13.5	3	1012.5	225	
p14	6	5	30	3.5	105	13.5	9	1417.5	945	
p15	5	5	25	3.5	87.5	13.5	14.5	1181.25	1268.75	
p16	5	5	25	3.5	87.5	13.5	19.5	1181.25	1706.25	
p17	5	5	25	3.5	87.5	13.5	24.5	1181.25	2143.75	
p18	5	5	25	3.5	87.5	13.5	29.5	1181.25	2581.25	
p19	2.9	5	14.5	2.5	36.25	13.5	33.45	489.375	1212.56	
p20	6	6	36	3.5	126	19	9	2394	1134	
p21	5	6	30	3.5	105	19	14.5	1995	1522.5	
p22	5	6	30	3.5	105	19	19.5	1995	2047.5	
p23	5	6	30	3.5	105	19	24.5	1995	2572.5	
Sp1	1.95	6	11.7	2.5	29.25	5.975	3	174.769	87.75	
sp2	2.9	2.9	8.41	2.5	21.025	8.55	4.55	179.764	95.6638	
sp3	1.15	6	6.9	2.5	17.25	10.425	3	179.831	51.75	
S4	3.28	5	16.4	2.5	41	2.5	7.64	102.5	313.24	
C1	6	1.5	9	1.5	13.5	22.75	3	307.125	40.5	
C7	2.9	1.5	4.35	1.5	6.525	22.75	33.45	148.444	218.261	
					SUM	2513.55		24891.4	47772.1	
			for columns							
column	length,ly	width,lx	depth(m)	volume	un.wt(KN	weight(KN	center of mass		moment	
							xi	yi	Wi*xi	Wi*yi
C1	2.78	0.3	0.4	0.3336	25	8.34	0	0	0	0
C2	2.78	0.4	0.3	0.3336	25	8.34	0	6	0	50.04
C3	2.78	0.4	0.3	0.3336	25	8.34	0	12	0	100.08
C4	2.78	0.4	0.3	0.3336	25	8.34	0	17	0	141.78
C5	2.78	0.4	0.3	0.3336	25	8.34	0	22	0	183.48

C6	2.78	0.4	0.3	0.3336	25	8.34	0	27	0	225.18
C7	2.78	0.4	0.3	0.3336	25	8.34	0	32	0	266.88
C8	2.78	0.4	0.3	0.3336	25	8.34	0	34.9	0	291.066
C9	2.78	0.3	0.4	0.3336	25	8.34	5	0	41.7	0
C10	2.78	0.4	0.3	0.3336	25	8.34	5	6	41.7	50.04
C11	2.78	0.4	0.3	0.3336	25	8.34	5	12	41.7	100.08
C12	2.78	0.4	0.3	0.3336	25	8.34	5	17	41.7	141.78
C13	2.78	0.4	0.3	0.3336	25	8.34	5	22	41.7	183.48
C14	2.78	0.4	0.3	0.3336	25	8.34	5	27	41.7	225.18
C15	2.78	0.4	0.3	0.3336	25	8.34	5	32	41.7	266.88
C16	2.78	0.4	0.3	0.3336	25	8.34	5	34.9	41.7	291.066
C17	2.78	0.4	0.3	0.3336	25	8.34	11	0	91.74	0
C18	2.78	0.4	0.3	0.3336	25	8.34	11	6	91.74	50.04
C19	2.78	0.4	0.3	0.3336	25	8.34	11	12	91.74	100.08
C20	2.78	0.4	0.3	0.3336	25	8.34	11	17	91.74	141.78
C21	2.78	0.4	0.3	0.3336	25	8.34	11	22	91.74	183.48
C22	2.78	0.4	0.3	0.3336	25	8.34	11	27	91.74	225.18
C23	2.78	0.4	0.3	0.3336	25	8.34	11	32	91.74	266.88
C24	2.78	0.4	0.3	0.3336	25	8.34	11	34.9	91.74	291.066
C25	2.78	0.4	0.3	0.3336	25	8.34	17	0	141.78	0
C26	2.78	0.4	0.3	0.3336	25	8.34	17	6	141.78	50.04
C27	2.78	0.4	0.3	0.3336	25	8.34	17	12	141.78	100.08
C28	2.78	0.4	0.3	0.3336	25	8.34	17	17	141.78	141.78
C29	2.78	0.4	0.3	0.3336	25	8.34	17	22	141.78	183.48
C30	2.78	0.4	0.3	0.3336	25	8.34	17	27	141.78	225.18
C31	2.78	0.4	0.3	0.3336	25	8.34	17	32	141.78	266.88
C32	2.78	0.4	0.3	0.3336	25	8.34	17	34.9	141.78	291.066
C33	2.78	0.4	0.3	0.3336	25	8.34	23	0	191.82	0
C34	2.78	0.4	0.3	0.3336	25	8.34	23	6	191.82	50.04
C35	2.78	0.4	0.3	0.3336	25	8.34	23	12	191.82	100.08
C36	2.78	0.4	0.3	0.3336	25	8.34	23	17	191.82	141.78
C37	2.78	0.4	0.3	0.3336	25	8.34	23	22	191.82	183.48
C38	2.78	0.4	0.3	0.3336	25	8.34	23	27	191.82	225.18
C39	2.78	0.4	0.3	0.3336	25	8.34	23	32	191.82	266.88
C40	2.78	0.4	0.3	0.3336	25	8.34	23	34.9	191.82	291.066
					sum	333.6			3736.32	6292.53
					overall weight	9606.65			102929	173550

mass calculation for ground floor										
solid slab	length,ly	width,lx	depth(m)	volume	un.wt(KN)	weight(KN)	center of mass		moment	
							xi	yi	Wi*xi	Wi*yi
p1	6	5	0.22	6.6	25	165	2.5	3	412.5	495
p2	5	5	0.22	5.5	25	137.5	2.5	14.5	343.75	1993.75
p3	6	6	0.22	7.92	25	198	8	9	1584	1782
p4	5	6	0.22	6.6	25	165	8	14.5	1320	2392.5
p5	5	5	0.22	5.5	25	137.5	2.5	19.5	343.75	2681.25
p6	5	6	0.22	6.6	25	165	8	19.5	1320	3217.5
p7	5	5	0.22	5.5	25	137.5	2.5	24.5	343.75	3368.75
p8	5	6	0.22	6.6	25	165	8	24.5	1320	4042.5
p9	5	5	0.22	5.5	25	137.5	2.5	29.5	343.75	4056.25
p10	5	6	0.22	6.6	25	165	8	29.5	1320	4867.5
p11	2.9	5	0.22	3.19	25	79.75	2.5	33.45	199.375	2667.64
p12	2.9	6	0.22	3.828	25	95.7	8	33.45	765.6	3201.17
p13	6	5	0.22	6.6	25	165	13.5	3	2227.5	495
p14	6	5	0.22	6.6	25	165	13.5	9	2227.5	1485
p15	5	5	0.22	5.5	25	137.5	13.5	14.5	1856.25	1993.75
p16	5	5	0.22	5.5	25	137.5	13.5	19.5	1856.25	2681.25
p17	5	5	0.22	5.5	25	137.5	13.5	24.5	1856.25	3368.75
p18	5	5	0.22	5.5	25	137.5	13.5	29.5	1856.25	4056.25
p19	2.9	5	0.22	3.19	25	79.75	13.5	33.45	1076.63	2667.64
p20	6	6	0.22	7.92	25	198	19	9	3762	1782
p21	5	6	0.22	6.6	25	165	19	14.5	3135	2392.5
p22	5	6	0.22	6.6	25	165	19	19.5	3135	3217.5
p23	5	6	0.22	6.6	25	165	19	24.5	3135	4042.5
Sp1	1.95	6	0.22	2.574	25	64.35	5.975	3	384.491	193.05
sp2	2.9	2.9	0.22	1.8502	25	46.255	8.55	4.55	395.48	210.46
sp3	1.15	6	0.22	1.518	25	37.95	10.425	3	395.629	113.85
SP2,1	4.52	4.5	0.22	4.4748	25	111.87	18.26	3	2042.75	335.61
SP2,2	1.48	6	0.22	1.9536	25	48.84	21.26	3	1038.34	146.52
SP3,1	4.52	4.5	0.22	4.4748	25	111.87	18.26	3.675	2042.75	411.122
SP3,2	1.48	6	0.22	1.9536	25	48.84	21.26	3	1038.34	146.52
S4	3.28	5	0.22	3.608	25	90.2	2.5	29.325	225.5	2645.12
C1	6	1.5	0.22	1.98	25	49.5	22.75	3	1126.13	148.5
C2	6	1.5	0.22	1.98	25	49.5	22.75	9	1126.13	445.5
C3	5	1.5	0.22	1.65	25	41.25	22.75	14.5	938.438	598.125
C4	5	1.5	0.22	1.65	25	41.25	22.75	19.5	938.438	804.375
C5	5	1.5	0.22	1.65	25	41.25	22.75	24.5	938.438	1010.63
C6	5	1.5	0.22	1.65	25	41.25	22.75	29.5	938.438	1216.88
C7	2.9	1.5	0.22	0.957	25	23.925	22.75	33.45	544.294	800.291
					sum	4249.3			49853.7	72174.5
			for beams		sum	1166.03			15289.5	21274.4
			shear wall		sum	107.066			900.22	156.081
			column		sum	333.6			3736.3	6292.5

walls	length,ly	width,lx	depth(m)	volume	un.wt(KN	weight	xi	yi	Wi*xi	Wi*yi
1XW1	0.85	0.2	2.78	0.4726	14	6.6164	0.575	-0.2	3.80443	-1.3233
1XW1,2	0.85	0.2	2.78	0.4726	14	6.6164	4.425	-0.2	29.2776	-1.3233
1XW3	4.6	0.2	2.78	2.5576	14	35.8064	13.5	0	483.386	0
1XW4	5.6	0.2	2.78	3.1136	14	43.5904	19	0	828.218	0
1XW5	1.3	0.2	2.78	0.7228	14	10.1192	22.85	0	231.224	0
CYW1	5.7	0.2	2.78	3.1692	14	44.3688	10.9	0	483.62	0
CYW2	5.7	0.2	2.78	3.1692	14	44.3688	10.9	8.95	483.62	397.101
CYW3	4.7	0.2	2.78	2.6132	14	36.5848	10.9	14.55	398.774	532.309
CYW4	4.7	0.2	2.78	2.6132	14	36.5848	10.9	19.55	398.774	715.233
CYW5	4.7	0.2	2.78	2.6132	14	36.5848	10.9	24.55	398.774	898.157
CYW6	4.7	0.2	2.78	2.6132	14	36.5848	10.9	29.55	398.774	1081.08
CYW7	2.6	0.2	2.78	1.4456	14	20.2384	10.9	34.55	220.599	699.237
3XW1	4.6	0.2	2.78	2.5576	14	35.8064	2.5	12.075	89.516	432.362
3XW2,1	1.07	0.15	2.78	0.44619	14	6.24666	6.735	12.075	42.0713	75.4284
3XW2,2	1.53	0.15	2.78	0.63801	14	8.93214	10.035	12.75	89.634	113.885
CYW3	5.7	0.2	2.78	3.1692	14	44.3688	4.9	3	217.407	133.106
CYW4	6.05	0.15	2.78	2.52285	14	35.3199	8.375	3.175	295.804	112.141
4XW1	4.6	0.15	2.78	1.9182	14	26.8548	2.5	15.925	67.137	427.663
4XW2	3.6	0.15	2.78	1.5012	14	21.0168	9	15.925	189.151	334.693
5XW1	4.6	0.15	2.78	1.9182	14	26.8548	2.5	19.275	67.137	517.626
5XW2	3.6	0.15	2.78	1.5012	14	21.0168	9	19.25	189.151	404.573
5XW3	4.6	0.2	2.78	2.5576	14	35.8064	13.5	22.075	483.386	790.426
5XW4	5.6	0.2	2.78	3.1136	14	43.5904	19	22.075	828.218	962.258
6XW1	4.6	0.15	2.78	1.9182	14	26.8548	2.5	25.575	67.137	686.812
6XW2	3.6	0.15	2.78	1.5012	14	21.0168	9	25.575	189.151	537.505
6-7XW1	4.6	0.15	2.78	1.9182	14	26.8548	2.5	28.645	67.137	769.256
6-7XW2	3.6	0.15	2.78	1.5012	14	21.0168	9	28.645	189.151	602.026
7XW1	4.2	0.15	2.78	1.7514	14	24.5196	2.3	32.075	56.3951	786.466
7XW2	3.6	0.15	2.78	1.5012	14	21.0168	9	32.075	189.151	674.114
7XW3	4.6	0.2	2.78	2.5576	14	35.8064	13.5	32.075	483.386	1148.49
7XW4	5.6	0.2	2.78	3.1136	14	43.5904	19	32.075	828.218	1398.16
8XW1	4.6	0.2	2.78	2.5576	14	35.8064	2.5	34.975	89.516	1252.33
8XW2	3.59	0.2	2.78	1.99604	14	27.9446	8.995	34.975	251.361	977.361
8XW3	1.65	0.2	2.78	0.9174	14	12.8436	14.975	34.975	192.333	449.205
8XW4	5.6	0.2	2.78	3.1136	14	43.5904	19	34.975	828.218	1524.57
8XW5	1.3	0.2	2.78	0.7228	14	10.1192	22.85	34.975	231.224	353.919
DYW3	2.32	0.15	2.78	0.96744	14	13.5442	5.125	14.95	69.4138	202.485
DYW4	1.56	0.15	2.78	0.65052	14	9.10728	5.125	19.05	46.6748	173.494
DYW5	2.04	0.15	2.78	0.85068	14	11.9095	5.125	24.95	61.0363	297.143
DYW6	1.91	0.15	2.78	0.79647	14	11.1506	5.125	29.05	57.1467	323.924
CD3	0.52	0.15	2.78	0.21684	14	3.03576	7.275	13.05	22.0852	39.6167
CD4	1.55	0.15	2.78	0.64635	14	9.0489	7.275	18.05	65.8307	163.333
CD5	2.17	0.15	2.78	0.90489	14	12.6685	7.275	23.05	92.163	292.008
CD6	1.91	0.15	2.78	0.79647	14	11.1506	7.275	28.65	81.1205	319.464
CD7	1.72	0.15	2.78	0.71724	14	10.0414	7.8	32.86	78.3226	329.959
D1Y7,1	1.25	0.15	2.78	0.52125	14	7.2975	1.624	33.45	11.8511	244.101

D1Y7,2	1.05	0.15	2.78	0.43785	14	6.1299	4.325	33.525	26.5118	205.505
7-8XW1	4.25	0.1	2.78	1.1815	14	16.541	2.125	33.5	35.1496	554.124
7-8XW2	3.55	0.15	2.78	1.48035	14	20.7249	12.775	33.45	264.761	693.248
1YW1	1.025	0.2	2.78	0.5699	14	7.9786	0	5.3375	0	42.5858
1YW7	0.804	0.2	2.78	0.44702	14	6.25834	-0.2	33.45	-1.2517	209.341
BCYW1	0.846	0.15	2.78	0.35278	14	4.93895	12.625	33.375	62.3542	164.837
BCYW2	1.1	0.15	2.78	0.4587	14	6.4218	15.55	33.502	99.859	215.143
2XW2	2.2	0.15	2.78	0.9174	14	12.8436	12.3	6.075	157.976	78.0249
1-2XW1	2.6	0.15	2.78	1.0842	14	15.1788	12.5	3.575	189.735	54.2642
1-2XW2	1.2	0.15	2.78	0.5004	14	7.0056	11.8	1.575	82.6661	11.0338
CBW1	1.4	0.15	2.78	0.5838	14	8.1732	12.075	0.85	98.6914	6.94722
CBW2	1.4	0.15	2.78	0.5838	14	8.1732	13.175	0.85	107.682	6.94722
						SUM	1234.18		12289.6	24412.4
			Imposed load							
panle	Ly(m)	Lx(m)	area(m2)	load	weight	center of mass		moment		
						Xi	Yi	wixi	wiyi	
p1	6	5	30	2.5	75	2.5	3	187.5	225	
p2	5	5	25	4.5	112.5	2.5	14.5	281.25	1631.25	
p3	6	6	36	3.25	117	8	9	936	1053	
p4	5	6	30	4.5	135	8	14.5	1080	1957.5	
p5	5	5	25	4.5	112.5	2.5	19.5	281.25	2193.75	
p6	5	6	30	4.5	135	8	19.5	1080	2632.5	
p7	5	5	25	4.5	112.5	2.5	24.5	281.25	2756.25	
p8	5	6	30	4.5	135	8	24.5	1080	3307.5	
p9	5	5	25	4.5	112.5	2.5	29.5	281.25	3318.75	
p10	5	6	30	4.5	135	8	29.5	1080	3982.5	
p11	2.9	5	14.5	4.5	65.25	2.5	33.45	163.125	2182.61	
p12	2.9	6	17.4	7.5	130.5	8	33.45	1044	4365.23	
p13	6	5	30	2.5	75	13.5	3	1012.5	225	
p14	6	5	30	3.5	105	13.5	9	1417.5	945	
p15	5	5	25	3.5	87.5	13.5	14.5	1181.25	1268.75	
p16	5	5	25	3.5	87.5	13.5	19.5	1181.25	1706.25	
p17	5	5	25	3.5	87.5	13.5	24.5	1181.25	2143.75	
p18	5	5	25	3.5	87.5	13.5	29.5	1181.25	2581.25	
p19	2.9	5	14.5	2.5	36.25	13.5	33.45	489.375	1212.56	
p20	6	6	36	3.5	126	19	9	2394	1134	
p21	5	6	30	3.5	105	19	14.5	1995	1522.5	
p22	5	6	30	3.5	105	19	19.5	1995	2047.5	
p23	5	6	30	3.5	105	19	24.5	1995	2572.5	
Sp1	1.95	6	11.7	2.5	29.25	5.975	3	174.769	87.75	
sp2	2.9	2.9	8.41	2.5	21.025	8.55	4.55	179.764	95.6638	
sp3	1.15	6	6.9	2.5	17.25	10.425	3	179.831	51.75	
S4	3.28	5	16.4	2.5	41	2.5	7.64	102.5	313.24	
C1	6	1.5	9	2.5	22.5	22.75	3	511.875	67.5	
C2	6	1.5	9	2.5	22.5	22.75	9	511.875	202.5	
C3	5	1.5	7.5	2.5	18.75	22.75	14.5	426.563	271.875	

C4	5	1.5	7.5	2.5	18.75	22.75	19.5	426.563	365.625	
C5	5	1.5	7.5	2.5	18.75	22.75	24.5	426.563	459.375	
C6	5	1.5	7.5	2.5	18.75	22.75	29.5	426.563	553.125	
C7	2.9	1.5	4.35	2.5	10.875	22.75	33.45	247.406	363.769	
				SUM	2624.4			27413.3	49797.1	
				for stairs						
flight 1,1	3.089	1.7	0.15	0.7877	25	19.6924	3.2145	10.975	63.3011	216.124
flight 1,2	3.089	1.7	0.15	0.7877	25	19.6924	3.2145	8.975	63.3011	176.739
landing 1,1	0.75	3.75	0.15	0.42188	25	10.5469	4.745	9.975	50.0449	105.205
landing 1,2	1.83	3.75	0.15	1.02938	25	25.7344	0.835	9.975	21.4882	256.7
flight 2,1	3.089	1.25	0.15	0.57919	25	14.4797	19.3245	34.225	279.813	495.567
flight 2,2	3.089	1.25	0.15	0.57919	25	14.4797	19.3245	32.625	279.813	472.4
landing 2,1	1.78	3.05	0.15	0.81435	25	20.3588	16.89	33.525	343.859	682.527
landing 2,2	1.72	3.05	0.15	0.7869	25	19.6725	21.38	33.525	420.598	659.521
flight 1	4.242	1.4	0.15	0.89082	25	22.2705	18.441	0.85	410.69	18.9299
flight 1	4.242	1.4	0.15	0.89082	25	22.2705	18.441	31.25	410.69	695.953
				SUM	189.198				2343.6	3779.67
				OVER ALL WEIGHT	9903.77				111826	177887

mass calculation for Basement floor											
solid slab	length,ly	width,lx	depth(m)	volume	un.wt(KN	weight(KN	center of mass		moment		
							xi	yi	Wi*xi	Wi*yi	
p1	6	5	0.22	6.6	25	165	2.5	3	412.5	495	
p2	5	5	0.22	5.5	25	137.5	2.5	14.5	343.75	1993.75	
p3	6	6	0.22	7.92	25	198	8	9	1584	1782	
p4	5	6	0.22	6.6	25	165	8	14.5	1320	2392.5	
p5	5	5	0.22	5.5	25	137.5	2.5	19.5	343.75	2681.25	
p6	5	6	0.22	6.6	25	165	8	19.5	1320	3217.5	
p7	5	5	0.22	5.5	25	137.5	2.5	24.5	343.75	3368.75	
p8	5	6	0.22	6.6	25	165	8	24.5	1320	4042.5	
p9	5	5	0.22	5.5	25	137.5	2.5	29.5	343.75	4056.25	
p10	5	6	0.22	6.6	25	165	8	29.5	1320	4867.5	
p11	2.9	5	0.22	3.19	25	79.75	2.5	33.45	199.375	2667.64	
p12	2.9	6	0.22	3.828	25	95.7	8	33.45	765.6	3201.17	
p13	6	5	0.22	6.6	25	165	13.5	3	2227.5	495	
p14	6	5	0.22	6.6	25	165	13.5	9	2227.5	1485	
p15	5	5	0.22	5.5	25	137.5	13.5	14.5	1856.25	1993.75	
p16	5	5	0.22	5.5	25	137.5	13.5	19.5	1856.25	2681.25	
p17	5	5	0.22	5.5	25	137.5	13.5	24.5	1856.25	3368.75	
p18	5	5	0.22	5.5	25	137.5	13.5	29.5	1856.25	4056.25	
p19	2.9	5	0.22	3.19	25	79.75	13.5	33.45	1076.63	2667.64	
p20	6	6	0.22	7.92	25	198	19	9	3762	1782	
p21	5	6	0.22	6.6	25	165	19	14.5	3135	2392.5	

p22	5	6	0.22	6.6	25	165	19	19.5	3135	3217.5
p23	5	6	0.22	6.6	25	165	19	24.5	3135	4042.5
Sp1	1.95	6	0.22	2.574	25	64.35	5.975	3	384.491	193.05
sp2	2.9	2.9	0.22	1.8502	25	46.255	8.55	4.55	395.48	210.46
sp3	1.15	6	0.22	1.518	25	37.95	10.425	3	395.629	113.85
SP2,1	4.52	4.5	0.22	4.4748	25	111.87	18.26	3	2042.75	335.61
SP2,2	1.48	6	0.22	1.9536	25	48.84	21.26	3	1038.34	146.52
SP3,1	4.52	4.5	0.22	4.4748	25	111.87	18.26	3.675	2042.75	411.122
SP3,2	1.48	6	0.22	1.9536	25	48.84	21.26	3	1038.34	146.52
S4	3.28	5	0.22	3.608	25	90.2	2.5	29.325	225.5	2645.12
p1	6	7.64	0.22	10.0848	25	252.12	22.75	3	5735.73	756.36
p2	6	7.64	0.22	10.0848	25	252.12	22.75	9	5735.73	2269.08
p3	5	7.64	0.22	8.404	25	210.1	22.75	14.5	4779.78	3046.45
p4	5	7.64	0.22	8.404	25	210.1	22.75	19.5	4779.78	4096.95
p5	5	7.64	0.22	8.404	25	210.1	22.75	24.5	4779.78	5147.45
p6	5	7.64	0.22	8.404	25	210.1	22.75	29.5	4779.78	6197.95
p7	2.9	7.64	0.22	4.87432	25	121.858	22.75	33.45	2772.27	4076.15
					sum	5427.87			76666.2	92740.6
			Imposed load							
panle	Ly(m)	Lx(m)	area(m2)	load	weight	center of mass		moment		
						Xi	Yi	wixi	wiyi	
p1	6	5	30	7.5	225	2.5	3	562.5	675	
p2	5	5	25	7.5	187.5	2.5	14.5	468.75	2718.75	
p3	6	6	36	7.5	270	8	9	2160	2430	
p4	5	6	30	7.5	225	8	14.5	1800	3262.5	
p5	5	5	25	7.5	187.5	2.5	19.5	468.75	3656.25	
p6	5	6	30	7.5	225	8	19.5	1800	4387.5	
p7	5	5	25	7.5	187.5	2.5	24.5	468.75	4593.75	
p8	5	6	30	7.5	225	8	24.5	1800	5512.5	
p9	5	5	25	7.5	187.5	2.5	29.5	468.75	5531.25	
p10	5	6	30	7.5	225	8	29.5	1800	6637.5	
p11	2.9	5	14.5	4.5	65.25	2.5	33.45	163.125	2182.61	
p12	2.9	6	17.4	7.5	130.5	8	33.45	1044	4365.23	
p13	6	5	30	7.5	225	13.5	3	3037.5	675	
p14	6	5	30	7.5	225	13.5	9	3037.5	2025	
p15	5	5	25	7.5	187.5	13.5	14.5	2531.25	2718.75	
p16	5	5	25	7.5	187.5	13.5	19.5	2531.25	3656.25	
p17	5	5	25	7.5	187.5	13.5	24.5	2531.25	4593.75	
p18	5	5	25	7.5	187.5	13.5	29.5	2531.25	5531.25	
p19	2.9	5	14.5	7.5	108.75	13.5	33.45	1468.13	3637.69	
p20	6	6	36	7.5	270	19	9	5130	2430	
p21	5	6	30	7.5	225	19	14.5	4275	3262.5	
p22	5	6	30	7.5	225	19	19.5	4275	4387.5	
p23	5	6	30	7.5	225	19	24.5	4275	5512.5	
Sp1	1.95	6	11.7	2.5	29.25	5.975	3	174.769	87.75	
sp2	2.9	2.9	8.41	2.5	21.025	8.55	4.55	179.764	95.6638	

sp3	1.15	6	6.9	2.5	17.25	10.425	3	179.831	51.75	
S4	3.28	5	16.4	2.5	41	2.5	7.64	102.5	313.24	
				SUM	4703.03			49264.6	84931.4	
				SHEAR WALL						
CDYW1	2.9	0.15	2.98	1.2963	25	32.4075	7.175	1.65	232.524	53.4724
CDYW2	2.9	0.15	2.98	1.2963	25	32.4075	9.725	1.65	315.163	53.4724
CDXW1	2.4	0.15	2.98	1.0728	25	26.82	8.45	0.275	226.629	7.3755
CDXW2	0.85	0.15	2.98	0.37995	25	9.49875	7.675	3.025	72.9029	28.7337
CDXW3	0.25	0.15	2.98	0.11175	25	2.79375	9.525	3.025	26.6105	8.45109
					sum	103.928			873.829	151.505
				for stairs						
flight 1,1	3.089	1.7	0.15	0.7877	25	19.6924	3.2145	10.975	63.3011	216.124
flight 1,2	3.089	1.7	0.15	0.7877	25	19.6924	3.2145	8.975	63.3011	176.739
landing 1,1	0.75	3.75	0.15	0.42188	25	10.5469	4.745	9.975	50.0449	105.205
landing 1,2	1.83	3.75	0.15	1.02938	25	25.7344	0.835	9.975	21.4882	256.7
flight2	4.54	5.5	0.15	3.7455	25	93.6375	27.29	3.1	2555.37	290.276
flight3	4.13	6.2	0.15	3.8409	25	96.0225	27.1	30.45	2602.21	2923.89
					SUM	265.326			5355.71	3968.93
				for beams						
BX1	24.35	0.4	0.2	1.948	25	48.7	12.175	0	592.923	0
BX2	24.35	0.4	0.2	1.948	25	48.7	12.175	6	592.923	292.2
BX3	24.35	0.4	0.2	1.948	25	48.7	12.175	12	592.923	584.4
BX4	24.35	0.4	0.2	1.948	25	48.7	12.175	17	592.923	827.9
BX5	24.35	0.4	0.2	1.948	25	48.7	12.175	22	592.923	1071.4
BX6	24.35	0.4	0.2	1.948	25	48.7	12.175	27	592.923	1314.9
BX7	24.35	0.4	0.2	1.948	25	48.7	12.175	32	592.923	1558.4
BX8	24.35	0.4	0.2	1.948	25	48.7	12.175	34.9	592.923	1699.63
BY1	35.25	0.2	0.2	1.41	25	35.25	0	17.625	0	621.281
BYD	35.25	0.4	0.2	2.82	25	70.5	5	17.625	352.5	1242.56
BYC	35.25	0.4	0.2	2.82	25	70.5	11	17.625	775.5	1242.56
BYB	35.25	0.4	0.2	2.82	25	70.5	16	17.625	1128	1242.56
BYA	35.25	0.4	0.2	2.82	25	70.5	22	17.625	1551	1242.56
BY2	35.25	0.4	0.2	2.82	25	70.5	23.3	17.625	1642.65	1242.56
					sum	777.35			10193	14182.9
				for columns						
column	length,ly	width,lx	depth(m)	volume	un.wt(KN	weight(KN	center of mass		moment	
							xi	yi	Wi*xi	Wi*yi
C1	2.98	0.3	0.4	0.3576	25	8.94	0	0	0	0
C2	2.98	0.4	0.3	0.3576	25	8.94	0	6	0	53.64
C3	2.98	0.4	0.3	0.3576	25	8.94	0	12	0	107.28
C4	2.98	0.4	0.3	0.3576	25	8.94	0	17	0	151.98
C5	2.98	0.4	0.3	0.3576	25	8.94	0	22	0	196.68
C6	2.98	0.4	0.3	0.3576	25	8.94	0	27	0	241.38
C7	2.98	0.4	0.3	0.3576	25	8.94	0	32	0	286.08
C8	2.98	0.4	0.3	0.3576	25	8.94	0	34.9	0	312.006
C9	2.98	0.3	0.4	0.3576	25	8.94	5	0	44.7	0

C10	2.98	0.4	0.3	0.3576	25	8.94	5	6	44.7	53.64
C11	2.98	0.4	0.3	0.3576	25	8.94	5	12	44.7	107.28
C12	2.98	0.4	0.3	0.3576	25	8.94	5	17	44.7	151.98
C13	2.98	0.4	0.3	0.3576	25	8.94	5	22	44.7	196.68
C14	2.98	0.4	0.3	0.3576	25	8.94	5	27	44.7	241.38
C15	2.98	0.4	0.3	0.3576	25	8.94	5	32	44.7	286.08
C16	2.98	0.4	0.3	0.3576	25	8.94	5	34.9	44.7	312.006
C17	2.98	0.4	0.3	0.3576	25	8.94	11	0	98.34	0
C18	2.98	0.4	0.3	0.3576	25	8.94	11	6	98.34	53.64
C19	2.98	0.4	0.3	0.3576	25	8.94	11	12	98.34	107.28
C20	2.98	0.4	0.3	0.3576	25	8.94	11	17	98.34	151.98
C21	2.98	0.4	0.3	0.3576	25	8.94	11	22	98.34	196.68
C22	2.98	0.4	0.3	0.3576	25	8.94	11	27	98.34	241.38
C23	2.98	0.4	0.3	0.3576	25	8.94	11	32	98.34	286.08
C24	2.98	0.4	0.3	0.3576	25	8.94	11	34.9	98.34	312.006
C25	2.98	0.4	0.3	0.3576	25	8.94	17	0	151.98	0
C26	2.98	0.4	0.3	0.3576	25	8.94	17	6	151.98	53.64
C27	2.98	0.4	0.3	0.3576	25	8.94	17	12	151.98	107.28
C28	2.98	0.4	0.3	0.3576	25	8.94	17	17	151.98	151.98
C29	2.98	0.4	0.3	0.3576	25	8.94	17	22	151.98	196.68
C30	2.98	0.4	0.3	0.3576	25	8.94	17	27	151.98	241.38
C31	2.98	0.4	0.3	0.3576	25	8.94	17	32	151.98	286.08
C32	2.98	0.4	0.3	0.3576	25	8.94	17	34.9	151.98	312.006
C33	2.98	0.4	0.3	0.3576	25	8.94	23	0	205.62	0
C34	2.98	0.4	0.3	0.3576	25	8.94	23	6	205.62	53.64
C35	2.98	0.4	0.3	0.3576	25	8.94	23	12	205.62	107.28
C36	2.98	0.4	0.3	0.3576	25	8.94	23	17	205.62	151.98
C37	2.98	0.4	0.3	0.3576	25	8.94	23	22	205.62	196.68
C38	2.98	0.4	0.3	0.3576	25	8.94	23	27	205.62	241.38
C39	2.98	0.4	0.3	0.3576	25	8.94	23	32	205.62	286.08
C40	2.98	0.4	0.3	0.3576	25	8.94	23	34.9	205.62	312.006
					sum	357.6			4005.12	6745.23
walls	length,ly	width,lx	depth(m)	volume	un.wt(KN	weight	xi	yi	Wi*xi	Wi*yi
1XW1	0.85	0.2	2.98	0.5066	14	7.0924	0.575	-0.2	4.07813	-1.4185
1XW1,2	0.85	0.2	2.98	0.5066	14	7.0924	4.425	-0.2	31.3839	-1.4185
1XW3	4.6	0.2	2.98	2.7416	14	38.3824	13.5	0	518.162	0
1XW4	5.6	0.2	2.98	3.3376	14	46.7264	19	0	887.802	0
1XW5	7.44	0.5	2.98	11.0856	14	155.198	25.72	0	3991.7	0
3XW1	4.6	0.2	2.98	2.7416	14	38.3824	2.5	12.075	95.956	463.467
7XW1	4.6	0.15	2.98	2.0562	14	28.7868	2.5	32.075	71.967	923.337
8XW1	4.6	0.2	2.98	2.7416	14	38.3824	2.5	34.975	95.956	1342.42
8XW2	5.6	0.2	2.98	3.3376	14	46.7264	8	34.975	373.811	1634.26
8XW3	4.6	0.2	2.98	2.7416	14	38.3824	13.5	34.975	518.162	1342.42
8XW4	5.6	0.2	2.98	3.3376	14	46.7264	19	34.975	887.802	1634.26

8XW5	7.44	0.5	2.98	11.0856	14	155.198	25.72	34.975	3991.7	5428.06
D1Y7,1	1.25	0.15	2.98	0.55875	14	7.8225	1.624	33.45	12.7037	261.663
D1Y7,2	1.05	0.15	2.98	0.46935	14	6.5709	4.325	33.525	28.4191	220.289
7-8XW1	4.25	0.1	2.98	1.2665	14	17.731	2.125	33.5	37.6784	593.989
1YW1	1.025	0.2	2.98	0.6109	14	8.5526	0	5.3375	0	45.6495
1YW7	0.804	0.2	2.98	0.47918	14	6.70858	-0.2	33.45	-1.3417	224.402
BW1	5.6	0.15	2.98	2.5032	14	35.0448	16.125	2.95	565.097	103.382
2W3	3.7	0.15	2.98	1.6539	14	23.1546	13.05	6.075	302.168	140.664
2W2	1.15	0.15	2.98	0.51405	14	7.1967	10.225	6.075	73.5863	43.72
CDWY	2.9	0.15	2.98	1.2963	14	18.1482	9.725	4.55	176.491	82.5743
					SUM	778.007			12663.3	14481.7
					OVER ALL WEIGHT	12413.11			159021.8	217202.3

Table A. 12 beam reinforcement

beam on axis A to H												
beam ID	b(mm)	d(mm)	fcd	moment(KNm)	usds	Kz	As,min	As,max	As,cal	As,prov.	Nreq.	Npro.
B1,Span	400	252	11.33	20.32	0.1	0.951	144.14	4032	243.767	243.76	1.21301	2Ø20mm
supp	400	252	11.33	24.66	0.1	0.946	144.14	4032	297.395	297.395	1.47987	2Ø20mm
B2,span	400	252	11.33	28.88	0.1	0.94	144.14	4032	350.511	350.51	1.74418	2Ø20mm
supp	400	252	11.33	13.76	0	0.961	144.14	4032	163.353	163.35	0.81286	2Ø16mm
B3,span	400	252	11.33	7.8	0	0.974	144.14	4032	91.3625	91.36	0.45463	2Ø16mm
supp	400	252	11.33	22.22	0.1	0.949	144.14	4032	267.122	267.122	1.32923	2Ø20mm
B4,span	400	252	11.33	9.34	0	0.968	144.14	4032	110.079	144.14	0.54777	2Ø16mm
supp	400	252	11.33	21.78	0.1	0.978	144.14	4032	254.173	254.17	1.26479	2Ø20mm
B5,span	400	252	11.33	10.32	0	0.978	144.14	4032	120.385	144.14	0.59905	2Ø16mm
supp	400	252	11.33	19.37	0.1	0.959	144.14	4032	230.456	230.45	1.14678	2Ø20mm
B6,span	400	252	11.33	13.82	0	0.976	144.14	4032	161.544	161.54	0.80386	2Ø16mm
supp	400	252	11.33	17.78	0.1	0.98	144.14	4032	207.048	207.048	1.03029	2Ø20mm
B7span	400	252	11.33	2.55	0	0.979	144.14	4032	29.7312	144.14	0.14795	2Ø16mm
supp	400	252	11.33	9.64	0	0.969	144.14	4032	113.509	144.14	0.56483	2Ø16mm
B1,Span	400	252	11.33	18.71	0.1	0.978	144.14	4032	218.167	218.167	1.08563	2Ø20mm
supp	400	252	11.33	33.09	0.1	0.98	144.14	4032	385.254	385.25	1.91707	2Ø20mm
B2,span	400	252	11.33	13.7	0	0.978	144.14	4032	159.797	159.79	0.79517	2Ø16mm
supp	400	252	11.33	21.92	0.1	0.969	144.14	4032	258.104	258.103	1.28435	2Ø20mm
B3,span	400	252	11.33	8.08	0	0.978	144.14	4032	94.2166	144.14	0.46883	2Ø16mm
supp	400	252	11.33	22.24	0.1	0.98	144.14	4032	258.932	258.93	1.28847	2Ø20mm
B4,span	400	252	11.33	9.42	0	0.978	144.14	4032	109.875	144.14	0.54675	2Ø16mm
supp	400	252	11.33	21.78	0.1	0.969	144.14	4032	256.455	256.4551	1.27615	2Ø20mm
B5,span	400	252	11.33	10.34	0	0.978	144.14	4032	120.606	144.14	0.60015	2Ø16mm

supp	400	252	11.33	19.09	0.1	0.978	144.14	4032	222.598	222.5983	1.10767	2Ø20mm
B6,span	400	252	11.33	13.39	0	0.977	144.14	4032	156.389	156.3894	0.77821	2Ø16mm
supp	400	252	11.33	11.7	0	0.975	144.14	4032	136.847	144.14	0.68097	2Ø16mm
B7span	400	252	11.33	2.35	0	0.979	144.14	4032	27.3965	144.14	0.13633	2Ø16mm
supp	400	252	11.33	12.87	0	0.97	144.14	4032	151.385	151.3854	0.75331	2Ø16mm
B1,Span	400	252	11.33	50	0.2	0.983	144.14	4032	580.413	580.4132	2.8882	3Ø20mm
supp	400	252	11.33	77.27	0.3	0.989	144.14	4032	891.438	891.4376	4.4359	5Ø20mm
B2,span	400	252	11.33	40	0.1	0.977	144.14	4032	467.326	467.3262	2.32547	3Ø20mm
supp	400	252	11.33	73.5	0.3	0.978	144.14	4032	857.132	857.132	4.26519	5Ø20mm
B3,span	400	252	11.33	24.12	0.1	0.978	144.14	4032	281.366	281.3655	1.40011	2Ø20mm
supp	400	252	11.33	54.42	0.2	0.979	144.14	4032	634.368	634.3682	3.15669	4Ø20mm
B4,span	400	252	11.33	27.23	0.1	0.988	144.14	4032	314.557	314.5567	1.56527	2Ø20mm
supp	400	252	11.33	53.38	0.2	0.983	144.14	4032	619.586	619.5861	3.08313	4Ø20mm
B5,span	400	252	11.33	28.66	0.1	0.988	144.14	4032	331.076	331.0759	1.64747	2Ø20mm
supp	400	252	11.33	52.45	0.2	0.983	144.14	4032	608.791	608.7915	3.02942	4Ø20mm
B6,span	400	252	11.33	33.91	0.1	0.988	144.14	4032	391.723	391.7231	1.94926	2Ø20mm
supp	400	252	11.33	48.91	0.2	0.983	144.14	4032	567.702	567.7024	2.82495	3Ø20mm
B7span	400	252	11.33	5.41	0	0.982	144.14	4032	62.8583	144.14	0.31279	2Ø16mm
supp	400	252	11.33	18.02	0.1	0.979	144.14	4032	210.014	210.0143	1.04506	2Ø20mm
B1,Span	400	252	11.33	44.37	0.2	0.977	144.14	4032	518.275	518.2755	2.579	3Ø20mm
supp	400	252	11.33	68.08	0.2	0.987	144.14	4032	787.327	787.3265	3.91783	4Ø20mm
B2,span	400	252	11.33	34.95	0.1	0.979	144.14	4032	407.325	407.3252	2.0269	3Ø20mm
supp	400	252	11.33	47.67	0.2	0.977	144.14	4032	556.651	556.651	2.76996	3Ø20mm
B3,span	400	252	11.33	21.47	0.1	0.975	144.14	4032	251.249	251.249	1.25024	2Ø20mm
supp	400	252	11.33	47.9	0.2	0.972	144.14	4032	562.272	562.2719	2.79793	3Ø20mm
B4,span	400	252	11.33	24.74	0.1	0.975	144.14	4032	289.516	289.5157	1.44066	2Ø20mm
supp	400	252	11.33	46.3	0.2	0.972	144.14	4032	543.49	543.4903	2.70447	3Ø20mm
B5,span	400	252	11.33	24.3	0.1	0.975	144.14	4032	284.367	284.3666	1.41504	2Ø20mm
supp	400	252	11.33	44.08	0.2	0.972	144.14	4032	517.431	571.431	2.5748	3Ø20mm
B6,span	400	252	11.33	30.58	0.1	0.975	144.14	4032	357.857	357.8573	1.78074	2Ø20mm
supp	400	252	11.33	7.1	0	0.972	144.14	4032	83.343	144.14	0.41472	2Ø16mm
B7span	400	252	11.33	2.19	0	0.975	144.14	4032	25.6281	144.14	0.12753	2Ø16mm
supp	400	252	11.33	13.82	0	0.972	144.14	4032	162.225	162.2254	0.80725	2Ø16mm
B1,Span	400	252	11.33	30.84	0.1	0.978	144.14	4032	359.903	359.9031	1.79092	2Ø20mm
supp	400	252	11.33	67.31	0.2	0.978	144.14	4032	785.187	785.1871	3.90718	4Ø20mm
B2,span	400	252	11.33	44.63	0.2	0.959	144.14	4032	530.99	530.9896	2.64226	3Ø20mm
supp	400	252	11.33	52.33	0.2	0.976	144.14	4032	611.693	61106925	3.04385	4Ø20mm
B3,span	400	252	11.33	23.31	0.1	0.98	144.14	4032	271.445	271.4448	1.35074	2Ø20mm
supp	400	252	11.33	48.77	0.2	0.979	144.14	4032	568.623	568.6229	2.82953	3Ø20mm
B4,span	400	252	11.33	26.37	0.1	0.969	144.14	4032	310.501	310.5014	1.54509	2Ø20mm
supp	400	252	11.33	52.33	0.2	0.978	144.14	4032	610.192	610.1923	3.03639	4Ø20mm

B5,span	400	252	11.33	30.388	0.1	0.98	144.14	4032	353.796	353.7959	1.76053	2Ø20mm
supp	400	252	11.33	39.83	0.1	0.978	144.14	4032	464.579	464.5789	2.3118	3Ø20mm
B6,span	400	252	11.33	20.8	0.1	0.969	144.14	4032	244.916	244.9158	1.21873	2Ø20mm
supp	400	252	11.33	9.13	0	0.978	144.14	4032	106.46	144.14	0.52976	2Ø16mm
B7span	400	252	11.33	5.98	0	0.98	144.14	4032	69.6229	144.14	0.34645	2Ø16mm
supp	400	252	11.33	18.19	0.1	0.978	144.14	4032	212.169	212.169	1.05578	2Ø20mm
B1,Span	400	252	11.33	50.71	0.2	0.969	144.14	4032	597.1	597.1	2.97124	3Ø20mm
supp	400	252	11.33	64.48	0.2	0.978	144.14	4032	752.098	752.0976	3.74252	4Ø20mm
B2,span	400	252	11.33	34.59	0.1	0.978	144.14	4032	403.336	403.3356	2.00704	3Ø20mm
supp	400	252	11.33	54.14	0.2	0.977	144.14	4032	632.332	632.3318	3.14656	4Ø20mm
B3,span	400	252	11.33	20.03	0.1	0.975	144.14	4032	234.278	234.2775	1.16579	2Ø20mm
supp	400	252	11.33	40.53	0.1	0.979	144.14	4032	472.502	472.5022	2.35123	3Ø20mm
B4,span	400	252	11.33	22.08	0.1	0.97	144.14	4032	259.719	259.7195	1.29239	2Ø20mm
supp	400	252	11.33	40.67	0.1	0.983	144.14	4032	472.108	472.1081	2.34926	3Ø20mm
B5,span	400	252	11.33	22.87	0.1	0.989	144.14	4032	263.843	263.8434	1.31291	2Ø20mm
supp	400	252	11.33	35.9	0.1	0.977	144.14	4032	419.425	419.4253	2.08711	3Ø20mm
B6,span	400	252	11.33	30.34	0.1	0.978	144.14	4032	353.815	353.8148	1.76062	2Ø20mm
supp	400	252	11.33	10.8	0	0.978	144.14	4032	125.985	144.14	0.62691	2Ø16mm
B7span	400	252	11.33	8.98	0	0.979	144.14	4032	104.679	144.14	0.52089	2Ø16mm
supp	400	252	11.33	10	0	0.988	144.14	4032	115.518	144.14	0.57483	2Ø16mm
GB1,Span	400	252	11.33	22.9	0.1	0.983	144.14	4032	265.802	265.8022	1.32266	2Ø20mm
supp	400	252	11.33	37.16	0.1	0.988	144.14	4032	429.267	429.2665	2.13608	3Ø20mm
GB2,span	400	252	11.33	22.29	0.1	0.983	144.14	4032	258.722	258.7219	1.28743	2Ø20mm
supp	400	252	11.33	24.96	0.1	0.988	144.14	4032	288.334	288.334	1.43478	2Ø20mm
GB3,span	400	252	11.33	15.2	0.1	0.983	144.14	4032	176.428	176.4277	0.87792	2Ø16mm
supp	400	252	11.33	24.92	0.1	0.982	144.14	4032	289.543	289.5431	1.4408	2Ø20mm
GB4,span	400	252	11.33	15.44	0.1	0.979	144.14	4032	179.946	179.9457	0.89543	2Ø16mm
supp	400	252	11.33	24.36	0.1	0.977	144.14	4032	284.543	284.5434	1.41592	2Ø20mm
GB5,span	400	252	11.33	15.33	0.1	0.987	144.14	4032	177.287	177.2872	0.8822	2Ø16mm
supp	400	252	11.33	23.73	0.1	0.979	144.14	4032	276.562	276.5616	1.3762	2Ø20mm
GB6,span	400	252	11.33	15.99	0.1	0.977	144.14	4032	186.718	186.7181	0.92913	2Ø16mm
supp	400	252	11.33	8.03	0	0.975	144.14	4032	93.9697	144.14	0.4676	2Ø16mm
GB7span	400	252	11.33	4.52	0	0.972	144.14	4032	53.0578	144.14	0.26402	2Ø16mm
supp	400	252	11.33	8.03	0	0.975	144.14	4032	93.9697	144.14	0.4676	2Ø16mm

Table A. 13 design of column

16.220 16.204 16.65 16.870		44.502	45.672	0.54811	0.023601184	0.02422	0
		49.98194519	50.9017351	0.54811	0.026507418	0.02700	0

Not slender	Not slender	49.98194519	50.9017351	0.54811	0.026507418	0.02700	0
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Combo 2=minimum

Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω
16.220 15.03	16.204 15.391		61.7838	62.3338	0.70171	0.032766412		0.03306
			65.02116562	65.04715697	0.70171	0.034483316		0.03450
Slender	Slender		65.02116562	65.04715697	0.70171	0.034483316		0.03450

R reinforcement calculation	
$\omega=$	0 0
$A_{s,tot}=$	0 Revise

Combo 3

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω
$\lambda=$	16.220			228.7476	140.4376	0.34941	0.12131397	0.07448	0.2
$\lambda_{min}=$	16.204 31.60 70.343			228.7476	140.4376	0.34941	0.12131397	0.07448	0.2
Remark	Not slender	Not slender		228.7476	140.4376	0.34941	0.12131397	0.07448	0.2
Reinforcement calculation					Tie reinforcement				
			$\omega=$	0.2	7		$\emptyset[mm]=$	8	
			$A_{s,tot}=$	1971.26949	use8#20		Spacing=	400mm	
			Remark	Ok!					

Edge column 1b

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω
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λ =	16.220 16.204			133.9928		290.4428	0.77724	0.071061723	0.15403	0.35
λ_{min} =	40.68 47.164			133.9928		290.4428	0.77724	0.071061723	0.15403	0.35
Remark	Not slender	Not slender		133.9928		290.4428	0.77724	0.071061723	0.15403	0.35
				Reinforcement calculation				Tie reinforcement		
			ω =	0.35 3449.721607		11 use12#20		\emptyset [mm]=	8 400mm	
			As,tot=					Spacing=		
			Remark	Ok!						

1c

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω	
λ =	16.220 16.204 42.93			128.207	288.707	0.65815	0.067993282	0.15311	0.45	
λ_{min} =	51.253			128.207	288.707	0.65815	0.067993282	0.15311	0.45	
Remark	Not slender	Not slender		128.207	288.707	0.65815	0.067993282	0.15311	0.45	
				Reinforcement calculation				Tie reinforcement		
			ω =	0.45 4435.356352		15 use16#20		\emptyset [mm]=	8 400mm	
			As,tot=					Spacing=		

1d

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω	
λ =	16.220 16.204 45.57			121.22	289.82	0.63034	0.064287798	0.15370	0.4	
λ_{min} =	52.372			121.22	289.82	0.63034	0.064287798	0.15370	0.4	

Remark	Not slender	Not slender		121.22		289.82	0.63034	0.064287798	0.15370	0.4
				Reinforcement calculation			Tie reinforcement			
			ω =	0.4	13			\emptyset [mm]=	8	
			As,tot=	3942.53898	use14#20			Spacing=	400mm	

3a

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω	
λ =	16.220	16.204		116.8868	305.1268	0.86072	0.061989729	0.16182	0.45	
λ_{min} =	42.60	44.818		119.1441261	305.1268	0.86072	0.063186879	0.16182	0.45	
Remark	Not slender	Not slender		119.1441261	305.1268	0.86072	0.063186879	0.16182	0.45	
				Reinforcement calculation			Tie reinforcement			
			ω =	0.45	15			\emptyset [mm]=	8	
			As,tot=	4435.356352	use16#20			Spacing=	400mm	

Story 2

Eage

2a

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω
λ =	16.220	16.204		132.068	244.568	0.87022	0.070040925	0.12970	0.4
λ_{min} =	11.59	20.485		140.0853125	244.568	0.87022	0.074292825	0.12970	0.4

Remark	Slender	Not slender		140.0853125	244.568	0.87022	0.074292825	0.12970	0.4
				Reinforcement calculation			Tie reinforcement		
			ω =	0.4	13		\emptyset [mm]=	8	
			As,tot=	3942.53898	use14#20		Spacing=	400mm	

3a

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω	Kr	1/rx	1/ry	e2,x	e2,y	
λ =	20.769	20.742		132.068		244.568	1.052	0.09322471	0.17264	0.5	0.40	3.49029 E-06	3.49029 E-06	3.143908	3.1358586
λ_{min} =	10.53	18.623		141.3435348		244.568	1.052	0.099771907	0.17264	0.5	0.40	3.49029 E-06	3.49029 E-06	3.143908	3.1358586
Remark	Slender	Slender		141.3435348		244.568	1.052	0.099771907	0.17264	0.5	0.40	3.49029 E-06	3.49029 E-06	3.143908	3.1358586
				Reinforcement calculation			Tie reinforcement								
			ω =	0.5	13		\emptyset [mm]=	8							
			As,tot=	4072.870847	use14#20		Spacing=	400mm							
			Remark	Ok!											

1b

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω
λ =	20.769	20.742 36.01		101.56		222.03	0.88518	0.071689412	0.5
λ_{min} =	44.195			101.56		222.03	0.88518	0.071689412	0.5
Remark	Not slender	Not slender		101.56		222.03	0.88518	0.071689412	0.5

				Reinforcement calculation				Tie reinforcement			
			ω =	0.5	13			\emptyset [mm]=	8		
			A_s ,tot=	4072.870847	use14#20			Spacing=	400mm		

Corner

1a

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	ψ ed,x	ψ ed,y	ω	
λ =	20.769	20.742		132.986	224.056	0.74569	0.093872471	0.15816	0.5	
λ min=	47.59	21.167		142.5341487	224.056	0.74569	0.10061234	0.15816	0.5	
Remark	Not slender	Not slender		142.5341487	224.056	0.74569	0.10061234	0.15816	0.5	
				Reinforcement calculation				Tie reinforcement		
			ω =	0.5	13			\emptyset [mm]=	8	
			A_s ,tot=	4072.870847	use14#20			Spacing=	400mm	
			Remark	Ok!						

Interior

3e

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	ψ ed,x	ψ ed,y	ω	
λ =	19.946	19.924		107.748	237.148	0.92535	0.057143059	0.12577	0.35	
λ min=	16.45	43.225		107.748	237.148	0.92535	0.057143059	0.12577	0.35	
Remark	Slender	Not slender		107.748	237.148	0.92535	0.057143059	0.12577	0.35	
				Reinforcement calculation				Tie reinforcement		

			ω =	0.35	11	\emptyset [mm]=	8
			A_s ,tot=	3449.721607	use 12#20	Spacing=	400mm

3d

check slenderness			Design moment, MEd											
	Axis x-x	Axis y-y	axis x-x	axis y-y	Ved	$\varphi_{ed,x}$	$\varphi_{ed,y}$	ω	Kr	1/rx	1/ry	e2,x	e2,y	
λ =	19.946	19.924	115.782	209.222	0.73275	0.06140381	0.11096	0.1	0.52465	4.05529E-06	4.05529E-06	4.07661	4.0676199	
λ_{min} =	23.734	23.734	119.6068518	209.222	0.73275	0.063432281	0.11096	0.35	0.64974	5.0222E-06	5.0222E-06	5.048604	5.0374705	
Remark	Slender	Not slender	122.0485983	209.222	0.73275	0.064727236	0.11096	0.1	0.52465	4.05529E-06	4.05529E-06	4.07661	4.0676199	
			Reinforcement calculation				Tie reinforcement							
			ω =	0.1	4	\emptyset [mm]=	8							
			A_s ,tot=	985.6347449	use 4#20	Spacing=	400mm							
			Remark	Ok!										

3c

	Axis x-x	Axis y-y	axis x-x	axis y-y	Ved	$\varphi_{ed,x}$	$\varphi_{ed,y}$	ω		
λ =	19.946	19.924	115.658	220.088	0.78111	0.061338047	0.11672	0.3		
λ_{min} =	40.61	47.047	115.658	220.088	0.78111	0.061338047	0.11672	0.3		
Remark	Not slender	Not slender	115.658	220.088	0.78111	0.061338047	0.11672	0.3		
			Reinforcement calculation				Tie reinforcement			
			ω =	0.3	10	\emptyset [mm]=	8			
			A_s ,tot=	2956.904235	use 10#20	Spacing=	400mm			

2b

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\Psi_{ed,x}$	$\Psi_{ed,y}$	ω	
$\lambda=$	19.946 19.924 39.27			100.14		239.59	0.78989	0.053108234	0.12706	0.3
$\lambda_{min}=$	21.826			100.14		239.59	0.78989	0.053108234	0.12706	0.3
Remark	Not slender	Not slender		100.14		239.59	0.78989	0.053108234	0.12706	0.3
				Reinforcement calculation			Tie reinforcement			
			$\omega=$	0.3	10		\emptyset [mm]=	8		
			As,tot=	2956.904235	use10#20		Spacing=	400mm		

3a and 2c

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\Psi_{ed,x}$	$\Psi_{ed,y}$	ω	
$\lambda=$	19.946 19.924			105.368		253.048	0.92535	0.05588085	0.13420	0.45
$\lambda_{min}=$	41.66 20.026			113.5569116		253.048	0.92535	0.060223757	0.13420	0.45
Remark	Not slender	Not slender		113.5569116		253.048	0.92535	0.060223757	0.13420	0.45
				Reinforcement calculation			Tie reinforcement			
			$\omega=$	0.45	15		\emptyset [mm]=	8		
			As,tot=	4435.356352	use16#20		Spacing=	400mm		

Story 3

Interior

3b

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\varphi_{ed,x}$	$\varphi_{ed,y}$	ω	Kr	1/rx	1/ry	e2,x	e2,y	
$\lambda =$	17.638 13.88	17.620 13.247		75.51	128.81	0.66373	0.040045963	0.06831	0.1	0.62324	4.81735E-06	4.81735E-06	3.78675	3.7789615	
$\lambda_{min} =$				83.08674895	137.3690269	0.66373	0.044064215	0.07285	0.1	0.62324	4.81735E-06	4.81735E-06	3.78675	3.7789615	
Remark	Slender	Slender		83.08674895	137.3690269	0.66373	0.044064215	0.07285	0.1	0.62324	4.81735E-06	4.81735E-06	3.78675	3.7789615	
				Reinforcement calculation			Tie reinforcement								
			$\omega =$	0.1	4		$\varnothing [mm] =$	8							
				985.6347449	use4#20			400mm							
			$A_s, tot =$				Spacing =								
			Remark	Ok!											

3e

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\varphi_{ed,x}$	$\varphi_{ed,y}$	ω	Kr	1/rx	1/ry	e2,x	e2,y	
$\lambda =$	17.638 12.49	17.620 13.194		98.776	157.276	0.74491	0.05238485	0.08341	0.3	0.61677	4.76733E-06	4.76733E-06	3.747432	3.7397242	
$\lambda_{min} =$				108.3461907	164.3465078	0.74491	0.057460304	0.08716	0.3	0.61677	4.76733E-06	4.76733E-06	3.747432	3.7397242	
Remark	Slender	Slender		108.3461907	164.3465078	0.74491	0.057460304	0.08716	0.3	0.61677	4.76733E-06	4.76733E-06	3.747432	3.7397242	
				Reinforcement calculation			Tie reinforcement								
			$\omega =$	0.3	10		$\varnothing [mm] =$	8							
				2956.904235	use10#20			400mm							
			$A_s, tot =$				Spacing =								

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\Psi_{ed,x}$	$\Psi_{ed,y}$		
$\lambda=$	18.424	18.401 42.81		98.774	160.074	0.90131	0.069722824	0.11299		
$\lambda_{min}=$	11.770			104.8351141	166.874471	0.90131	0.074001257	0.11779		
Remark	Not slender	Slender		104.8351141	166.874471	0.90131	0.074001257	0.11779		
			Reinforcement calculation				Tie reinforcement			
			$\omega=$	0.5	13		\emptyset [mm]=	8		
			$A_{s,tot}=$	4072.870847	use14#20		Spacing=	400mm		

3e, 3b,3c, 3d, 2b,2c

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\Psi_{ed,x}$	$\Psi_{ed,y}$	ω	
$\lambda=$	18.424	18.401		109.432	199.632	0.79645	0.077246118	0.14092	0.4	
$\lambda_{min}=$	45.50	46.591		114.9631605	199.632	0.79645	0.081150466	0.14092	0.4	
Remark	Not slender	Not slender		114.9631605	199.632	0.79645	0.081150466	0.14092	0.4	
			Reinforcement calculation				Tie reinforcement			
			$\omega=$	0.4	11		\emptyset [mm]=	8		
			$A_{s,tot}=$	3258.296677	use12#20		Spacing=	400mm		

Corner 1a

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\Psi_{ed,x}$	$\Psi_{ed,y}$	ω
$\lambda=$	18.424	18.401		79.579	114.379	0.61022	0.056173412	0.08074	0.35
$\lambda_{min}=$	17.86	53.228		81.21511729	114.379	0.61022	0.057328318	0.08074	0.35
Remark	Slender	Not slender		81.21511729	114.379	0.61022	0.057328318	0.08074	0.35

				Reinforcement calculation			Tie reinforcement			
			ω =	0.35	10		\emptyset [mm]=	8		
			As,tot=	2851.009593	use10#20		Spacing=	400mm		

Story 4

Corner 1a 4f

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω	
λ =	18.424			116.628	179.718	0.4902	0.082325647	0.12686	0.35	
	18.401 19.97									
λ_{min} =	59.388			116.628	179.718	0.4902	0.082325647	0.12686	0.35	
Remark	Not slender	Not slender		116.628	179.718	0.4902	0.082325647	0.12686	0.35	
				Reinforcement calculation			Tie reinforcement			
			ω =	0.35	10		\emptyset [mm]=	8		

Eadge

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω	
λ =	18.424			128.058	175.408	0.69191	0.090393882	0.12382	0.4	
	18.401 20.83									
λ_{min} =	49.987			128.058	175.408	0.69191	0.090393882	0.12382	0.4	
Remark	Not slender	Not slender		128.058	175.408	0.69191	0.090393882	0.12382	0.4	
				Reinforcement calculation			Tie reinforcement			
			ω =	0.4	11		\emptyset [mm]=	8		
			As,tot=	3258.296677	use12#20		Spacing=	400mm		

Interior

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω
$\lambda=$	18.424	18.401 18.74		128.172	247.772	0.74598	0.090474353	0.17490	0.5
$\lambda_{min}=$	48.142			128.172	247.772	0.74598	0.090474353	0.17490	0.5
Remark	Not slender	Not slender		128.172	247.772	0.74598	0.090474353	0.17490	0.5
				Reinforcement calculation			Tie reinforcement		
			$\omega=$	0.5	13		$\emptyset[\text{mm}]=$	8	
				4072.870847	use14#20			400mm	
			$A_{s,tot}=$				Spacing=		

Story 5

Edge

	Axis x-x	Axis y-y		axis x-x	axis y-y	Ved	$\psi_{ed,x}$	$\psi_{ed,y}$	ω
$\lambda=$	19.159	19.133		90.6	147.7	0.64924	0.087726943	0.14302	0.3
	13.72	21.523							
$\lambda_{min}=$				96.42353344	147.7	0.64924	0.093365803	0.14302	0.3
Remark	Slender	Not slender		96.42353344	147.7	0.64924	0.093365803	0.14302	0.3
				Reinforcement calculation			Tie reinforcement		
			$\omega=$	0.3	5		$\emptyset[\text{mm}]=$	8	
				1979.415232	use5#20			400mm	
			$A_{s,tot}=$				Spacing=		

Corner

	Axis x-x	Axis y-y		axis x-x	axis y-y		Ved	$\Psi_{ed,x}$	$\Psi_{ed,y}$	ω
$\lambda =$	19.159	19.133 21.76		103.336		114.936	0.44523	0.100059066	0.11129	0.2
$\lambda_{min} =$	23.586			103.336		114.936	0.44523	0.100059066	0.11129	0.2
Remark	Not slender	Not slender		103.336		114.936	0.44523	0.100059066	0.11129	0.2
				Reinforcement calculation				Tie reinforcement		
			$\omega =$	0.2	4			$\text{\O}[\text{mm}] =$	8	
				1319.610154	Use4#20				400mm	
			$A_{s,tot} =$					Spacing =		

Interior

	Axis x-x	Axis y-y		axis x-x	axis y-y		Ved	$\Psi_{ed,x}$	$\Psi_{ed,y}$	ω
$\lambda =$	19.159	19.133		106.234		162.834	0.69791	0.102865166	0.15767	0.55
	14.08	49.772								
$\lambda_{min} =$				110.6338898		162.834	0.69791	0.107125529	0.15767	0.55
Remark	Slender	Not slender		110.6338898		162.834	0.69791	0.107125529	0.15767	0.55
				Reinforcement calculation				Tie reinforcement		
			$\omega =$	0.55	4			$\text{\O}[\text{mm}] =$	8	
				3628.927925	use4#20				400mm	
			$A_{s,tot} =$					Spacing =		

Table A. 14 foundation reaction and moment

support reaction at the base				
Story	Point	FZ	MX	MY
BASE	1	1728	6	8
BASE	2	3037	4	0
BASE	3	2920	3	-1
BASE	4	2663	3	1
BASE	5	2691	3	1
BASE	7	2573	4	2
BASE	8	2021	5	-3
BASE	9	1276	8	0
BASE	10	2274	1	4
BASE	11	4498	-2	1
BASE	12	4288	-1	2
BASE	13	3909	-1	3
BASE	14	3928	-1	3
BASE	15	3749	-3	4
BASE	16	2822	-3	2
BASE	17	1530	2	1
BASE	18	1605	-4	1
BASE	19	1672	2	1

BASE	20	1039	-8	-1
BASE	21	3757	1	-1
BASE	22	4231	-2	3
BASE	23	3908	-2	3
BASE	24	3929	-2	3
BASE	25	3967	-2	3
BASE	26	3304	-3	2
BASE	27	2885	-1	4
BASE	28	3813	12	-1
BASE	29	4136	-3	3
BASE	30	4174	-3	3
BASE	31	4203	-3	3
BASE	32	3485	-2	2
BASE	33	1347	-8	6
BASE	34	513	4	6
BASE	35	1567	-6	7
BASE	36	2057	-2	0
BASE	37	2143	-1	0
BASE	38	2339	-1	2
BASE	39	2323	-1	1
BASE	41	2333	-1	1

BASE	42	1835	-3	-2
BASE	63	440	0	0
BASE	64	1040	0	0
BASE	65	462	0	0
BASE	66	1122	0	0
BASE	67	90	0	0
BASE	68	37	0	0