Civil Engineering Department
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Structural design of G+3 mixed use building using EBCS 2015

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Executive Summary

This project is about the structural design and analysis of G+3 mixed use building considering all the effects of the building receives in its service year. Structural design involves proportioning of structural members of the building and providing reinforcement which results economical.

The roof is composed of a flat RC roof the design deals with analysis of the external forces that the roof is exposed. Then, we have selected an adequate strength against the applied wind, dead and live loads that they are subjected to. Our buildings floors are all constructed with a solid RC slab. We have designed the slab for serviceability and ultimate limit state. Therefore, we have selected a depth which satisfies deflection requirement and provided flexural and shear reinforcement. The circulation system in our building is facilitated by a u-shaped stair type. It was designed by analyzing the stair for all loads it is subjected to through its service life and provided with adequate reinforcement.

The lateral load analysis includes both wind and earthquake. It consists of lateral load analysis which are wind and earthquake load analysis by ETABS 2016 by using several load combinations and also frames were analyzed using ETABS 2016.

After generating the analysis result, we designed beams and columns by assuming a preliminary cross section and finally checking with respect to the maximum shear and flexural moments. Finally, we have provided shear and flexural reinforcements. We have also designed a shear wall for wall and also, we have designed isolated footing type is selected for our foundation. The design was made by taking the loads coming from the super structure through columns. Finally, we generated detail drawings for slab reinforcement, beam reinforcement, column reinforcement and footing reinforcement. Throughout the whole process, different software will be used such as: MS – office (word, excel,) Analysis & design software (ETABS 2016)

Acknowledgment

First and for most we would like to express our gratitude to Wolkite university department of civil engineering for giving us this chance, which enables us to change the theoretical knowledge in to practical understanding.

We would like to thank our advisers MR. Yossef Gemessa and MR. Yohanis for supervision, advice and guidance from the very early stage of this project as well as telling us lot of experiences throughout the work

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Acronyms

A Accidental action
A Cross sectional area
A_c Cross sectional area of concrete
A_p Area of a prestressing tendon or tendons
A_s Cross-sectional area of reinforcement
$A_{s,min}$ minimum cross sectional area of reinforcement
A_{sw} Cross sectional area of shear reinforcement
$E_{c,eff}$ Effective modulus of elasticity of concrete
E_{cd} Design value of modulus of elasticity of concrete
E_{cm} Secant modulus of elasticity of concrete
$E_c(t)$ Tangent modulus of elasticity of normal weight concrete at a stress of $\sigma_c = 0$ and at time t
F Action
F_d Design value of an action
F_k Characteristic value of an action
G_k Characteristic permanent action
I Second moment of area of concrete section
L Length
M Bending moment
M_{Ed} Design value of the applied internal bending moment
N Axial force
N_{Ed} Design value of the applied axial force (tension or compression)
P Prestressing force
P_0 Initial force at the active end of the tendon immediately after stressing
Q_k Characteristic variable action
R Resistance
ULS Ultimate limit state
V Shear force
V_{Ed} Design value of the applied shear force

Latin lower-case letters

b Overall width of a cross-section, or actual flange width in a T or L beam
b_w Width of the web on T, I or L beams
d Diameter; Depth
d Effective depth of a cross-section
d_g Largest nominal maximum aggregate size
e Eccentricity
f_c Compressive strength of concrete
f_{cd} Design value of concrete compressive strength
f_{ck} Characteristic compressive cylinder strength of concrete at 28 days
f_{cm} Mean value of concrete cylinder compressive strength
f_{ctk} Characteristic axial tensile strength of concrete
f_{ctm} Mean value of axial tensile strength of concrete
f_p Tensile strength of prestressing steel
f_{pk} Characteristic tensile strength of prestressing steel
f_t Tensile strength of reinforcement
f_{tk} Characteristic tensile strength of reinforcement
f_y Yield strength of reinforcement

f_{yd}	Design yield strength of reinforcement
f_{yk}	Characteristic yield strength of reinforcement
f_{ywd}	Design yield of shear reinforcement
h	Height
k	Coefficient; Factor
l	(or l or L) Length; Span
r	Radius
1/r	Curvature at a particular section
t	Thickness
t	Time being considered
u	Perimeter of concrete cross-section, having area Ac
u,v,w	Components of the displacement of a point
x	Neutral axis depth
x,y,z	Coordinates
z	Lever arm of internal forces

Greek lower-case letters

α	Angle; ratio
β	Angle; ratio; coefficient
γ	Partial factor
γ_A	Partial factor for accidental actions A
γ_C	Partial factor for concrete
γ_G	Partial factor for permanent actions, G
γ_Q	Partial factor for variable actions, Q
γ_S	Partial factor for reinforcing or prestressing steel
δ	Increment/redistribution ratio
ε_c	Compressive strain in the concrete
ε_{c1}	Compressive strain in the concrete at the peak stress f_c
ε_{cu}	Ultimate compressive strain in the concrete
ε_u	Strain of reinforcement or prestressing steel at maximum load
ε_{uk}	Characteristic strain of reinforcement or prestressing steel at maximum load
θ	Angle
λ	Slenderness ratio
μ	Coefficient of friction between the tendons and their ducts
ν	Poisson's ratio
ν	Strength reduction factor for concrete cracked in shear
ρ	Oven-dry density of concrete in kg/m^3
ρ_l	Reinforcement ratio for longitudinal reinforcement
ρ_w	Reinforcement ratio for shear reinforcement
σ_c	Compressive stress in the concrete
σ_{cp}	Compressive stress in the concrete from axial load or prestressing
σ_{cu}	Compressive stress in the concrete at the ultimate compressive strain ε_{cu}
τ	Torsional shear stress
ϕ	Diameter of a reinforcing bar or of a prestressing duct
ϕ_n	Equivalent diameter of a bundle of reinforcing bars
$\varphi(t,t_0)$	Creep coefficient, defining creep between times t and t_0, related to elastic deformation at 28 days
$\varphi(\infty,t_0)$	Final value of creep coefficient

ψ Factors defining representative values of variable actions

ψ_0 for combination values

ψ_1 for frequent values

ψ_2 for quasi-permanent value

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. I 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.

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 casualty. It must extend the time for evacuation of people inside a building. This requirement of structural design is accomplished by the principle called ductility. Ductility allows yielding of steel reinforcement prior to the collapse of the building. Yielding of steel bars warns the start of failure of a structure or its part.

1.2. Background of the project

Our project have G+3 it is mixed used building that is around 224.4 m^2 area, Ground floor used for pharmacy, bank, store and office, first floor used for store, café and restaurant, second floor used for supermarket, shops (6 shops) and store and the third floor used for bed rooms (have 6 bed rooms and also our project roof type is RC solid slab.

This project deals about the structural analysis and design of G+3 building for mixed used considering all the internal and external effects according to ES EN 2015.

Then the structure designed exposed to a worst-case combination or existence scenario of these loads, analyzed for internal actions that was develop in the different structural parts. Finally, these parts are proportioned geometrically and reinforced with steel bars in such a way that they could resist the internal actions shear, bending moment and axial force.

The primary aim of all structural design is to ensure that the structure was performs satisfactorily during its design life. Specifically, the design must check that the structure can carry the loads safely and that it was not deform excessively due to the applied loads. This requires the designer to make realistic estimates of the strengths of the materials composing the structure and the loading to which it may be subjected during its design life. Furthermore, the designer was needed a basic understanding of structural behavior.

1.3 Objectives

1.3.1. Main objective

As a structural engineer we are expected to convert architectural engineers' (given drawing) drawing in to real world building with safe and durable structural design with in architectural engineers' (given drawing) dimension so, our main objective is to design safe, durable and economical structural design.

1.3.2. Sub-objectives

The design project will have the following specific objectives

- ✓ To work and practice the general concepts of designing methodology.
- ✓ Knowing design code and design criteria.
- ✓ To consider the load consideration and calculation.
- ✓ To know heavy loads and how to design this loads.
- ✓ To design seismic (wind and earth quake) impact on the building.
- ✓ To design the reinforcement detailing.
- ✓ Overall safety of the structure.

1.4. Structural Design Philosophy and Methods

For a given structural system, the design problem consists of the following steps:

1. Idealization of structure for analysis
    a. Dimension of members,
    b. Support condition of structure and etc.
2. Estimation of loadings
3. Analysis of idealized structural model to determine stress-resultants
    a. Axial forces,
    b. Shear forces,
    c. Bending moments and their effects (deformations)
4. Design of structural elements (if assumed dimensions are adequate)
5. Detailed structural drawings and schedule of reinforcing bars

To achieve safe and economic structures, three philosophies of design had been adopted by codes of practices.

1. Working Stress Design (WSD) or Elastic Design Method
2. Ultimate Strength Design (USD) Method, and
3. Limit State Design (LSD) Method

1.4.1. Methods for slab analysis & design

Coefficient method is based up on elastic theory but the reinforcement for slabs is calculated by strength methods the account for the actual inelastic behavior of structural members at the factored load stage. This method limits the structural usefulness of the material up to ascertain load at which the maximum stress in extreme fibers reaches the yield stress of the material in bending. The exactness of this method is restricted to square or rectangular slabs with symmetric supports and for uniform distribution loads.

Yield line and Hillerborg strip methods are the alternative plastic theories to the usual coefficient and finite element method of analysis and design method. Plastic analysis methods are derived from the general theory structural plasticity, which states that the collapse load of a structure lies between two limits, an upper and a lower bound of the true collapse load λ. Yield line method which is one of the plastic theories can be applied to the slabs of any shape, any support condition, uniformly or partially and none uniformly distributed loads, slabs with holes of any size and so on.

1.5. Design Specifications and Constants

1.5.1. Material used and Properties

Material selected to design the slab structure

- ✓ Concrete grade of C20/25
- ✓ Steel grade of S-400

Design strength value

a) Concrete

$\Upsilon c=1.5$ $fcd = \alpha cc * \frac{fck}{\gamma c} = 0.85 * \frac{25}{1.5} = 11.33MPa$

fctd=1.03mpa Elastic modulus of concrete, Ec=29Gpa

fctm=2.2

b) Steel

$\Upsilon s=1.15$

$fyd = \frac{fyk}{\gamma s} = \frac{400}{1.15} = 347.83MPa$

Elastic modulus of steel, Es=200Gpa

1.6. Basic Structural Components

All buildings have two zones a structural above ground level is called Super- Structure the underground part is Sub-Structure. The buildings super- structure & sub-structure are defined by geometry, i.e., points, lines, surfaces, spaces, and bodies (solids). In an ordinary building the super structural part is composed of: the nodes or the point elements of joints (connections), the linear members of beams &columns, the surface elements of slabs and walls.

A. Joints/Connections

At the intersections of every building component occurs a joint. Joints range from the large scale of a building joining the ground to the beam-column connection and to the small scale of mortar joints bonding brick together in certain patterns in a masonry wall.

B. Beams

Beams are linear members and they are distinguished in shape, cross sections material and Support conditions. The effect of load action (eccentric versus concentric) on beam behavior in response to member shape and profile may be in

- ✓ Simple bending
- ✓ Biaxial bending, or
- ✓ Unsymmetrical bending
- ✓ Unsymmetrical bending

C. Columns

Columns support loads in compression. Whereas hangers, ties and thin brace do so in tension. They carried bending and axial compression where the bending can be more important than the axial load. Columns are the primary components of skeleton structures. They may carry an entire building. It can be

- ✓ Short or long
- ✓ Slender or stocky

D. Slab

Slabs are plate elements forming floors & roofs in building which normally carry uniformly distributed loads. Slabs may be simply supported or continuous over one or more supports and are classified according to the method of support as follows.

a) . Spanning one –way between beams or walls
b) spanning two-way between the support beams or walls
c) Flat slabs carried on columns and edge beams or walls with no interior beams.

E. Foundation

It is the part of the structure that is usually placed below the surface of the ground and that transmits the load to the underlying soil or rock. All soils compress noticeably when loaded and causes the supported structure to settle

1.5 Structural layout

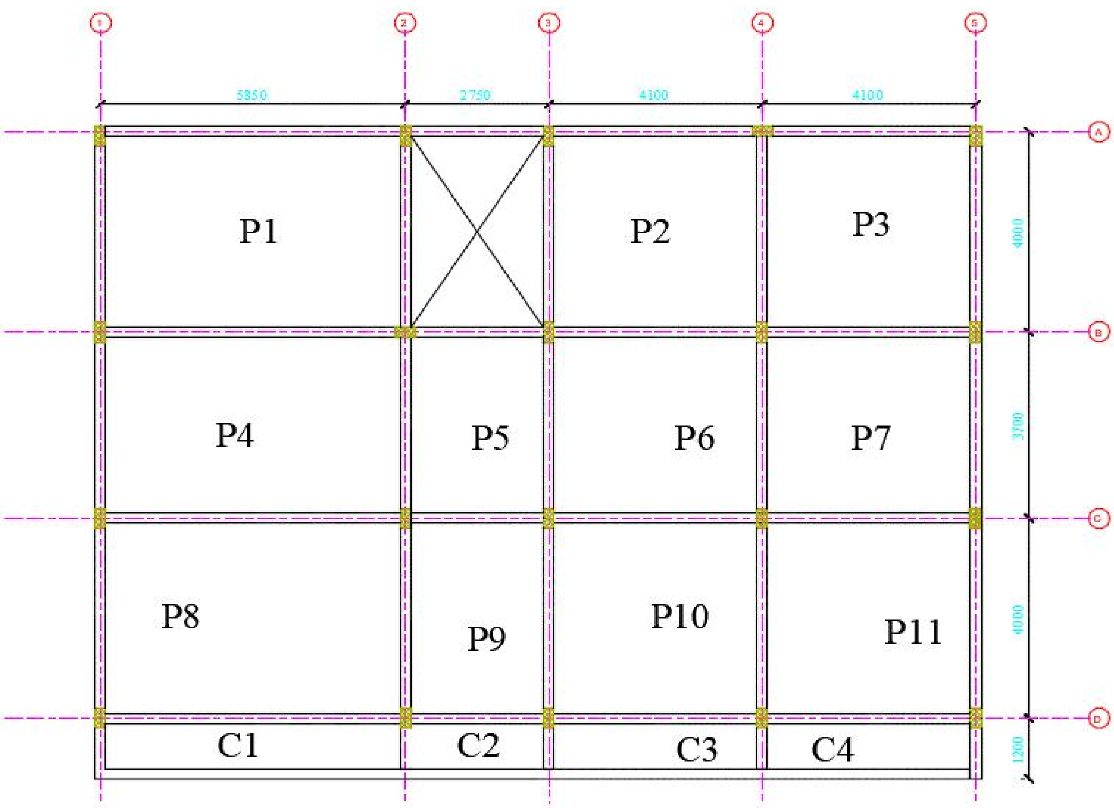


Figure1- 1. 3rd floor slab layout

1.7. Design Aids used for this paper

Method (Approach) = Limit state design method

References (Code) = ES EN (Ethiopian Standard based on Euro Norms)

Euro Code nowadays is being practiced in most countries around the world and Ethiopia is one of those countries who has shown interest in adopting the Euro Code.

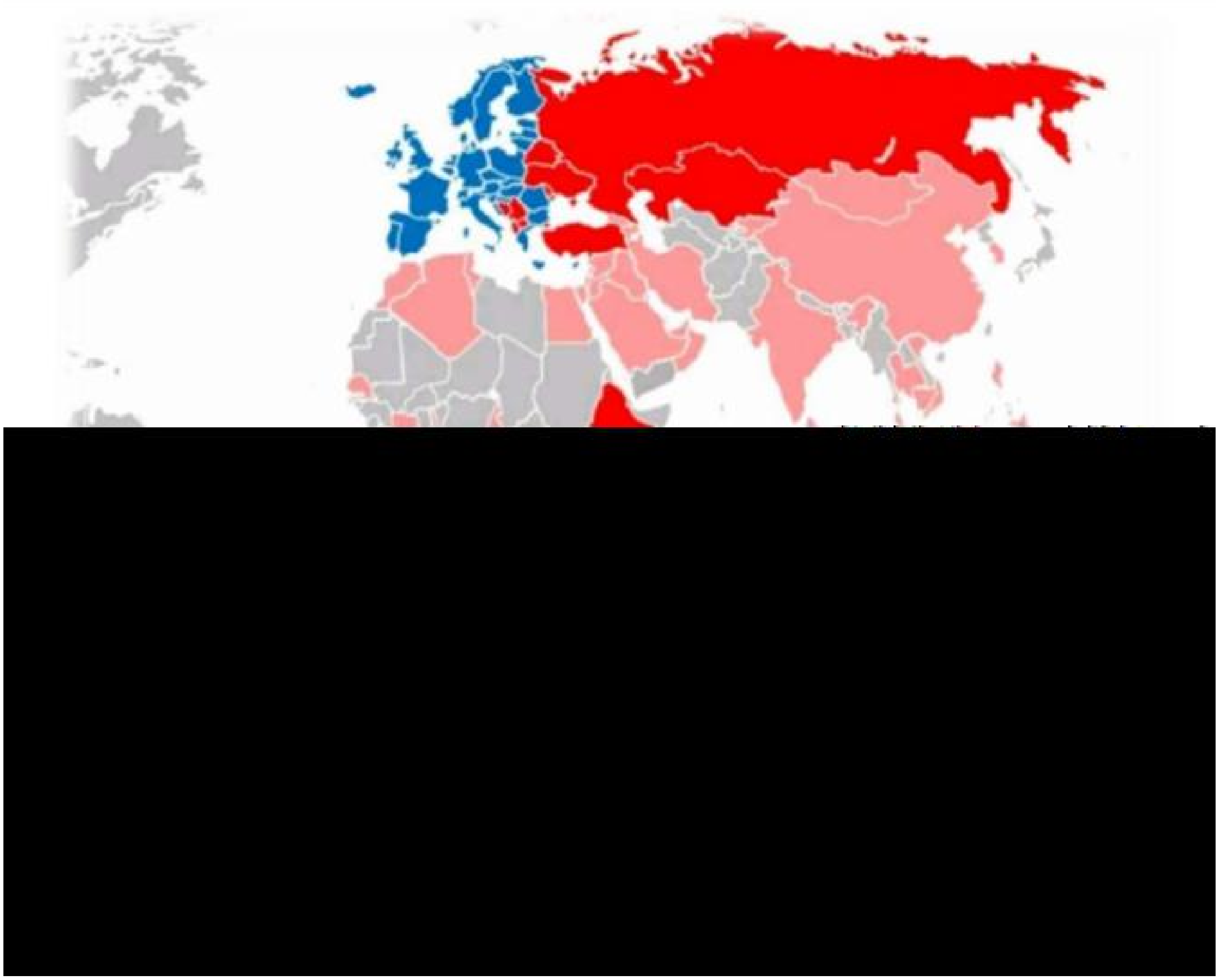


Figure 1- 2-euro code practice around the world

The 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 analysis 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.

ES EN 1990	ES EN 0	Basis of design (ES EN 0)
ES EN 1991	ES EN 1	Actions on structures (ES EN 1)
	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
ES EN 1992	ES EN 2	Design of concrete structures (ES EN 2)
	Part 1-1	General rules and rules for buildings
	Part 1-2	General rules – structural fire design
	Part 2	Reinforced and prestresses concrete bridges
	Part 3	Liquid retaining and containing structures
ES EN 1993	ES EN 3	Design of steel structures
ES EN 1994	ES EN 4	Design of composite steel and concrete structures
ES EN 1995	ES EN 5	Design of timber structures
ES EN 1996	ES EN 6	Design of masonry structures
ES EN 1997	ES EN 7	Geotechnical design
ES EN 1998	ES EN 8	Earthquake resistant design of structures
ES EN 1999	ES EN 9	Design of aluminum alloy structures

Figure 1- 3. ES-EN with all parts

2. ROOF DESIGN

2.1. General

Roof is the most upper part of a building that protects from any kind of weather. It is subjected to Different kinds of loads such as wind load, its own self weight, the loads of the persons who goes on the roof for maintenance and snow loads too. Most type of roof consists of network of frames of Wood or steel and covering materials.

2.2. Wind load analysis

Structures deflect or stop the wind, converting the wind's kinetic energy into potential energy of pressure-creating load. The action of wind can be of the type of suction or pressure to our structures both externally or internally. We consider in our structural project, the design of G+3 building for mixed use purpose located in Addis Ababa.

Analysis of wind load Wind loads is dynamic loads and there are two methods of analysis of dynamic loads.

Quasi- Static method: This is applied to stiff structures in which the movement of the structure with wind is negligible.

Dynamic analysis: in which the movement of structures with wind loads in considered

The magnitude of the wind load depends on the roof shape, wind direction and location of the building. Appropriate fasteners and holding down bolts or anchors must be used.

Table 1-necessary data of wind load analysis

Roof type	**Flat roof**
Location	**Addis Ababa**
Building height	**13.2M**
Terrain category	**Category IV**: - Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15m.
Directional factor	**1:** recommended value
Seasonal factor	**1**: recommended value
Basic Wind velocity	**22m/s**: for Ethiopia
Z_0	**1m**: for terrain category IV from table
$Z_{0.II}$	**0.05m:** for (terrain category II)
Minimum height	**10m:** for terrain category IV from table
$maximum\ height$	**200m**
$orography\ factor$	**1:** recommended
Turbulence factor	**1:** recommended
Air density	**1.25 kg/m^3:** recommended
Reference height	**13.2m:** for h < b

a) determination of basic wind velocity V_b

$$V_b = C_{dir} C_{sea} V_{bo}$$

Where $C_{dir} = directional\ factor$

$C_{sea} = seasonal\ factor$

$V_{bo} = fundamental\ basic\ wind\ velocity$

$$V_b = 1 * 1 * 22\ m/s = \mathbf{22\ m/s}$$

b) Roughness factor $C_r(Z)$

$$C_r(Z) = \begin{cases} K_r \ln\left(\frac{Z}{Z_0}\right) & for\, Z_{min} \le Z \le Z_{max} \\ C_r(Z_{min}) & for\, Z \le Z_{min} \end{cases}$$

Where $K_r = 0.19\left(\frac{Z_0}{Z_{0.II}}\right)^{0.07}$

$Z_{0.II}$ = 0.05m (terrain category II)

$Z_{min} = minimum\ height$

$Z_{max} = maximum\ height\ taken\ as\ 200m$

$$K_r = 0.19\left(\frac{1}{0.05}\right)^{0.07} = \mathbf{0.234}$$

$$C_r(Z) = K_r \ln\left(\frac{Z}{Z_0}\right) \qquad for\ 10 \le 13.2 \le 200$$

$$= 0.234 ln\left(\frac{13.2}{1}\right)$$

$$C_r(Z) = \mathbf{0.604}$$

c) Mean wind velocity $V_m(Z)$

$$V_m(Z) = C_r(Z) * C_0(Z) * V_b$$

$where\ C_r(Z) = Roughness\ factor$

$C_0(Z) = orography\ factor\ taken\ as\ 1$

$V_b = basic\ wind\ velocity$

$V_m(Z) = 0.604 * 1 * 22\ m/s$ = **13.3 m/s**

d) Wind turbulence $l_V(Z)$

$$l_V(Z) = \frac{K_l}{C_0(Z) * \ln\left(\frac{Z}{Z_0}\right)} \qquad for\, Z_{min} \leq Z \leq Z_{max}$$

Where $K_l = turbulence\, factor$

$C_0(Z) = orography\, factor$

$$l_V(Z) = \frac{1}{1 * ln\left(\frac{13.2}{1}\right)} = \mathbf{0.388}$$

e) Peak velocity pressure $q_p(Z)$

$$q_p(Z) = [1 + 7 * l_V(Z)] * \frac{1}{2} * \rho * {V_m}^2(Z)$$

$where\; \rho = air\; density$

$$q_p(Z) = [1 + 7 * 0.388] * \tfrac{1}{2} * 1.25 * 13.3^2 = \mathbf{410.83\; N = 0.411 KN}$$

f) Wind pressure

- External pressure

$W_e = q_p(Z).C_{pe}$

$where,\; q_p(Z_e)\; peak\; velocity\; pressure$

$Z_e\; reference\; height$

$C_{pe}\; external\; pressure\; coefficient$

- ✓ External pressure coefficient

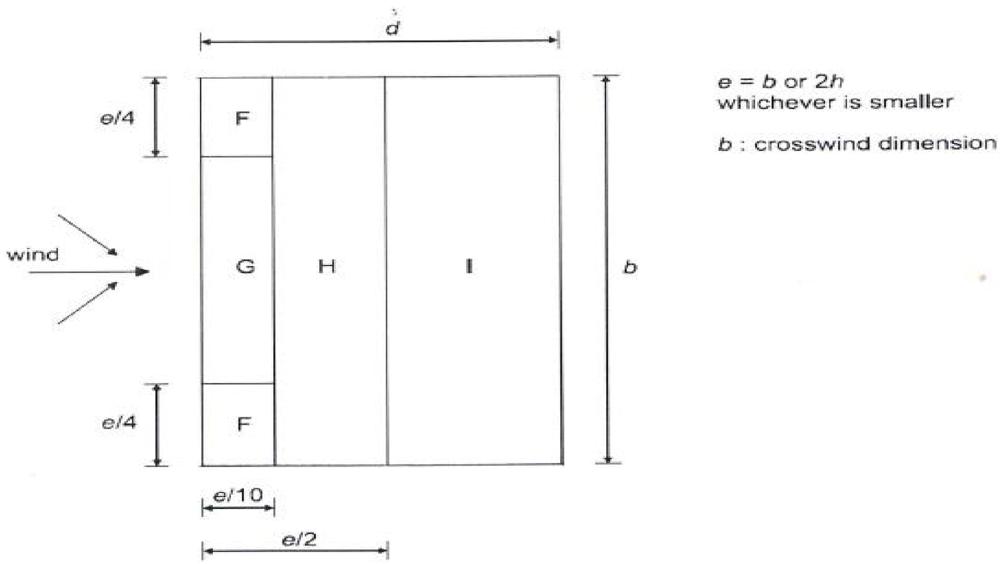
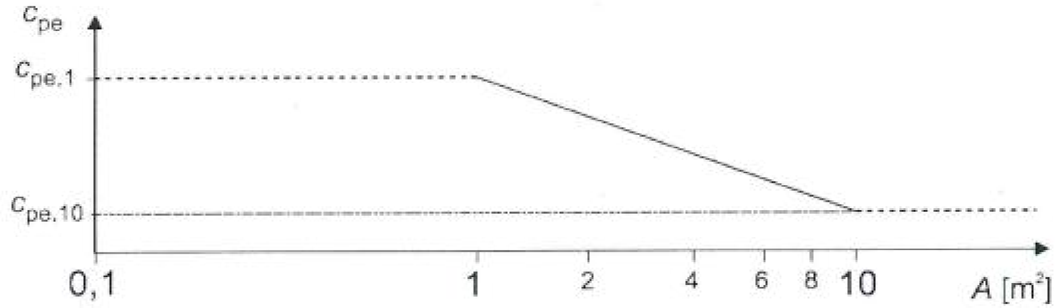


Figure 2- 1.key for flat roof



The figure is based on the following:

for 1 m^2 < A < 10 m^2 $c_{pe} = c_{pe,1} - (c_{pe,1} - c_{pe,10}) \log_{10} A$

Figure 2- 2.Recommended procedure for determining the external pressure coefficient cpe for building with a loaded area A between 1m² and 10m²

Table 2.External pressure coefficients for flat roofs

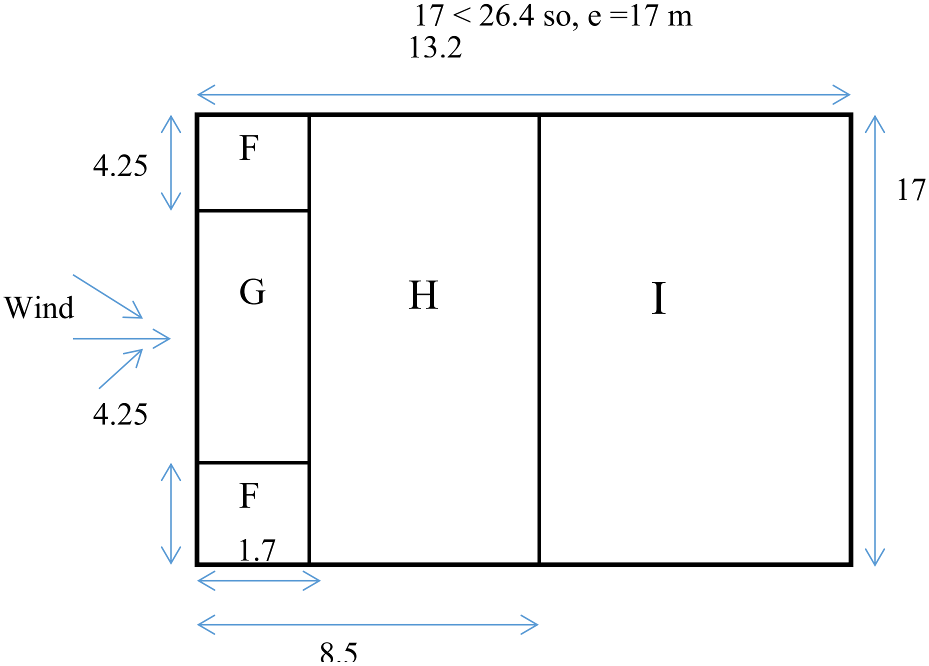
Roof type	Zone							
	F		G		H		I	
	cpe,10	cpe,1	cpe,10	cpe,1	cpe,10	cpe,1	cpe,10	cpe,1
Sharp eaves	-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	+0,2 -0,2	

Case -1 ϴ=90°

b = 17, d = 13.2m, h=13.2

2h= 26.4

17 < 26.4 so, e =17 m



$Cp_e = Cpe1+(Cpe10-Cpe1)logA$

$A_F=1.7*4.25=7.225M^2$

$A_G=8.5*1.7=14.45M^2$

$A_H=6.8*17=115.6M^2$

$A_I=17*4.7=79.9M^2$

Zone	Cpe values		Cpe	We
F	Cpe10	-1.8		
	Cpe1	-2.5	-1.9	-0.781
G	Cpe10	-1.2		
	Cpe1	-2	-1.072	-0.441
H	Cpe10	-0.7		
	Cpe1	-1.2	-0.17	-0.07
I	Cpe10	0.2		
	Cpe1	-0.2	0.56	0.23
	Cpe10	-0.2		
	Cpe1	0.2	-0.56	-0.23

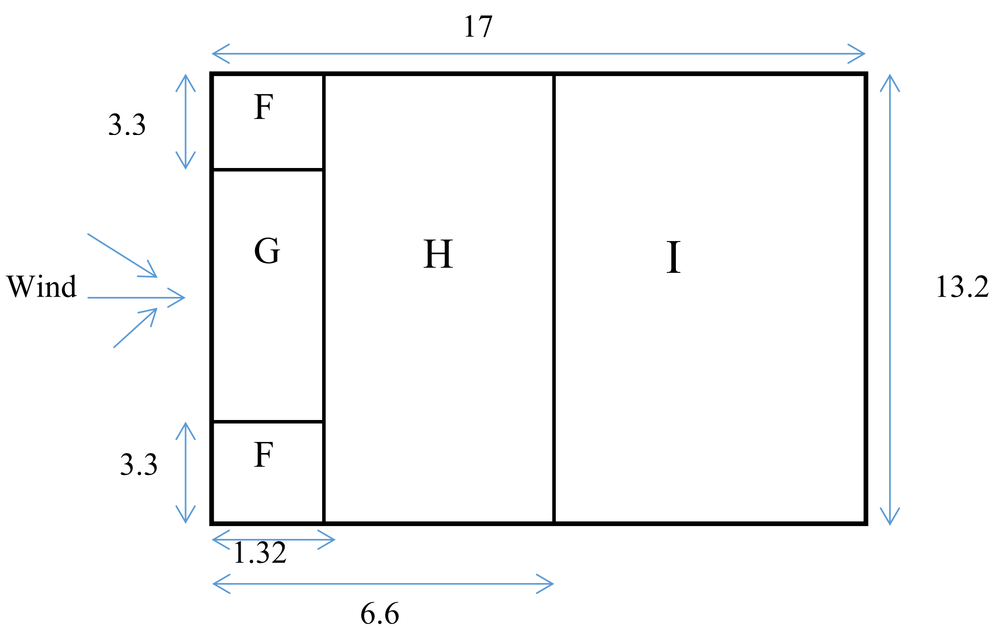
Critical external pressures: - pressure = **0.23** and suction = **-0.781**

Case -2 ϴ=0º

b = 13.2m, d = 17m, h=13.2

2h= 26.4

13.2 < 26.4 so, e =13.2m



Zone	Cpe values		Cpe	We
	Cpe10	-1.8		
F	Cpe1	-2.5	-2.05	-0.843
	Cpe10	-1.2		
G	Cpe1	-2	-1.25	-0.514
	Cpe10	-0.7		
H	Cpe1	-1.2	-1.078	-0.443
	Cpe10	0.2		
	Cpe1	-0.2	0.655	0.27
	Cpe10	-0.2		
I	Cpe1	0.2	-0.655	-0.27

Critical external pressures: - pressure = **0.27**and suction = **-0.843**

- Internal pressure

$W_i = q_p(Z).C_{pi}$

$where,\ q_p(Z_i)\ peak\ velocity\ pressure$

$Z_i\ reference\ height$

$C_{pi}\ internal\ pressure\ coefficient$

Internal Wind Pressure Coefficient

Take Cpi = 0.8 for pressure and Cpi = -0.5 for suction as per code.

$$W_i = 0.411 * 0.8 = 0.3288$$

$$= 0.411 * -0.5 = -0.2055$$

g) Net pressure = $W_e - W_i$

We	Wi	Wnet
0.23	0.3288	-0.0988
-0.781	-0.2055	-0.5755
0.27	0.3288	-0.0588
-0.843	-0.2055	-0.6375

Hence, the critical net wind loads on the roof are

Critical Pressures are (KN/m2)

Wnet min= -0.0588

Wnet max= -0.6375

2.3. Concrete cover design for roof slab

Cover is designed according to (*ES EN 1992-1-1*, 2015)

a. Exposure class = XC4, for cyclic wet and dry, concrete surfaces subject to water contact, not within exposure class XC2.(*ES EN 1992-1-1*, 2015)

b. Minimum strength class = C30/37, for exposure class XC4

c. Minimum concrete cover for bond/durability

$$Cmin = max\begin{Bmatrix} Cmin,B \\ Cmin,Dur \\ 10mm \end{Bmatrix} = max\begin{Bmatrix} 10mm \\ 15mm \\ 10mm \end{Bmatrix} = 15mm$$

$C_{min,\,b}$= 10 mm, assumed diameter of bar

Cmin, dur = 15 mm,

The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N(*ES EN 1992-1-1*, 2015).

The recommended minimum Structural Class is S1

Cmin=15mm

d. Nominal cover

$C_{nom} = C_{min} + \delta c_{dev} = 15mm + 10mm = 25mm$

Δc_{dev} = 10mm, the value of Δcdev for use in a Country may be found in its National Annex. The recommended value is 10 mm.

e) Minimum cover Design for Fire = 20mm

f) Governing concrete cover = 25mm

3. SLABE DESIGN

3.1 SOLID SLAB ANALYSIS AND DESIGN

A reinforced concrete slab is a broad, flat plate, usually horizontal, with top and bottom surface parallel or nearly so. It is used to provide flat surface mainly for roofs and floors of building parking lots, air fields, road ways, bridges etc. It may be supported by reinforced concrete beam, by masonry or reinforced concrete walls, by structural steel members, directly by columns, or continuously by the ground. But we are intending to design beam supported floor and roofs solid slabs of the building accordingly (*ES EN 1992-1-1*, 2015, p. 2015)

Beam support slab can be classified as one-way slab and two-way slab based on the longer to shorter length ratio.

A. One-way slabs: - main reinforcement in each element runs in one direction only. The slab is one way if the slab panel longer length to shorter length ratio is greater than two (i.e. $Ly/Lx > 2$).

B. Two-way slab: -main reinforcement runs in both directions where ratio of long to short span is less than or equal to two (i.e. $Ly/Lx \leq 2$).

Panel selection

Before proceeding to the analysis and design of solid slab we should select type of panel. The selection of panels is depending on:

- ✓ Boundary condition
- ✓ Shorter (Lx) and longer (Ly) length of the panel
- ✓ Function of panel.

Material selected to design the slab structure

- ✓ Concrete grade of C20/25
- ✓ Steel grade of S-400
- ✓ Deformed reinforcement bar of diameter with Ø10 for both span and support

Design strength value

c) Concrete

Υc=1.5 $fcd = \alpha cc * \frac{fck}{\gamma c}$ =0.85 $* \frac{25}{1.5}$= 11.33MPa

fctd=1.03mpa Elastic modulus of concrete, Ec=29Gpa

d) Steel

Υs=1.15

$fyd = \frac{fyk}{\gamma s} = \frac{400}{1.15}$ = 347.83MPa

Elastic modulus of steel, Es=200Gpa

e) Determine Geometry/Dimension

Consider one-meter strip width, b=1000mm

D=effective depth (d) +cover +Lon/2

In order to determine the depth of the slab, first it is needed to find concrete cover and effective depth.

3.1.1 Concrete cover design

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

The nominal cover shall be specified on the drawings.it is defined as Cmin

Cnom=Cmin+ΔCdev

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

Minimum cover, shall provide in order to ensure

Safe transmission of bond force

a) Corrosion resistance/ Durability

Fire resistance

Cmin=max$\begin{cases} Cmin, b \\ Cmin, dur + \Delta Cdur, \boldsymbol{B} - \Delta Cdur, st - \Delta Cdur, add \\ 100 \end{cases}$

Where;

Cmin, b -minimum cover due to bond requirement, see (*ES EN 1992-1-1*, 2015)

Cmin,dur - minimum cover due to environmental conditions, see(*ES EN 2 Part 1-1*, n.d.) ES EN 1992:2015 Art 4.4.1.2 (5)

ΔCdur,γ- additive safety element, see(*ES EN 1992-1-1*, 2015) ES EN1992:2015 Art 4.4.1.2 (6)

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

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

But; the recommended value of ΔCdur,γ , ΔCdur,st , and ΔCdur,add is zero see Art. 4.4.1.2

(6, 7, and 8).

b) Cover Design for Bond

$$Cmin = max\begin{Bmatrix} Cmin,B \\ Cmin,Dur \\ 10mm \end{Bmatrix} = max\begin{Bmatrix} 12mm \\ 10mm \\ 10mm \end{Bmatrix} = 12mm$$

i. $C_{min,\,b}$= 12 mm, assumed diameter of bar [Appendix B, Table B-1]

ii. Cmin, dur = 10 mm,

1. Cmin=12mm

iii. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1

c) Nominal cover

a. $C_{nom} = C_{min} + \delta c_{dev} = 10mm + 10mm = 20mm$

i. Δc_{dev} = 10mm, the value of Δc_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.

d) Minimum cover Design for Fire = 20mm

e) Governing concrete cover = 20mm

3.1.2 Effective Depth Determination: Serviceability requirement

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

f) by limiting the span/depth ratio, according to 7.4.2 or
g) by comparing a calculated deflection, according to 7.4.3, with a limit value

$$\frac{l}{d} = K\left(11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho} * 3.2 * \sqrt{fck\left(\frac{\rho o}{\rho} - 1\right)^{\frac{3}{2}}}\right) * F1 * F2 * F3 \text{ if } \rho > \rho o$$

$$\frac{l}{d} = K(11 + 1.5\sqrt{fck} * \rho o/(\rho - \rho') + 1/12 * \sqrt{fck}\sqrt{(\rho o/\rho)}) * F1 * F2 * F3 \text{if } \rho < \rho o$$

Where;

- l/d - is the limit span/depth
- K -is the factor to take into account the different structural systems
- ρo - is the reference reinforcement ratio = $10^{-3}\sqrt{fck}$
- ρ - is the required tension reinforcement ratio at mid -span to resist the moment due to the design loads (at support for cantilevers)
- ρ' - is the required compression reinforcement ratio at mid -span to resist the moment due to design loads (at support for cantilevers) fck is in MPa units.

- F1=300/σs =500/(fyk*As,req/As,pro)
- F2=0.8, for flanged sections where the ratio of the flange breadth to the rib breadth exceeds 3. Otherwise; F2=1 for other cases.
- F3=7/leff , For beams and slabs, other than flat slabs, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflections (leff in meters, see Art. 5.3.2.2 (1)). Or

F3=8.5/leff, For flat slabs where the greater span exceeds 8.5 m, and which support partitions liable to be damaged by excessive deflections (leff

in meters). Otherwise; F3=1 for both cases.

Assumption

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

As,req = As,provided.

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

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

$\rho o=10^{-3}\sqrt{fck}=10^{-3}\sqrt{20}=0.447\%$

N=11+1.5$\sqrt{fck}$=11 + 1.5$\sqrt{20}$=17.7

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

F2=1 and F3 = 1 (because span of slab ≤ 7m)

For end span of continuous beam or one way continuous slab or two way spanning slab continuous over one long sided ,K=1.3

l/d=K*N*F1*F2*F3=1.3*17.71*1.25*1*1=28.78

Depth, D=d+d' Where: d' = cover +Φlon/2 use Φlon=10mm

d' = cover +Φlon/2= 20mm +10mm/2 = 25mm

For end span of two way slab. (Example – S1 Fig 3.2.1) l/ d =28.78mm

l=lx=4000 mm

d=4000/28.78=138.98mm

Similarly for the rest panels d will be calculated

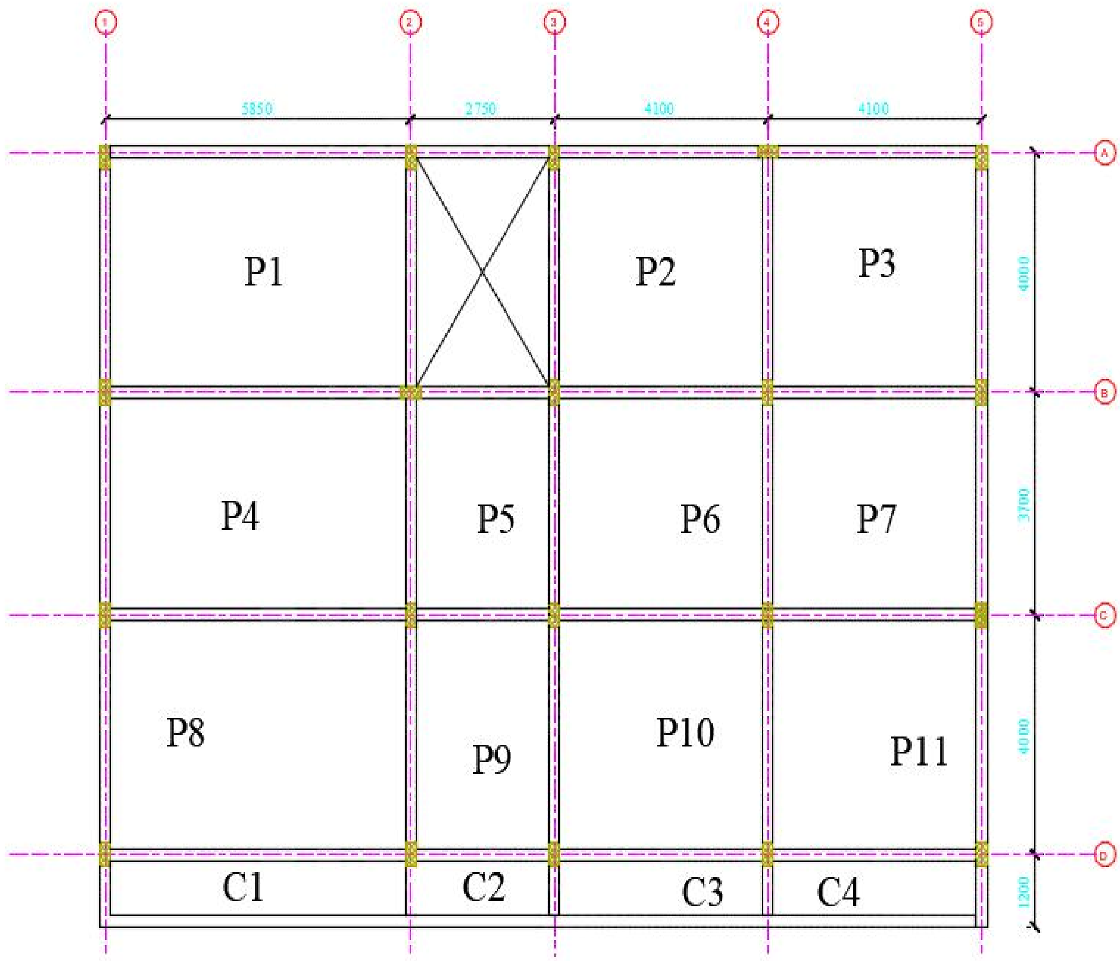


Figure 3- 1. 3rd floor slab layout

3.1.3. Depth determination of 3rd floor slab

Φbar= 10
Cnom= 20
ϒs = 1.15 fcd= $\frac{0.85*fck}{ϒc}$=11.33 fyd= $\frac{fyk}{ϒs}$= 347.83
ϒc = 1.5
fck = 20
fyk= 400 unit weight 25
F1= 1.25
F2= 1
F3= 1
d'= 25

Table 3. slab depth determination

panel	Ly	Lx	Ly/Lx	Type	support	N	K	Lx/d	d	D	D provided
p1	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	200
p2	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
p3	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
p4	5850	3700	1.58	two way	End	17.71	1.3	28.78	128.6	153.58	
p5	3700	2750	1.35	two way	Interior	17.71	1.5	33.20	82.8	107.82	
p6	4100	3700	1.11	two way	Interior	17.71	1.5	33.20	111.4	136.44	
p7	4100	3700	1.11	two way	End	17.71	1.3	28.78	128.6	153.58	
p8	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	
p9	4000	2750	1.45	two way	Interior	17.71	1.5	33.20	82.8	107.82	
p10	4100	4000	1.03	two way	Interior	17.71	1.5	33.20	120.5	145.47	
p11	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
C1	5850	1200	4.88	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C2	2750	1200	2.29	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C3	4100	1200	3.42	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C4	4100	1200	3.42	cantilever	End	17.71	0.4	8.85	135.5	160.53	

3.1.4 Load Calculations

The slab is loaded with both DL and LL. Dead load comes from self-weight of slab, floor finish, cement screed, plastering and partition load.

Live loads are either movable or moving load without any acceleration or impact.

Dead and live loads are calculated depending on the service of the slabs and self- weight. Ignoring any localized effects caused by concentrated load, the partition loads are distributed over the area of the slab. The design loads are factored according to the following formula.

Pd = 1.35DL + 1.5LL

Where: Pd - total factored design load

DL - total dead load on slab

LL - total live load on slab

a) Dead load (DL)

Dead loads include self-weight of slab, floor finish, celling plaster and partition walls. The unit weight of each material shown in the table below as per (*ES EN 1992-1-1*, 2015)

Example for P1

DL= self-weight of slab + floor finish + celling plaster + partition load

Self-weight of slab = thickness of slab*unit weight = 0.2m*25KN/m^3= **5.0KN/m^2**

Floor finish = thickness * unit weight

For this panel our floor finish is cement screed with unit weight of 23KN/m^3

30mm cement screed=0.03*23=**0.69**KN/m

For ceramic= 0.005m * 23KN/m^3 = **0.12KN/m^2**

Celling plaster = thickness * unit weight

Celling plaster = 0.03m * 23KN/m^3 = **0.69KN/m^2**

Partition load: - the partition load on the slab acts at wall thickness but for design purpose the load will be distributed uniformly over whole panel.

Due to partition wall

The unit specific weight of 15cm thickness Pumice Hollow Block is taken the average value of 12KN/m3. And plastering of 25mm both sides

Weight due to partition wall = (0.15*12*2.85) + (0.025*23*2.85*2)

= 8.41KN/m

Pannel-1

8.41KN/m x (0.95+0.729+4.26+0.45+0.451+0.875+1.36)=76.32KN

==> $\frac{76.32KN}{5.85m*4m}=3.26KN/m^2$

Panel-2

8.41(1.275)=10.72KN

==> $\frac{10.72KN}{4.1m*4m}=0.65KN/m^2$

Pannel-3

8.41(1.275)=10.72 KN

==> $\frac{10.72KN}{4.1m*4m}=0.65KN/m^2$

Pannel-4

8.41(1.8+1.44+1.2+1.926)=53.54 KN

==> $\frac{53.54KN}{5.85m*3.7m}=2.47KN/m^2$

Pannel-6

8.41(3)=25.23 KN

==> $\frac{25.23KN}{4.1m*3.7m}=1.56KN/m^2$

Pannel-7

8.41(3)=25.23 KN

==> $\frac{25.23KN}{4.1m*3.7m} = 1.56KN/m^2$

Pannel-8

8.41(4.2)=35.32KN

==> $\frac{35.32KN}{5.85m*4m} = 1.51KN/m^2$

Note: Wall loads along beams (on top of beam) and reasonably near beams are intentionally left since they will be added later to their respective beams.

Total DL=**5KN/m^2 + 0.69KN/m^2 + 0.12KN/m^2 +0.69KN/m^2 +3.26KN/m^2 = 9.76 KN/m^2**

Dead load for cantilever slab C1

Self-weight of slab = thickness of slab*unit weight = 0.2m*25KN/m^3= **5.0KN/m^2**

Floor finish = thickness * unit weight

For this panel our floor finish is cement screed with unit weight of 23KN/m^3

30mm cement screed=0.03*23=0.69KN/m

For ceramic= 0.005m * 23KN/m^3 = **0.12KN/m^2**

Celling plaster = thickness * unit weight

Celling plaster = 0.03m * 23KN/m^3 = **0.69KN/m^2**

Total dead load=5+0.12+0.69= 5.81KN/m^2

b) Live load

live load is selected with respect to functional use of panel

For panel1 is done in tabular form below and other panels in appendix A1

Table 4.Design load calculation

P1			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			ceramic	23	0.005	0.115
partition wall	load of pa 76.32	Area 23.4	HCB	14	0.15	3.261538462
LIVE LOAD Q_k (category A)				3	KN/m^2	
DEAD LOAD Gk				9.76	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				17.67	KN/m^2	

3.1.5 Moment analysis of 3rd floor slab

Slabs with side ratio less than 2 are treated as two-way slabs and analysis can be made by means of coefficients on the basis of the following assumptions and procedures.

- The slab is composed of rectangular panels, supported at all four edges by walls or beams, stiff enough to be treated as an unyielding.
- Slabs are subjected to uniform load or concentrated load which can be converted to equivalent uniform load not exceeding 20% of the total load.

a) Coefficient method moment analysis

Moments for each panels with edge either simply supported or fully fixed are calculated from,

$mi=\beta siPdLx^2$

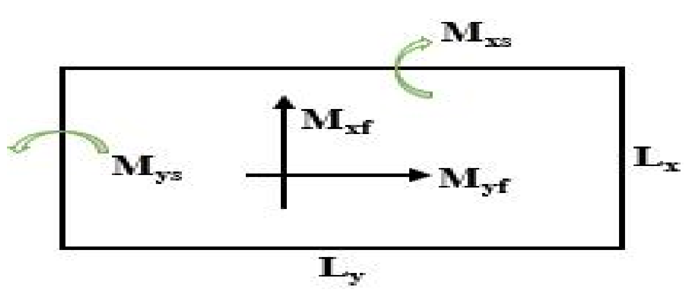


Figure 3- 2. Support and span moment notations

Where: -

mi = design BM per unit width at point of reference.

βsi = Coefficient given in table, as a function of Ly/Lx

Pd = Design uniform load (KN/m2)

L x, L y = shorter and longer spans of the panel, respectively

The Notation of Critical Moments

The subscripts for moments are:

s = support

f = field (span)

x & y = are directions of shorter & longer span, respectively calculated in tabular form below

β value is depend on support condition and Ly/Lx ratio

sample calculation For P1

P1 support condition is Type 2 and Ly/Lx=1.46 then the β is calculated below

$\beta_{sx,sup}$=0.057 $\beta_{sx,span}$=0.042

$\beta_{sy,sup}$=0.039 $\beta_{sy,span}$=0.029

$M_{xs}=\beta_{sx,sup}*pd*Lx^2=0.057*17.67*4^2=16.12$

$M_{xf}=\beta_{sx,span}*pd*Lx^2=0.042*17.67*4^2=11.88$

$M_{ys} = \beta_{sy,sup}*pd*Lx^2=0.039*17.68*4^2=11.03$

$M_{yf} = \beta_{sy,span}*pd*Lx^2=0.029*17.68*4^2=8.2$

Then the rest of panels calculated in tabular below

b) Strip method moment analysis

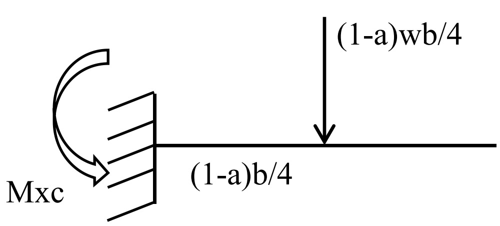
Strip method is a lower bound approach, based on the satisfaction of equilibrium requirements everywhere in the slab. By the strip method, a moment field is first determined that fulfills equilibrium requirements, after which the reinforcement of the slab at each point is designed for this moment field.

If a distribution of moment can be found that satisfies both equilibrium and boundary conditions for a given external loading, and if the yield moment capacity of the slab is nowhere exceeded, then the given external loading will represent a lower bound of the true carrying capacity.

Advantages:

- The strip method gives results on the safe side, which is certainly preferable in practice.
- The strip method is a design method by which the needed reinforcement can be calculated.

Design of slab near to stair case and cantilever slab is designed by strip method



Mxc=(1-a)wb/4*(1-a)b/4

$W=pd =1.35DL + 1.5LL = 1.35*5.81+ 1.5*4=13.84KN/M^2$

Field moment and Support moment

Mxf=Mxc/3

Mxs=2Mxc/3

For cantilever slab C1

a =1.2m b=5.85m

Mxc= (1-1.2)*13.84*5.85/4*(1-1.2)*5.85/4=1.18

Mxf= 1.18/3=0.4

Mxs= 2*1.18/3=0.8

For cantilever slab C2

a =1.2m b=2.75m

Mxc= (1-1.2)*13.84*2.75/4*(1-1.2)*2.75/4=1.05

Mxf= 1.05/3=0.35

Mxs= 2*1.05/3=0.7

For cantilever slab C3 and C4

a =1.2m b=4.1m

Mxc= (1-1.2)*13.84*4.1/4*(1-1.2)*4.1/4=0.58

Mxf= 0.58/3=0.2

Mxs= 2*0.58/3=0.39

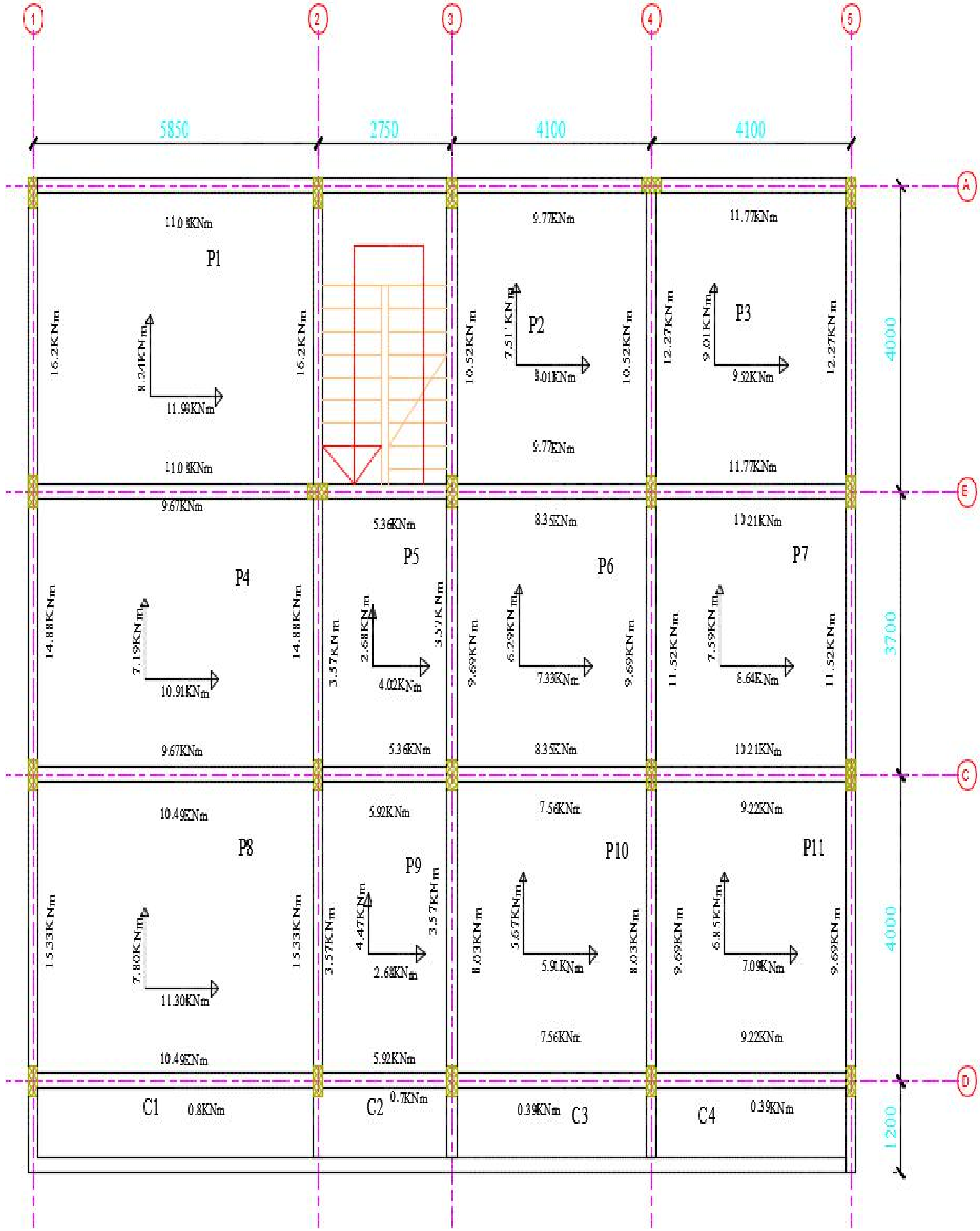


Figure 3- 3. 3rd slab unadjusted moment

3.1.6 Moment adjustment of 3rd floor slab

a) Support moment adjustment

Compute change M=M_{large}-M_{small} on each support

When change M is less than 20% of large moment use M=$\left(\frac{M\text{large}+M\text{small}}{2}\right)$

When % of change M is greater than 20% use moment distribution

Moment distribution method: -the method begins by assuming each joint of a structure is fixed. Then, by unlocking and locking each joint in succession, the internal moments at the joints are "distributed" and balanced until the joints have rotated to their final or nearly final positions. Generally, we distribute the analyzed support moment by multiplying with distribution factor according to support condition and span length.

Relative bending stiffness K

$$K = \begin{cases} \frac{I}{L}\ if\ far\ end\ of\ member\ is\ fixed \\ \frac{3I}{4L} if\ far\ end\ of\ member\ hinged \end{cases}$$

Distribution factor Df

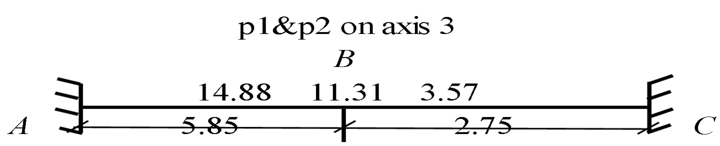
$Df = \frac{k}{\Sigma k}$ (Kassimali, 2011)

For panel P4&P5 on axis-2

M_{xs}=14.88KNm, M_{xs}=3.57KNm

Δm_x=11.31KN.m

20% 14.88KNm=2.98 since Δm_x>2.98, we opt for moment distribution method



Relative bending stiffness K

$K_{AB}=\frac{I}{L}=\frac{I}{5.85}=0.17\,I$

$K_{BC}=\frac{I}{L}=\frac{I}{2.75}=0.36\,I$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.320$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.680$$

Balanced support moments

Mys1=0.32(14.88-3.57) +3.57=7.2KN-m

Mys2=14.88-0.68(14.88-3.57) =7.2 KN-m

For panel P1&P4 on axis-B

M_{ys}=11.08KNm, M_{ys}=9.67KNm

Δm_x=1.41KN.m

20% 11.08KNm=2.22 since Δm_x<2.22, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (11.08+9.67)/2 = 10.4KNm

For panel P2&P3 on axis-4

M_{xs}=12.27KNm, M_{xs}=10.52KNm

Δm_x=1.75KN.m

20% 12.27KNm=2.45 since Δm_x<2.45, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (12.27+10.52)/2= 11.4KNm

b) Field moment adjustment

If the support moment is decreased, then the span moment Mxf and Myf are increased which allows for the changes of support moments.

(Mxf)d=Mxf + CxΔMxs + CxΔMys

(Myf)d=Myf + CyΔMxs + CyΔMys

When ΔM:-the difference between the initial support moment and adjusted support moment.

Cx and Cy: - coefficients for adjusting span moment from table if the support moment increased no adjustment made to span moment.

If ΔMxs and ΔMxs are getting zero no need adjustment on the field but one of them are greater than zero needed field adjustment.

For Panel 1

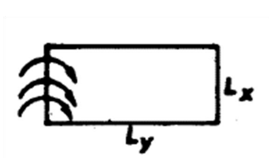
M_{xf} = 11.93 M_{Xs} = 16.2 $M_{Xs,adj}$ = 16.2

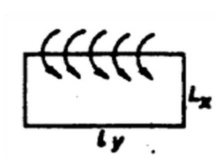
M_{yf} = 8.24 M_{ys} = 11.08 $M_{ys,adj}$ = 10.4

Ly/Lx= 1.46

ΔM_{xs} = Mxs- M_Xs,adj = 0.00 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.68





Cx = 0.31 Cx = 0.41

Cy = 0.1 Cy = 0.32

ΔMxf = CxΔMxs + CxΔMys = 0.279

Mxf ,adj = ΔMxf + Mxf = **12.209**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.218

Myf ,adj = ΔMyf + Myf = **8.458**

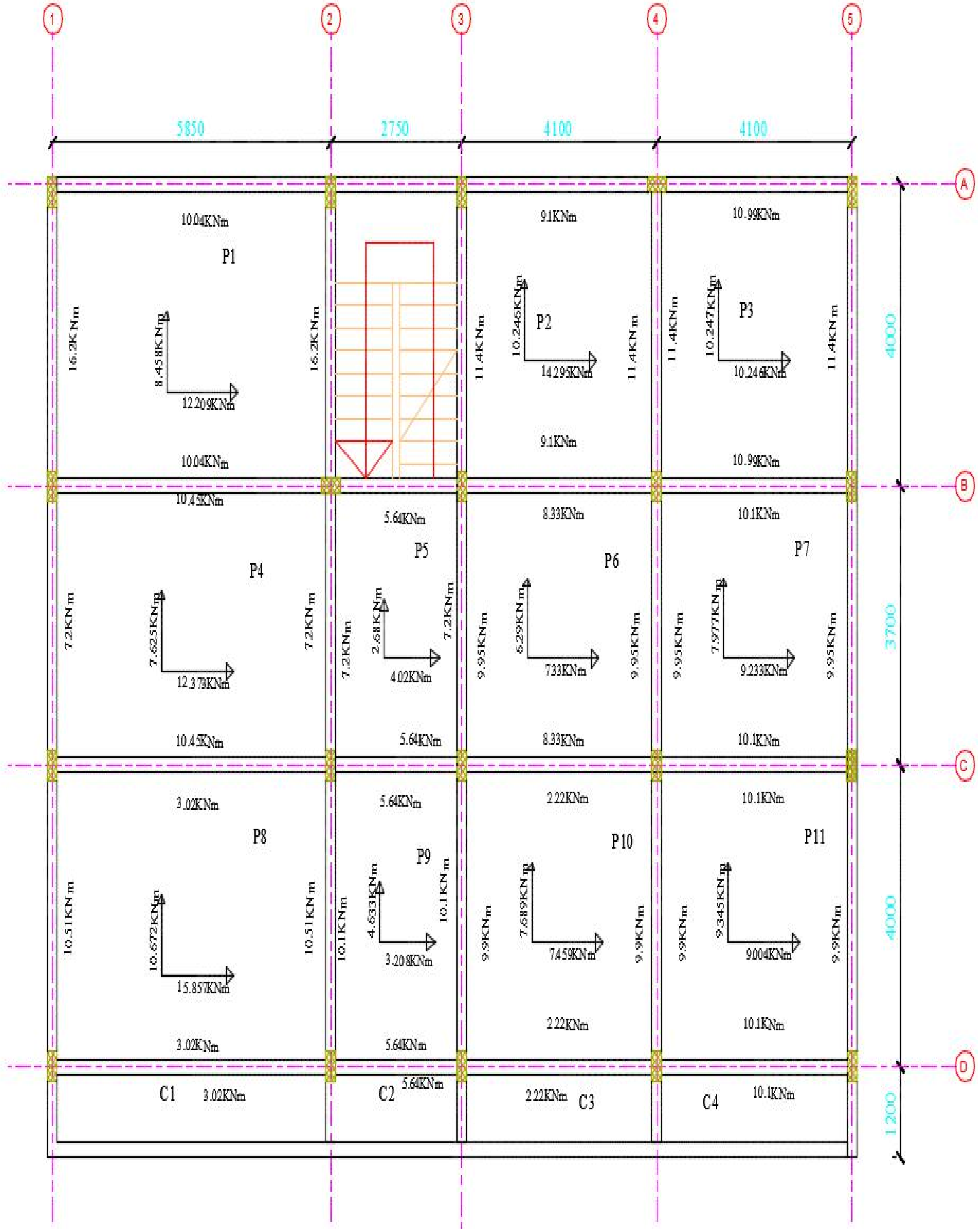


Figure 3- 4. 3rd floor slab adjusted moment

3.1.7 Load Transfer to beam

Two-way slab shear

The shear force for each panel calculated by coefficient method

$V_x = \beta_{vx} P_d L_x$

$V_y = \beta_{vx} P_d L_y$

Bvx and βvy are coefficients based on Ly/Lx ration and support condition

$\beta_{vxc} = 0.46$ $\beta_{vxd} = 0$ (in type 2 β_{vxd} is zero from table)

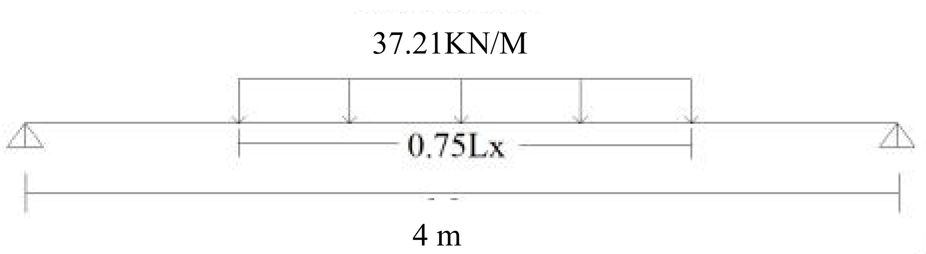
$\beta_{vyc} = 0.36$ $\beta_{vyd} = 0.24$

$V_x = \beta_{vxc} P_d L_x = 0.46*17.67*4 = 32.51$

$V_{yc} = \beta_{vyc} P_d L_y = 0.36*17.67*5.85 = 37.21$

$V_{yd} = \beta_{vyd} P_d L_y = 0.24*17.67*5.85 = 24.81$

The loads of slab transferred in to beam 90% across the entire length and 100% across 0.75 span length.



3.1.8. Check shear strength

$$v_{Rd,c} = \left[0.12K(100\rho_1 f_{CK})^{\frac{1}{3}}\right] \text{bd} \geq V_{min}$$

$$K = 1 + \left(\frac{200}{d}\right)^{0.5} \leq 2$$

d= 140

k= 2.20 not < 2,so k=2

$$\rho_1 = \frac{A_{sl}}{bd} \leq 0.02$$

taking minimum reinforcment ø 10 c/c 300

$$\rho_1 = 0.0019$$

$$V_{min} = [0.035K^{\frac{3}{2}} f_{ck}{}^{\frac{1}{2}}]bd$$

$$V_{min} = 61.981$$

$v_{Rd,c} = 41.867 \leq V_{min}$ so, $v_{Rd,c} = 61.981$

maximum shear force transfererd to beams is V_i= V_i=37.21

V_i= 37.21 KN/m

61.981>37.21,slab thickness is sufficient to support design load

3.1.9 Reinforcement calculation of 3rd floor slab

- Area of steel

$$A_{st,min} = \max \begin{cases} 0.26\frac{f_{ctm}}{fyk}bd = 0.26\frac{2.2}{400}1000 * 140 = 200.2\, mm^2 \\ 0.0013bd = 0.0013 * 1000 * 140 = 182\, mm^2 \end{cases}$$

$A_{st,min}$ = 200 mm^2

$A_{st,max}$=0.04bd=0.04*1000*140=5600mm^2

Area of steel calculated

$$Ast,cal = \frac{m_{sd}}{Z * fyd}$$

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

Z = 0.9d = 0.9*140 mm = 126 mm

Example for panel 1 support moment = 10.4 KN-m

$Ast = \frac{m_{sd}}{Z*fyd} = \frac{10.4*1000*1000N-mm}{126mm*347.83N/mm^2} = 237.3\ mm^2$ > 200 mm^2 OK!!

a) Spacing

$S_{max} = \max \begin{cases} 3D = 3*200 = 600mm \\ 400mm \end{cases}$

S_{max} = 600 mm

S_{min}=300mm on ES EN 2015

Let the ø of reinforcement be 10mm

$a_{st} = \frac{\pi d^2}{4} = \frac{3.14*10^2}{4} = 78.54\ mm^2$

Spacing of bars =S = $\frac{ba_s}{A_s} = \frac{1000*78.54}{237.3} = 330.97$ mm

$330.97mm \approx 350$ mm used < 600mm OK!!

panel	moment	Ast=Msd/Z*fyd		ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	16.2	369.64	10	212.37	220	ø	10	c/c	220
	M_{xf} =	12.209	278.58	10	281.79	280	ø	10	c/c	280
	M_{ys} =	10.4	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	8.458	250.00	10	314.00	300	ø	10	c/c	300
P-2	M_{xs} =	11.4	260.12	10	301.79	300	ø	10	c/c	300
	M_{xf} =	14.295	326.17	10	240.67	240	ø	10	c/c	240
	M_{ys} =	9.1	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	10.246	250.00	10	314.00	300	ø	10	c/c	300
P-3	M_{xs} =	11.4	260.12	10	301.79	300	ø	10	c/c	300
	M_{xf} =	10.619	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	10.99	250.76	10	313.05	300	ø	10	c/c	300
	M_{yf} =	10.247	250.00	10	314.00	300	ø	10	c/c	300
P-4	M_{xs} =	7.2	250.00	10	314.00	250	ø	10	c/c	250
	M_{xf} =	12.373	282.32	10	278.06	280	ø	10	c/c	280
	M_{ys} =	10.45	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.625	250.00	10	314.00	300	ø	10	c/c	300
P-5	M_{xs} =	7.2	250.00	10	314.00	260	ø	10	c/c	260
	M_{xf} =	4.02	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	5.64	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	2.68	250.00	10	314.00	300	ø	10	c/c	300
P-6	M_{xs} =	9.95	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	7.33	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	8.33	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	6.29	250.00	10	314.00	300	ø	10	c/c	300
P-7	M_{xs} =	9.95	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	9.233	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	10.1	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.977	250.00	10	314.00	300	ø	10	c/c	300
P-8	M_{xs} =	10.51	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	15.857	361.81	10	216.96	220	ø	10	c/c	220
	M_{ys} =	3.02	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	10.672	250.00	10	314.00	300	ø	10	c/c	300
P-9	M_{xs} =	10.1	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	3.208	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	5.64	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	4.633	250.00	10	314.00	300	ø	10	c/c	300
P-10	M_{xs} =	9.9	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	7.459	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	2.22	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.689	250.00	10	314.00	300	ø	10	c/c	300
P-11	M_{xs} =	9.9	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	9.004	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	10.1	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	9.345	250.00	10	314.00	300	ø	10	c/c	300

strip reinforcement									
middle x-d:	moments(KN.m)	ρmin	Asmin	S=b*as/AS	Spro	Reinforcement			
Mxs=	3.01	0.00125	250	314	300	ø	10	c/c	300
Mxf=	4.2	0.00125	250	314	300	ø	10	c/c	300
edge x-dxn									
Mxs=	0.75	0.00125	250	314	300	ø	10	c/c	300
Mxf=	1.05	0.00125	250	314	300	ø	10	c/c	300
middle y-dxn									
Mys=	12.04	0.00125	250	314	300	ø	10	c/c	300
Myf=	6.18	0.00125	250	314	300	ø	10	c/c	300
edge x-dxn									
Mys=	0.75	0.00125	250	314	300	ø	10	c/c	300
Myf=	1.05	0.00125	250	314	300	ø	10	c/c	300

3.1.10 Lap length

$l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd} \geq l_{0,min}$

$$l_{o,min} \geq max \begin{cases} 0.3\alpha_6 l_{b,rqd} \\ 15\emptyset \\ 200mm \end{cases}$$

Where: $l_{b,rqd} = \frac{\emptyset}{4} * \frac{\sigma sd}{fbd}$

$$\sigma sd = \frac{fyk}{\gamma_s}$$

$$fbd = 2.25\eta_1\eta_2 f_{ctd}$$

$$\eta_1 = 1\ for\ good\ quality\ of\ bond$$

$$\eta_1 = 0.7\ for\ other\ cases$$

$$\eta_2 = 1\ for\ \emptyset \leq 32mm$$

$$\eta_2 = \frac{132-\emptyset}{100}\ for\ \emptyset > 32mm$$

$$fctd = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, recommended\ value$$

Sample calculation for $\emptyset 10$ reinforcement bar

$$fctd = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, recommended\ value$$

$f_{ctk} =\ 1.mpa, from\ ESEN$ 1992 Table 3.1

$fctd = \frac{0.85*1.5}{1.5} = 0.85$

$fbd = 2.25\eta_1\eta_2 f_{ctd}$

$\eta_1 = 1$, *our* slab D is < 250mm, good condition of bond

$\eta_2 = 1$,$for\ \emptyset \leq 32mm$

$f_{ctd} = 0.85$

`$fbd = 2.25 * 1 * 1 * 0.85 = 1.91$

$\sigma sd = \frac{fyk}{\gamma_s} = \frac{400}{1.15} = 347.83$

$l_{b,rqd} = \frac{\emptyset}{4} * \frac{\sigma sd}{fbd} = \frac{10}{4} * \frac{347.83}{1.91} = 455.3mm$

$l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd} \geq l_{0,min}$

α_1, α_2 & $\alpha_3 = 1$, $\alpha_4 = 0.7$, for reinforcement bar in compression

$\alpha_6 = \left(\frac{\rho_1}{25}\right)^{0.5}$ but not exceeding 1.5 nor less than 1.0, where ρ_1 is the percentage of reinforcement lapped within 0.65 l0 from the center of the lap length considered

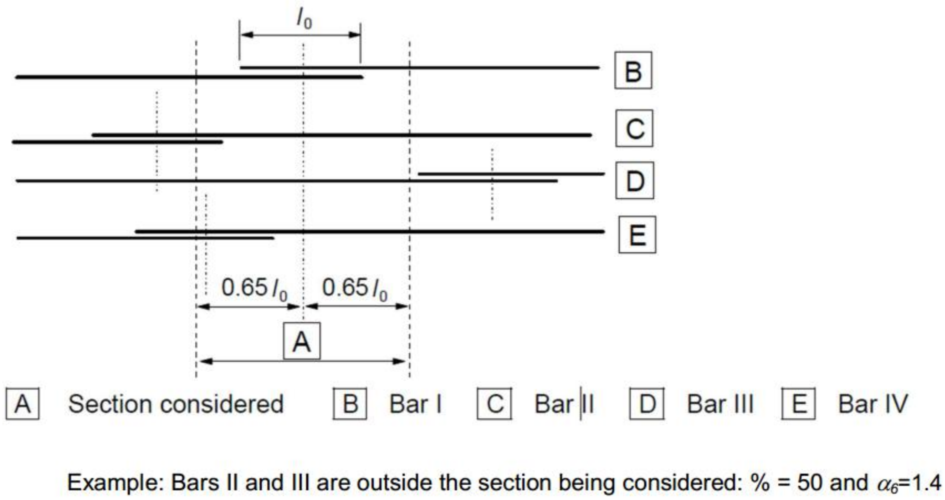


Figure 3- 5. percentage of lapped bars in one lapped section

Let all reinforcement in slab lapped on the same center, therefore $\alpha_6 = 1.5$ from the table value for $\rho_1 > 50$ %,

$$l_{o,min} \geq max \begin{cases} 0.3\alpha_6 l_{b,rqd} = 0.3 * 1.5 * 455.3 = 204.89mm \\ 15\emptyset = 15 * 10 = 150mm \\ 200mm \end{cases}$$

$l_{o,min}$= 204.89mm

$l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd} \geq l_{0,min}$

l_0 = 1*1*1*1.5* 455.3mm = 682.95 ≥ 204.89mm

lap length, l_0 = 683mm

3.1.11. Anchorage length

$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \ l_{b,rqd} \geq l_{b,min}$

for anchorages in tension

$$l_{b,min} \geq max\begin{cases} 0.3l_{b,rqd=} \\ 10\emptyset \\ 100mm \end{cases}$$

$$l_{b,min} \geq max\begin{cases} 0.3 * 455.3 = 136.59 \\ 10 * 10 \\ 100mm \end{cases}$$

$$l_{b,min} = 136.59mm$$

for anchorages in compression

$$l_{b,min} \geq max\begin{cases} 0.6l_{b,rqd} \\ 10\emptyset \\ 100mm \end{cases}$$

$$l_{b,min} \geq max\begin{cases} 0.6 * 455.3 = 273.18 \\ 10 * 10 \\ 100mm \end{cases}$$

$$l_{b,min} = 273.18mm$$

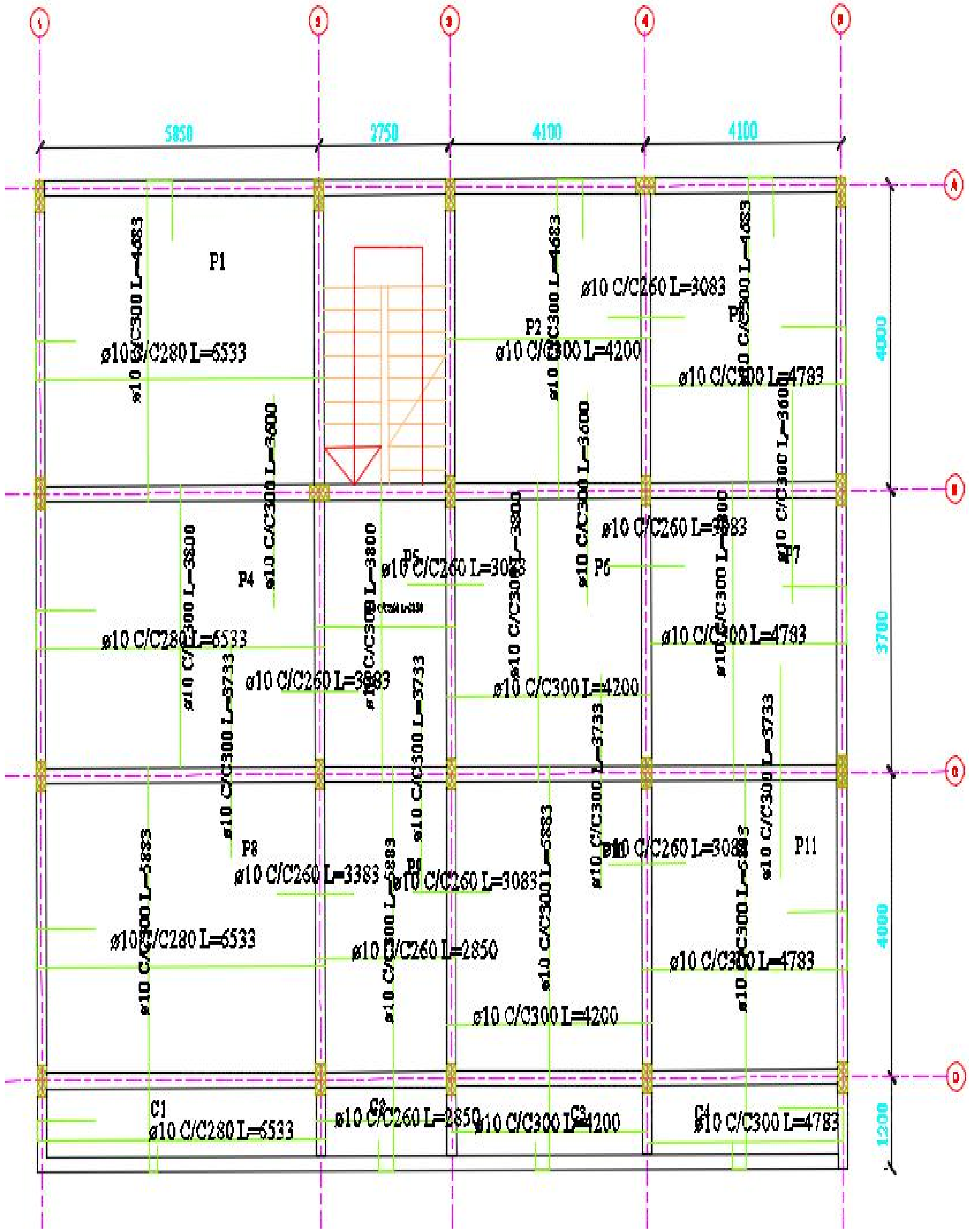


Figure 3- 6. slab reinforcement detailing

3.1.10. Re -Check Depth for Deflection: Serviceability Requirement

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

Checking depth of slab

$$N = \left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho}\right)^{\frac{3}{2}}\right] \; if\ \rho \leq$$

$$A_{st.pro} = \frac{bd}{s_{Pr0}} \qquad A_{st.pro} = \frac{1000*175}{220} = 795.5$$

$$\rho = \frac{A_{st.pro}}{bd} \qquad \rho = \frac{795.5}{1000*175} = 0.0043$$

$$\rho_o = \frac{\sqrt{f_{ck}}}{1000} = \frac{\sqrt{20}}{1000} = 0.00447$$

$$N = \left[11 + 1.5\sqrt{20}\frac{0.00447}{0.0043} + 3.2\sqrt{20}\left(\frac{0.00447}{0.0043}\right)^{\frac{3}{2}}\right] \; if\ \rho \leq \rho_0$$

N= 33.1412

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

$$F1 = \frac{500}{\left(f_{yk} * \frac{As\ cal}{As\ prov}\right)} = \frac{500}{\left(400 * \frac{354}{795.5}\right)} = 2.81$$

$$\frac{5000}{d} = 1.3 * 33.1412 * 2.81 * 1 * 1$$

d= 41.3002 mm

d ≥ 41.300 it safe

4. STAIR CASE

4.1. General

Stairs are structures which provide access to different floor levels. They are one of the structural elements constructed with steps rising to landing between floors with a serious of steps rising further from the landing to floor above. They are sloping one-way spanning slab.

Guide lines and types of stair

There are different types of staircases. Ours is categorized under half turn (scissor type).

The type of stair and its layout is governed essentially by the available size of staircase room and positions beams and columns along the boundary of stair case. Following are some useful guide lines in deciding the layout and type of stair:

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

- ✓ Heavy dead load due to inclined length of slab acting over horizontal span and due to additional weight of steps,
- ✓ Greater live load in stair than on floors. Therefore, longer span for flight should be avoided as far as possible.

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

a) Pinned – pinned support

We have two basic ways in which stairs are planned

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

c) Open well stairs, where a space or well exists between flights the stairs mentioned above are generally free-standing ones (simple straight flights of stairs).

4.2. Design procedure

- ✓ Determination of depth for deflection: which is a function of design tensile strength of steel, effective span length of the shortest span in which more load is expected to transfer and support condition
- ✓ Loading: which determines the total load in the stair and landing
- ✓ Analysis: determines moment and shear forces based on the analyzed moment
- ✓ Check depth for flexure: this step helps to cross check the design depth as it is safe for
- ✓ Flexure or not, if not revise the depth determined in step 1 and also the loads.
- ✓ Reinforcement provision: using the computed moments, number and area of
- ✓ Reinforcement bars determined.
- ✓ Detailing: the arrangement of reinforcement

STEP 1 Geometrical Data

- ✓ Riser height = 15cm
- ✓ Tread width = 30cm
- ✓ Number of risers = 10
- ✓ Number of treads = 10

$\theta = \tan^{-1}(\frac{1.5}{2.7}) = 29.05^0 = 30^0$

STEP 2 Determination of minimum depth for deflection

$$\frac{l}{d} = k\left[11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho} + 3.2\sqrt{fck}(\frac{\rho o}{\rho})^{\frac{3}{2}}\right] \quad \text{if } \rho \leq \rho o$$

$$\frac{l}{d} = k\left[11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho} + \frac{1}{12}\sqrt{fck} * \sqrt{\frac{\rho}{\rho}}\right] \quad \text{if } \rho > \rho o$$

Table 6. basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural System	K	Concrete highly stressed $\rho = 1.5\%$	Concrete lightly stressed $\rho = 0.5\%$
Simply supported beam, one – or twoway spanning simply supported slab	11	14	20
End span of continuous beam or oneway continuous slab or two-way spanning slab continuous over one long side	11.3	18	26
Interior span of beam or one-way or two-way spanning slab	11.5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	11.2	17	24
Cantilever	0.4	6	8

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

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

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

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

Where L= effective length of the beam

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

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

Therefore,

End span, 26*1.25=32.5

End span d=$\frac{L}{32.5} = \frac{2700mm}{32.5}$=83.08mm

Drequired=drequired+stirrup+diameter of $\frac{bar}{2}$ =83.08mm+8mm+20mm+10/2=116mm

Use D=130mm

STEP 3 Load Determination

Table 7. material unit weight and thickness

Material	Unit Weight (KN/m^3)	Thickness (cm)
Marble	27	3
Cement Screed	23	3
Concrete	25	-
Plastering	23	2

4.2.1. Loads in stair

a) Soffit slab

Self-weight of slab = 0.14mx1mx25KN/m^3/cos30^0 =4.04KN/m

Plastering = 0.02mx1mx25 KN/m^3/cos30^0=0.58KN/m

Total Dead Load =4.04 + 0.58= 4.62KN/m

b) Thread

Self-weight of thread = ($\frac{1}{2}$) x0.3x1x25 = 3.75KN/m

Thread Floor Finish = 0.03x27x1 = 0.81KN/m

Total Dead Load = 4.56KN/m

c) Riser

DL of riser=$\frac{number\ of\ riser(hcsxtcsx\delta cs)}{projected\ length}$

Riser finish=$\frac{number\ of\ riser(hcsxtcsx\delta cs)}{projected\ length}$ =$\frac{10(0.02x23x1x0.15)}{10x0.3}$ $= 0.23\text{KN/m}$

Cement screed=$\frac{number\ of\ riser(hcsxtcsx\delta cs)}{projected\ length}$ =$\frac{10(0.03x23x1x0.15)}{10x0.3}$ $= 0.345\text{KN/m}$

Total Dead Load =0.575KN/m

Total Dead Load On Stair =4.62 KN/m+4.56 KN/m+0.575 KN/m=9.755 KN/m

Live Load On Stair = 4KN/m^2 x 1m = 4KN/m

Pd=1.35DD+1.5LL=1.35x9.755+1.5x4 = 19.17 KN/m

4.2.2. Loads in landing

DL = Finishing DL + Cement Screed + Slab Self weight + Plastering

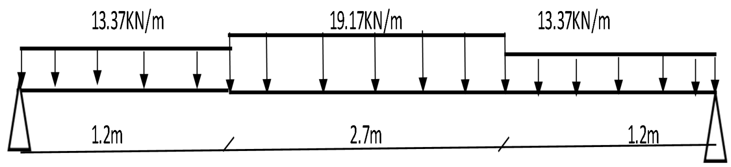
= (0.03x1x27) + (0.03x23x1) + (0.14x1x25) + (0.02x1x23) = 5.46KN/m

Live Load On Stair = 4KN/m2x1m = 4KN/m

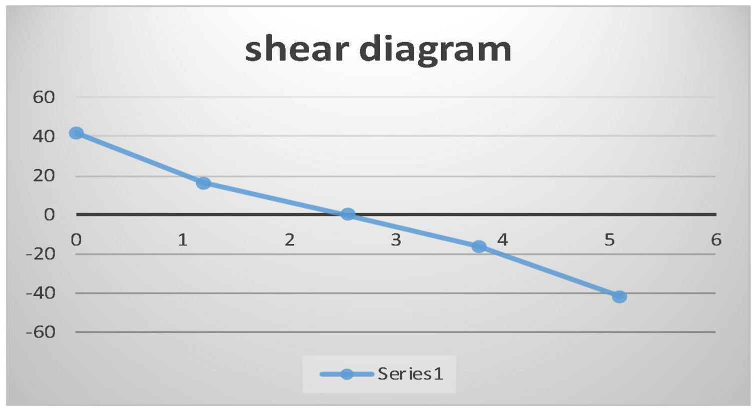
Pd=1.35DD+1.5LL=1.35x5.46+1.5x4 = 13.37 KN/m

STEP 4

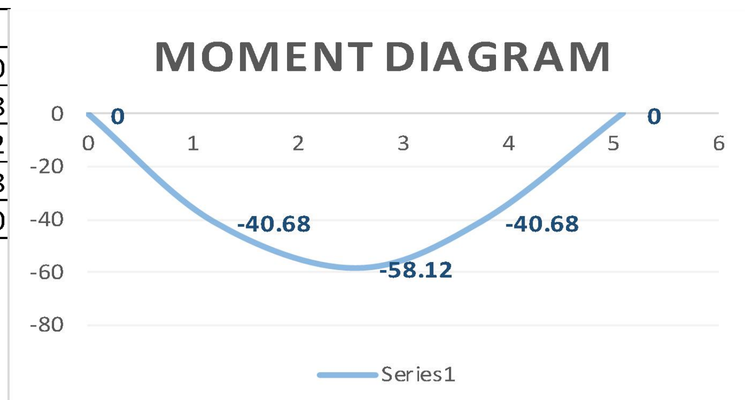
4.2.3. Moment Analysis



Length	LOAD
0	41.923
1.2	16.044
2.55	0
3.77	-16.044
5.1	-41.923



Length	Load
0	0
1.2	-40.68
2.55	-58.12
3.75	-40.68
5.1	0



STEP 5 Check depth for flexure

Maximum moment from all floor is 58.12 and take k value=0.22

dmin= $\sqrt{\frac{Msd}{kfckb}}$= $\sqrt{\frac{58.12*10^{6}}{0.22*25*1000}}$=102.8mm<130mm

Therefore, depth of 200 satisfy flexural requirement for all floors

STEP 6 Reinforcement

fcd=14.16MPa

fctm=2.6Mpa

fyd=347.83MPa

b=1000mm

D=140mm

Effective depth $d=D-\frac{10}{2}-20=105mm$

$As=\frac{Msd}{fydz}$

$z=0.5\ d\ (1+\sqrt{(1-3.53k)})$

$K=Msd/fckbd^2=58.12/(20x1000x105^2)=\underline{0.26}$

$Z=0.5*0.105(1+\sqrt{(1-3.53*0.26)})=67.55$

$As=(58.12x10^6)/(347.83x67.55)=2473.62mm^2=2474mm^2$

$S=\frac{bxas}{As}=\frac{1000x78.5}{2473.62}=31.73=30$

Secondary Reinforcement

$\geq 20\% Asmin =$

Spacing for the distribution bars,

Smax$\leq\{3.5h = 3.5x130 = 455mm\ and 450mm$

Therefor Smax=450mm

Finally, let's check the minimum reinforcement recommendation as per ES EN 1992 – 2015

Principal reinforcement

Asmin=Max$\left\{\frac{0.26fctmxbd}{fyk}\right.$ =211.11mm^2 and0.0013bd=182mm^2

Therefore Asmin =211.11mm^2

The spacing of the bars shouldn't exceed Smax Recommended value in ES EN 1992-2015,

For principal reinforcement, 3h = 400mm

For secondary reinforcement, 3.5h≤ 450mm

stair	Moment	d	k	z	As	Asprov	Scal	Sprov
support	0	105	0	0	0	0	0	0
span	58.12	105	0.22	83.9	2473.63	2474	31.7	θ12c/c300mm
support	40.68	105	0.18	84.2	1388.9	1389	56.5	θ12c/c500mm

4.2.4. Stair Detailing

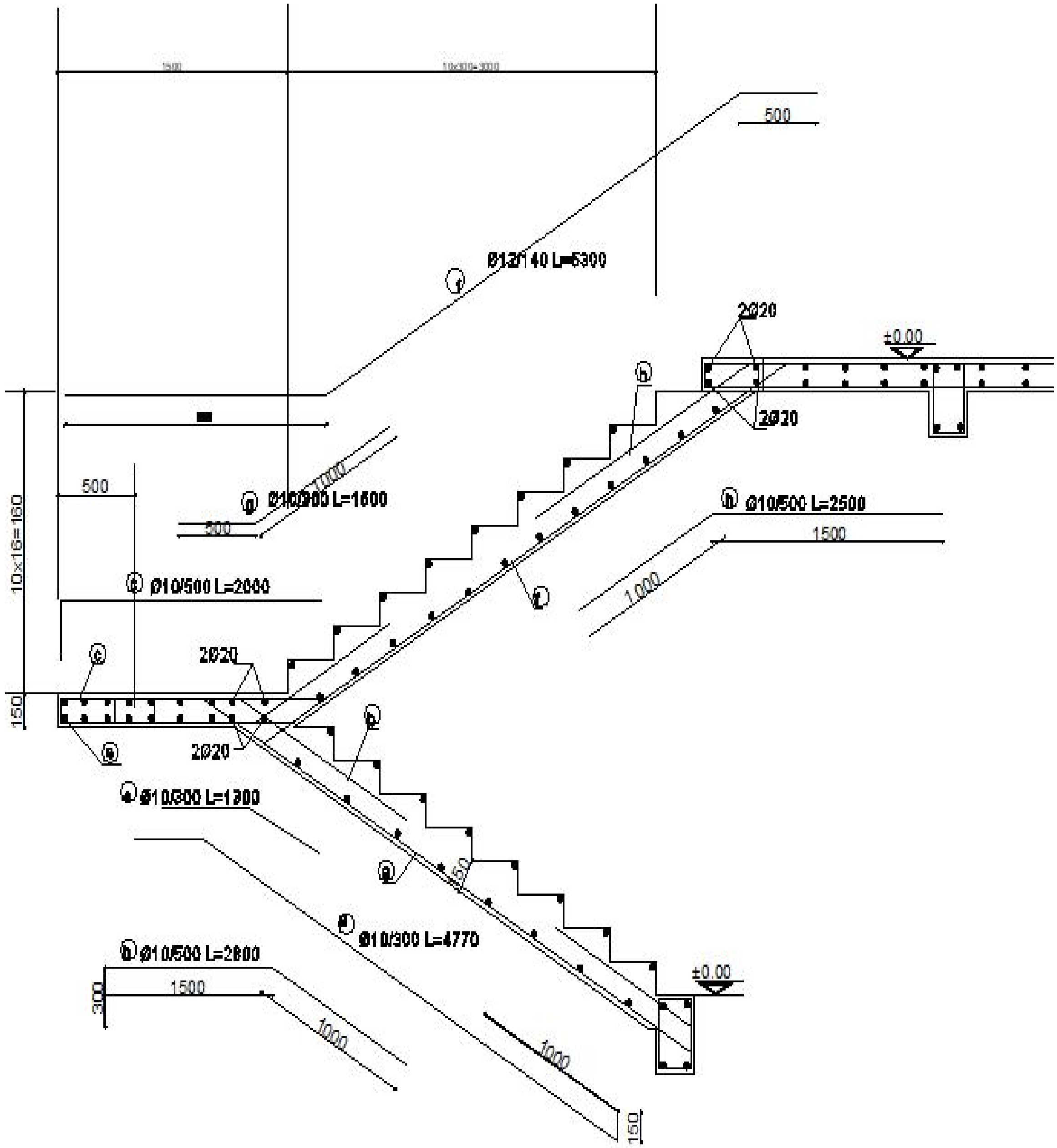


Figure 4- 1. Stair case detailing

5. LATERAL LOAD ANALYSIS

5.1. General

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

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

Design Basis:

Ground condition: (Assume sub soil class B)

Seismic zone: (Addis Ababa, Zone II)

5.2.Wind load Analysis

The wind load analysis is done by ETABS 2016 software. The input data from (*ES EN 1991-1-1*, 2015)is as follows

From the three different procedures of determining design wind actions, we select Static procedures of analysis because it is Suitable for the design of low-and mid-rise buildings

- Wind velocity=22m/s (*Wind Actions,ES EN 1991:2015*, 2018)
- Terrain category = III, by assuming the building is in area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest (*ES EN 1991-1-4*, 2015).
- Orthography factor = 1(*ES EN 1991-1-4*, 2015) for $\varphi < 0.05$, by assuming Lu > 0.72m, φ= H/Lu

A. Cs & Cd (*ES EN 1991-1-4*, 2015)
   - Cs = Cd = 0.85

- Turbulence factor =1 the recommended value from (*ES EN 1991-1-4*, 2015) (section 4.4)
- Air density = 1.25kg/m^3 (*ES EN 1991-1-4*, 2015) (section 4.5)

5.2.1. Wind load analysis on external wall (on building)

Buildings are exposed to wind load especially the roof and the walls. So, the effect of the wind on the building should be analyzed.

a) Wind in X direction

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

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = II

Wind Direction = 0;90 degrees

Basic Wind Velocity, V_b [EC 4.2(2)]	$V_b = 22 \frac{\text{meter}}{\text{sec}}$
Windward Coefficient, $C_{p,wind}$	$C_{p,wind} = 0.8$
Leeward Coefficient, $C_{p,lee}$	$C_{p,lee} = 0.5$
Air Density, ρ	$\rho = 1.25$

Top Story = Story4

Bottom Story = Base

Include Parapet = No

Factors and Coefficients

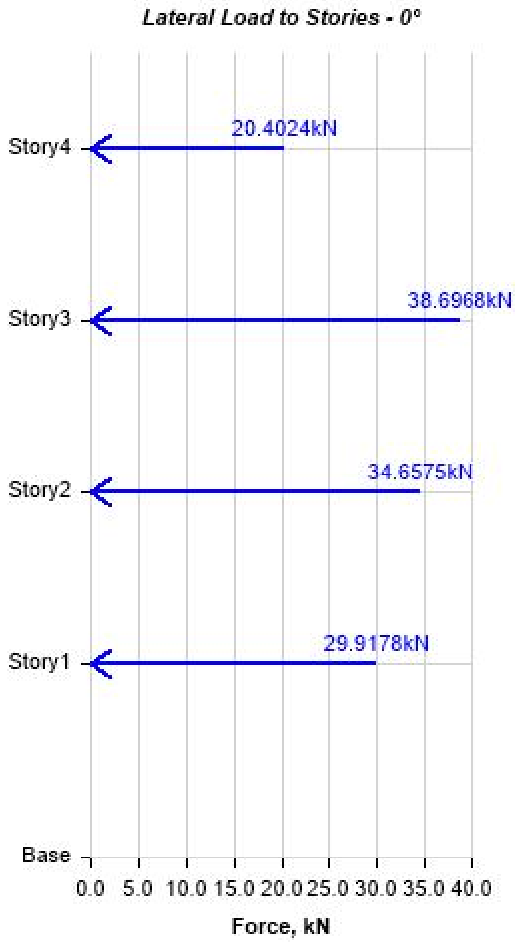
Structural Factor, cs<c$_d$> [EC 6.2(1)]	$c_s c_d = 1$
Elevation, z_0	$z_0 = 0.05$
Minimum Elevation, z_{min}	$z_{min} = 2$
Maximum Elevation, z_{max}	$z_{max} = 200$
Turbulence Factor, k_l [EC 4.4(1)]	$k_l = 1$
Orography Factor, c_o [EC 4.3.3]	$c_o = 1$

Turbulence Intensity, I_v [EC 4.4(1)]	$I_v = \frac{k_l}{c_o(z)\ln(\frac{z}{z_0})}$ for$z_{min} \leq z \leq z_{max}$
	$= I_v(z_{min})$ for zz_{min}
Terrain Factor, k_r [EC 4.3.2(1) Eq. 4.5]	$k_r = 0.19(^{\frac{z_0}{0.05}}$ $k_r = 0.19$
Roughness Factor, <$c_{r>(z)}$ [EC 4.3.2(1) Eq. 4.4]	$c_r(z) = k_r\ln(\frac{z}{z_0})$ for$z_{min} \leq z \leq z_{max}$
	$= c_r(z_{min})$ for zz_{min}

Lateral Loading

Peak Velocity Pressure, <$q_{p>(z)}$ [EC 4.5(1) Eq. 4.8]	$q_p(z) = [1 + 7I_v(z)]\frac{1}{2}\rho[c_r(z)c_o(z)v_b]^2$
Wind Pressure, w [EC 5.2(1) Eq. 5.1]	$w = q_p(z)c_s c_d(c_{p,wind} + c_{p,lee})$

Applied Story Forces



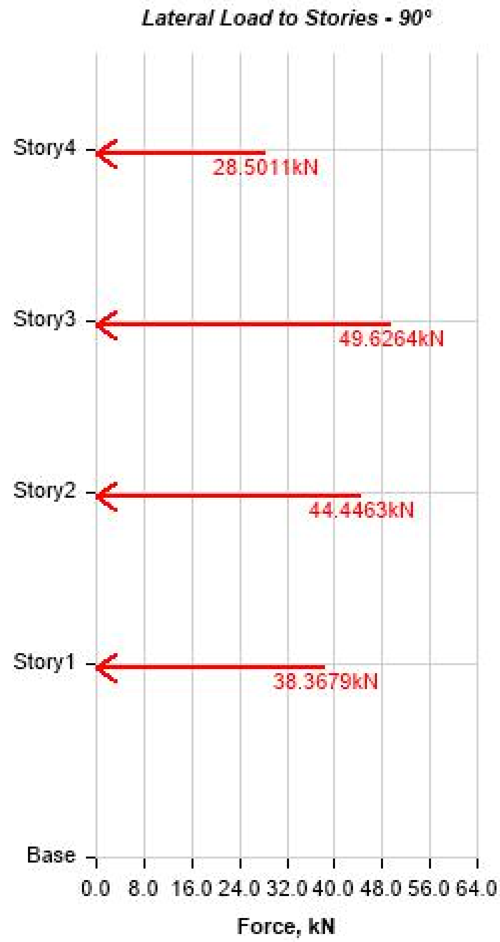


Figure 5- 1. wind load along y-axis

Table 8.wind story forces

Story	**Elevation**	**X-Dir**	**Y-Dir**
	M	**kN**	**kN**
Roof	13.2	20.4024	0
G+3	10	38.6968	0
G+2	6.8	34.6575	0
G+1	3.6	29.9178	0
Base	0	0	0

Story	**Elevation**	**X-Dir**	**Y-Dir**
	m	**kN**	**kN**
Roof	13.2	0	28.501
G+3	10	0	49.6264
G+2	6.8	0	44.4463
G+1	3.6	0	38.3679
Base	0	0	0

B) Wind y direction

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

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = II

Wind Direction = 0;90 degrees

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

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

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

Air Density, ρ $\rho = 1.25$

Top Story = Story4

Bottom Story = Base

Include Parapet = No

Factors and Coefficients

Structural Factor, cs<cd> [EC 6.2(1)] $c_s c_d = 1$

Elevation, z_0 $z_0 = 0.05$

Minimum Elevation, z_{min} $z_{min} = 2$

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

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

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

Turbulence Intensity, I_v [EC 4.4(1)] $I_v = \frac{k_l}{c_o(z)\ln(\frac{z}{z_0})}$ for$z_{min} \leq z \leq z_{max}$

$= I_v(z_{min})$ for zz_{min}

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

Roughness Factor, <cr>(z) [EC 4.3.2(1) Eq. 4.4] $c_r(z) = k_r \ln(\frac{z}{z_0})$ for$z_{min} \leq z \leq z_{max}$

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

Lateral Loading

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

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

Applied Story Forces

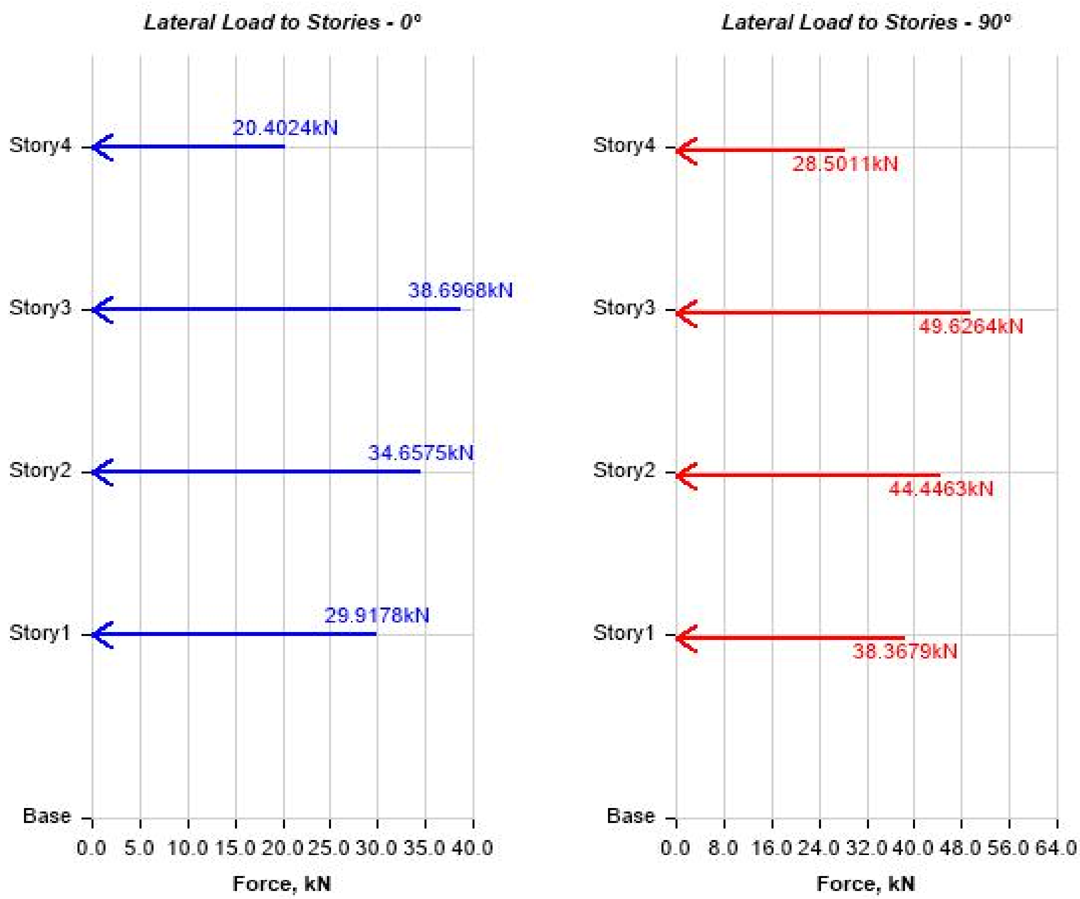


Figure 5- 2. wind load along x-axis

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story4	13.2	20.4024	0
Story3	10	38.6968	0
Story2	6.8	34.6575	0
Story1	3.6	29.9178	0
Base	0	0	0

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story4	13.2	0	28.5011
Story3	10	0	49.6264
Story2	6.8	0	44.4463
Story1	3.6	0	38.3679
Base	0	0	0

5.3.Earthquake load analysis

An earthquake is the vibration of earth produced by the rapid release of accumulated energy in elastically strained rock. The building is to be constructed in Wolkite city, which is located in. it is categorized seismic zone three.

Analysis of earthquake loads on building on ES EN 2015

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

Distribution of the base shear over the height of the building

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

Seismic load analysis (earth quake load analysis) also analyzed by ETABS software, the required parameters derived from ES EN 1998-1:2015, Design of Structures for Earthquake Resistance - Part 1: General rules - seismic actions and rules for buildings

A. Ground type = B, by assuming stratigraphic profile of Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.

B. Spectrum type = 2 If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M,, not greater than 5.5, it is recommended that the Type 2 spectrum is adopted.(*ES EN 1998-1*, 2015)

C. Seismic zone = 2 (*ES EN 1998-1*, 2015)

D. Ground acceleration = 0.07 for zone 2

- Soil factor = 1.2 , for spectrum type 2 & ground B
- Spectrum period, for spectrum type 2 & ground type B
  - Tb = 0.15
  - Tc = 0.5
  - Td = 2

- Behavioral factor = 4.9, for frame system structures 3.0αu/α1, αu/α1= 1.3 for multistory building.(*ES EN 1998-1*, 2015) (Appendix D, Table D4)
- Correction factor = 1, damping correction factor with a reference value of η= 1 for 5% viscous damping (*ES EN 1998-1*, 2015)
- Lower bound factor = 0.2 , recommended value from (*ES EN 1998-1*, 2015)

5.3.1. Base shear force

(1)P The seismic base shear force F_b, for each horizontal direction in which the building is analysed, shall be determined using the following expression:

$$F_b = S_d (T_1) \cdot m \qquad (4.5)$$

where

$S_d (T_1)$ is the ordinate of the design spectrum (see **3.2.2.5**) at period T_1;

T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered;

m is the total mass of the building, above the foundation or above the top of a rigid basement, computed in accordance with **3.2.4(2)**;

λ is the correction factor, the value of which is equal to: λ= 0.85 if $T_1 \leq 2\ T_C$ and the building has more than two storeys, or λ = 1.0 otherwise.

NOTE The factor λ accounts for the fact that in buildings with at least three storeys and translational degrees of freedom in each horizontal direction, the effective modal mass of the 1st (fundamental) mode is smaller, on average by 15%, than the total building mass.

(2) For the determination of the fundamental period of vibration T_1 of the building, expressions based on methods of structural dynamics (for example the Rayleigh method) may be used.

(3) For buildings with heights of up to 40 m the value of T_1 (ins) may be approximated by the following expression:

$$TT_1 = C_t \cdot H^{3/4} \quad (4.6)$$

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.

For concrete moment resisting frame Ct=0.075, H=34.94m

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

$T1=0.075*34.94^{0.75}=1.078sec$

T1≤ min (4Tc(s) =4*0.5=2sec, 2sec), T1=2sec < 1sec…. not ok Take T1=1sec

Table 9. Importance classes for buildings

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

NOTE Importance classes I, II and III or IV correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EBCS EN 1990:2013, Annex B.

Distribution of the horizontal seismic forces

(1) The fundamental mode shapes in the horizontal directions of analysis of the building may be calculated using methods of structural dynamics or may be approximated by horizontal displacements increasing linearly along the height of the building.

(2)P The seismic action effects shall be determined by applying, to the two planar models, horizontal forces F_i to all storeys.

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j} \quad (4.10)$$

where

F_i is the horizontal force acting on storey i;

F_b is the seismic base shear in accordance with expression (4.5);

s_i, s_j are the displacements of masses m_i, m_j in the fundamental mode shape;

m_i, m_j are the storey masses computed in accordance with **3.2.4(2)**.

(3) When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i should be taken as being given by:

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \qquad (4.11)$$

where

z_i, z_j are the heights of the masses m_i,m_j above the level of application of the seismic action (foundation or top of a rigid basement).

(4)P The horizontal forces F_i determined in accordance with this clause shall be distributed to the lateral load resisting system assuming the floors are rigid in their plane.

NOTE The value to be ascribed to β for use is found in the National Annex. The recommended value for β is 0,2.

For frame system and DCM, qo = 3 α u /α 1

For buildings which are not regular in elevation, the value of qo should be reduced by 20%

qo = 3 α u /α 1 - 0.2*3 α u /α 1 = 0.8 * 3 α u /α 1

α1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant;

αu is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor αu may be obtained from a nonlinear static (pushover) global analysis.

When the multiplication factor αu/α1 has not been evaluated through an explicit calculation, for buildings which are regular in plan the following approximate value of αu/α1 may be

Frames or frame-equivalent dual systems.

− One-storey buildings: αu/α1=1,1;

− multistorey, one-bay frames: αu/α1=1,2;

− multistorey, multi-bay frames or frame-equivalent dual structures: αu/α1=1,3.

α u /α 1 = 1.3

For buildings which are not regular in plan (see 4.2.3.2), the approximate value of αu/α1 that may be not performed for its evaluation are equal to the average of (a) 1,0 and of (b) the value given in (5) of this subclause.

The factor kw reflecting the prevailing failure mode in structural systems with walls shall be 1.0 for frame and frame equivalent dual system.

$$T_D \le T: \quad S_d(T) = \begin{cases} = a_g \cdot S \cdot \dfrac{2.5}{q} \cdot \left[\dfrac{T_C \cdot T_D}{T^2}\right] \\ \ge \beta \cdot a_g \end{cases}$$

$S_d(T)$ is the design spectrum;
q is the behaviour factor;
β is the lower bound factor for the horizontal design spectrum

$q=q_o k_w \ge 1.5=0.8*3*1.3*1=3.12$

Sd(T) = Max {**0.2638** ,**0.1962**

Sd(T) =0.2638

Table 10.center of mass and rigidity

Story	Diaphragm	Mass X kg	Mass Y Kg	XCM m	YCM m	Cumulative X kg	Cumulative Y kg	XCCM m	YCCM m
Roof	D1	79370.92	79370.92	8.6303	4.8179	79370.92	79370.92	8.6303	4.8179
G+3	D1	171465.89	171465.89	8.6841	5.2038	171465.89	171465.89	8.6841	5.2038
G+2	D1	164844.81	164844.81	8.7292	5.2356	164844.81	164844.81	8.7292	5.2356
G+1	D1	192407.12	192407.12	8.453	4.987	192407.12	192407.12	8.453	4.987

$F_b = S_d(T_1) \cdot W_{tot}$=0.2638*1216177.48=320827.6192KN

Ft=0.07T1Fb=0.07*1*320827.6192=22457.9334KN

a) Earth quake x direction

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

Direction and Eccentricity

Direction = X + Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2] $C_t = 0.075$m

Structure Height Above Base, H H = 13.2 m

Factors and Coefficients

Country = Other

Design Ground Acceleration, a_g $a_g = 0.07$g

Ground Type [EC Table 3.1] = B

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

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

Seismic Response

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

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

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

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

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

Equivalent Lateral Forces

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

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X + Ecc. Y	0.685	14092.1043	1079.6529

Applied Story Forces

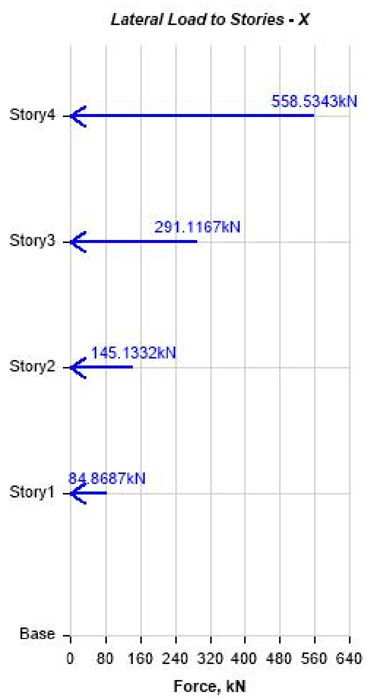


Figure 5- 3. lateral load to stories x

Table 11. lateral load to stories x

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story4	13.2	558.5343	0
Story3	10	291.1167	0
Story2	6.8	145.1332	0
Story1	3.6	84.8687	0
Base	0	0	0

b) Earth quake y direction

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

Direction and Eccentricity

Direction = Y - Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075m$
Structure Height Above Base, H	$H = 13.2\ m$

Factors and Coefficients

Country = Other

Design Ground Acceleration, a_g	$a_g = 0.07g$

Ground Type [EC Table 3.1] = B

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

Seismic Response

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

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

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

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

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

Equivalent Lateral Forces

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

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
Y - Ecc. X	0.698	14092.1043	1060.3826

Applied Story Forces

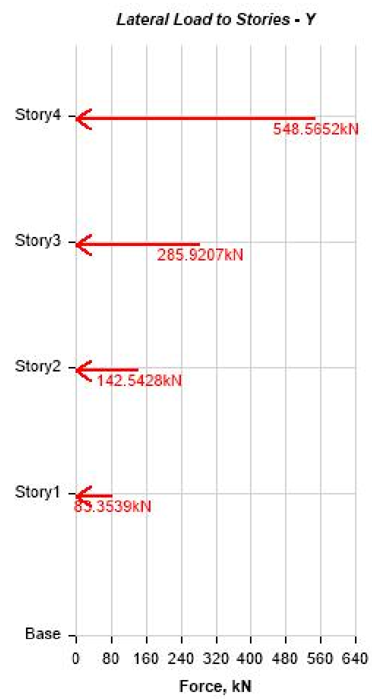


Figure 5- 4.lateral load on story y

Table 12.lateral load on story y

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story4	13.2	0	548.5652
Story3	10	0	285.9207
Story2	6.8	0	142.5428
Story1	3.6	0	83.3539
Base	0	0	0

6. Frame analysis

6.1. General

Frames are often used in buildings and composed of beams and columns that are either pin or fixed connected. The loading on a frame causes bending of its member and if it has right joint connection. This structure is generally indeterminate the strength of such frame is derived from the moment inter connection between the beams and the columns at the rigid joints.

Frame can be classified in to two

- ✓ Non sway frame
- ✓ There is no horizontal movement
- ✓ When the frame is symmetric with geometric load
- ✓ Deflection is zero (Δ=0)
- ✓ Bending does not cause the joint to have a linear displacement
- ✓ Sway frame: It is lateral displacement and have horizontal movement

  Generally, frame will be sway

  - ✓ If the frame is not symmetric with geometric load / eccentricity
  - ✓ If we have inclined structure
  - ✓ Un symmetric shape of frame
  - ✓ d/t end condition of the columns
  - ✓ non uniform section of the frame
  - ✓ horizontal loading
  - ✓ settlement of support
  - ✓ combination of the above

On sway frame deflection is not equal to zero. Since our frame is subjected to un symmetric load the analysis procedure is carried out as sway frame.

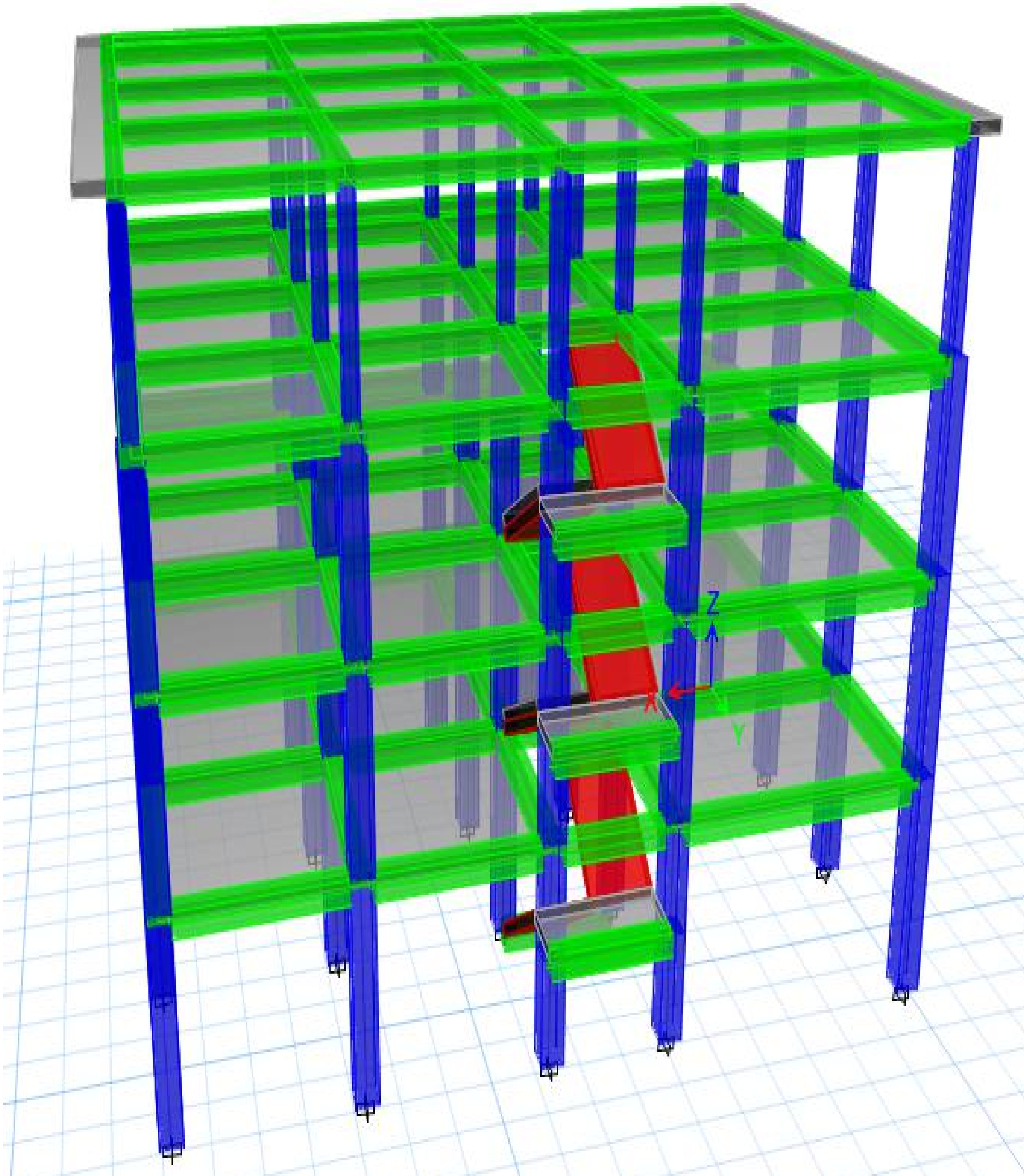


Figure 6- 1.3D of the building

Frame is analyzed to get maximum shear and moments in the structure by using ETABs 2016 software.

6.2.Analysis procedure

The steps to analyze the structure described below.

1. Select the Base Units and Design Codes: - we select the base unit to metric SI and design codes of Euro Code
2. Set up Grid Lines: - set up number and spacing of grid lines in both x and y direction by using Arctectural drawing data.
3. Define Story Levels: - define story height and label them.
4. Define material Properties: - define material properties that we used in overall design process. For initial we define C-25 concrete and S-400 rebar according to material data's on (*ES EN 1992-1-1*, 2015)

A. Concrete: -

Grade - C25

Modules of elasticity – 31 GPa

Weight per unit volume- 25kg/m3.

Poisson's ratio, u – is the ratio between lateral strains and longitudinal strains in a material subjected to loading, Poisson's ratio of concrete varies between 0.1 for high strength mix and 0.2 for weak mixes. It is normally taken as 0.15 for strength design and 0.2 for serviceability criteria.

Coefficient of thermal expansion: - when free to deform concrete will expand and contract due to fluctuations in temperature. An average value for coefficient of the coefficient of thermal expansion of concrete is about 10 millionths per degree Celsius (10*10-6 /c).

The software itself calculates shear modulus, the formula for shear modulus=E/2(1+U).

B. Rebar

Grade – S400

Weight per unit volume = 78.6KN/m3

Modulus of elasticity =200GPa=200000MPa

Coefficient of Thermal expansion=0.0000117

5. Define Section properties: - for initial we define column, beam and slab sections.

   a. Beam: - Cross section 400x400 and top tie beam 300x300

   b. Column: - Cross section 500x500 and 300x300

   c. Slab: - depth of 200mm

6. Draw Structural Objects: - draw the structural objects included in architectural drawing

7. Define Load Patterns: - define the loads on the structure

Table 13.load patterns

Name	Type	Self Weight Multiplier	Auto Load
Dead	Dead	1	
Live	Live	0	
beam partition	Dead	0	
EQXT	Seismic	0	EUROCODE8 2004
EQXB	Seismic	0	EUROCODE8 2004
EQYR	Seismic	0	EUROCODE8 2004
EQYL	Seismic	0	EUROCODE8 2004
Wind X	Wind	0	EUROCODE1 2005
Wind Y	Wind	0	EUROCODE1 2005
Supper dead load	Dead	0	
stair case	Dead	0	
Slab load	Dead	0	

8. Define load combinations.

Table 14.load combinations

comb 1	1.35D	1.5LL			linear add
comb 2	DL	LL			linear add
comb 3	DL	0.6LL	EQXL	0.3EQYL	linear add
comb 4	DL	0.6LL	EQXR	0.3EQYL	linear add
comb 5	DL	0.6LL	-EQXL	0.3EQYL	linear add
comb 6	DL	0.6LL	-EQXR	0.3EQYL	linear add
comb 7	DL	0.6LL	EQXL	0.3EQYR	linear add
comb 8	DL	0.6LL	EQXR	0.3EQYR	linear add
comb 9	DL	0.6LL	-EQXL	0.3EQYR	linear add
comb 10	DL	0.6LL	-EQXR	0.3EQYR	linear add
comb 11	DL	0.6LL	EQXL	-0.3EQYL	linear add
comb 12	DL	0.6LL	EQXR	-0.3EQYL	linear add
comb 13	DL	0.6LL	-EQXL	-0.3EQYL	linear add
comb 14	DL	0.6LL	-EQXR	-0.3EQYL	linear add
comb 15	DL	0.6LL	EQXL	-0.3EQYR	linear add
comb 16	DL	0.6LL	EQXR	-0.3EQYR	linear add
comb 17	DL	0.6LL	-EQXL	-0.3EQYR	linear add
comb 18	DL	0.6LL	-EQXR	-0.3EQYR	linear add
comb 19	DL	0.6LL	EQYL	0.3EQXL	linear add
comb 20	DL	0.6LL	EQYR	0.3EQXL	linear add
comb 21	DL	0.6LL	-EQYL	0.3EQXL	linear add
comb 22	DL	0.6LL	-EQYR	0.3EQXL	linear add
comb 23	DL	0.6LL	EQYL	0.3EQXR	linear add
comb 24	DL	0.6LL	EQYR	0.3EQXR	linear add
comb 25	DL	0.6LL	-EQYL	0.3EQXR	linear add
comb 26	DL	0.6LL	-EQYR	0.3EQXR	linear add
comb 27	DL	0.6LL	EQYL	-0.3EQXL	linear add
comb 28	DL	0.6LL	EQYR	-0.3EQXL	linear add
comb 29	DL	0.6LL	-EQYL	-0.3EQXL	linear add
comb 30	DL	0.6LL	-EQYR	-0.3EQXL	linear add
comb 31	DL	0.6LL	EQYL	-0.3EQXR	linear add
comb 32	DL	0.6LL	EQYR	-0.3EQXR	linear add
comb 33	DL	0.6LL	-EQYL	-0.3EQXR	linear add
comb 34	DL	0.6LL	-EQYR	-0.3EQXR	linear add
comb 35	1.35DL	1.5LL	0.9WL		linear add
comb 36	1.35DL	1.5LL	-0.9WL		linear add
comb 37	1.35DL	1.5WL			linear add
comb 38	comb 3-18				envelope
comb 39	comb 19-34				envelope
comb 40	comb 35-39				envelope
comb 43	comb 1-40				envelope

9. Load the structural model

- calculate the units of load for all defined load patters by hand and find other loading properties from the code like live load
- Example on shell loads(uniform)
    - For super dead load (floor finish and celling plaster)

Assume marble floor finish with unit weight of 23KN/m^3 and thickness of 20mm

Load of floor finish= 23 *0.002=0.046 KN/m^2

Assume celling plaster thickness=30mm and unit weight of 23 KN/m^3

Load of celling plaster=23*0.003=0.069 KN/m^2

Super dead load= 0.046 + 0.069= **0.115 KN/m^2**

- Example on frame loads (distributed)
    - For beam partition load
    - Assume partition thickness 15cm, height 3m, made of HCB unit weight 14 KN/m^3

Partition load=0.15m*3m*14KN/m^3 =**6.3 KN/m**

10. Assign Joint restraint: - assign joint on the bottom of the foundation column to fixed support.
11. Assign Joint diaphragm: - assign joint diaphragm for the whole structure to calculate center of mass.
12. Analyze the model:- analysis the model to get moment and shear results.

6.3.ETABS out puts

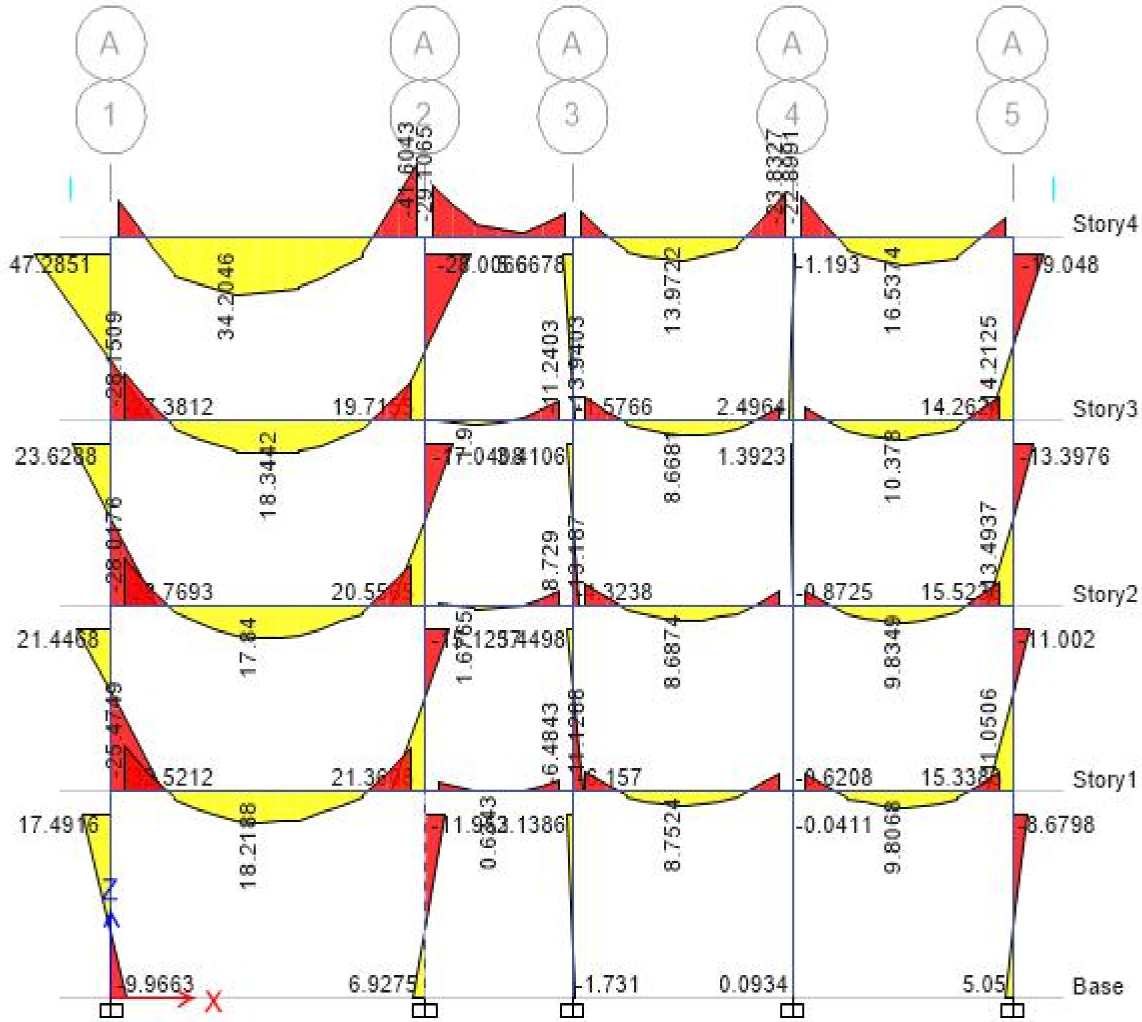


Figure 6- 2bending moment diagram

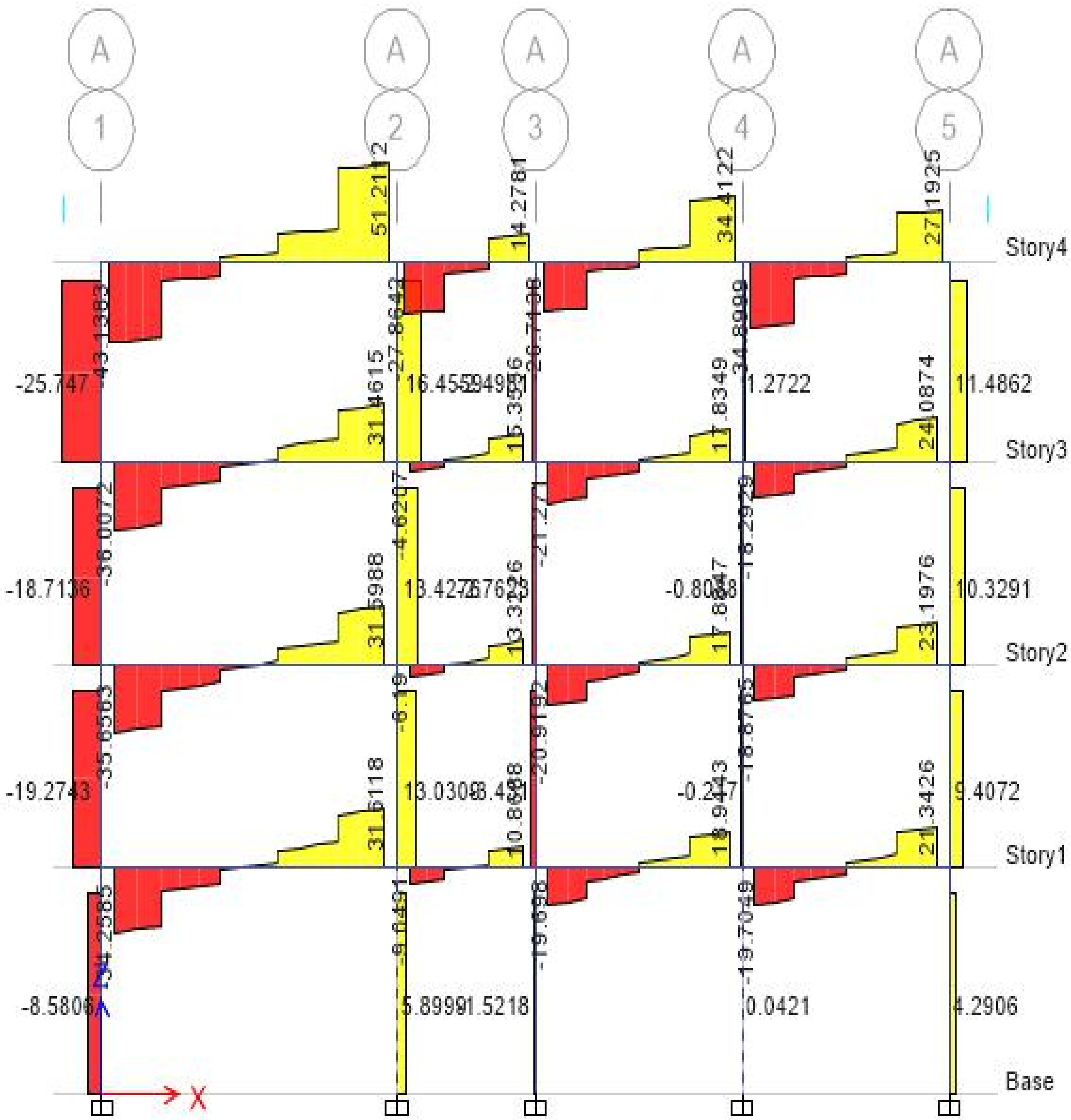


Figure 6- 3shear force diagram

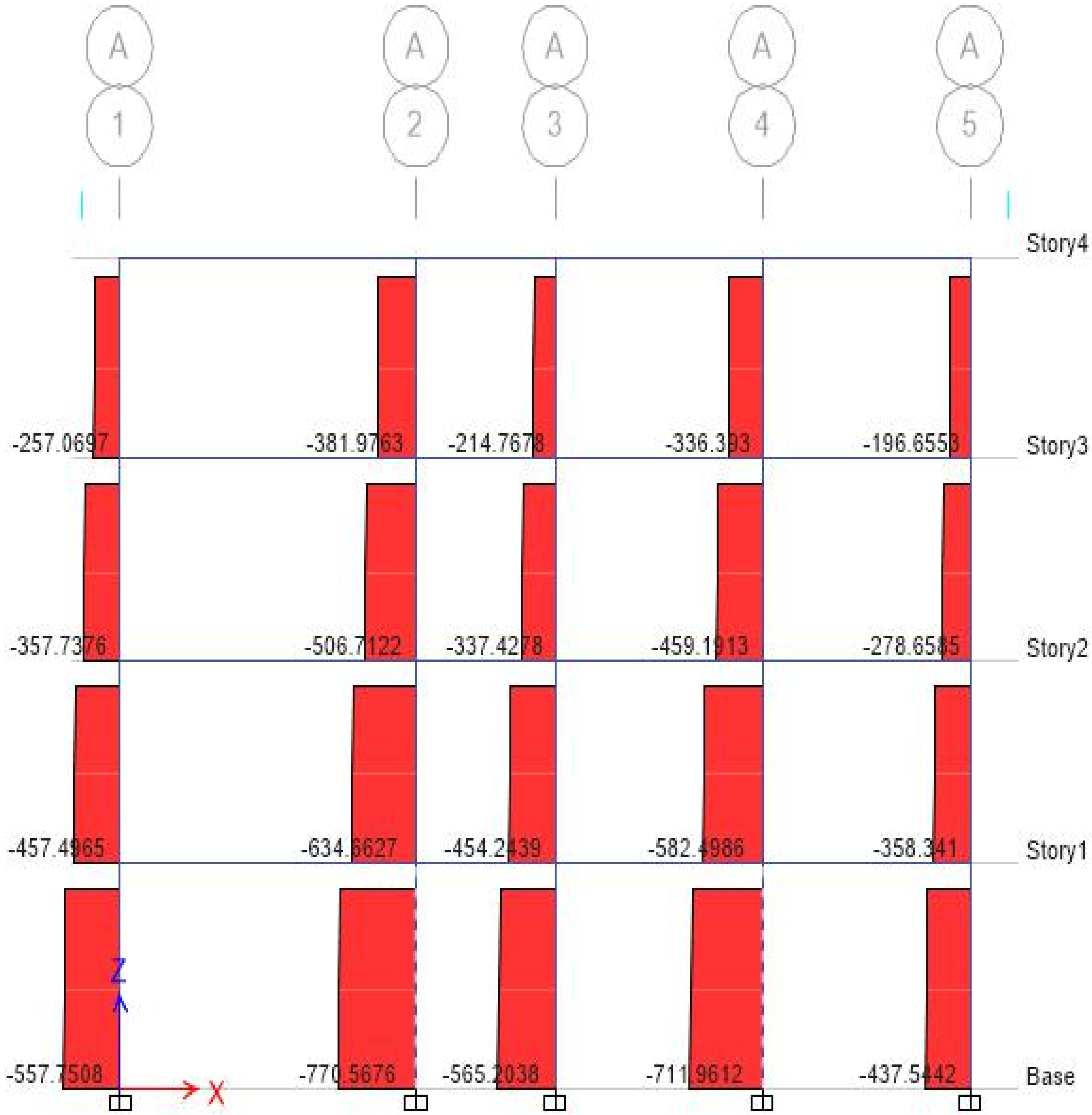


Figure 6- 4Axial load diagram

7. BEAM DESIGN

7.1. General

Beam is a horizontal structural member which offer resistance to bending due to applied loads. As any members, beam is also subjected to different kinds of loadings. Its mode of deflection is primarily by bending. The loads applied to the beam result in reaction forces at the beam's support points. The total effect of all the forces acting on the beam is to produce shear forces and bending moments within the beam, that in turn induce internal stresses, strains and deflections of the beam. Beams are characterized by their manner of support, profile (shape of cross-section), length, and their material.

7.2. types of beam

Singly reinforced beam is reinforced with steel bars only at the tension zone.

Doubly reinforced beam is reinforced in both zones (tension & compression zone).

If a section of a beam is limited in depth, it cannot develop the compressive force required to resist the applied bending moment. If it’s a small increase in moment, over reinforced beam section can be used (not recommended in design). In such case, providing a reinforcement in the compression zone assists the concrete in resisting compressive force. This kind of beam section is called doubly reinforced beam. If the required depth is also unacceptable, doubly reinforced beam is provided.

7.3. Beam Design

7.3.1 Design Data

Dimension – 300x300mm

Material used

A -Concrete

- ✓ Grade of concrete C-25
- ✓ Concrete cube strength, *fcu*=25MPA
- ✓ Concrete characteristic strength=*0.8fcu*=0.8*25=20MPA
- ✓ Concrete design strength, $fcd = \alpha cc * \frac{fck}{\gamma c} = 0.85 * \frac{25}{1.5} = 11.33$MPa

B -Steel S-400

- ✓ Steel tensile strength, 400MPA
- ✓ Steel tensile design strength, $fyd = \frac{fyk}{\gamma s} = \frac{400}{1.15} = 347.83$MPa
- ✓ Elastic modulus of steel, Es=200Gpa

7.3.2. Concrete Cover

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

Cnom=Cmin+ΔCdev

Where,

a) min, Minimum concrete cover

ΔCdev, is allowance in design for deviation.

Minimum concrete cover shall be provided in order to ensure:

- ✓ The safe transmission of bond forces
- ✓ An adequate fire resistance
- ✓ The protection of the steel against corrosion (durability)

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

$$C_{min}=\max\begin{cases} C_{min,b} \\ C_{min,dur}+\Delta C_{dur,\boldsymbol{B}}-\Delta C_{dur,st}-\Delta C_{dur,add} \\ 100 \end{cases}$$

Where;

- ✓ Cmin, b -minimum cover due to bond requirement, see ES EN1992:2015 Art. 4.4.1.2 (3).
- ✓ Cmin,dur - minimum cover due to environmental conditions, see ES EN 1992:2015 Art 4.4.1.2 (5)
- ✓ ΔCdur,γ- additive safety element, see ES EN1992:2015 Art 4.4.1.2 (6)
- ✓ ΔCdur,st -reduction of minimum cover for use of stainless steel, see ES EN 1992:2015 Art 4.4.1.2 (7)
- ✓ ΔCdur,add -reduction of minimum cover for use of additional protection, see ES EN1992:2015 Art 4.4.1.2 (8)

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

a) C min, b minimum cover due to bond requirement

Therefore, As the bars are arranged separately, the minimum cover due to bond requirement, C min, b is 20mm (diameter of the bar).

b) C min, dur minimum cover due to environmental conditions

Therefore, As the structural class is S1 & Exposure class is XC1, the minimum cover due to environmental requirement, C min, dur is 10mm.

c) ΔCdur, γ additive safety element

The concrete cover should be increased by the additive safety element ΔCdur, .

Note: The value of ΔCdur γ for use in a Country may be found in its National Annex. The recommended value is 0mm.

Therefore, the concrete cover for safety element, ΔCdur, γ is 0mm.

d) ΔC dur,st reduction of minimum cover for use of stainless steel

Where stainless steel is used or where other special measures have been taken, the minimum cover may be reduced by ΔCdur, γ. For such situations the effects on all relevant material properties should be considered, including bond.

Note: The recommended value ΔCdur ,st. , without further specification, is 0mm.

Therefore, the reduction of minimum cover for use of stainless steel, ΔCdur ,st is 0mm.

e) ΔC dur, add reduction of minimum cover for use of additional protection

For concrete with additional protection (e.g. coating) the minimum cover may be reduced by Δc dur,add.

Note: The recommended value, for Δc dur,add without further specification, is 0 mm.

Finally, $Cmin=max\begin{cases} Cmin,b \\ Cminb,dur+\Delta Cdur,\boldsymbol{B}-\Delta Cdur,st-\Delta Cdur,add \\ 10 \end{cases}$

$Cmin=max\begin{cases} 20,10+0,0-0-0,10 \end{cases}$

Cmin=20mm

f) Allowance in design for deviation

To calculate the nominal cover, Cnom an addition to the minimum cover shall be made in design to allow for the deviation (ΔCdev). The required minimum cover shall be increased by the absolute value of the accepted negative deviation.

Note: The value of ΔCdev for use in a Country may be found in its National Annex. The recommended value is 10 mm. In certain situations, the accepted deviation and hence allowance, (ΔCdev) may be reduced.

Note: The reduction in ΔCdev in such circumstances for use in a Country may be found in its National Annex. The recommended values are:

- ✓ Where fabrication is subjected to a quality assurance system, in which the monitoring includes measurements of the concrete cover, the allowance in design for deviation ΔC dev may be reduced:

10 mm ≥ Δc dev ≥ 5 mm

- ✓ Where it can be assured that a very accurate measurement device is used for monitoring and non-conforming members are rejected (e.g. precast elements), the allowance in design for deviation ΔCdev may be reduced:

10 mm ≥ Δc dev ≥ 0 mm

Therefore, we have taken ΔCdev = 10mm

Cnom=Cmin+ΔCdev =20mm+10mm=30mm

7.3.3. Check depth for deflection

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

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

axial compression

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

Where L= effective length of the beam

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

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

Therefore,

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

End span,d=L/26=5800/32.5=178.5

Interior span,d=L/30= 5800/37.5=154.7

$$Drequired = drequired + stirrup + \frac{\O bar}{2} = 178.5 + 8 + \frac{20}{2} = 197mm$$

Therefore, D provided=500>Drequired …ok

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

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

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

Where $K = \frac{Msd}{bd2fck} = \frac{31.61}{300*0.5*20} = 0.026$

7.3.4. provide reinforcement

$$Z = \frac{d}{2}(1 + \sqrt{1 - 3.53K} = \frac{450}{2}(1 + \sqrt{1 - 3.53 * 0.026}) = 439.42$$

$$Ast,cal = \frac{Msd}{Zfyd} = \frac{31.61}{439.42 * 347.83} = 206.81$$

check the minimum and maximum reinforcement

$$A_{st},min = max \begin{cases} 0.26\frac{f_{ctm}}{fyk}bd = 0.26\frac{2.2}{400} * 300 * 450 = 193.05\,mm^2 \\ 0.0013bd = 0.0013 * 300 * 450 = 175.5\,mm^2 \end{cases}$$

A_{st},min=193.05mm^2

A_{st},max=0.04bd=0.04*300*450=5400mm^2

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

$$no\ of\ bar = \frac{Aspro}{Ast} = \frac{206.8155}{314} = 0.66$$

Use2 Ø 20

The rest of beams is calculated in tabular form

7.3.5. Lap length

l_0= α_1 α_2 α_3 α_5 α_6 $l_{b,rqd} \geq l_{0,min}$ (*ES EN 1992-1-1*, 2015)

$$l_{o,min} \geq max \begin{cases} 0.3\alpha_6 l_{b,rqd} \\ 15Ø \\ 200mm \end{cases}$$

Where: $l_{b,rqd} = \frac{Ø}{4} * \frac{\sigma sd}{fbd}$

$\sigma sd = \frac{fyk}{\gamma_s}$

$fbd = 2.25\eta_1\eta_2 f_{ctd}$

$\eta_1 = 1\ for\ good\ quality\ of\ bond$

$\eta_1 = 0.7\ for$ *other cases* (*ES EN 1992-1-1*, 2015) Art, 8.4.2

$\eta_2 = 1\ for\ \emptyset \leq 32mm$

$\eta_2 = \frac{132-\emptyset}{100}\ for\ \emptyset > 32mm$

$fctd = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$

$\alpha_{ct} = 0.85, recommended\ value$

Sample calculation for $\emptyset 20$ reinforcement bar

$$fctd = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$\alpha_{ct} = 0.85, recommended\ value$

$f_{ctk} =$ 1.$mpa, from\ ESEN$ 1992 Table 3.1

$fctd = \frac{0.85*1.5}{1.5}$= **0.85**

$fbd = 2.25\eta_1\eta_2 f_{ctd}$

$\eta_1 = 1$, *our* slab D is < 250mm, good condition of bond

$\eta_2 = 1\ ,for\ \emptyset \leq 32mm$

$f_{ctd} =$ 0.85

$fbd = 2.25 * 1 * 1 * 0.85 = \mathbf{1.91}$

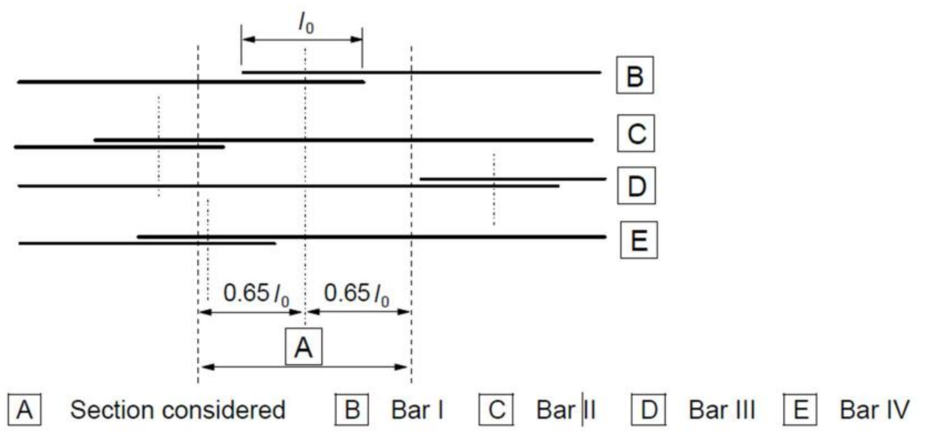
$$\sigma sd = \frac{fyk}{\gamma_s} = \frac{400}{\mathbf{1.15}} = \mathbf{347.83}$$

$$l_{b,rqd} = \frac{\emptyset}{4} * \frac{\sigma sd}{fbd} = \frac{20}{4} * \frac{347.83}{1.91} = \mathbf{910mm}$$

$l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd} \geq l_{0,min}$

α_1, α_2 & α_3 = 1, α_4 = 0.7, for reinforcement bar in compression

$\alpha_6 = \left(\frac{\rho_1}{25}\right)^{0.5}$ but not exceeding 1.5 nor less than 1.0, where ρ_1 is the percentage of reinforcement lapped within 0.65 l0 from the center of the lap length considered



Example: Bars II and III are outside the section being considered: % = 50 and α_6=1.4

Figure 8.8: Percentage of lapped bars in one lapped section

Let all reinforcement in slab lapped on the same center, therefore **α_6 = 1.5** from the table value for $\rho_1 > 50\,\%$,

$$l_{o,min} \geq max \begin{cases} 0.3\alpha_6 l_{b,rqd} = 0.3 * 1.5 * 910 = 409.5mm \\ 15\emptyset = 15 * 20 = 300mm \\ 200mm \end{cases}$$

$l_{o,min}$= 409.5mm

$l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd} \geq l_{0,min}$

$l_0 = 1*1*1*1.5*910mm = 1365 \geq 286.82$ mm

lap length, l_0 = **1365 mm**

7.3.6. Anchorage length

$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \ l_{b,rqd} \geq l_{b,min}$

for anchorages in tension

$$l_{b,min} \geq max\begin{cases}0.3l_{b,rqd}\\ 10\emptyset\\ 100mm\end{cases}$$

$$l_{b,min} \geq max\begin{cases}0.3 * 910 = 273\\ 10\emptyset = 200\\ 100mm\end{cases}$$

$$l_{b,min} = \mathbf{273mm}$$

α_1 α_2 α_3 α_5 coefficients

$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \ l_{b,rqd} \geq l_{b,min}$

$l_{bd} = 1*1*1*\mathbf{910}mm = \mathbf{910mm} \geq$ **191.24 mm**

for anchorages in compression

$$l_{b,min} \geq max\begin{cases}0.6l_{b,rqd}\\ 10\emptyset\\ 100mm\end{cases}$$

$$l_{b,min} \geq max\begin{cases}0.6 * 910 = 546\\ 10\emptyset = 200\\ 100mm\end{cases}$$

$$l_{b,min} = \mathbf{546mm}$$

$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \ l_{b,rqd} \geq l_{b,min}$

$l_{bd} = 1*1*1*\ 910mm = \mathbf{910mm} \geq 273$**mm**

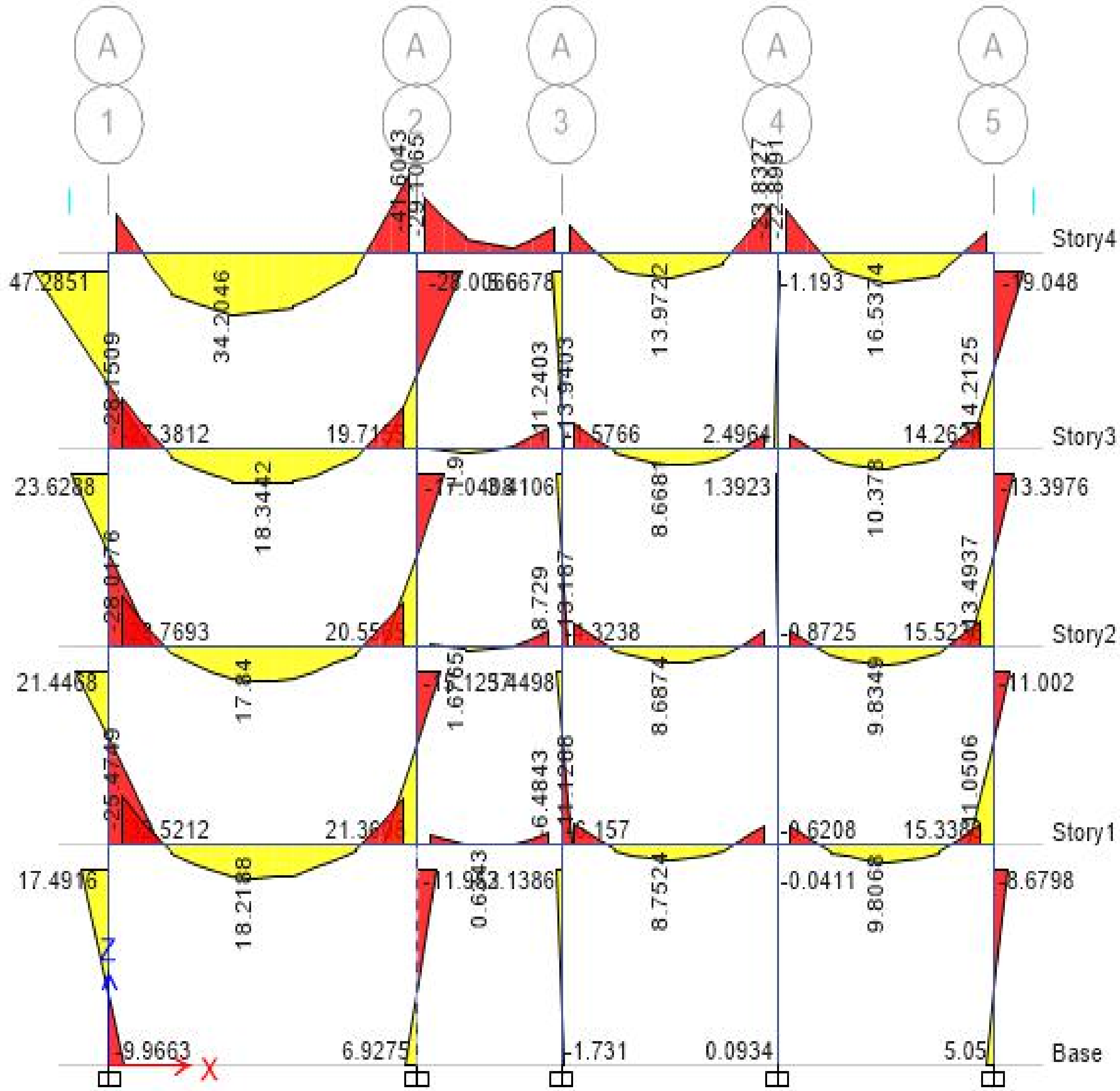


Figure 7- 1.moment 3-3 diagram

Table 15.beam reinforcement

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	47.28	300	300	0.0876	274.6832	single	128.7	3600	494.861	494.861	1.58	2Ø20
span	1-2	34.2	300	300	0.0633	282.1732	single	128.7	3600	348.4562	348.4562	1.11	2Ø20
support	2	41.6	300	300	0.0770	277.9896	single	128.7	3600	430.2319	430.2319	1.37	2Ø20
span	2-3	6.67	300	300	0.0124	296.6934	single	128.7	3600	64.63322	128.7	0.41	2Ø20
support	3	8.56	300	300	0.0159	295.7428	single	128.7	3600	83.2142	128.7	0.41	2Ø20
span	3-4	13.97	300	300	0.0259	292.9869	single	128.7	3600	137.0838	137.0838	0.44	2Ø20
support	4	23.83	300	300	0.0441	287.8224	single	128.7	3600	238.0331	238.0331	0.76	2Ø20
span	4-5	13.97	300	300	0.0259	292.9869	single	128.7	3600	137.0838	137.0838	0.44	2Ø20
support	5	23.83	300	300	0.0441	287.8224	single	128.7	3600	238.0331	238.0331	0.76	2Ø20
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark

The others is in appendix F

7.4. Beam shear Design

For the verification of the shear resistance the following symbols are defined:

- $V_{Rd,c}$ is the design shear resistance of the member without shear reinforcement.
- $V_{Rd,s}$ is the design value of the shear force which can be sustained by the yielding shear reinforcement.
- $V_{Rd,max}$ is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.

$V_{RD} = V_{RD,S}$

In regions of members where $V_{ED} \leq V_{RD,C}$,no calculated shear reinforcement is necessary is design shear force in the section considered resulting from the external loading.

In regions where, $V_{Ed} > V_{Rd,c}$ sufficient shear reinforcement should be provided in order that $V_{Ed} \leq V_{Rd}$

The design shear force should not exceed the permitted maximum value $V_{RD,MAX}$, anywhere in the member.

For members subject to predominantly uniformly distributed loading, the design shear force need not be checked at a distance less that d from the face of the support. Any shear reinforcement required should continue to the support. In addition it should be verified that the shear at the support does not exceed $V_{RD,}$max

Members not requiring design shear reinforcement

The design value for the shear resistance $V_{Rd\ c}$ is given by:

$$v_{Rd,c} = \left[0.12K(100\rho_1 f_{CK})^{\frac{1}{3}} + K_1\sigma_{cp}\right]\text{bwd}$$

With minimum of

$$v_{Rd,c} = \left[V_{min} + K_1\sigma_{cp}\right]\text{bwd}$$

Where:

f_{CK} is in MPa

$K = 1 + \left(\frac{200}{d}\right)^{0.5} \leq 2 \ with\ d\ in\ mm$

$\rho_1 = \frac{A_{sl}}{bwd} \leq 0.02$

A_{sl} is the area of the tensile reinforcement, which extends $\geq$ (l_{bd} + d) beyond the section considered.

bw is the smallest width of the cross-section in the tensile area (mm)

σ_{cp} $= \frac{N_{ED}}{A_c} < 0.2 f_{cd}\ [MPa]$

N_{ED} is the axial force in the cross-section due to loading or prestressing in newtons ($N_{ED} > 0$ for compression). The influence of imposed deformations on $N_E$may be ignored.

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

$v_{Rd,c}$ is in newtons

The values of $C_{Rd\ c}$ V_{min} and k1 for use in a country may be found in its National Annex. The recommended value for $C_{Rd\ c}$ is $0.18/\gamma_c$, that for V_{min} is 0.035 $k^{3/2}.f_{ck}^{1/2}$ and that for $k_1$is 0.15.The design of members with shear reinforcement is based on a truss model. The angle θ should be limited. The limiting values of cotΘ for use in a country may be found in its National Annex.

The recommended limits are $1 \leq \cot\Theta \leq 2.5$.

Members requiring design shear reinforcement

For members with vertical shear reinforcement, the shear resistance, V_{Rd} is the smaller value of

$V_{RD,S} = \frac{A_{SW}}{S} Z f_{ywd} cot\theta$, and

$V_{RD,max} = \alpha_c b_W Z V f_{cd}/(cot\theta + \tan\theta)$

Where

A_{SW} is the cross-sectional area of the shear reinforcement

S is the spacing of the stirrups

f_{ywd} is the design yield strength of the shear reinforcement

V follows from the expression below

For reinforced and prestressed members, if the design stress of the shear reinforcement is below 80% of the characteristic yield stress f_{yk}, V may be taken as:

V=0.6

$V=0.9-f_{ck}/200\ 0.5$ for $f_{ck} \geq 60MPa$

The value of $\tan\alpha_c$ for use in a Country may be found in its National Annex. The recommended value is 1 for non-prestressed structures.

Minimum area and maximum spacing of shear reinforcement

The ratio of shear reinforcement is given by

$\boldsymbol{\rho_W} = A_{sw}/(\boldsymbol{s.b_w.sin\alpha})$

Where

ρ_W is the shear reinforcement ratio ρ_W should not be less than ρ_{Wmin}

A_{sw} is the area of shear reinforcement within length s

S is the spacing of the shear reinforcement measured along the longitudinal axis of the member

b_w is the breadth of the web of the member

$\boldsymbol{\alpha}$ is the angle between shear reinforcement and the longitudinal axis

When, on the basis of the design shear calculation, no shear reinforcement is required, minimum shear reinforcement should nevertheless be provided. The minimum shear reinforcement may be omitted in members such as slabs (solid, ribbed or hollow core slabs) where transverse redistribution of loads is possible. Minimum reinforcement may also be omitted in members of minor importance which do not contribute significantly to the overall resistance and stability of the structure.

The value of $\rho_{w,min}$ for beams for use in a Country may be found in its National Annex. The recommended value is $\rho_{w,min} = \left(0.08\sqrt{f_{ck}}\right)/f_{yk}$

The maximum longitudinal spacing between shear assemblies should not exceed $S_{l,max}$

The value of $S_{l,max}$ for use in a country may be found in its National Annex. The recommended value is $S_{max} = 0.75d(1 + cot\alpha)$

where α is the inclination of the shear reinforcement to the longitudinal axis of the beam.

7.4.1. Procedure for design

- Minimum area and maximum spacing of shear reinforcement

Step 1: Determine maximum applied shear force at the face of support, ved

Step 2: Determine vrd,max with cot θ=2.5

Step 3: If V_{RD},max > ved cotθ=2.5, go to step 6 and calculate required shear reinforcement

Step 4 If v_{rd},max < v_{ed} calculate required strut angle:

$$\Theta = 0.5\sin^{-1}\left[\left(\frac{2V_{ED}}{\alpha cbwZ\upsilon fcd}\right)\right]$$

Step 5: If Cotθ is less than 1, re-size element, otherwise go to step 6

Step 6: Calculate V Rd,c

Step 7: If $v_{rd,c}$ > v_{ed}, Minimum shear reinforcement is sufficient

Step 8: If vrd,c < ved , Calculate amount of shear reinforcement required

Step 9: Check min shear reinforcement and maximum spacing

Sample calculation for shear design

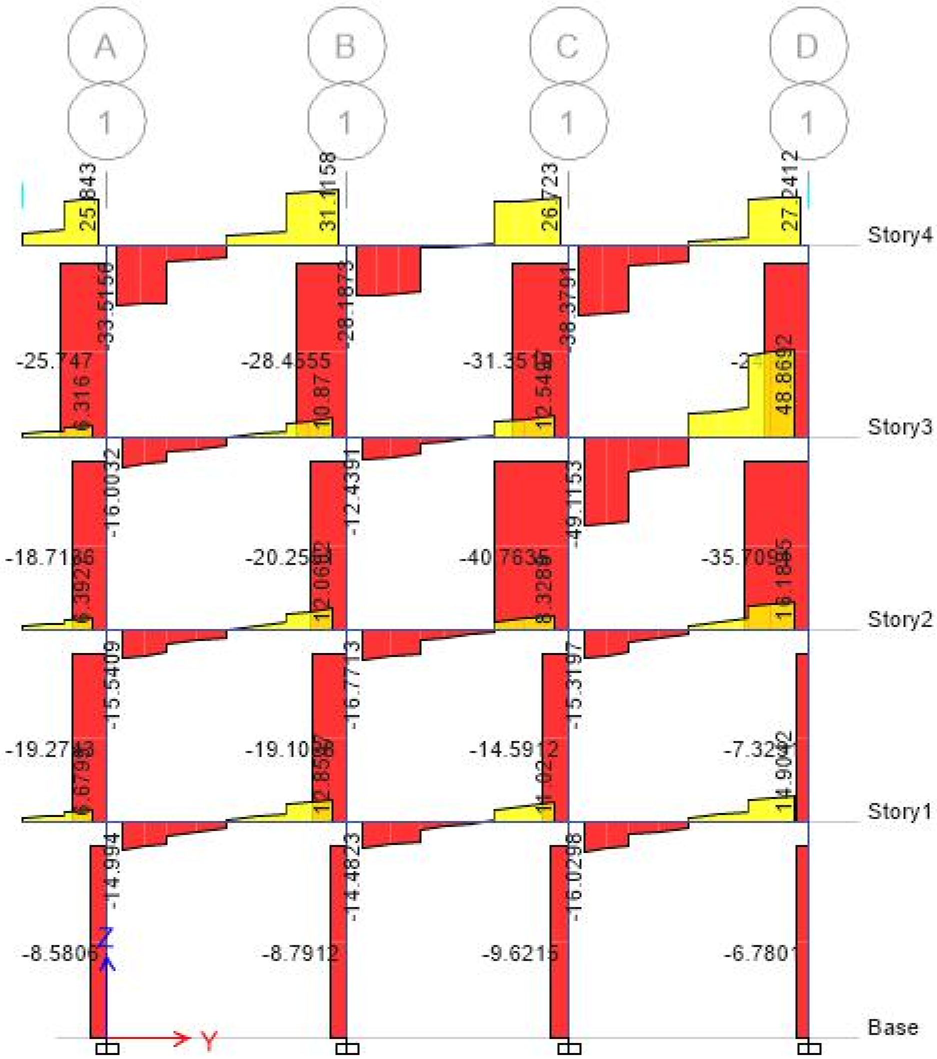
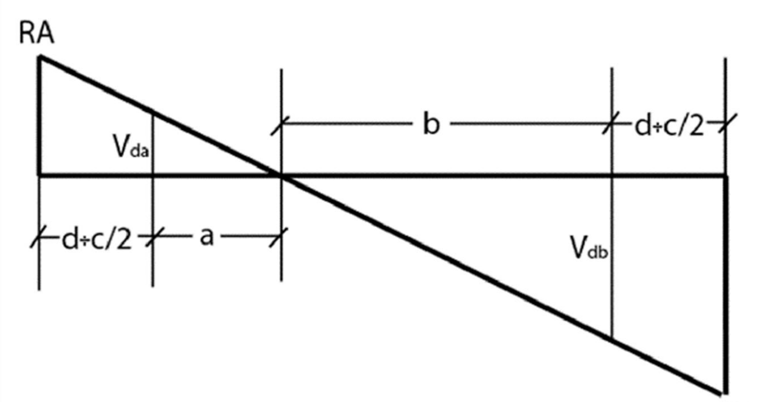


Figure 7- 2.shear 2-2 diagram

RA=	33.51	KN
RB	31.11	KN
L=	5.85	m
d=	300	mm
C=	300	mm
D+c/2	0.45	m
a=	2.583635	m
b=	2.366365	m
VEda=	28.53923	KN
VEdb=	26.13923	KN



Concrete capacity check

$$v_{Rd,c} = \left[0.12K(100\rho_1 f_{CK})^{\frac{1}{3}}\right] \text{bd} \geq V_{min}$$

$$K = \mathbf{1} + \left(\frac{200}{d}\right)^{0.5} \leq 2$$

d= 300

b= 300

Asl= 314

k= 1.82

$$\rho_1 = \frac{A_{sl}}{bd} \leq \mathbf{0.02}$$

taking minimum reinforcment ø 8

@ 260

$\boldsymbol{\rho}_1 =$ 0.0035

$\boldsymbol{v}_{Rd,c} =$ 37.488

$\boldsymbol{v}_{Rd,c} =$ VRdc <= VEd, shear reinforcment design required

longtudinal bar diam.= 16

minimum spacing = 240

diagonal compression check

$$V_{Rd,max} = \propto_{cw} b_w Z V_1 f_{cd} \left(\frac{\mathbf{1}}{cot\theta + tan\theta}\right)$$

$$v_{Rd,c} = \left[0.12K(100\rho_1 f_{CK})^{\frac{1}{3}}\right] \text{bd} \geq V_{min}$$

$$V_1 = \mathbf{0.6}\left(\mathbf{1} - \frac{f_{ck}}{250}\right)$$

V1= 0.552

fcd= 11.33

$\boldsymbol{z} = \mathbf{0.9}\boldsymbol{d}$

Z= 270

$\boldsymbol{cot\theta} =$ 2.5

$\mathbf{tan}\boldsymbol{\theta} =$ 0.4

$\boldsymbol{V}_{Rd,max} =$ 174.6852

VRd max>VEd, so this section does not requre shear reinforcment for diagonal compr

Calculate the required shear reinforcement

The ratio of shear reinforcement is given by;

$$\rho w = \frac{A_{sw}}{(s * b_w - sin\alpha)}$$

Where

ρw is the shear reinforcement ratio

ρw should not be less than ρw_{min}

A_{sw} is the area of shear reinforcement within length s

s is the spacing of the shear reinforcement measured along the longitudinal axis of the member

b_w is the breadth of the web of the member

α is the angle between shear reinforcement and the longitudinal axis.

$$\rho w_{min} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} = \frac{0.08\sqrt{20}}{400} = 0.00089$$

- The maximum longitudinal spacing of bent-up bars should not exceed $S_{b,max}$

$S_{b,max} = 0.75d(1 + cot\alpha) = 0.75 * 300(1 + cot90)$ =225mm

$$S_{min} = \frac{A_{sw}}{\rho_w b_w sin\alpha} = \frac{50.24}{0.00089x300x1} = 188.16$$

Required shear reinforcement

$$\frac{A_{sw}}{s} = \frac{V_{ED}}{0.9df_{yk}cot\theta}$$

$S_{calc} = \frac{A_{sw}0.9df_{yk}cot\theta}{V_{ED}} = \frac{50.26*0.9*300*400}{31.11} =$ 174.48mm

$$use\ s = 175mm$$

Then the remain beam spacing in tabular form below and appendix

Table 16.shear reinforcement

Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	33.51	31.11	225	174.5	188.2	174.48	200
B-C	28.18	26.72	225	203.1	188.2	203.15	200
C-D	38.37	27.24	225	199.3	188.2	199.27	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	16	10.87	225	499.4	188.2	225.00	200
B-C	12.54	12.43	225	436.7	188.2	225.00	200
C-D	49.11	48.86	225	111.1	188.2	111.09	200

The remaining stories are in appendix F

7.5. Beam reinforcement detailing

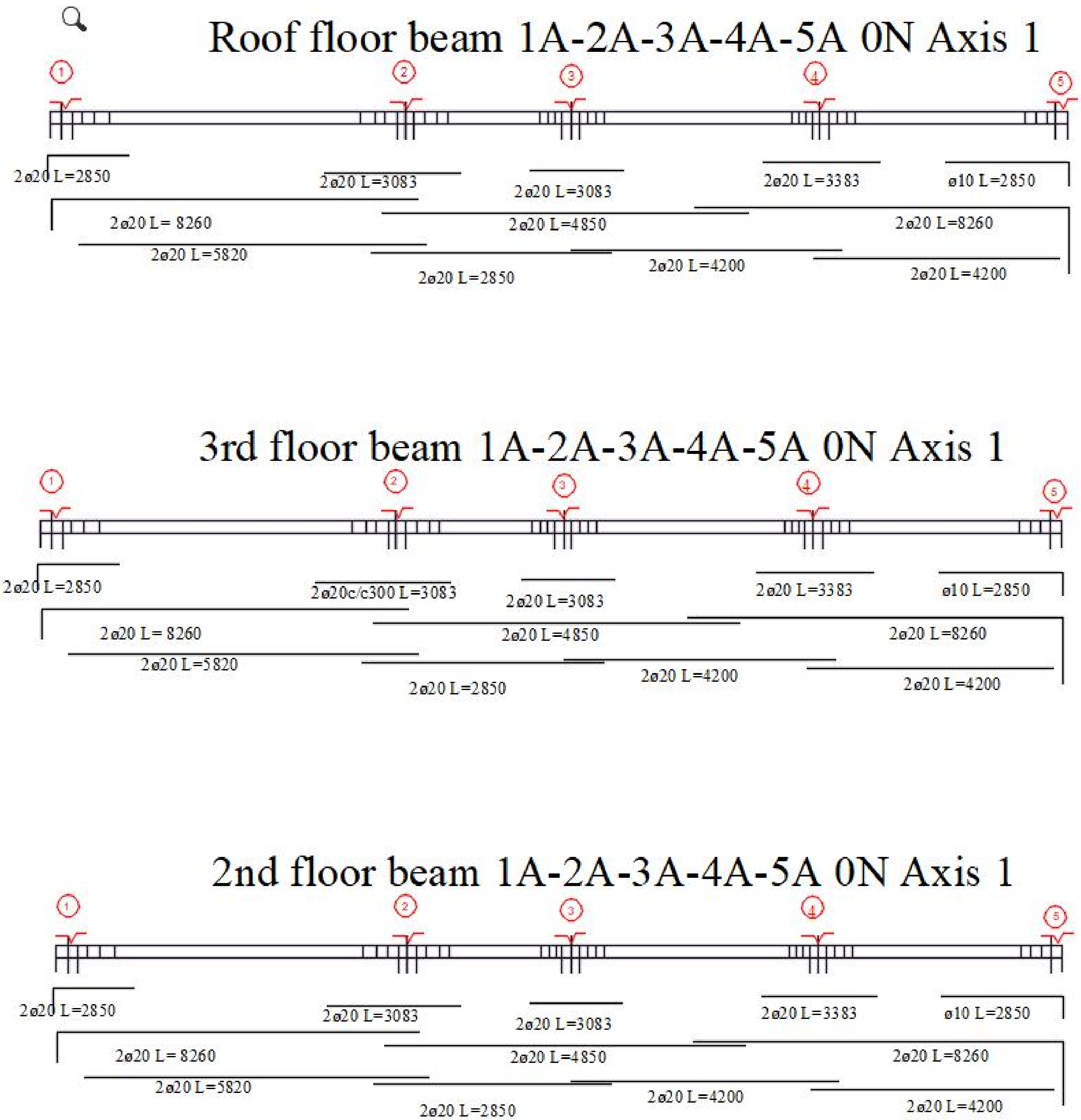
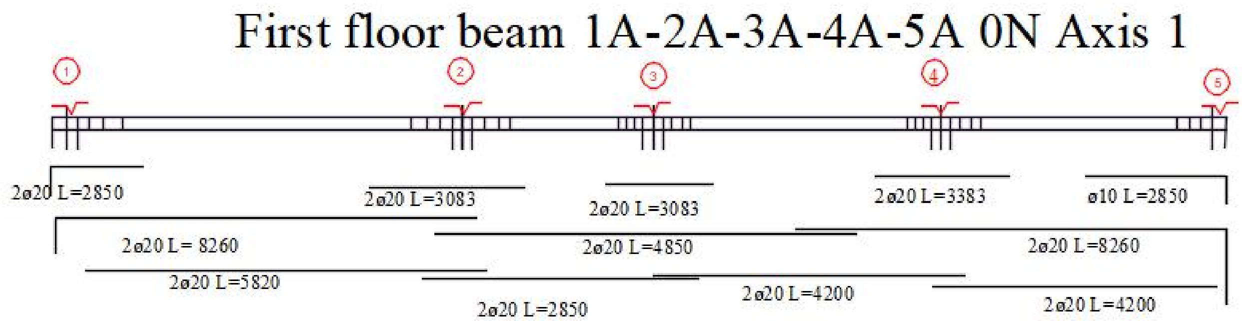


Figure 7- 3.longitudinal reinforcement detailing



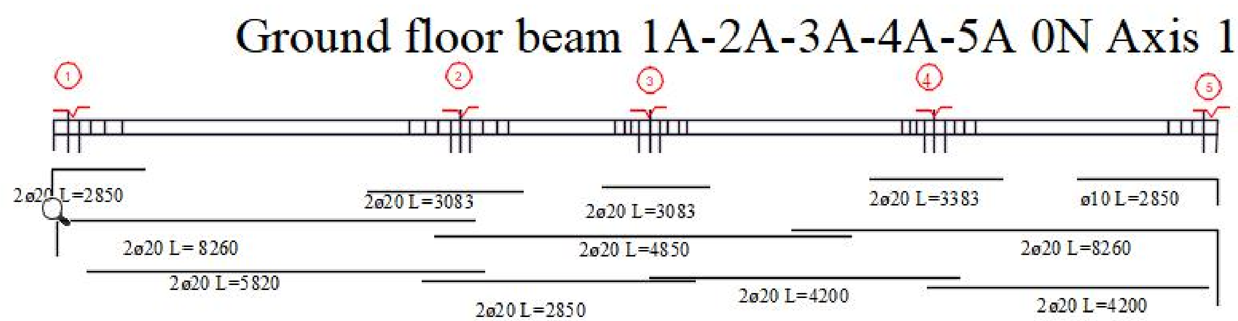


Figure 7- 4.longtiuidinal reinforcement detailing

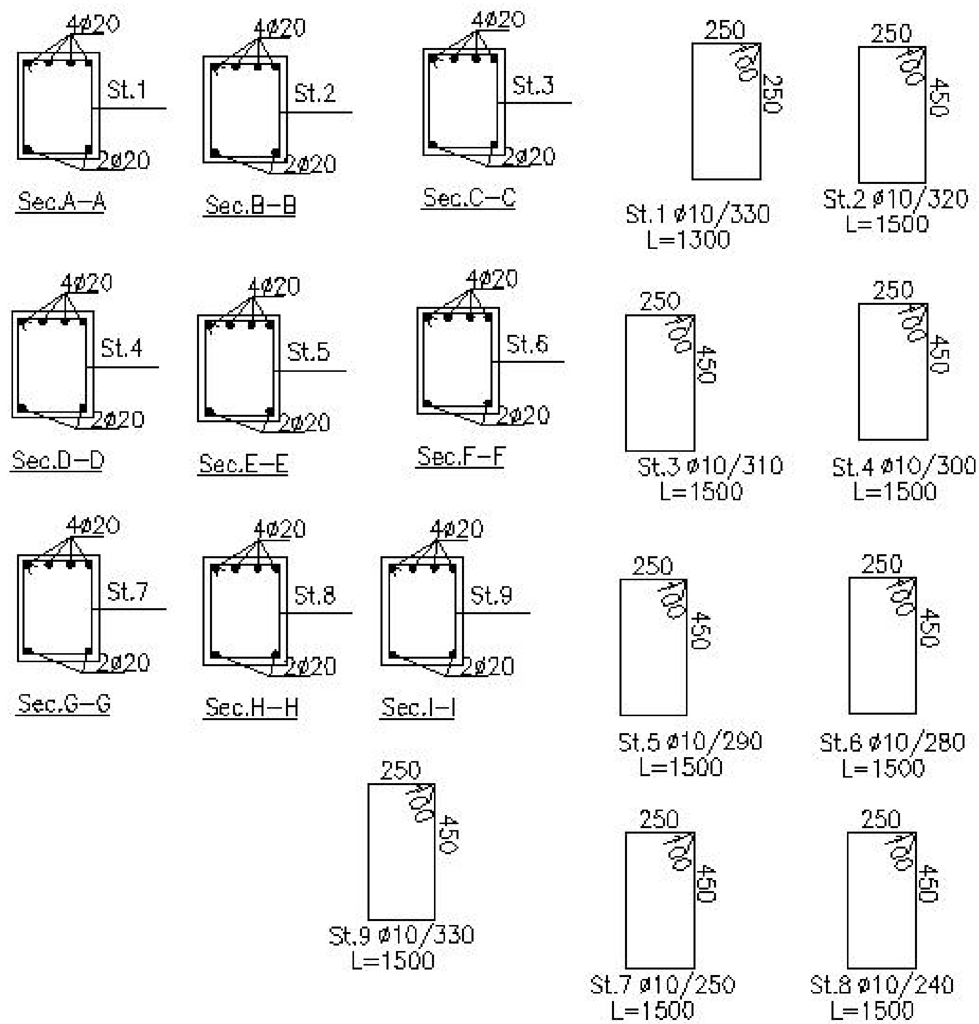


Figure 7- 5.stirrup reinforcement detailing

8. Column design

8.1. General

Columns are defined as members that carry loads chiefly in compression, even though the bending action may produce tensile forces over part of their cross section. Columns are designed to transfer the load from beams and slabs (in flat slab) down to the foundation without buckling or crashing. To determine the nature of the frame, we substitute the beams and columns by one substitute frame.

The value of the axial force on each substitute frame column obtained by adding the axial load of each column for the story including self-weight. A column is special case of a compression member that is vertical. Almost all compression members in RC structure subjected to moment in addition to axial loads these may be due to misalignment of load on column, may result from column resisting portion of the unbalanced moment at the ends of the beams supported by the columns/ unbalanced moments from beams.

The bending moments can be converted in to un-equivalent axial load applied at eccentricity. Therefore, columns should be designed for the design moment obtained from the total eccentricity.

Column may be classified based the following criteria

- ✓ On the basis of lateral reinforcement; tied columns, spiral columns
- ✓ On the basis of manner by which lateral stability is provided to the structure as a whole; braced and unbraced columns
- ✓ On the basis of sensitivity to second order effect due to lateral displacements; sway and non-sway columns
- ✓ On the basis of degree of slenderness; short and slender columns
- ✓ On the basis of loading; axially loaded column, column under uniaxial bending, column under bi axial bending.

Axial column; column subjected to axial loads accompanied by bending about one axis whereas biaxial, for column support axial force and bending about two perpendicular axes. A braced structure is one which contain bracing elements.

These are vertical elements usually walls, which are so stiff relative to other vertical elements that may be assumed to be attract all horizontal forces. Braced structure may be defined as one where the bracing elements attract and transmits to the foundations, at least 90% of all horizontal forces applied to the structure.

Braced structure is one where side sway of the whole structure is unlikely to be significant while in unbraced structure sideways likely to be significant, and defined as lateral displacement of the ends of the column increasing the critical bending moment by more than 10% above calculated by ignoring the displacement.

8.2. Design procedure

8.2.1. Concrete cover design

Cover is designed according to (*ES EN 1992-1-1*, 2015)

a. Exposure class = XC1, for Dry or permanently wet Environment like Concrete inside buildings with low air humidity and Concrete permanently submerged in water. [Appendix B, Table B-2](*ES EN 1992-1-1*, 2015)

b. Minimum strength class = C20/25, for exposure class XC1[Appendix B, Table B-3]

c. Minimum concrete cover for bond/durability

d. $Cmin = max\begin{Bmatrix} Cmin,B \\ Cmin,Dur \\ 10mm \end{Bmatrix} = max\begin{Bmatrix} 20mm \\ 10mm \\ 10mm \end{Bmatrix} = 20mm$

   i. $C_{min,\,b}$= 20 mm, assumed diameter of bar [Appendix B, Table B-1]

   ii. Cmin, dur = 10 mm,

      1. Cmin=20mm

   iii. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1

e. Nominal cover

a. $C_{nom} = C_{min} + \delta c_{dev} = 20mm + 10mm = 30mm$

i. Δc_{dev} = 10mm, the value of Δcdev for use in a Country may be found in its National Annex. The recommended value is 10 mm.

f. Minimum cover Design for Fire = 20mm

g. Governing concrete cover = 30mm

8.2.2. Effective length determination

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

Effective length (lo) for braced member

From (*ES EN 1992-1-1*, 2015) Art, 5.8.

$$l_o = 0.5l\sqrt{\left(1+\frac{k_1}{0.45+k_1}\right)\cdot\left(1+\frac{k_2}{0.45+k_2}\right)}$$

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

l-length of column

K_1-stiffness at end 1

K_2-stiffness at end 2

Stiffness at each end (K) =column stiffness/∑beam stiffness

$$K = \frac{\text{Column stiffness}}{\sum \text{Beam stiffness}}$$

$$K_i = \frac{\left(\frac{EI}{l}\right)c}{\left(\frac{2*EI}{l}\right)b}$$

8.2.3. Check slenderness limit

Slenderness ratio $\lambda = \frac{l_o}{i}$ Where lo-effective length of column

i Is radius of gyration of the uncracked concrete In both directions

$$i = \sqrt{\frac{I_c}{A}}$$

The minimum limiting value of slenderness is

$$\lambda\text{lim} = \frac{20 * \text{ABC}}{\sqrt{n}}$$

where:

A = 1 / (1+0.2 φ_{ef}) (if φ_{ef} is not known, A = 0.7 may be used)

B = $\sqrt{1+2\omega}$ (if ω is not known, B = 1.1 may be used)

C = 1.7 - r_m (if r_m is not known C = 0.7 may be used)

φ_{ef} effective creep ratio; see 5.8.4:

ω = $A_s f_{yd} / (A_c f_{cd})$; mechanical reinforcement ratio;

A_s is the total area of longitudinal reinforcement

n = $N_{Ed} / (A_c f_{cd})$; relative normal force

r_m =M_{01}/M_{02}; moment ratio

M_{01}, M_{02} are the first order end moments, $|M_{02}| \geq |M_{01}|$

If the end moments M_{01} and M_{02} give tension on the same side, r_m should be taken positive (i.e. $C \leq 1.7$), otherwise negative (i.e. $C > 1.7$).

In the following cases, r_m should be taken as 1.0 (i.e. $C = 0.7$):

- for braced members in which the first order moments arise only from or predominantly due to imperfections or transverse loading
- for unbraced members in general (*ES EN 1992-1-1*, 2015) Art 5.8.3.1

$$M01 = min\{|Mtop|,|Mbottom|\} + e_i + NED$$

$$M02 = max\{|Mtop|,|Mbottom|\} + e_i + NED$$

Accidental eccentricity

$$e_i = max\begin{Bmatrix} \frac{l_o}{400} \\ \frac{h}{30} \\ 20mm \end{Bmatrix}$$ (*ES EN 1992-1-1*, 2015) Art 6.1.4

In the both direction

8.2.4. Calculate reinforcement

$$\boldsymbol{A_{s,tot}} = \frac{\omega * A_c * f_{cd}}{f_{yd}}$$

$\boldsymbol{\omega}$, is taken design chart by using, V_{sd}, $\mu_{sd,x}$, $\mu_{sd,y}$ from (*ES EN 1992-1-1*, 2015)

$$V_{sd} = \frac{N_{sd}}{A_c f_{cd}}$$

$$\mu_{sd,x} = \frac{M_{EDx}}{f_{cd} * A_c * h}$$

$$\mu_{sd,y} = \frac{M_{EDy}}{f_{cd} * A_c * h}$$

If the column is slender column

$$M_{ED} = max\{M_{02};M_{Oe} + M_2;M_{O1} + 0.5M_2;e_i N_{ED}\}$$

If the column is not slender column (short column)

$$M_{ED} = max\{M_{02};e_i N_{ED}\}$$

$M_{Oe} = 0.6M_{02} + \ 0.4M_{O1} \geq 0.4M_{02}$

Minimum and maximum area of reinforcement

$$A_{s,min} = max\begin{Bmatrix} \left(0.1 * \dfrac{NED}{f_{yd}}\right) \\ 0.002 * Ac \end{Bmatrix}$$

$$A_{s,max} = 0.04Ac$$

For shear reinforcement

$$\text{Spacing} = min \begin{Bmatrix} 20 * \text{ølong} \\ lesser\ dimension\ of\ column \\ 400mm \end{Bmatrix}$$

Sample calculation

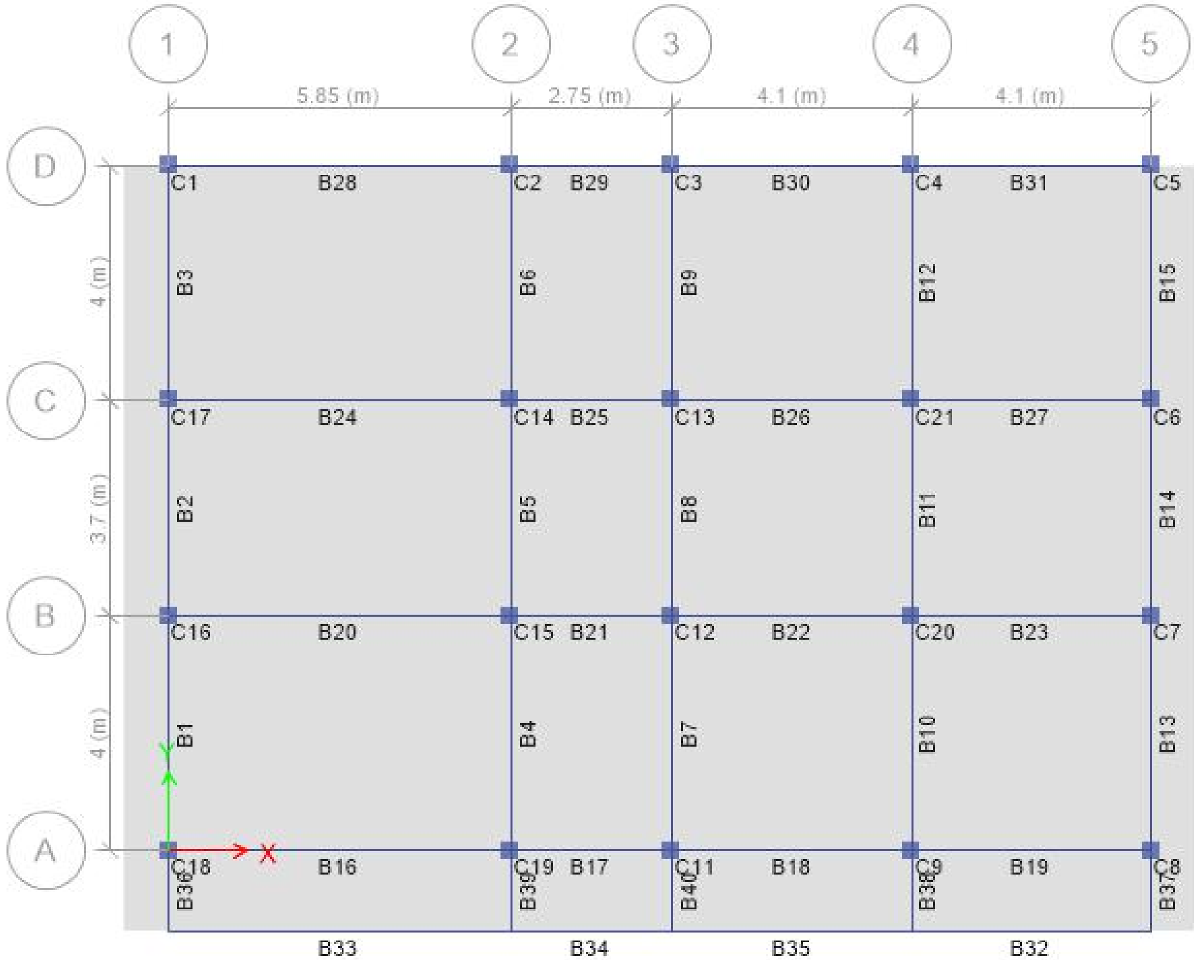


Figure 8- 1.naming of column and beams

Effective length determination

Let calculate for C-21 on roof elevation

Given data

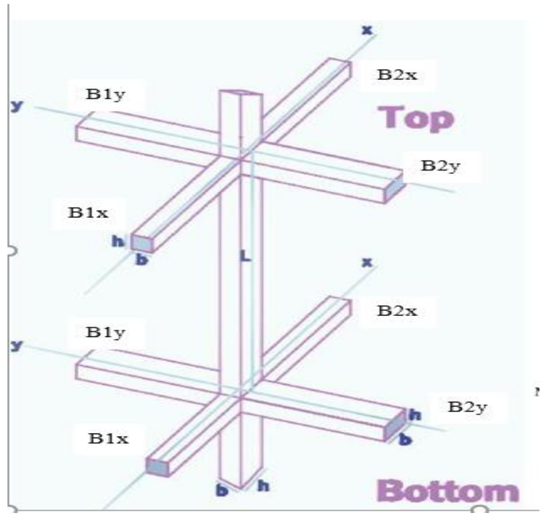


Figure 8- 2. center column cross-section

X-SECTIONAL DATA			
Element	Depth 'h' (Y-dxn) (n	width 'b' (X-dxn)(mm)	length 'L' (mm)
column	500	500	3000
Top beam			
B1 (x-x) (continous)	300	300	4100
B2 (x-x) (continous)	300	300	4100
B1 (y-y) (continous)	300	300	4000
B2 (y-y) (continous)	300	300	3700
Bott beam			
B1 (x-x) (continous)	400	400	4100
B2 (x-x) (continous)	400	400	4100
B1 (y-y) (continous)	400	400	4000
B2 (y-y) (continous)	400	400	3700

$$l_o = 0.5l\sqrt{\left(1+\frac{k_1}{0.45+k_1}\right)\cdot\left(1+\frac{k_2}{0.45+k_2}\right)}$$

$$K_{1x} = \frac{\left(\frac{\frac{500*500^3}{12}}{3000}\right)}{\left(\frac{\frac{300*300^3}{12}}{4100}\right) + \left(\frac{\frac{400*400^3}{12}}{4100}\right)} = \frac{1736111.11}{164634.14 + 520325.2} = \mathbf{2.53}$$

Same for $K_{2x} = \mathbf{2.53}$

$$K_{1y} = \frac{\left(\frac{\frac{500*500^3}{12}}{3000}\right)}{\left(\frac{\frac{300*300^3}{12}}{4000}\right) + \left(\frac{\frac{400*400^3}{12}}{4000}\right)} = \frac{1736111.11}{168750 + 533333.33} = \mathbf{2.47}$$

$$K_{2y} = \frac{\left(\frac{\frac{500*500^3}{12}}{3000}\right)}{\left(\frac{\frac{300*300^3}{12}}{3700}\right) + \left(\frac{\frac{400*400^3}{12}}{3700}\right)} = \frac{1736111.11}{182432.43 + 576576.58} = \mathbf{2.28}$$

$l_{ox} = 0.5 * 3000\sqrt{\left(1 + \frac{2.53}{0.45+2.53}\right) \cdot \left(1 + \frac{2.53}{0.45+2.53}\right)}$=**2884.52m**

$l_{oy} = 0.5 * 3000\sqrt{\left(1 + \frac{2.47}{0.45+2.47}\right) \cdot \left(1 + \frac{2.28}{0.45+2.28}\right)}$=**2760.65m**

Check slenderness limit

Slenderness ratio $\lambda = \frac{l_o}{i}$

$$i = \sqrt{\frac{I_c}{A}} = \sqrt{\frac{\frac{400*400^3}{12}}{400*400}} = 115.47$$

$$\lambda_x = \frac{l_{ox}}{i} = \frac{2884.52}{115.47} = 24.98$$

$$\lambda_y = \frac{l_{oy}}{i} = \frac{2760.65}{115.45} = 23.9$$

The minimum limiting value of slenderness is

$$\lambda\text{lim} = \frac{20 * \text{ABC}}{\sqrt{n}}$$

φ_{ef} is not known

A=0.7 B=1.1 C= 1.7 – rm

$$rm = \frac{M_{01}}{M_{02}}$$

$$M01 = min\{|Mtop|,|Mbottom|\} + e_i * NED$$

$$M02 = max\{|Mtop|,|Mbottom|\} + e_i * NED$$

Accidental eccentricity

$$e_{ix} = max\begin{Bmatrix} \frac{2884.52}{400} = 7.21 \\ \frac{500}{30} = 16.67 \\ 20mm \end{Bmatrix} = \mathbf{20\ mm}$$

$$e_{iy} = max\begin{Bmatrix} \frac{2760.65}{400} = 6.9 \\ \frac{500}{30} = 16.67 \\ 20mm \end{Bmatrix} = \mathbf{20\ mm}$$

$M01x = min\{|22.08|,|18.93|\} + 0.02 * 380.11$= 26.53KNm

$M02x = max\{|22.08|,|18.93|\} + 0.02 * 380.11$= 29.68KNm

$M01y = min\{|2.15|,|-1.04|\} + 0.02 * 380.11$= 8.64KNm

$M02y = min\{|2.15|,|-1.04|\} + 0.02 * 380.11$= 9.75KNm

$$n = \frac{NED}{A_c f_{cd}} = \frac{380.11}{500 * 500 * 11.33} = 0.00134$$

$$\lambda xlim = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{26.53}{29.68})}{\sqrt{0.134}}$$

$\lambda xlim = \mathbf{33.914}$

$$\lambda ylim = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{8.64}{9.75})}{\sqrt{0.134}}$$

$\lambda ylim = \mathbf{48.912}$

24.98 < 33.914, column is not slender

23.9 < 48.912 column is not slender

$M_{ED} = max\{M_{02}; e_i N_{ED}\}$

$M_{EDx} = max\{29.68; 0.02 * 380.11 = 29.68\}$

$M_{EDy} = max\{9.75; 0.02 * 380.11 = 9.75\}$

Calculate reinforcement

$A_{s,tot} = \frac{\omega * A_c * f_{cd}}{f_{yd}}$

ω , is taken design chart by using, V_{sd}, $\mu_{sd,x}$, $\mu_{sd,y}$ from (*ES EN 1992-1-1*, 2015)

$V_{sd} = \frac{N_{sd}}{A_c f_{cd}} = \frac{380.11 * 1000}{500 * 500 * 11.33}$ =0.134

$\mu_{sd,x} = \frac{M_{EDx}}{f_{cd} * A_c * h} = \frac{29.68 * 1000000}{11.33 * 500 * 500 * 500} = 0.02$

$\mu_{sd,y} = \frac{M_{EDy}}{f_{cd} * A_c * h} = \frac{9.75 * 1000000}{11.33 * 500 * 500 * 500} = 0.01$

ω , is taken from biaxial chart on [appendix E] for this sample ω= 0.15

Minimum and maximum area of reinforcement

$$A_{s,min} = max\begin{Bmatrix}\left(0.1 * \frac{NED}{f_{yd}}\right)\\ 0.002 * Ac\end{Bmatrix}$$

$$A_{s,min} = max\begin{Bmatrix}\left(0.1 * \frac{380.11}{\frac{400}{1.15}} * 1000\right) = 109.28\\ 0.002 * 500 * 500 = 500\end{Bmatrix}$$

$$A_{s,min} = 500mm2$$

$A_{s,max} = 0.04Ac$ = 0.04*500*500= 10000

$$A_{s,tot} = \frac{\omega * A_c * f_{cd}}{f_{yd}} = \frac{0.15*500*500*\frac{20}{1.5}}{\frac{400}{1.15}} = 1087\text{mm2}$$

Let us use ø 20 bar

No of bar $= \frac{A_{st}}{a_{st}} = \frac{1087}{\frac{3.14*20^2}{4}} = 3.46$

Use 4 ø20 bar

For shear reinforcement

Take ø 8mm

$$\text{Spacing} = min\begin{Bmatrix}20 * \text{ølong} = 20 * 20 = 400\\ lesser\ dimension\ of\ column = 500\\ 400mm\end{Bmatrix}$$

Use ø 10c/c 400

8.2.5. Lap Length

The design lap length is:

$l_o = \alpha_1\alpha_2\alpha_3\alpha_4\alpha_5\alpha_6 l_{b,rqd} \geq l_{o,min}$

Where;

lb,rqd is basic anchorage length

$l_{o,min} \geq$ max {$0.3\alpha_6 l_{b,rqd}$; 15Ø;200 mm}

Values of $\alpha_1\alpha_2\alpha_3$ and α_5 may be taken from table and α_6 are given in the table

$$l_{o,min} \geq max \begin{cases} 0.3\alpha_6 l_{b,rqd} \\ 15\emptyset \\ 200mm \end{cases}$$

Where: $l_{b,rqd} = \frac{\emptyset}{4} * \frac{\sigma sd}{fbd}$

$$\sigma sd = \frac{fyk}{\gamma_s}$$

$$fbd = 2.25\eta_1\eta_2 f_{ctd}$$

$$\eta_1 = 1\ for\ good\ quality\ of\ bond$$

$$\eta_1 = 0.7\ for\ other\ cases$$

$$\eta_2 = 1\ for\ \emptyset \leq 32mm$$

$$\eta_2 = \frac{132-\emptyset}{100}\ for\ \emptyset > 32mm$$

$$fctd = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, recommended\ value$$

Sample calculation for 20 reinforcement bar

$$fctd = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$\alpha_{ct} = 0.85, recommended\ value$

$f_{ctk} = 1.mpa, from\ ESEN$ 1992 Table 3.1

$fctd = \frac{0.85*1.5}{1.5} =$ **0.85**

$fbd = 2.25\eta_1\eta_2 f_{ctd}$

$\eta_1 = 1$, our Slab D is < 250mm, good condition of bond

$\eta_2 = 1$,$for\ \emptyset \leq 32mm$

$f_{ctd} = 0.85$

$fbd = 2.25 * 1 * 1 * 0.85 = \mathbf{1.91}$

$\sigma sd = \frac{fyk}{\gamma_s} = \frac{400}{\mathbf{1.15}} = \mathbf{347.83}$

$l_{b,rqd} = \frac{\emptyset}{4} * \frac{\sigma sd}{fbd} = \frac{20}{4} * \frac{347.83}{1.91} = \mathbf{910mm}$

$l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd} \geq l_{0,min}$

α_1, α_2 & $\alpha_3 = 1$, $\alpha_4 = 0.7$, for reinforcement bar in compression

$\alpha_6 = \left(\frac{\rho_1}{25}\right)^{0.5}$ but not exceeding 1.5 nor less than 1.0, where ρ_1 is the percentage of reinforcement lapped within 0.65 l0 from the center of the lap length considered

Let all reinforcement in slab lapped on the same center, therefore $\boldsymbol{\alpha_6 = 1.4}$ from the table value for $\rho_1 > 50\,\%$,

$$l_{o,min} \geq max\begin{cases} 0.3\alpha_6 l_{b,rqd} = 0.3 * 1.4 * \mathbf{910} = \mathbf{382.2}mm \\ 15\emptyset = 15 * 20 = 300mm \\ 200mm \end{cases}$$

$$l_{o,min} = \mathbf{382.2}\boldsymbol{mm}$$

$l_o = \alpha_1\alpha_2\alpha_3\alpha_4\alpha_5\alpha_6 l_{b,rqd}$=1*0.7*0.7*0.7*1.4*910=**437mm ≈450mm**

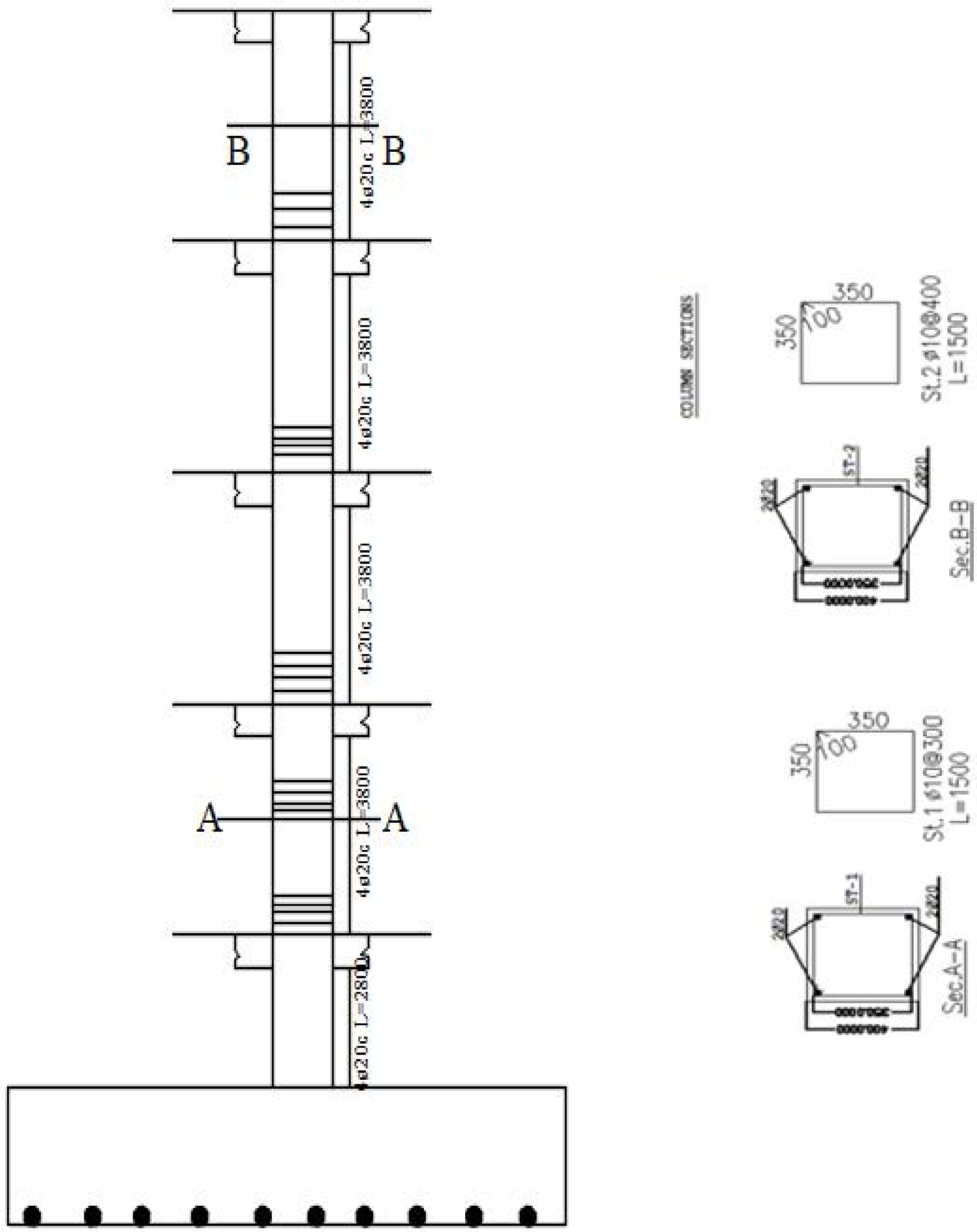


Figure 8- 3.column reinforcement detailing

9. FOUNDATION DESIGN

9.1. General principles of foundation

- ✓ To prepare a plan of the base structure showing the various column load -bearing wall with estimated loads including dead load, live load, moments and torque coming in to the foundation units
- ✓ Study of allowable bearing pressures for various strata below ground level, as given by the soil investigation report.
- ✓ Determining the required foundation depth. This depth is minimum

The main objectives of a foundation are the following

- ✓ Foundation is part of a structure which transfers the load of the structure to soil on which it rests.
- ✓ To distribute the weight of the structure over larger area so as to avoid over loading of the soil beneath.
- ✓ To load the sub structure evenly so as to avoid unequal settlement.
- ✓ To provide a level surface for building operations
- ✓ To take the sub structure deep into the ground and thus increase its stability and avoid Overturning.

9.2. Design procedure

9.2.1. Concrete cover design

a) Cover is designed according to (*ES EN 1992-1-1*, 2015)

Exposure class = XC1, for Dry or permanently wet Environment like Concrete inside buildings with low air humidity and Concrete permanently submerged in water. [Appendix B, Table B-2](*ES EN 1992-1-1*, 2015)

Minimum strength class = C20/25, for exposure class XC1[Appendix B, Table B-3]

c) Minimum concrete cover for bond/durability

$$Cmin = max\begin{Bmatrix} Cmin,B \\ Cmin,Dur \\ 10mm \end{Bmatrix} = max\begin{Bmatrix} 20mm \\ 10mm \\ 10mm \end{Bmatrix} = 20mm$$

i. $C_{min,\,b}$= 20 mm, assumed diameter of bar [Appendix B, Table B-1]

ii. Cmin, dur = 10 mm,

1. Cmin=20mm

iii. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1

d) Nominal cover

a. $C_{nom} = C_{min} + \delta c_{dev}$ = 20mm + 10mm = 30mm

i. Δc_{dev} = 10mm, the value of Δcdev for use in a Country may be found in its National Annex. The recommended value is 10 mm.

e) Minimum cover Design for Fire = 20mm

f) Governing concrete cover = 30mm

9.2.2. Determination of footing size

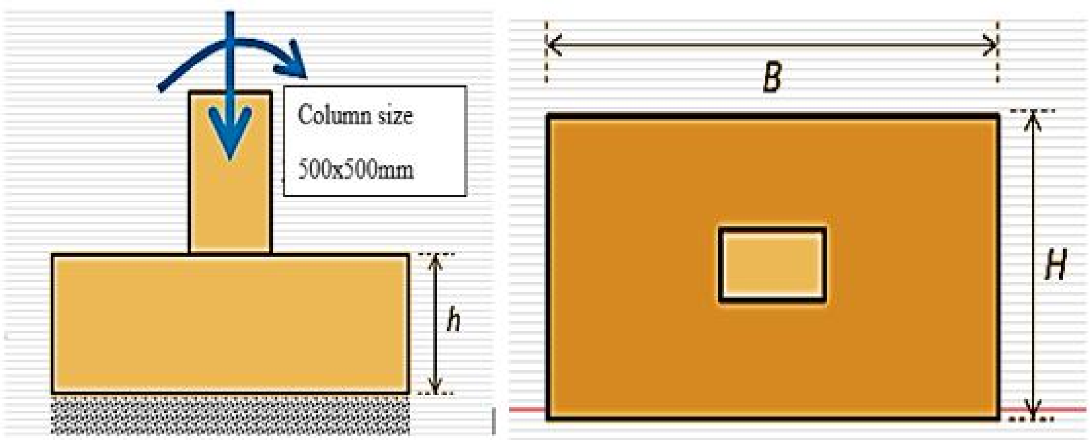


Figure 9- 1. footing size

Service axial N=2550KN

Service moment, Mx=2.8KN.m , My=6.3KN.m

Assumed self-weight 10% of service load, W=255KN

Take γ_{soil}=150N/mm2

ultimate bearing capacity 350KN/m2

$$\text{Area of footing required} = \frac{(N+W)}{\gamma soil} = \frac{(2550+255)}{150} = 18.7m2$$

- ❖ Try square footing size, B*H*h=3*3*0.7

$$Ixx = \frac{bh3}{12} = \frac{3*33}{12} = 6.75m4$$

$$y = \frac{H}{2} = \frac{3}{2} = 1.5m$$

$$\text{maximum soil pressure, } P = \frac{(N+W)}{A} + \frac{My}{I} = \frac{(2550+255)}{9} + \frac{6.3}{6.75} = 313$$

Self-weight, W=A*h*unit weight of concrete=3*3*0.7*25=157.5KN

Design square footing

The foundation column(C-2) is 500mm * 500mm.

9.2.3. Eccentricity

ex = My /p = 6.3/2550 = 0.0025m,

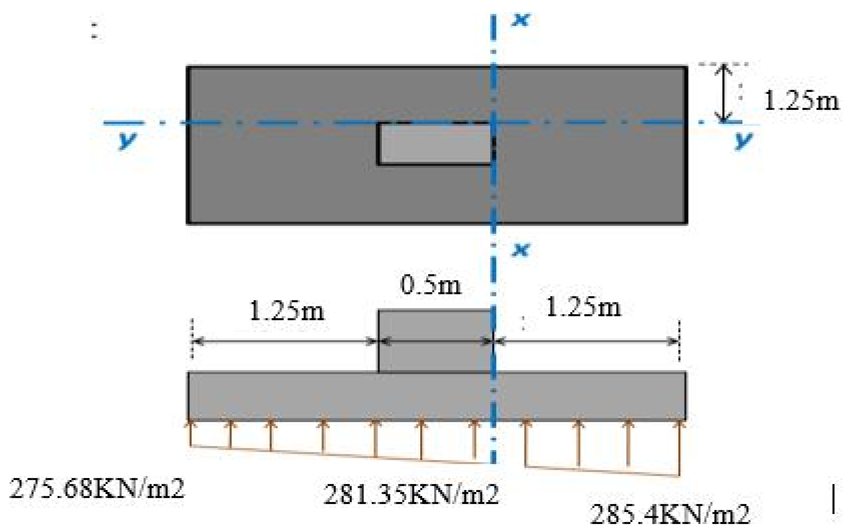
e y = Mx /p = 2.8/2550 = 0.0011m

Contact pressure ρ max

ρ max$=\frac{P}{A}*\left(1\pm\frac{6ex}{B}\pm\frac{6ey}{B}\right)$

ρ max $=\frac{2550}{9}*\left(1+\frac{6(0.0025)}{3}+\frac{6(0.0011)}{3}\right)$ =285.4KN/m^2<350KN/m2 ok

ρ min $=\frac{2550}{9}*\left(1-\frac{6(0.0025)}{3}-\frac{6(0.0011)}{3}\right)$ =275.68KN/m^2



$$\text{Mxx} = (281.35*\frac{1.25*1.25}{2}+(285.4-275.68)\left(\frac{1.25}{2}\right)*\left(1.25*\frac{2}{3}\right)=225KNm$$

$$\text{Myy}=\frac{(275.68+285.4)}{2}*\left(\frac{(1.25*1.25)}{2}\right)=220kNm$$

Effective depth

$$\text{Dx} = \text{h} - \text{c} - 0.5\text{Øbar} = 700 - 30 - 0.5*24 = 658mm$$

$$\text{Dy} = \text{h} - \text{c} - 1.5\text{Øbar} = 700 - 30 - 1.5*24 = 634mm$$

9.2.4. Main reinforcement -longitudinal bar

$$K = \frac{Mxx}{fcdbd2} = \frac{225}{20 * 3 * 700 * 700} = 0.000007 < Kbal = 0.167\ k = 0.167$$

Compression reinforcement is not required

$$Z = \frac{d}{2}\left\lfloor 1 + \sqrt{1 - 3.53K} \right\rfloor \leq 0.95d\ = \frac{d}{2}\left\lfloor 1 + \sqrt{1 - 3.53 * 0.167} \right\rfloor = 0.82d \leq 0.95d$$

$$z = 0.95 * 700 = 665$$

$$As,req = \frac{Mxx}{fydZ} = \frac{255}{347.82 * 665} = 1102.5mm2$$

$$As,min = \frac{0.26 * fctm * bd}{fyk} = \frac{0.26 * 2.2 * 3 * 0.7}{400} = 3003mm2$$

$$\geq 0.0013bd = 0.0013 * 3 * 0.7 = 2730mm2 where\ fck\ \geq 25$$

$$As,max = 0.04Ac = 0.04 * bh = 0.04 * 3000 * 700 = 84000mm2$$

Since As,req $<$ As,min use As,min = 3003mm2

$$as = \frac{\Pi d2}{4} = \frac{3.14 * 24 * 24}{4} = 452.16mm2$$

$$no\ of\ bar = \frac{As,req}{as} = \frac{3003}{452.16} = 6.6$$

$$use\ 7Ø24$$

9.2.5. Main reinforcement- transverse bar

$$K = \frac{Mxx}{fcdbd2} = \frac{225}{20 * 3 * 700 * 700} = 0.000007 < Kbal = 0.167\ k = 0.167$$

Compression reinforcement is not required

$$Z = \frac{d}{2}\left\lfloor 1 + \sqrt{1 - 3.53K} \right\rfloor \leq 0.95d\ = \frac{d}{2}\left\lfloor 1 + \sqrt{1 - 3.53 * 0.167} \right\rfloor = 0.82d \leq 0.95d$$

$$z = 0.95 * 700 = 665$$

$$As,req = \frac{Mxx}{fydZ} = \frac{255}{347.82 * 665} = 1102.5mm2$$

$$As,min = \frac{0.26 * fctm * bd}{fyk} = \frac{0.26 * 2.2 * 3 * 0.7}{400} = 3003mm2$$

$$as = \frac{\Pi d2}{4} = \frac{3.14 * 24 * 24}{4} = 452.16mm2$$

$$no\ of\ bar = \frac{As,req}{as} = \frac{3003}{452.16} = 6.6$$

$$use\ 7\emptyset 24$$

$$\geq 0.0013bd = 0.0013 * 3 * 0.7 = 2730mm2 where\ fck\ \geq 25$$

$$As,max = 0.04Ac = 0.04 * bh = 0.04 * 3000 * 700 = 84000mm2$$

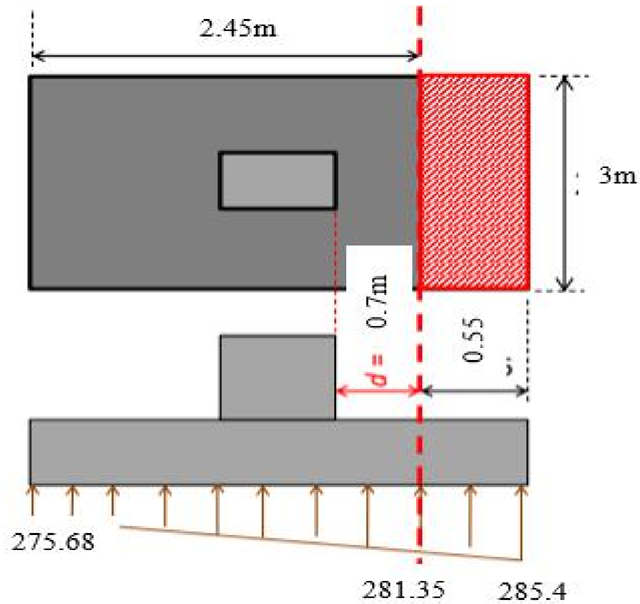
$$\text{Since As,req } < \text{As,min use As,min} = 3003\text{mm2}$$

9.2.6. Vertical shear (wide beam shear) check

Critical shear at 1.0d from face of column

Average pressure at critical section

$$= 275.68 + \left(\frac{2.45}{3.0}\right) * 9.72 = 283.68$$



Design shear force, VEd $= 283.68 * 0.55 * 3 = 468.072KN$

Note bar extend beyond critical section at=550-30=520>(lbd+d)=36Ø+d=36*24+700=1564 then Asl=0

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

ρ l=Asl/bd=0

Vrdc=0

Vmin=$\lfloor 0.035 * K^{3/2}\sqrt{fck} \rfloor bd = \lfloor 0.035 * 1.535^{3/2}\sqrt{20} \rfloor 3000 * 700 = 625KN$

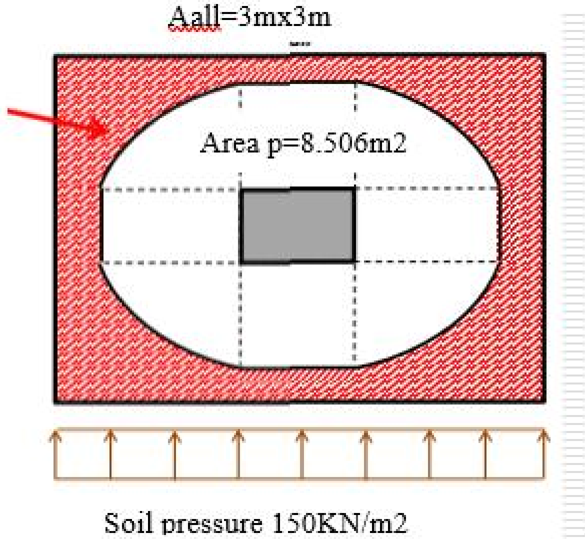
VEd<Vmin---ok!

9.2.7. Punching shear check

Critical shear at 2d from face of column

Average $d = \frac{D+dy}{2} = \frac{700+634}{2} = 667m$

$2d = 2 * 667m = 1334m$



Control perimeter

$\text{U} = 2(500 + 500) + (2\Pi * 1334) = 10377.52$

Area with in perimeter

$\text{A} = (0.5 * 0.5) + (2 * 0.5 * 1.334) + (2 * 0.5 * 1.334) + (3.14 * 1.334 * 1.334) = 8.506\text{m2}$

Average punching shear force at control perimeter

$$VEd = 150 * ((3 * 3) - 8.506) = 74.1KN$$

Punching shear stress

$$vEd = \frac{\beta VEd}{ud}$$

Where

$\beta = 1 + k(\frac{MEd}{VEd})(\frac{u1}{w1})$

$\text{w1} = 0.5{C_1}^2 + C_1 C_2 + 4C2_2\text{d} = 16\text{d2} + 2\,\Pi\text{d}C_1$

$\text{w1} = 0.5 * 500^2 + 500 * 500 + 4 * 500 * 667 + 16 * 667 * 667 + 2\,\Pi * 667 * 500$

$\text{w1} = 10921604\text{mm4}$

$$\beta = 1 + 0.6\left(\frac{6.38 * 1000}{74.1}\right)\left(\frac{10377.52}{10921604}\right) = 1.05$$

$$vEd = \frac{\beta VEd}{ud} = \frac{1.05 * 74.1 * 1000}{10377 * 700} = 0.011N/mm2$$

Punching shear resistance

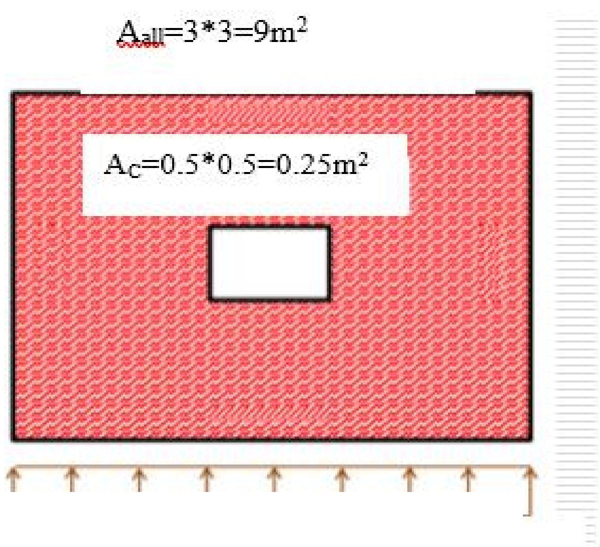
$K = 1 + \sqrt{\frac{200}{d}}$=1 + $\sqrt{\frac{200}{700}}$=1.535<2.....................................ok

$V_{Rd,c} = \text{ Vmin} = \lfloor 0.035 * \text{K}3/2\sqrt{fck} \rfloor$

$V_{Rd,c} = \lfloor 0.035 * 1.535^{3/2}\sqrt{20} \rfloor$=0.298N/mm^2>$V_{Ed}$=0.011N/mm^2

a) Maximum punching shear at column perimeter

$V_{Ed,max}$=2550kN



Column column perimeter (V_0)

V_0=2(500+500)=2000mm

Punching shear stress

$$V_{Ed} = \frac{\beta VEd}{ud}$$

W1=0.5$C_1{}^2$+C_1C_2=0.5*500²+500*500=0.375*10⁶mm²

$$\beta = 1 + 0.6 * \left(\frac{6.38 * 1000}{2550 * 10^3}\right)\left(\frac{2000}{0.375 * 106}\right) = 1$$

$$V_{Ed} = \frac{1 * 2550 * 10^3}{2000 * 667} = 1.91N/mm2$$

Maximum shear resistaqnce

$V_{Ed,max}$=0.5 $\left[0.6\left(1 - \frac{f_{ck}}{500}\right)\right]$*$\frac{f_{ck}}{1.5}$

$V_{Ed,max}$=0.5 $\left[0.6\left(1 - \frac{20}{500}\right)\right] * \frac{20}{1.5} = 3.84\text{N/mm2} > V_{Ed}$..............................ok

9.2.8. Reinforcement Detailing

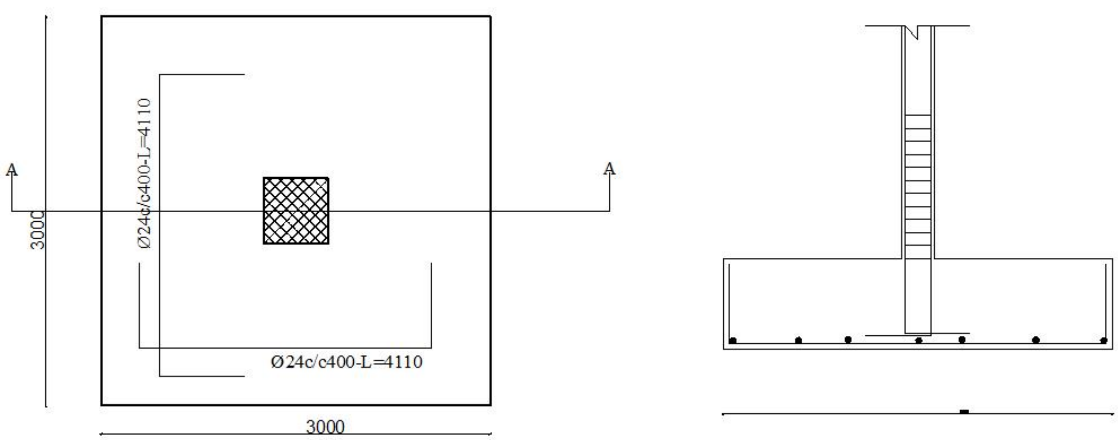


Figure 9- 2. footing detailing

Reference

Chair of Concrete Materials and Structures. (2017). *Design of RC Beams According to ES EN 1992:2015*. AAU-AAiT, School of Civil and Environmental Engineering.

ES EN 1992-1-1:2015 Ethiopian Standard – Based on European Norm, Design of Concrete Structures, Part 1-1: General rules and rules for buildings. (n.d.). Ministry of construction, Federal Democratic Republic of Ethiopia.

ES EN 1992-1-1:2015 Ethiopian Standard – Based on European Norm, Design of Concrete Structures, Part 1-1: General rules and rules for buildings. (2015). Ministry of construction, Federal Democratic Republic of Ethiopia.

10. Appendix

Appendix A1: determination of loading 3rd floor

Table 17. 3rd floor design load

P2 & P3			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				0.653658537
	10.72	16.4	HCB	14	0.15	
LIVE LOAD Q_k (category A)				4	KN/m^2	
DEAD LOAD Gk				7.15	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				15.65	KN/m^2	

P4			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				2.472979215
	53.54	21.65	HCB	14	0.15	
LIVE LOAD Q_k (category A)				4	KN/m^2	
DEAD LOAD Gk				8.97	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				18.11	KN/m^2	

P5			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			ceramic	23	0.005	0.115
LIVE LOAD Q_k (category A)				4	KN/m^2	
DEAD LOAD Gk				6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				14.77	KN/m^2	

P6 & P7			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			procelain	23	0.005	0.115
	load of pa	Area				3.234615385
partition wall	25.23	7.8	HCB	14	0.15	
LIVE LOAD Q_k (category A)				4	KN/m^2	
DEAD LOAD Gk				9.73	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				19.13	KN/m^2	

P8			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			procelain	23	0.005	0.115
	load of pa	Area				1.509401709
partition wall	35.32	23.4	HCB	14	0.15	
LIVE LOAD Q_k (category A)				4	KN/m^2	
DEAD LOAD Gk				8.00	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				16.81	KN/m^2	

P9	Material	Unit weig	Dimensio	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	ceramic	23	0.005	0.115
LIVE LOAD Q_k (category A)		4	KN/m^2	
DEAD LOAD Gk		6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		14.77	KN/m^2	

P10 & P11	Material	Unit weig	Dimensio	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	procelain	23	0.005	0.115
LIVE LOAD Q_k (category A)		4	KN/m^2	
DEAD LOAD Gk		6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		14.77	KN/m^2	

Appendix A2: moment analysis of 3rd floor

Table 18. moment analysis of 3rd floor

panel	support conditio	ly/lx	lx	lx^2	Pd	moment coeficient		moment	moment
						location	value	location	$Mi=\alpha_i P_d$
P-1	Type 2	1.46	4	16.00	17.76	$\beta_{sx,sup}$=	0.057	M_{xs} =	16.20
						$\beta_{sx,span}$=	0.042	M_{xf} =	11.93
						$\beta_{sy,sup}$=	0.039	M_{ys} =	11.08
						$\beta_{sy,span}$=	0.029	M_{yf} =	8.24
P-2	Type 3	1.03	4.00	16.00	15.65	$\beta_{sx,sup}$=	0.042	M_{xs} =	10.52
						$\beta_{sx,span}$=	0.032	M_{xf} =	8.01
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.77
						$\beta_{sy,span}$=	0.03	M_{yf} =	7.51
P-3	Type 4	1.03	4	16	15.65	$\beta_{sx,sup}$=	0.049	M_{xs} =	12.27
						$\beta_{sx,span}$=	0.038	M_{xf} =	9.52
						$\beta_{sy,sup}$=	0.047	M_{ys} =	11.77
						$\beta_{sy,span}$=	0.036	M_{yf} =	9.01
P-4	Type 2	1.58	3.7	13.69	18.11	$\beta_{sx,sup}$=	0.060	M_{xs} =	14.88
						$\beta_{sx,span}$=	0.044	M_{xf} =	10.91
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.67
						$\beta_{sy,span}$=	0.029	M_{yf} =	7.19
P-5	Type 1	1.35	2.75	7.56	14.77	$\beta_{sx,sup}$=	0.048	M_{xs} =	5.36
						$\beta_{sx,span}$=	0.036	M_{xf} =	4.02
						$\beta_{sy,sup}$=	0.032	M_{ys} =	3.57
						$\beta_{sy,span}$=	0.024	M_{yf} =	2.68
P-6	Type 1	1.10	3.7	13.69	19.13	$\beta_{sx,sup}$=	0.037	M_{xs} =	9.69
						$\beta_{sx,span}$=	0.028	M_{xf} =	7.33
						$\beta_{sy,sup}$=	0.032	M_{ys} =	8.38
						$\beta_{sy,span}$=	0.024	M_{yf} =	6.29
P-7	Type 2	1.10	3.7	13.69	19.13	$\beta_{sx,sup}$=	0.044	M_{xs} =	11.52
						$\beta_{sx,span}$=	0.033	M_{xf} =	8.64
						$\beta_{sy,sup}$=	0.039	M_{ys} =	10.21
						$\beta_{sy,span}$=	0.029	M_{yf} =	7.59
P-8	Type 2	1.46	4	16.00	16.81	$\beta_{sx,sup}$=	0.057	M_{xs} =	15.33
						$\beta_{sx,span}$=	0.042	M_{xf} =	11.30
						$\beta_{sy,sup}$=	0.039	M_{ys} =	10.49
						$\beta_{sy,span}$=	0.029	M_{yf} =	7.80
P-9	Type 1	1.50	2.75	7.56	14.77	$\beta_{sx,sup}$=	0.053	M_{xs} =	5.92
						$\beta_{sx,span}$=	0.04	M_{xf} =	4.47
						$\beta_{sy,sup}$=	0.032	M_{ys} =	3.57
						$\beta_{sy,span}$=	0.024	M_{yf} =	2.68
P-10	Type 1	1.03	4	16.00	14.77	$\beta_{sx,sup}$=	0.034	M_{xs} =	8.03
						$\beta_{sx,span}$=	0.025	M_{xf} =	5.91
						$\beta_{sy,sup}$=	0.032	M_{ys} =	7.56
						$\beta_{sy,span}$=	0.024	M_{yf} =	5.67
P-11	Type 2	1.03	4	16.00	14.77	$\beta_{sx,sup}$=	0.041	M_{xs} =	9.69
						$\beta_{sx,span}$=	0.03	M_{xf} =	7.09
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.22
						$\beta_{sy,span}$=	0.029	M_{yf} =	6.85

Appendix A3: support moment adjustment

For panel P2&P3 on axis-4

M_{xs}=12.27KNm, M_{xs}=10.52KNm

Δm_x=1.75KN.m

20% 12.27KNm=2.45 since Δm_x<2.45, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (12.27+10.52)/2= 11.4KNm

For panel P5& P6 on axis C

M_{ys}=7.2KNm, M_{xs}=9.69KNm

Δm_x=2.49KN.m

20% 9.69KNm=1.94 since Δm_x>1.94, we use the moments distribution method

Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{2.75}=0.36364\ I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{4.1}=0.24\ I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.600$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.400$$

Balanced support moments

Mys1=0.6(9.69-7.2) +7.2=8.69KN-m

Mxs2=9.69-0.4(9.69-7.2) =8.69 KN-m

For panel P6&P7 on axis-4

M_{xs}=8.69KNm, M_{xs}=11.52KNm

Δm_x=2.83KN.m

20% 11.52KNm=2.3 since Δm_x>2.3, we use the moments distribution method

Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4.1}=\underline{0.2439 \quad I}$$

$$K_{BC}=\frac{I}{L}=\frac{I}{4.1}=\underline{0.24 \quad I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.500$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.500$$

Balanced support moments

Mxs1=0.5(11.2-8.69) +8.69=9.95KN-m

Mxs2=11.2-0.5(11.2-8.69) =9.95 KN-m

For panel P2&P6 on axis-B

M_{ys}=9.77KNm, M_{ys}=8.38KNm

Δm_x=1.39KN.m

20% 9.77KNm=1.95 since Δm_x<1.95, the average of the moments is taken

ΔM<20% use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.77+8.38)/2= 9.1KNm

For panel P3&P7 on axis-B

M_{ys}=11.77KNm, M_{ys}=10.21KNm

Δm_x=1.56KN.m

20% 11.77KNm=2.35 since Δm_x<2.35, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (11.77+10.21)/2= 10.99KNm

For panel P4&P8 on axis-C

M_{ys}=10.4KNm, M_{ys}=10.49KNm

Δm_x=0.09KN.m

20% 10.49KNm=2.1 since Δm_x<2.1, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (10.4+10.49)/2= 10.45KNm

For panel P8&P9 on axis-2

M_{ys}=15.33KNm, M_{ys}=3.57KNm

Δm_x=11.76KN.m

20% 15.33KNm=3.07 since Δm_x>3.07, we use the moments distribution method

Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{2.75}=\underline{0.36364\ I}$$

$$K_{BC}=\frac{I}{L}=\frac{I}{4}=\underline{0.25\ I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.590$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.410$$

Balanced support moments

Mxs1=0.59(15.33-3.57) +3.57=10.51KN-m

Mxs2=15.33-0.41(15.33-3.57) =10.51 KN-m

For panel P5&P9 on axis-C

M_{xs}=5.36KNm, M_{xs}=5.92KNm

Δm_x=0.56KN.m

20% 5.92KNm=1.184 since Δm_x<1.184, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (5.36+5.92)/2= 5.64KNm

For panel P9&P10 on axis-3

M_{xs}=10.51KNm, M_{xs}=9.69KNm

Δm_x=0.82KN.m

20% 10.51KNm=2.102 since Δm_x<2.102, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (10.51+9.69)/2= 10.1KNm

For panel P6&P10 on axis-C

M_{xs}=9.1KNm, M_{xs}=7.56KNm

Δm_x=1.54KN.m

20% 9.1KNm=1.82 since Δm_x<1.82, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.1+7.56)/2= 8.33KNm

For panel P10&P11 on axis-4

M_{xs}=10.1KNm, M_{xs}=9.69KNm

Δm_x=0.41KN.m

20% 10.1KNm=2.02 since Δm_x<2.02, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M=(10.1+9.69)/2=9.9

M For panel P7&P11 on axis-C

M_{ys}=10.99KNm, M_{ys}=9.22KNm

Δm_x=1.77KN.m

20% 10.99KNm=2.198 since Δm_x<2.198, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (10.99+9.22)/2= 10.1KNm

For panel P8&P12 on axis-D

M_{xs}=10.45KNm, M_{ys}=0.8KNm

Δm_x=9.65KN.m

20% 10.45KNm=2.09 since Δm_x>2.09, we use moment distribution method

Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=\underline{0.25 \quad I}$$

$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=\underline{0.83 \quad I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.230$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.770$$

Balanced support moments

Mxs1=0.23(10.45-0.8) +0.8=3.02KN-m

Mxs2=10.45-0.77(10.45-0.8) = 3.02KN-m

For panel P9&P13 on axis-D

M_{xs}=5.64KNm, M_{ys}=0.7KNm

Δm_x=4.94KN.m

20% 10.45KNm=1.13 since Δm_x>1.13, we use moment distribution method

Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=\underline{0.25 \quad I}$$

$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=\underline{0.83 \quad I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.230$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.770$$

Balanced support moments

Mxs1=0.23(5.64-0.7) +0.7=1.84KN-m

Mxs2=5.64-0.77(5.64-0.7) = 1.84KN-m

For panel P10&P14 on axis-D

M_{ys}=8.33KNm, M_{ys}=0.39KNm

Δm_x=7.94KN.m

20% 8.33KNm=1.67 since Δm_x>1.67, we use moment distribution method

Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=0.25I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=0.83I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.230$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.770$$

Balanced support moments

Mxs1=0.23(8.33-0.39) +0.39=2.22KN-m

Mxs2=8.33-0.77(8.33-0.9) = 2.22KN-m

For panel P11&P15 on axis-D

M_{xs}=10.1KNm, M_{ys}=0.39KNm

Δm_x=9.71KN.m

20%10.1KNm=2.02 since Δm_x>2.02, we use moment distribution method

Relative bending stiffness K

$$K_{AB} = \frac{I}{L} = \frac{I}{4} = 0.25\ I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{1.2} = 0.83\ I$$

Distribution factor Df

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.230$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.770$$

Balanced support moments

Mxs1=0.23(10.1-0.39) +0.39=2.62KN-m

Mxs2=10.1-0.77(10.1-0.39) = 2.62KN-m

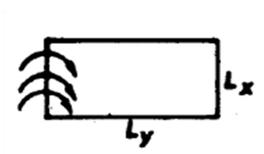
Appendix A4: Field moment adjustment

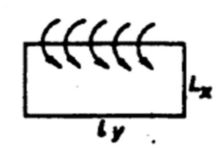
For Panel 2

M_{xf} = 11.93	M_{Xs} = 16.2	$M_{Xs,adj}$ = 11.4
M_{yf} = 8.24	M_{ys} = 11.08	$M_{ys,adj}$ = 9.1
Ly/Lx= 1.03		

ΔM_{xs} = Mxs- M_Xs,adj = 4.80

ΔM_{ys} = Mys- M_ys,adj = 1.98





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 2.365

Mxf ,adj = ΔMxf + Mxf = **14.295**

ΔM_{yf} = CyΔMxs + CyΔMys = 2.006

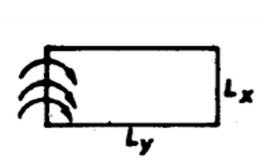
Myf ,adj = ΔMyf + Myf = **10.246**

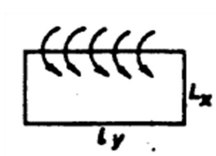
For Panel 3

M_{xf} = 9.52	M_{Xs} = 12.27	$M_{Xs,adj}$ = 11.4
M_{yf} = 9.01	M_{ys} = 11.77	$M_{ys,adj}$ = 9.1
Ly/Lx= 1.03		

ΔM_{xs} = Mxs- M_Xs,adj = 0.87

ΔM_{ys} = Mys- M_ys,adj = 2.67





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 1.099

Mxf ,adj = ΔMxf + Mxf = **10.619**

ΔM_{yf} = CyΔMxs + CyΔMys = 1.237

Myf ,adj = ΔMyf + Myf = **10.247**

For Panel 4

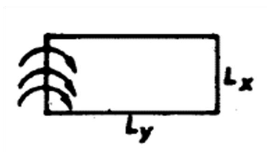
M_{xf} = 10.91 M_{Xs} = 16.2 $M_{Xs,adj}$ = 11.4

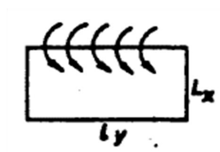
M_{yf} = 7.19 M_{ys} = 11.08 $M_{ys,adj}$ = 10.99

Ly/Lx= 1.58

ΔM_{xs} = Mxs- M_Xs,adj = 4.80

ΔM_{ys} = Mys- M_ys,adj = 0.09





Cx = 0.297 Cx = 0.416

Cy = 0.0852 Cy = 0.293

ΔMxf = CxΔMxs + CxΔMys = 1.463

Mxf ,adj = ΔMxf + Mxf = **12.373**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.435

Myf ,adj = ΔMyf + Myf = **7.625**

For panel 5 and panel 6 no need of adjustment, since change in Moment is zero

For Panel 7

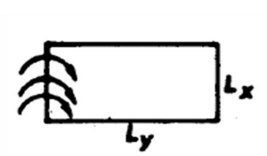
M_{xf} = 8.64 M_{Xs} = 11.52 $M_{Xs,adj}$ = 9.95

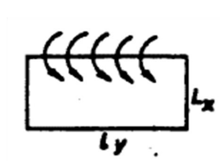
M_{yf} = 7.59 M_{ys} = 10.21 $M_{ys,adj}$ = 10.1

Ly/Lx= 1.10

ΔM_{xs} = Mxs- M_Xs,adj = 1.57

ΔM_{ys} = Mys- M_ys,adj = 0.11





Cx = 0.356 Cx = 0.314

Cy = 0.22 Cy = 0.374

ΔMxf = CxΔMxs + CxΔMys = 0.593

Mxf ,adj = ΔMxf + Mxf = **9.233**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.387

Myf ,adj = ΔMyf + Myf = **7.977**

For Panel 8

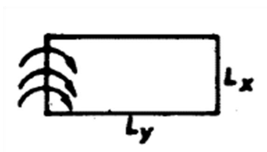
M_{xf} = 11.3 M_{Xs} = 15.33 $M_{Xs,adj}$ = 10.51

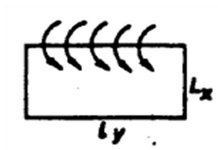
M_{yf} = 7.8 M_{ys} = 10.49 $M_{ys,adj}$ = 3.02

Ly/Lx= 1.46

ΔM_{xs} = Mxs- M_Xs,adj = 4.82

ΔM_{ys} = Mys- M_ys,adj = 7.47





Cx = 0.31 Cx = 0.41

Cy = 0.1 Cy = 0.32

ΔMxf = CxΔMxs + CxΔMys = 4.557

Mxf ,adj = ΔMxf + Mxf = **15.857**

ΔM_{yf} = CyΔMxs + CyΔMys = 2.872

Myf ,adj = ΔMyf + Myf = **10.672**

For Panel 9

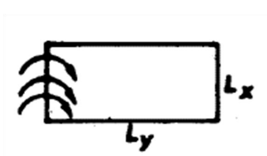
M_{xf} = 2.68 M_{Xs} = 3.57 $M_{Xs,adj}$ = 1.84

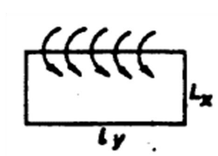
M_{yf} = 4.47 M_{ys} = 5.92 $M_{ys,adj}$ = 10.1

Ly/Lx= 1.50

ΔM_{xs} = Mxs- M_Xs,adj = 1.73

ΔM_{ys} = Mys- M_ys,adj = 0.00





Cx = 0.305 Cx = 0.421

Cy = 0.094 Cy = 0.31

ΔMxf = CxΔMxs + CxΔMys = 0.528

Mxf ,adj = ΔMxf + Mxf = **3.208**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.163

Myf ,adj = ΔMyf + Myf = **4.633**

For Panel 10

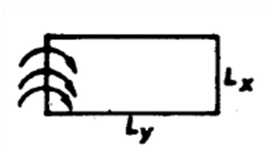
M_{xf} = 5.91 M_{Xs} = 8.03 $M_{Xs,adj}$ = 9.9

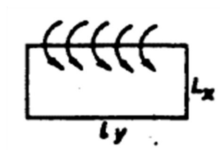
M_{yf} = 5.67 M_{ys} = 7.56 $M_{ys,adj}$ = 2.22

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.00

ΔM_{ys} = Mys- M_ys,adj = 5.34





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 1.549

Mxf ,adj = ΔMxf + Mxf = **7.459**

ΔM_{yf} = CyΔMxs + CyΔMys = 2.019

Myf ,adj = ΔMyf + Myf = **7.689**

For Panel 11

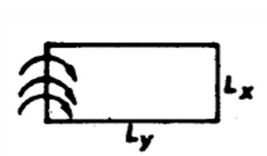
M_{xf} = 7.09 M_{Xs} = 9.69 $M_{Xs,adj}$ = 9.9

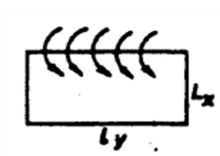
M_{yf} = 6.85 M_{ys} = 9.22 $M_{ys,adj}$ = 2.62

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.00

ΔM_{ys} = Mys- M_ys,adj = 6.60





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 1.914

Mxf ,adj = ΔMxf + Mxf = **9.004**

ΔM_{yf} = CyΔMxs + CyΔMys = 2.495

Myf ,adj = ΔMyf + Myf = **9.345**

Table 19. 3rd floor slab load transfer to beam

Appendix A4: 3rd floor load transfer to beam

panel	support condition	ly/lx	lx	pd	shear force coeficient Location value		shear force location	Shear force $V_i=\beta_{Vi}P_d$ Lx
P-1	Type 2	1.46	4.00	17.76	β_{vxc} =	0.46	V_{xc} =	32.678
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.36	V_{yc} =	25.574
					β_{vyd} =	0.24	V_{yd} =	17.050
P-2	Type 3	1.03	4.00	15.65	β_{vxc} =	0.372	V_{xc} =	23.287
					β_{vxd} =	0.249	V_{xd} =	16
					β_{vyc} =	0.36	V_{yc} =	22.536
					β_{vyd} =	0	V_{yd} =	0.000
P-3	Type 4	1.03	4.00	15.65	β_{vxc} =	0.412	V_{xc} =	25.791
					β_{vxd} =	0.269	V_{xd} =	17
					β_{vyc} =	0.4	V_{yc} =	25.040
					β_{vyd} =	0.26	V_{yd} =	16.276
P-4	Type 2	1.58	3.70	18.11	β_{vxc} =	0.48	V_{xc} =	32.163
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.36	V_{yc} =	24.123
					β_{vyd} =	0.24	V_{yd} =	16.082
P-5	Type 1	1.35	2.75	14.77	β_{vxc} =	0.42	V_{xc} =	17.059
					β_{vyc} =	0.33	V_{yc} =	13.404
P-6	Type 1	1.10	3.70	19.13	β_{vxc} =	0.43	V_{xc} =	30.436
					β_{vyc} =	0.4	V_{yc} =	28.312
P-7	Type 2	1.10	3.70	19.13	β_{vxc} =	0.38	V_{xc} =	26.897
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.36	V_{yc} =	25.481
					β_{vyd} =	0.24	V_{yd} =	17
P-8	Type 2	1.46	4.00	16.81	β_{vxc} =	0.49	V_{xc} =	32.948
					β_{vxd} =	0.32	V_{xd} =	21.517
					β_{vyc} =	0.4	V_{yc} =	26.896
					β_{vyd} =	0.24	V_{yd} =	16.138
P-9	Type 1	1.45	2.75	14.77	β_{vxc} =	0.49	V_{xc} =	19.903
					β_{vyc} =	0.4	V_{yc} =	16.247
P-10	Type 1	1.03	4.00	14.77	β_{vxc} =	0.49	V_{xc} =	28.949
					β_{vyc} =	0.4	V_{yc} =	23.632
P-11	Type 2	1.03	4.00	14.77	β_{vxc} =	0.49	V_{xc} =	28.949
					β_{vxd} =	0.32	V_{xd} =	18.906
					β_{vyc} =	0.4	V_{yc} =	23.632
					β_{vyd} =	0.24	V_{yd} =	14.179

Appendix B1: Second floor Depth determination

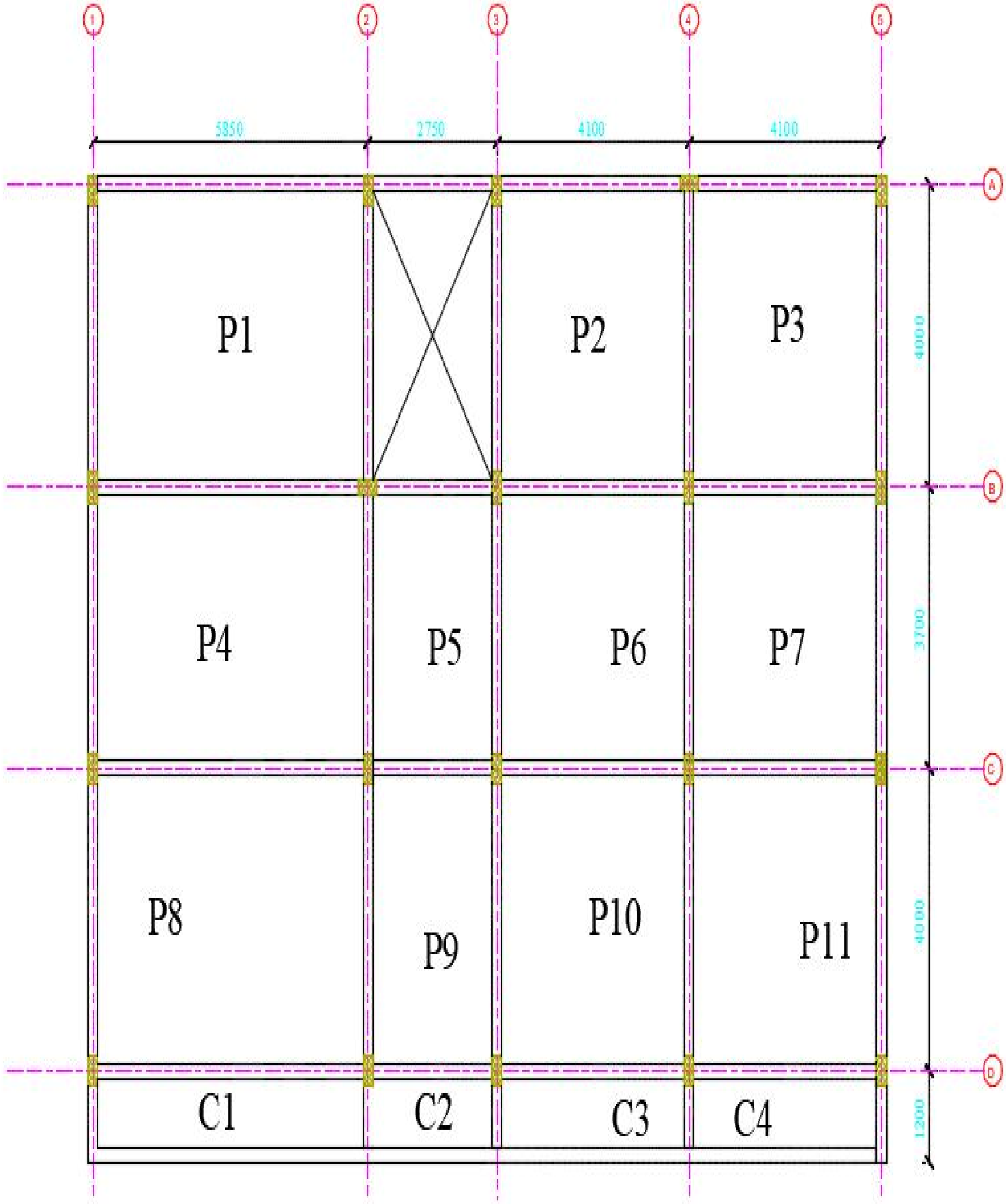


Figure 6- 1. 2nd floor slab layout

Table 20.second floor depth determination

Φbar= 10
Cnom= 20
ϒs = 1.15 fcd= $\frac{0.85*fck}{\Upsilon c}$=11.33 fyd= $\frac{fyk}{\Upsilon s}$= 347.83
ϒc = 1.5
fck = 20
fyk= 400 unit weight 25
F1= 1.25
F2= 1
F3= 1
d'= 25

panel	Ly	Lx	Ly/Lx	Type	support	N	K	Lx/d	d	D	D provided
p1	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	200
p2	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
p3	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
p4	5850	3700	1.58	two way	End	17.71	1.3	28.78	128.6	153.58	
p5	3700	2750	1.35	two way	Interior	17.71	1.5	33.20	82.8	107.82	
p6	4100	3700	1.11	two way	Interior	17.71	1.5	33.20	111.4	136.44	
p7	4100	3700	1.11	two way	End	17.71	1.3	28.78	128.6	153.58	
p8	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	
p9	4000	2750	1.45	two way	Interior	17.71	1.5	33.20	82.8	107.82	
p10	4100	4000	1.03	two way	Interior	17.71	1.5	33.20	120.5	145.47	
p11	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
C1	5850	1200	4.88	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C2	2750	1200	2.29	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C3	4100	1200	3.42	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C4	4100	1200	3.42	cantilever	End	17.71	0.4	8.85	135.5	160.53	

Appendix B2: Load calculation of 2nd floor slab

Table 21. load calculation of 2nd floor slab

P1			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Ceramic	23	0.005	0.115
partition wall	load of pa	Area				3.022649573
	70.73	23.4	HCB	14	0.15	
LIVE LOAD Q_k (category A)				2	KN/m^2	
DEAD LOAD Gk				9.52	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				15.85	KN/m^2	

P2 & P3	Material	Unit weig	Dimensio	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	Porcelain	23	0.005	0.115
LIVE LOAD Q_k (category D1)		5	KN/m^2	
DEAD LOAD Gk		6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		16.27	KN/m^2	

P4			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				2.909930716
	63	21.65	HCB	14	0.15	
LIVE LOAD Q_k (category D1)				5	KN/m^2	
DEAD LOAD Gk				9.40	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				20.20	KN/m^2	

P5			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area	GLASS	25	0.15	0.023813268
	0.2423	10.175				
LIVE LOAD Q_k (category C1)				3	KN/m^2	
DEAD LOAD Gk				6.52	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				13.30	KN/m^2	

P6 & P7			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area	GLASS	25	0.15	0.056558998
	0.858	15.17				
LIVE LOAD Q_k (category D1)				5	KN/m^2	
DEAD LOAD Gk				6.55	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				16.34	KN/m^2	

P8	Material	Unit weig	Dimensio	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	Porcelain	23	0.005	0.115
LIVE LOAD Q_k (category D1)		5	KN/m^2	
DEAD LOAD Gk		6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		16.27	KN/m^2	

P9			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				0.051818182
	0.57	11	GLASS	25	0.15	
LIVE LOAD Q_k (category D1)				5	KN/m^2	
DEAD LOAD Gk				6.55	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				16.34	KN/m^2	

P10 & P11			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				0.09652439
	1.583	16.4	GLASS	25	0.15	
LIVE LOAD Q_k (category D1)				5	KN/m^2	
DEAD LOAD Gk				6.59	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				16.40	KN/m^2	

C1			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				3.531908832
	24.794	7.02	STEEL	77	0.15	
LIVE LOAD Q_k (category D1)				5	KN/m^2	
DEAD LOAD Gk				10.03	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				21.04	KN/m^2	

C2			Material	Unit weigl	Dimensiol	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				2.102727273
	23.13	11	HCB	14	0.15	
LIVE LOAD Q_k (category D1)				5	KN/m^2	
DEAD LOAD Gk				8.60	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				19.11	KN/m^2	

C3 & C4			Material	Unit weigl	Dimensiol	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				1.535162602
	7.553	4.92	GLASS	25	0.15	
LIVE LOAD Q_k (category A)				4	KN/m^2	
DEAD LOAD Gk				8.03	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				16.84	KN/m^2	

Appendix B3: Moment analysis 2nd floor slab

Table 22.Moment analysis 2nd floor slab

panel	support conditio	ly/lx	lx	lx^2	Pd	moment coeficient		moment	moment
						location	value	location	$Mi=\alpha_i P_d$
P-1	Type 2	1.46	4	16.00	15.85	$\beta_{sx,sup}=$	0.057	$M_{xs}=$	14.46
						$\beta_{sx,span}=$	0.042	$M_{xf}=$	10.65
						$\beta_{sy,sup}=$	0.039	$M_{ys}=$	9.89
						$\beta_{sy,span}=$	0.029	$M_{yf}=$	7.35
P-2	Type 3	1.03	4.00	16.00	16.27	$\beta_{sx,sup}=$	0.042	$M_{xs}=$	10.93
						$\beta_{sx,span}=$	0.032	$M_{xf}=$	8.33
						$\beta_{sy,sup}=$	0.039	$M_{ys}=$	10.15
						$\beta_{sy,span}=$	0.03	$M_{yf}=$	7.81
P-3	Type 4	1.03	4	16	16.27	$\beta_{sx,sup}=$	0.049	$M_{xs}=$	12.76
						$\beta_{sx,span}=$	0.038	$M_{xf}=$	9.89
						$\beta_{sy,sup}=$	0.047	$M_{ys}=$	12.24
						$\beta_{sy,span}=$	0.036	$M_{yf}=$	9.37
P-4	Type 2	1.58	3.7	13.69	20.20	$\beta_{sx,sup}=$	0.060	$M_{xs}=$	16.59
						$\beta_{sx,span}=$	0.044	$M_{xf}=$	12.17
						$\beta_{sy,sup}=$	0.039	$M_{ys}=$	10.78
						$\beta_{sy,span}=$	0.029	$M_{yf}=$	8.02
P-5	Type 1	1.35	2.75	7.56	13.30	$\beta_{sx,sup}=$	0.048	$M_{xs}=$	4.83
						$\beta_{sx,span}=$	0.036	$M_{xf}=$	3.62
						$\beta_{sy,sup}=$	0.032	$M_{ys}=$	3.22
						$\beta_{sy,span}=$	0.024	$M_{yf}=$	2.41
P-6	Type 1	1.10	3.7	13.69	16.34	$\beta_{sx,sup}=$	0.037	$M_{xs}=$	8.28
						$\beta_{sx,span}=$	0.028	$M_{xf}=$	6.26
						$\beta_{sy,sup}=$	0.032	$M_{ys}=$	7.16
						$\beta_{sy,span}=$	0.024	$M_{yf}=$	5.37
P-7	Type 2	1.10	3.7	13.69	16.34	$\beta_{sx,sup}=$	0.044	$M_{xs}=$	9.84
						$\beta_{sx,span}=$	0.033	$M_{xf}=$	7.38
						$\beta_{sy,sup}=$	0.039	$M_{ys}=$	8.72
						$\beta_{sy,span}=$	0.029	$M_{yf}=$	6.49
P-8	Type 2	1.46	4	16.00	16.27	$\beta_{sx,sup}=$	0.057	$M_{xs}=$	14.84
						$\beta_{sx,span}=$	0.042	$M_{xf}=$	10.93
						$\beta_{sy,sup}=$	0.039	$M_{ys}=$	10.15
						$\beta_{sy,span}=$	0.029	$M_{yf}=$	7.55
P-9	Type 1	1.50	2.75	7.56	16.34	$\beta_{sx,sup}=$	0.053	$M_{xs}=$	6.55
						$\beta_{sx,span}=$	0.04	$M_{xf}=$	4.94
						$\beta_{sy,sup}=$	0.032	$M_{ys}=$	3.95
						$\beta_{sy,span}=$	0.024	$M_{yf}=$	2.97
P-10	Type 1	1.03	4	16.00	16.40	$\beta_{sx,sup}=$	0.034	$M_{xs}=$	8.92
						$\beta_{sx,span}=$	0.025	$M_{xf}=$	6.56
						$\beta_{sy,sup}=$	0.032	$M_{ys}=$	8.40
						$\beta_{sy,span}=$	0.024	$M_{yf}=$	6.30
P-11	Type 2	1.03	4	16.00	16.40	$\beta_{sx,sup}=$	0.041	$M_{xs}=$	10.76
						$\beta_{sx,span}=$	0.03	$M_{xf}=$	7.87
						$\beta_{sy,sup}=$	0.039	$M_{ys}=$	10.23
						$\beta_{sy,span}=$	0.029	$M_{yf}=$	7.61

Appendix B4: Moment adjustment of 2nd floor slab

For panel P1&P4 on axis-B

M_{ys}=9.89KNm, M_{ys}=10.78KNm

Δm_x=0.89KN.m

20% 10.78KNm=2.156   since Δm_x<2.156, the average of the moments is taken

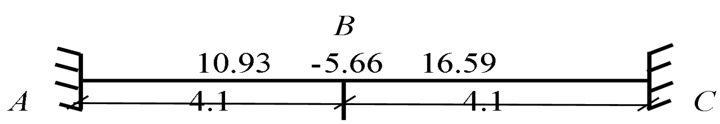
$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.89+10.78)/2 = 10.335KNm

For panel P2&P3 on axis-4

M_{xs}=10.93KNm, M_{xs}=16.59KNm, Δm_x=5.66KN.m

20% 16.59KNm=3.318   since Δm_x>3.318, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4.1}=\underline{0.2439\ I}$$

$$K_{BC}=\frac{I}{L}=\frac{I}{4.1}=\underline{0.24\ I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.500$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.500$$

Balanced support moments

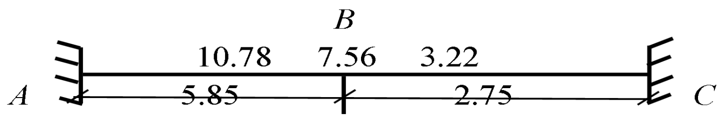
Mxs1=0.5(16.59-10.93) +10.93=13.76KN-m

Mxs2=16.59-0.5(16.59-10.93) =13.76 KN-m

For panel P4&P5 on axis-2

M_{ys}=10.78KNm, M_{ys}=3.22KNm, Δm_x=7.56KN.m

20% 10.78KNm=2.156   since Δm_x>2.156, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{5.85}=\underline{0.17094\ I}$$

$$K_{BC}=\frac{I}{L}=\frac{I}{2.75}=\underline{0.36\ \ I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.320$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.680$$

Balanced support moments

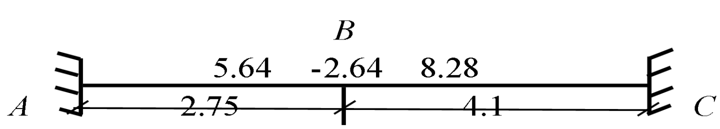
Mxs1=0.32(10.78-3.22) +3.22=5.64KN-m

Mxs2=10.78-0.68(10.78-3.22) =5.64 KN-m

For panel P5&P6 on axis-3

M_{xs}=5.64KNm, M_{xs}=8.28KNm, Δm_x=2.64KN.m

20% 8.28KNm=1.656   since Δm_x>1.656, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{2.75}=0.36364\,I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{4.1}=0.24\,I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.600$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.400$$

Balanced support moments

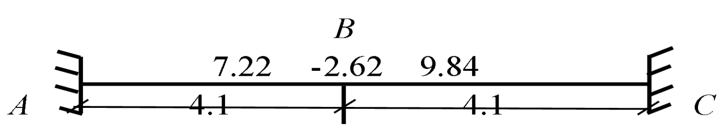
Mxs1=0.6(8.28-5.64) +5.64=7.224KN-m

Mxs2=8.28-0.4(8.28-5.64) =7.224 KN-m

For panel P6&P7 on axis-4

M_{xs}=7.224KNm, M_{xs}=9.84KNm, Δm_x=2.616KN.m

20% 9.84KNm=1.968, since Δm_x>1.968, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4.1}=0.2439\,I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{4.1}=0.24\,I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.500$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.500$$

Balanced support moments

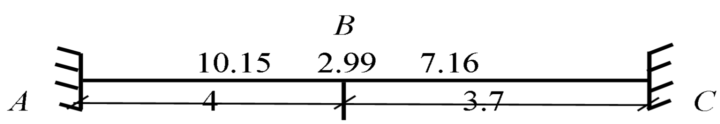
Mxs1=0.5(9.84-7.224) +7.224=8.532KN-m

Mxs2=9.84-0.5(9.84-7.224) =8.532KN-m

For panel P2&P6 on axis-B

M_{ys}=10.15KNm, M_{ys}=7.16KNm, Δm_x=2.99KN.m

20% 10.15KNm=2.03 since Δm_x>2.99, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}= 0.25\ I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{3.7}= 0.27\ I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.480$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.520$$

Balanced support moments

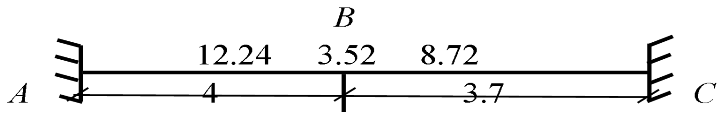
Mys1=0.48(10.15-7.16) +7.16=8.59KN-m

Mys2=10.15-0.52(10.15-7.16) = 8.59KN-m

For panel P3&P7 on axis-B

M_{ys}=12.24KNm, M_{ys}=8.72KNm, Δm_x=3.52KN.m

20% 12.24KNm=2.448   since Δm_x>3.52, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=0.25\ I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{3.7}=0.27\ I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.480$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.520$$

Balanced support moments

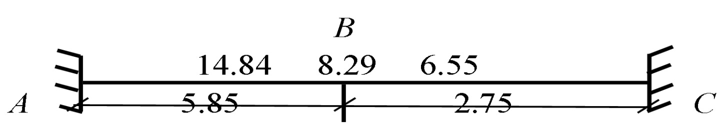
Mys1=0.48(12.24-8.72) +8.72=10.41KN-m

Mys2=12.24-0.52(12.24-8.72) =10.41 KN-m

For panel P8&P9 on axis-2

M_{xs}=14.84KNm, M_{xs}=6.55KNm, Δm_x=8.29KN.m

20% 14.84KNm=2.968   since Δm_x>2.968, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{5.85}=0.17094\ I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{2.75}=0.36\ I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.320$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.680$$

Balanced support moments

Mxs1=0.32(14.84-6.55) +6.55=9.2KN-m

Mxs2=14.84-0.68(14.84-6.55) =9.2KN-m

For panel P9&P10 on axis-3

M_{xs}=9.2KNm, M_{xs}=8.92KNm, Δm_x=0.28KN.m

20% 9.2KNm=1.84   since Δm_x<1.84, The average moment is used

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.2+8.92)/2 = 9.06KNm

For panel P10&P11 on axis-4

M_{xs}=9.06KNm, M_{xs}=10.76KNm, Δm_x=1.7KN.m

20% 10.76KNm=2.15   since Δm_x<1.7, The average moment is used

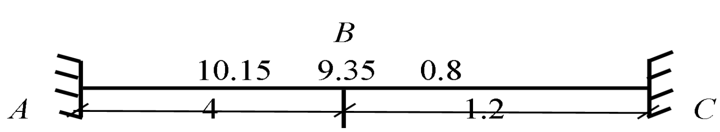
$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.06+10.76)/2 = 9.91KNm

For panel P8& C1 on axis-D

M_{xs}=10.15KNm, M_{xs}=0.8KNm, Δm_x=9.35KN.m

20% 10.15KNm=2.03   since Δm_x>2.03, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=\underline{0.25 \quad I}$$
$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=\underline{0.83 \quad I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.230$$
$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.770$$

Balanced support moments

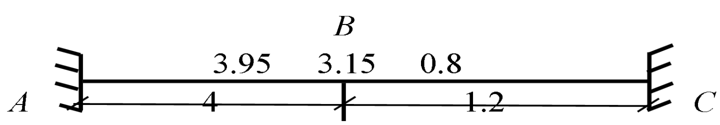
Mys1=0.23(10.15-0.8) +0.8=2.95KN-m

Mys2=10.15-0.77(10.15-0.8) =2.95KN-m

For panel P9& C2 on axis-D

M_{xs}=3.95KNm, M_{xs}=0.7KNm, Δm_x=3.25KN.m

20% 3.95KNm=0.79 since Δm_x>0.79, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=\underline{0.25 \quad I}$$
$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=\underline{0.83 \quad I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.230$$
$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.770$$

Balanced support moments

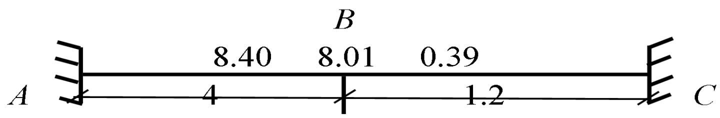
Mys1=0.23(3.95-0.7) +0.7=0.75KN-m

Mys2=3.95-0.77(3.95-0.7) =0.75KN-m

For panel P10& C3 on axis-D

M_{xs}=8.4KNm, M_{xs}=0.39KNm, Δm_x=8.01KN.m

20% KNm=1.68 since Δm_x>1.68, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=0.25\ I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=0.83\ I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.230$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.770$$

Balanced support moments

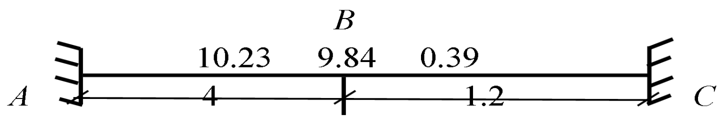
Mys1=0.23(8.4-0.39) +0.39=2.23KN-m

Mys2=8.4-0.77(8.4-0.39) =2.23KN-m

For panel P11& C4 on axis-D

M_{ys}=10.23KNm, M_{ys}=0.39KNm, Δm_x=8.01KN.m

20% KNm=1.68 since Δm_x>1.68, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=0.25\,I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=0.83\,I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.230$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.770$$

Balanced support moments

Mys1=0.23(10.23-0.39) +0.39=2.65KN-m

Mys2=10.23-0.77(10.23-0.39) =2.65KN-m

Second floor field adjustment

For panel 1 no need of adjustment

For Panel 2

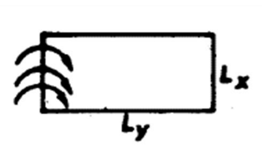
M_{xf} = 8.33 M_{Xs} = 10.93 $M_{Xs,adj}$ = 13.76

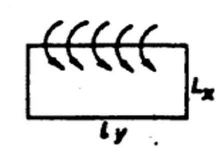
M_{yf} = 7.81 M_{ys} = 10.5 $M_{ys,adj}$ = 8.59

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.00

ΔM_{ys} = Mys- M_ys,adj = 1.91





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 0.554

Mxf ,adj = ΔMxf + Mxf = **8.884**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.722

Myf ,adj = ΔMyf + Myf = **8.532**

For Panel 3

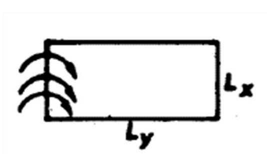
M_{xf} = 9.37 M_{Xs} = 12.76 $M_{Xs,adj}$ = 13.76

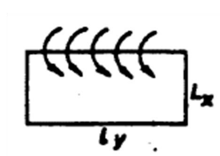
M_{yf} = 9.89 M_{ys} = 12.24 $M_{ys,adj}$ = 10.41

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.00

ΔM_{ys} = Mys- M_ys,adj = 1.83





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 0.531

Mxf ,adj = ΔMxf + Mxf = **9.901**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.692

Myf ,adj = ΔMyf + Myf = **10.582**

For Panel 4

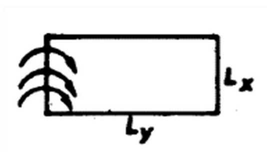
M_{xf} = 12.17 M_{Xs} = 16.59 $M_{Xs,adj}$ = 5.64

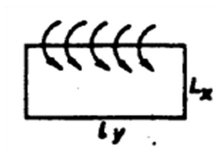
M_{yf} = 8.02 M_{ys} = 10.78 $M_{ys,adj}$ = 10.335

Ly/Lx= 1.58

ΔM_{xs} = Mxs- M_Xs,adj = 10.95

ΔM_{ys} = Mys- M_ys,adj = 0.44





Cx = 0.297 Cx = 0.416

Cy = 0.0852 Cy = 0.293

ΔMxf = CxΔMxs + CxΔMys = 3.437

Mxf ,adj = ΔMxf + Mxf = **15.607**

ΔM_{yf} = CyΔMxs + CyΔMys = 1.063

Myf ,adj = ΔMyf + Myf = **9.083**

For panel 5 and 6 no need of adjustment

For Panel 7

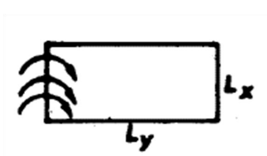
M_{xf} = 7.38 M_{Xs} = 9.84 $M_{Xs,adj}$ = 8.532

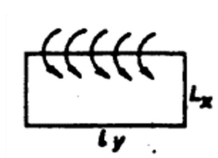
M_{yf} = 6.49 M_{ys} = 8.72 $M_{ys,adj}$ = 10.41

Ly/Lx= 1.10

ΔM_{xs} = Mxs- M_Xs,adj = 1.31

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.356 Cx = 0.314

Cy = 0.22 Cy = 0.374

ΔMxf = CxΔMxs + CxΔMys = 0.466

Mxf ,adj = ΔMxf + Mxf = **7.846**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.288

Myf ,adj = ΔMyf + Myf = **6.778**

For Panel 8

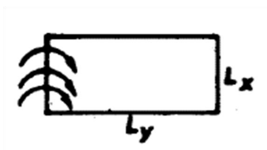
M_{xf} = 10.93 M_{Xs} = 14.84 $M_{Xs,adj}$ = 9.2

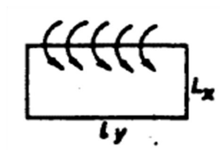
M_{yf} = 7.55 M_{ys} = 10.15 $M_{ys,adj}$ = 2.95

Ly/Lx= 1.46

ΔM_{xs} = Mxs- M_Xs,adj = 5.64

ΔM_{ys} = Mys- M_ys,adj = 7.20





Cx = 0.31 Cx = 0.41

Cy = 0.1 Cy = 0.32

ΔMxf = CxΔMxs + CxΔMys = 4.700

Mxf,adj = ΔMxf + Mxf = **15.630**

ΔM_{yf} = CyΔMxs + CyΔMys = 2.868

Myf,adj = ΔMyf + Myf = **10.418**

For Panel 9

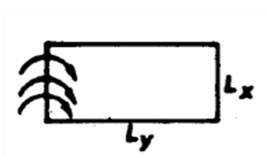
M_{xf} = 4.94 M_{Xs} = 6.55 $M_{Xs,adj}$ = 9.06

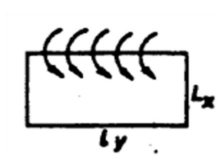
M_{yf} = 2.97 M_{ys} = 3.95 $M_{ys,adj}$ = 0.75

Ly/Lx= 1.50

ΔM_{xs} = Mxs- M_Xs,adj = 0.00

ΔM_{ys} = Mys- M_ys,adj = 3.20





Cx = 0.305 Cx = 0.421

Cy = 0.094 Cy = 0.31

ΔMxf = CxΔMxs + CxΔMys = 1.347

Mxf,adj = ΔMxf + Mxf = **6.287**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.992

Myf,adj = ΔMyf + Myf = **3.962**

For Panel 10

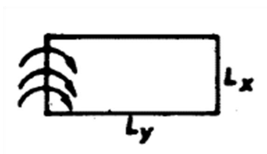
M_{xf} = 6.56 M_{Xs} = 8.92 $M_{Xs,adj}$ = 9.91

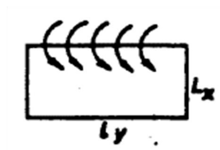
M_{yf} = 6.3 M_{ys} = 8.4 $M_{ys,adj}$ = 2.23

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.00 neglect

ΔM_{ys} = Mys- M_ys,adj = 6.17





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 1.789

Mxf,adj = ΔMxf + Mxf = **8.349**

ΔM_{yf} = CyΔMxs + CyΔMys = 2.332

Myf,adj = ΔMyf + Myf = **8.632**

For Panel 11

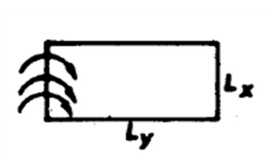
M_{xf} = 7.87 M_{Xs} = 10.76 $M_{Xs,adj}$ = 9.91

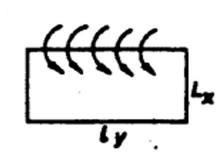
M_{yf} = 7.16 M_{ys} = 10.23 $M_{ys,adj}$ = 2.65

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.85

ΔM_{ys} = Mys- M_ys,adj = 7.58





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 2.515

Mxf,adj = ΔMxf + Mxf = **10.385**

ΔM_{yf} = CyΔMxs + CyΔMys = 3.088

Myf,adj = ΔMyf + Myf = **10.248**

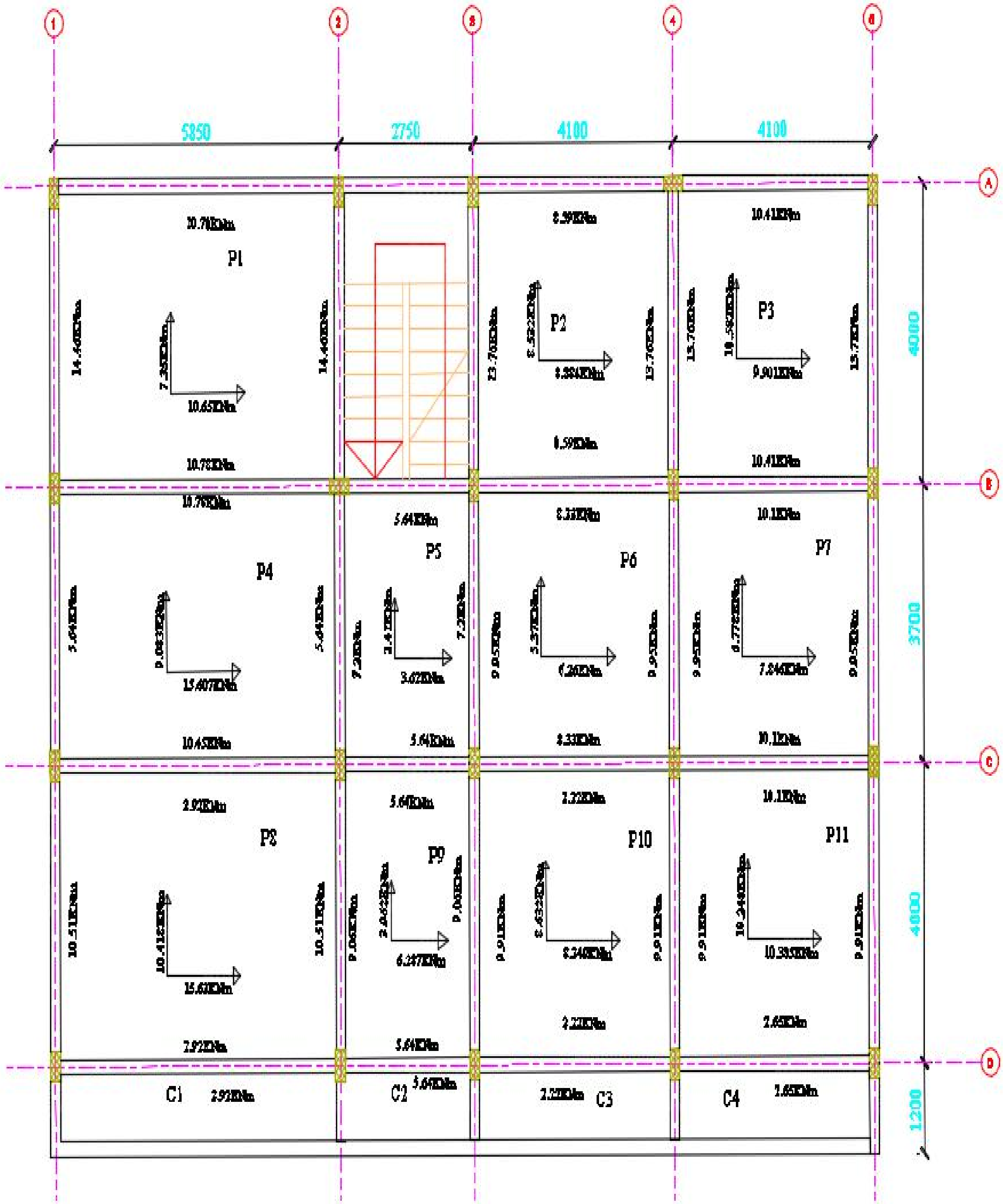


Figure 10- 2second floor slab adjusted moment

Appendix B5: load transfer of 2nd floor slab

Table 23. 2nd floor slab load transfer to beam

panel	support condition	ly/lx	lx	pd	shear force coeficient Location value		shear force location	Shear force $V_i=\beta_{Vi}P_d$ Lx
P-1	Type 2	1.46	4.00	15.85	β_{vxc} =	0.46	V_{xc} =	29.164
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.36	V_{yc} =	22.824
					β_{vyd} =	0.24	V_{yd} =	15.216
P-2	Type 3	1.03	4.00	16.27	β_{vxc} =	0.372	V_{xc} =	24.210
					β_{vxd} =	0.249	V_{xd} =	16
					β_{vyc} =	0.36	V_{yc} =	23.429
					β_{vyd} =	0	V_{yd} =	0.000
P-3	Type 4	1.03	4.00	16.27	β_{vxc} =	0.412	V_{xc} =	26.813
					β_{vxd} =	0.269	V_{xd} =	18
					β_{vyc} =	0.4	V_{yc} =	26.032
					β_{vyd} =	0.26	V_{yd} =	16.921
P-4	Type 2	1.58	3.70	20.20	β_{vxc} =	0.48	V_{xc} =	35.875
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.36	V_{yc} =	26.906
					β_{vyd} =	0.24	V_{yd} =	17.938
P-5	Type 1	1.35	2.75	13.30	β_{vxc} =	0.42	V_{xc} =	15.362
					β_{vyc} =	0.33	V_{yc} =	12.070
P-6	Type 1	1.10	3.70	16.34	β_{vxc} =	0.43	V_{xc} =	25.997
					β_{vyc} =	0.4	V_{yc} =	24.183
P-7	Type 2	1.10	3.70	16.34	β_{vxc} =	0.38	V_{xc} =	22.974
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.36	V_{yc} =	21.765
					β_{vyd} =	0.24	V_{yd} =	15
P-8	Type 2	1.46	4.00	16.27	β_{vxc} =	0.49	V_{xc} =	31.889
					β_{vxd} =	0.32	V_{xd} =	20.826
					β_{vyc} =	0.4	V_{yc} =	26.032
					β_{vyd} =	0.24	V_{yd} =	15.619
P-9	Type 1	1.45	2.75	16.34	β_{vxc} =	0.49	V_{xc} =	22.018
					β_{vyc} =	0.4	V_{yc} =	17.974
P-10	Type 1	1.03	4.00	16.34	β_{vxc} =	0.49	V_{xc} =	32.026
					β_{vyc} =	0.4	V_{yc} =	26.144
P-11	Type 2	1.03	4.00	16.34	β_{vxc} =	0.49	V_{xc} =	32.026
					β_{vxd} =	0.32	V_{xd} =	20.915
					β_{vyc} =	0.4	V_{yc} =	26.144
					β_{vyd} =	0.24	V_{yd} =	15.686

Appendix B6: shear capacity check

$$v_{Rd,c} = \left[0.12K(100\rho_1 f_{CK})^{\frac{1}{3}}\right] \mathrm{bd} \geq V_{min}$$

$$K = 1 + \left(\frac{200}{d}\right)^{0.5} \leq 2$$

d= 140

k= 2.20 not < 2,so k=2

$$\rho_1 = \frac{A_{sl}}{bd} \leq 0.02$$

taking minimum reinforcment ø 10 c/c 300

$\rho_1 =$ 0.0019

$$V_{min} = [0.035K^{\frac{3}{2}} f_{ck}^{\frac{1}{2}}]bd$$

$V_{min} = 61.981$

$v_{Rd,c} = 41.867 \leq V_{min}$ so, $v_{Rd,c} = 61.981$

maximum shear force transfererd to beams is Vi= Vi=37.21

Vi= 37.21 KN/m

61.981>37.21,slab thickness is sufficient to support design load

Appendix B7: Second floor Reinforcement calculation

Table 24.Second floor Reinforcement calculation

panel	moment	Ast=Msd/Z*fyc		ø	spacing	spacing used	Reinforcment			
P-1	$M_{xs} =$	14.46	329.94	10	237.92	220	ø	10	c/c	220
	$M_{xf} =$	10.65	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	10.78	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	7.35	250.00	10	314.00	300	ø	10	c/c	300
P-2	$M_{xs} =$	13.76	313.96	10	250.03	300	ø	10	c/c	300
	$M_{xf} =$	8.884	250.00	10	314.00	240	ø	10	c/c	240
	$M_{ys} =$	8.59	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	8.532	250.00	10	314.00	300	ø	10	c/c	300
P-3	$M_{xs} =$	13.76	313.96	10	250.03	300	ø	10	c/c	300
	$M_{xf} =$	9.901	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	10.41	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	10.582	250.00	10	314.00	300	ø	10	c/c	300
P-4	$M_{xs} =$	5.64	250.00	10	314.00	250	ø	10	c/c	250
	$M_{xf} =$	15.607	356.11	10	220.44	280	ø	10	c/c	280
	$M_{ys} =$	10.78	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	9.083	250.00	10	314.00	300	ø	10	c/c	300
P-5	$M_{xs} =$	7.224	250.00	10	314.00	260	ø	10	c/c	260
	$M_{xf} =$	3.62	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	3.22	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	2.41	250.00	10	314.00	300	ø	10	c/c	300
P-6	$M_{xs} =$	8.532	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	6.26	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	8.59	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	5.37	250.00	10	314.00	300	ø	10	c/c	300
P-7	$M_{xs} =$	8.532	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	7.846	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	10.41	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	6.778	250.00	10	314.00	300	ø	10	c/c	300
P-8	$M_{xs} =$	9.2	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	15.03	342.94	10	228.90	220	ø	10	c/c	220
	$M_{ys} =$	2.92	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	10.418	250.00	10	314.00	300	ø	10	c/c	300
P-9	$M_{xs} =$	9.06	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	6.287	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	0.75	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	3.962	250.00	10	314.00	300	ø	10	c/c	300
P-10	$M_{xs} =$	9.91	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	8.349	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	2.23	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	8.632	250.00	10	314.00	300	ø	10	c/c	300
P-11	$M_{xs} =$	9.91	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	10.385	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	2.65	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	10.248	250.00	10	314.00	300	ø	10	c/c	300

C-1		2.95	330.00	10	237.88	200	ø 10 c/c 200
C-2		0.75	330.00	10	237.88	200	ø 10 c/c 200
C-3		2.23	330.00	10	237.88	200	ø 10 c/c 200
C-4		2.65	330.00	10	237.88	200	ø 10 c/c 200

Appendix B8: Reinforcement Detailing

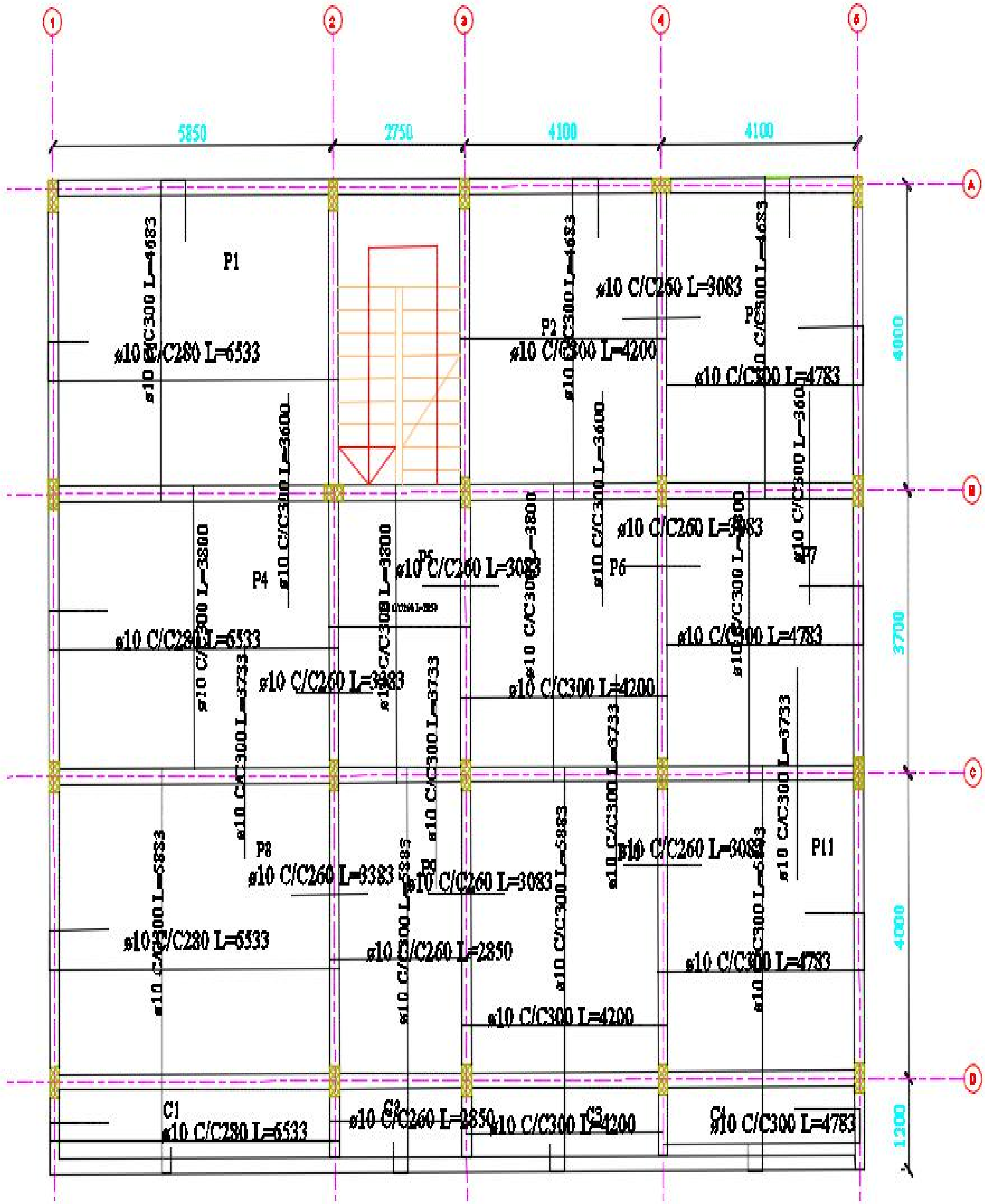


Figure 10- 3 second floor reinforecement detailing

Appendix C: first floor slab design

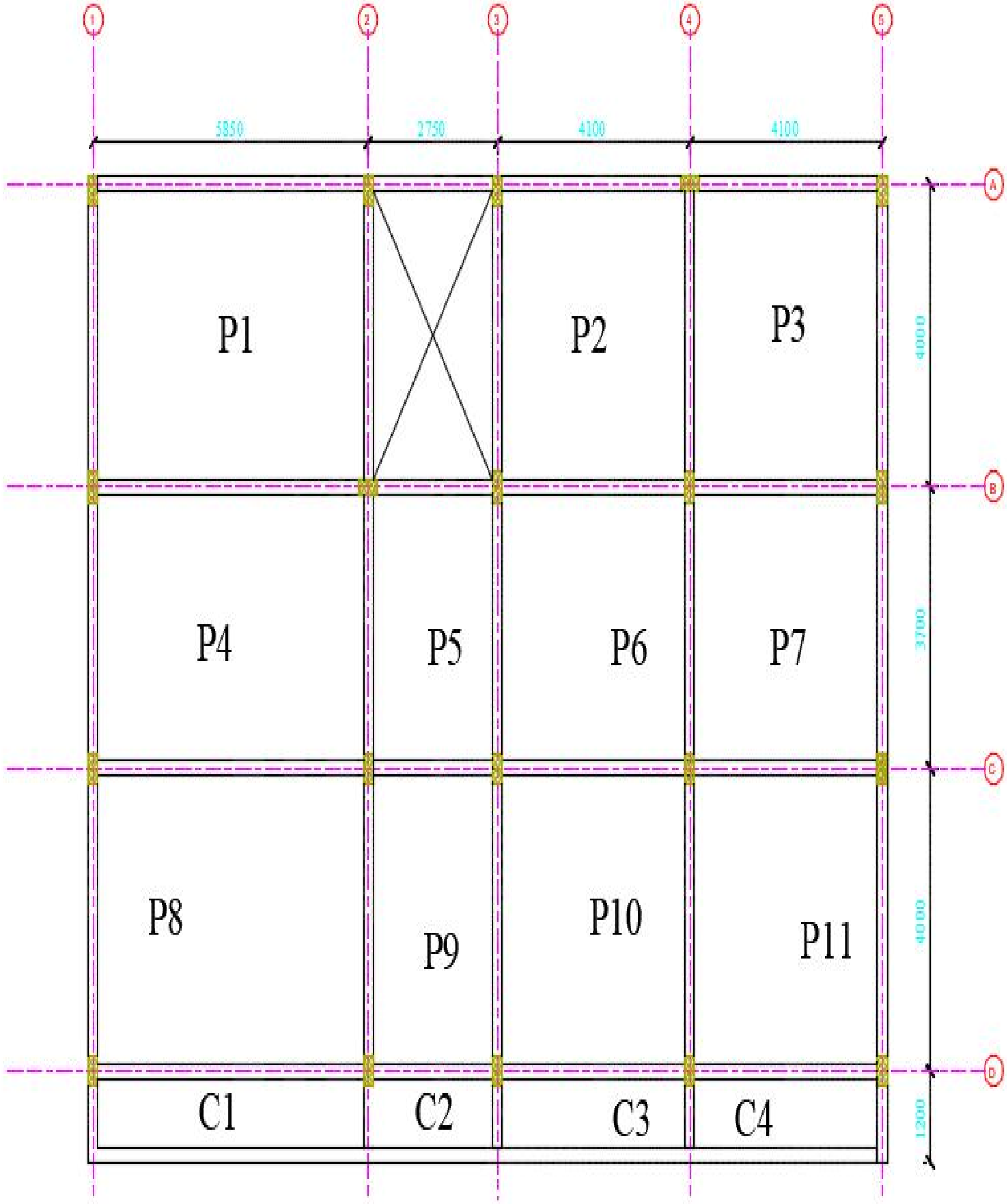


Figure 10- 4 first floor slab lay out

Appendix C1: First floor slab depth determination

Φbar= 10
Cnom= 20
ϒs = 1.15
ϒc = 1.5
fck = 20
fyk= 400
F1= 1.25
F2= 1
F3= 1
d'= 25

fcd= $\frac{0.85*fck}{\Upsilon c}$ = 11.33

fyd= $\frac{fyk}{\Upsilon s}$ = 347.83

unit weight 25

panel	Ly	Lx	Ly/Lx	Type	support	N	K	Lx/d	d	D	D provided
p1	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	200
p2	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
p3	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
p4	5850	3700	1.58	two way	End	17.71	1.3	28.78	128.6	153.58	
p5	3700	2750	1.35	two way	Interior	17.71	1.5	33.20	82.8	107.82	
p6	4100	3700	1.11	two way	Interior	17.71	1.5	33.20	111.4	136.44	
p7	4100	3700	1.11	two way	End	17.71	1.3	28.78	128.6	153.58	
p8	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	
p9	4000	2750	1.45	two way	Interior	17.71	1.5	33.20	82.8	107.82	
p10	4100	4000	1.03	two way	Interior	17.71	1.5	33.20	120.5	145.47	
p11	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
C1	5850	1200	4.88	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C2	2750	1200	2.29	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C3	4100	1200	3.42	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C4	4100	1200	3.42	cantilever	End	17.71	0.4	8.85	135.5	160.53	

Appendix C2: Load calculation of first floor slab

Table 25Load calculation of first floor slab

P1			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Ceramic	23	0.005	0.115
partition wall	load of pa	Area	HCB	14	0.15	3.022649573
	70.73	23.4				
LIVE LOAD Q_k (category A)				2	KN/m^2	
DEAD LOAD Gk				9.52	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				15.85	KN/m^2	

P2			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Ceramic	23	0.005	0.115
partition wall	load of pa	Area	HCB	14	0.15	1.717682927
	28.17	16.4				
LIVE LOAD Q_k (category A)				2	KN/m^2	
DEAD LOAD Gk				8.21	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				14.09	KN/m^2	

P3	Material	Unit weig	Dimensio	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	Porcelain	23	0.005	0.115
LIVE LOAD Q_k (category C1)		3	KN/m^2	
DEAD LOAD Gk		6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		13.27	KN/m^2	

P4			Material	Unit weigl	Dimensioi	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
	load of pa	Area				2.501616628
partition wall	54.16	21.65	HCB	14	0.15	
LIVE LOAD Q_k (category C1)				3	KN/m^2	
DEAD LOAD Gk				9.00	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				16.65	KN/m^2	

P5			Material	Unit weigl	Dimensioi	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Ceramic	23	0.005	0.115
	load of pa	Area				1.405110565
partition wall	14.297	10.175	HCB	14	0.15	
LIVE LOAD Q_k (category C1)				3	KN/m^2	
DEAD LOAD Gk				7.90	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				15.17	KN/m^2	

P6, P7, P8 & P10	Material	Unit weigl	Dimensioi	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	Porcelain	23	0.005	0.115
LIVE LOAD Q_k (category C1)		3	KN/m^2	
DEAD LOAD Gk		6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		13.27	KN/m^2	

P9			Material	Unit weigl	Dimensioi	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
	load of pa	Area				3.058181818
partition wall	33.64	11	HCB	14	0.15	
LIVE LOAD Q_k (category C1)				3	KN/m^2	
DEAD LOAD Gk				9.55	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				17.40	KN/m^2	

P11			Material	Unit weigl	Dimensioi	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
	load of pa	Area				0.368902439
partition wall	6.05	16.4	GLASS	25	0.15	
LIVE LOAD Q_k (category C1)				3	KN/m^2	
DEAD LOAD Gk				6.86	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				13.77	KN/m^2	

C1			Material	Unit weigl	Dimensioi	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
	load of pa	Area				3.531908832
partition wall	24.794	7.02	STEEL	77	0.15	
LIVE LOAD Q_k (category C1)				3	KN/m^2	
DEAD LOAD Gk				10.03	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				18.04	KN/m^2	

C2			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				2.102727273
	23.13	11	HCB	14	0.15	
LIVE LOAD Q_k (category D1)				5	KN/m^2	
DEAD LOAD Gk				8.60	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				19.11	KN/m^2	

C3 & C4			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				1.535162602
	7.553	4.92	GLASS	25	0.15	
LIVE LOAD Q_k (category A)				4	KN/m^2	
DEAD LOAD Gk				8.03	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				16.84	KN/m^2	

Appendix C3: First floor moment analysis

Table 26First floor moment analysis

panel	support conditio	ly/lx	lx	lx^2	Pd	moment coeficient location	value	moment location	moment $Mi=\alpha_i P_d$
P-1	Type 2	1.46	4	16.00	15.85	$\beta_{sx,sup}$=	0.057	M_{xs} =	14.46
						$\beta_{sx,span}$=	0.042	M_{xf} =	10.65
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.89
						$\beta_{sy,span}$=	0.029	M_{yf} =	7.35
P-2	Type 3	1.03	4.00	16.00	14.09	$\beta_{sx,sup}$=	0.042	M_{xs} =	9.47
						$\beta_{sx,span}$=	0.032	M_{xf} =	7.21
						$\beta_{sy,sup}$=	0.039	M_{ys} =	8.79
						$\beta_{sy,span}$=	0.03	M_{yf} =	6.76
P-3	Type 4	1.03	4	16	13.27	$\beta_{sx,sup}$=	0.049	M_{xs} =	10.40
						$\beta_{sx,span}$=	0.038	M_{xf} =	8.07
						$\beta_{sy,sup}$=	0.047	M_{ys} =	9.98
						$\beta_{sy,span}$=	0.036	M_{yf} =	7.64
P-4	Type 2	1.58	3.7	13.69	16.65	$\beta_{sx,sup}$=	0.060	M_{xs} =	13.68
						$\beta_{sx,span}$=	0.044	M_{xf} =	10.03
						$\beta_{sy,sup}$=	0.039	M_{ys} =	8.89
						$\beta_{sy,span}$=	0.029	M_{yf} =	6.61
P-5	Type 1	1.35	2.75	7.56	15.17	$\beta_{sx,sup}$=	0.048	M_{xs} =	5.51
						$\beta_{sx,span}$=	0.036	M_{xf} =	4.13
						$\beta_{sy,sup}$=	0.032	M_{ys} =	3.67
						$\beta_{sy,span}$=	0.024	M_{yf} =	2.75
P-6	Type 1	1.10	3.7	13.69	13.27	$\beta_{sx,sup}$=	0.037	M_{xs} =	6.72
						$\beta_{sx,span}$=	0.028	M_{xf} =	5.09
						$\beta_{sy,sup}$=	0.032	M_{ys} =	5.81
						$\beta_{sy,span}$=	0.024	M_{yf} =	4.36
P-7	Type 2	1.10	3.7	13.69	13.27	$\beta_{sx,sup}$=	0.044	M_{xs} =	7.99
						$\beta_{sx,span}$=	0.033	M_{xf} =	5.99
						$\beta_{sy,sup}$=	0.039	M_{ys} =	7.08
						$\beta_{sy,span}$=	0.029	M_{yf} =	5.27
P-8	Type 2	1.46	4	16.00	13.27	$\beta_{sx,sup}$=	0.057	M_{xs} =	12.10
						$\beta_{sx,span}$=	0.042	M_{xf} =	8.92
						$\beta_{sy,sup}$=	0.039	M_{ys} =	8.28
						$\beta_{sy,span}$=	0.029	M_{yf} =	6.16
P-9	Type 1	1.50	2.75	7.56	17.40	$\beta_{sx,sup}$=	0.053	M_{xs} =	6.97
						$\beta_{sx,span}$=	0.04	M_{xf} =	5.26
						$\beta_{sy,sup}$=	0.032	M_{ys} =	4.21
						$\beta_{sy,span}$=	0.024	M_{yf} =	3.16
P-10	Type 1	1.03	4	16.00	13.27	$\beta_{sx,sup}$=	0.034	M_{xs} =	7.22
						$\beta_{sx,span}$=	0.025	M_{xf} =	5.31
						$\beta_{sy,sup}$=	0.032	M_{ys} =	6.79
						$\beta_{sy,span}$=	0.024	M_{yf} =	5.10
P-11	Type 2	1.03	4	16.00	13.77	$\beta_{sx,sup}$=	0.041	M_{xs} =	9.03
						$\beta_{sx,span}$=	0.03	M_{xf} =	6.61
						$\beta_{sy,sup}$=	0.039	M_{ys} =	8.59
						$\beta_{sy,span}$=	0.029	M_{yf} =	6.39

Appendix C5: First floor moment adjustment

For panel P1&P4 on axis-B

M_{ys}=9.89KNm, M_{ys}=8.89KNm

Δm_x=1KN.m

20% 9.89KNm=1.978 since Δm_x<1.978, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.89+8.89)/2 = 9.39KNm

For panel P2&P3 on axis-4

M_{xs}=9.47KNm, M_{xs}=10.40KNm, Δm_x=0.93KN.m

20% 10.40KNm=2.08 since Δm_x<2.08, the average of the moments is taken

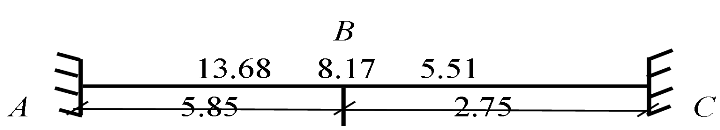
$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.47+10.40)/2 = 9.935KNm

For panel P4&P5 on axis-2

M_{xs}=13.68KNm, M_{xs}=5.51KNm, Δm_x=8.17KN.m

20% 13.68KNm=2.736 since Δm_x>2.736, Therefore we use moment distribution



Relative bending stiffness K

$K_{AB}=\frac{I}{L}=\frac{I}{5.85}=$ 0.17094 I

$K_{BC}=\frac{I}{L}=\frac{I}{2.75}=$ 0.36 I

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}} = 0.320$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}} = 0.680$$

Balanced support moments

Mxs1=0.32(13.68-5.51) +5.51=8.1244KN-m

Mxs2=13.68-0.68(13.68-5.51) =8.1244KN-m

For panel P5&P6 on axis-3

M_{xs}=8.1244KNm, M_{xs}=6.72KNm, Δm_x=1.4044KN.m

20% 8.1244KNm=1.625 since Δm_x<1.625, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (8.1244+6.72)/2 = 7.42KNm

For panel P6&P7 on axis-4

M_{xs}=7.42KNm, M_{xs}=7.99KNm, Δm_x=0.57KN.m

20% 7.99KNm=1.598, since Δm_x<1.598 , the average of the moments is taken

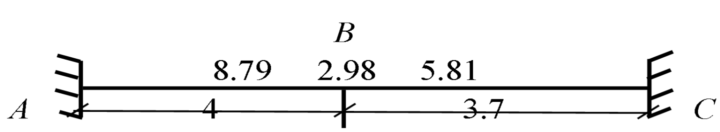
$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (7.42+7.99)/2 = 7.66KNm

For panel P2&P6 on axis-B

M_{ys}=8.79KNm, M_{ys}=5.81KNm, Δm_x=2.98KN.m

20% 8.79KNm=1.758 since Δm_x>1.758, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB} = \frac{I}{L} = \frac{I}{4} = 0.25\,I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{3.7} = 0.27\,I$$

Distribution factor Df

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.480$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.520$$

Balanced support moments

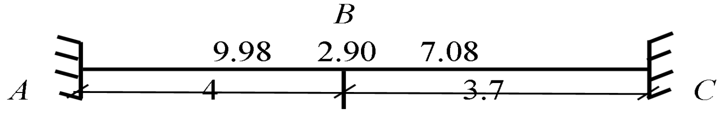
Mys1=0.48(8.79-5.81) +5.81=7.24KN-m

Mys2=8.79-0.52(8.79-5.81) = 7.24KN-m

For panel P3&P7 on axis-B

M_{ys}=9.98KNm, M_{ys}=7.08KNm, Δm_x=2.9KN.m

20% 9.98KNm=1.996 since Δm_x>1.996, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB} = \frac{I}{L} = \frac{I}{4} = 0.25\,I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{3.7} = 0.27\,I$$

Distribution factor Df

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.480$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.520$$

Balanced support moments

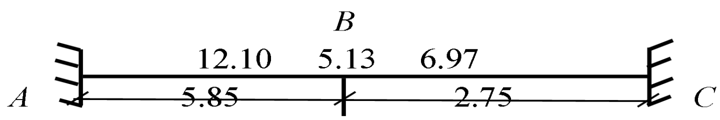
Mys1=0.48(9.98-7.08) +7.08=8.472KN-m

Mys2=9.98-0.52(9.98-7.08) =8.472KN-m

For panel P8&P9 on axis-2

M_{xs}=12.10KNm, M_{xs}=6.97KNm, Δm_x=5.13KN.m

20% 12.10KNm=2.42   since Δm_x>2.42, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{5.85}=\underline{0.17094\ I}$$

$$K_{BC}=\frac{I}{L}=\frac{I}{2.75}=\underline{0.36\ I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.320$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.680$$

Balanced support moments

Mxs1=0.32(12.10-6.97) +6.97=8.61KN-m

Mxs2=12.10-0.68(12.10-6.97) =8.61KN-m

For panel P9&P10 on axis-3

M_{xs}=8.61KNm, M_{xs}=7.22KNm, Δm_x=1.39KN.m

20% 8.61KNm=1.722   since Δm_x<1.722, The average moment is used

ΔM<20% use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (8.61+7.22)/2 = 7.92KNm

For panel P10&P11 on axis-4

M_{xs}=7.92KNm, M_{xs}=9.03KNm, Δm_x=1.11KN.m

20% 9.03KNm=1.806 since Δm_x<1.806, The average moment is used

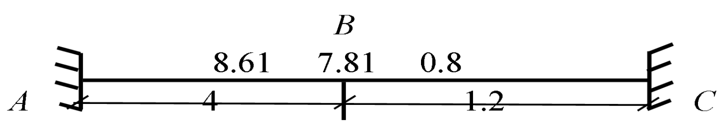
$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.03+7.92)/2 = 8.475KNm

For panel P8& C1 on axis-D

M_{xs}=8.61KNm, M_{xs}=0.8KNm, Δm_x=7.81KN.m

20% 8.61KNm=1.722 since Δm_x>1.722, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}= 0.25\ I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}= 0.83\ I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.230$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.770$$

Balanced support moments

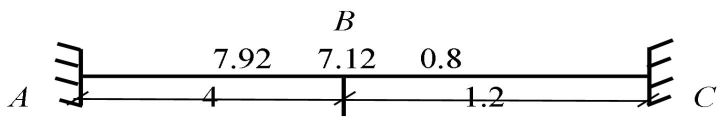
Mys1=0.23(8.61-0.8) +0.8=2.6KN-m

Mys2=8.61-0.77(8.61-0.8) =2.6KN-m

For panel P9& C2 on axis-D

M_{xs}=7.92KNm, M_{xs}=0.7KNm, Δm_x=7.22KN.m

20% 3.95KNm=0.79   since Δm_x>0.79, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=0.25\,I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=0.83\,I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.230$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.770$$

Balanced support moments

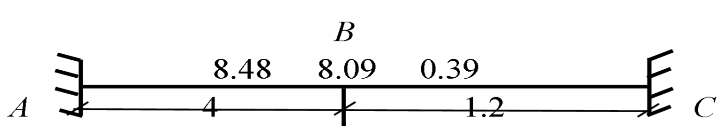
Mys1=0.23(7.92-0.7) +0.7=2.36KN-m

Mys2=7.92-0.77(7.92-0.7) =2.36KN-m

For panel P10& C3 on axis-D

M_{xs}=6.79KNm, M_{xs}=0.39KNm, Δm_x=8.09KN.m

20% 8.48KNm=1.696   since Δm_x>1.696, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=0.25\,I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=0.83\,I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.230$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.770$$

Balanced support moments

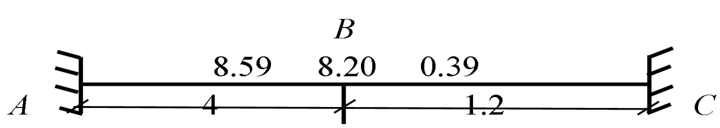
Mys1=0.23(8.48-0.39) +0.39=2.25KN-m

Mys2=8.48-0.77(8.48-0.39) =2.25KN-m

For panel P11& C4 on axis-D

M_{ys}=8.59KNm, M_{ys}=0.39KNm, Δm_x=8.2KN.m

20% KNm=1.68 since Δm_x>1.68, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=0.25\,I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{1.2}=0.83\,I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.230$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.770$$

Balanced support moments

Mys1=0.23(8.59-0.39) +0.39=2.276KN-m

Mys2=8.59-0.77(8.59-0.39) =2.276KN-m

Appendix C6 First floor field adjustment

For panel 1

For Panel 1

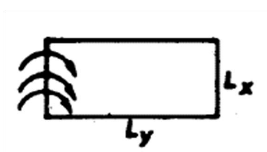
M_{xf} = 10.65 M_{Xs} = 14.46 $M_{Xs,adj}$ = 14.46

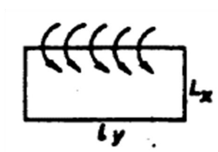
M_{yf} = 7.35 M_{ys} = 9.89 $M_{ys,adj}$ = 9.39

Ly/Lx= 1.46

ΔM_{xs} = Mxs- M_Xs,adj = 0.00 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.50





Cx = 0.31 Cx = 0.41

Cy = 0.1 Cy = 0.32

ΔMxf = CxΔMxs + CxΔMys = 0.205

Mxf ,adj = ΔMxf + Mxf = **10.855**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.160

Myf ,adj = ΔMyf + Myf = **7.510**

Panel 2 no need of adjustment

For Panel 3

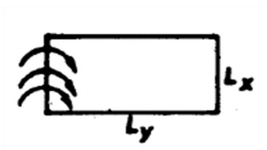
M_{xf} = 8.07 M_{Xs} = 10.4 $M_{Xs,adj}$ = 9.935

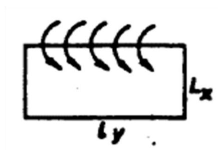
M_{yf} = 7.64 M_{ys} = 9.98 $M_{ys,adj}$ = 8.472

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.47

ΔM_{ys} = Mys- M_ys,adj = 1.51





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 0.611

Mxf ,adj = ΔMxf + Mxf = **8.681**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.692

Myf ,adj = ΔMyf + Myf = **8.332**

For Panel 4

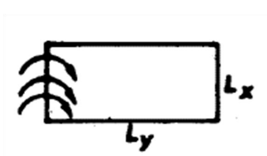
M_{xf} = 10.03 M_{Xs} = 13.68 $M_{Xs,adj}$ = 8.12

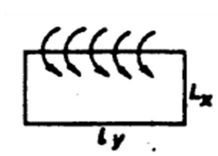
M_{yf} = 6.61 M_{ys} = 8.89 $M_{ys,adj}$ = 9.39

Ly/Lx= 1.58

ΔM_{xs} = Mxs- M_Xs,adj = 5.56

ΔM_{ys} = Mys- M_ys,adj = 0.00





Cx = 0.297 Cx = 0.416

Cy = 0.0852 Cy = 0.293

ΔMxf = CxΔMxs + CxΔMys = 1.651

Mxf ,adj = ΔMxf + Mxf = **11.681**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.474

Myf ,adj = ΔMyf + Myf = **7.084**

For panel 5 and panel 6 no need of adjustment

For Panel 7

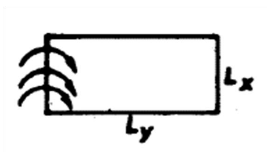
M_{xf} = 5.99 M_{Xs} = 7.99 $M_{Xs,adj}$ = 7.66

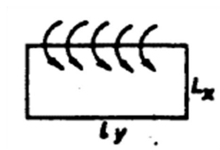
M_{yf} = 5.27 M_{ys} = 7.08 $M_{ys,adj}$ = 8.47

Ly/Lx= 1.10

ΔM_{xs} = Mxs- M_Xs,adj = 0.33

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.356 Cx = 0.314

Cy = 0.22 Cy = 0.374

ΔMxf = CxΔMxs + CxΔMys = 0.117

Mxf ,adj = ΔMxf + Mxf = **6.107**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.073

Myf ,adj = ΔMyf + Myf = **5.343**

For Panel 8

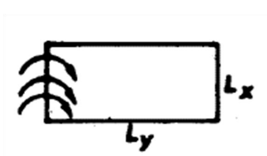
M_{xf} = 8.92 M_{Xs} = 12.1 $M_{Xs,adj}$ = 8.61

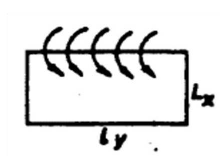
M_{yf} = 6.16 M_{ys} = 6.97 $M_{ys,adj}$ = 2.6

Ly/Lx= 1.46

ΔM_{xs} = Mxs- M_Xs,adj = 3.49

ΔM_{ys} = Mys- M_ys,adj = 4.37





Cx = 0.31 Cx = 0.41

Cy = 0.1 Cy = 0.32

ΔMxf = CxΔMxs + CxΔMys = 2.874

Mxf ,adj = ΔMxf + Mxf = **11.794**

ΔM_{yf} = CyΔMxs + CyΔMys = 1.747

Myf ,adj = ΔMyf + Myf = **7.907**

For Panel 9

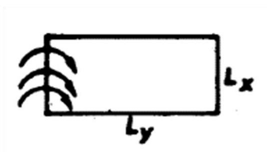
M_{xf} = 5.26 | M_{Xs} = 6.59 | $M_{Xs,adj}$ = 8.61

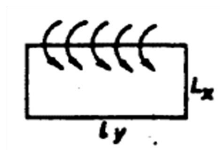
M_{yf} = 3.16 | M_{ys} = 4.21 | $M_{ys,adj}$ = 2.36

Ly/Lx= 1.50

ΔM_{xs} = Mxs- M_Xs,adj = 0.00

ΔM_{ys} = Mys- M_ys,adj = 1.85





Cx = 0.305 | Cx = 0.421

Cy = 0.094 | Cy = 0.31

ΔMxf = CxΔMxs + CxΔMys = 0.779

Mxf,adj = ΔMxf + Mxf = **6.039**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.574

Myf,adj = ΔMyf + Myf = **3.734**

For Panel 10

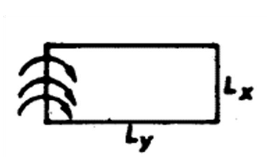
M_{xf} = 5.31 | M_{Xs} = 7.22 | $M_{Xs,adj}$ = 8.48

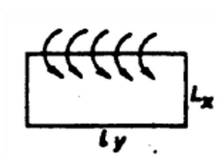
M_{yf} = 5.1 | M_{ys} = 6.79 | $M_{ys,adj}$ = 2.25

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.00 neglect

ΔM_{ys} = Mys- M_ys,adj = 4.54





Cx = 0.373 | Cx = 0.29

Cy = 0.262 | Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 1.317

Mxf,adj = ΔMxf + Mxf = **6.627**

ΔM_{yf} = CyΔMxs + CyΔMys = 1.716

Myf,adj = ΔMyf + Myf = **6.816**

For Panel 11

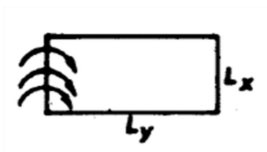
$M_{xf} = 6.61$ $M_{Xs} = 9.03$ $M_{Xs,adj} = 8.48$

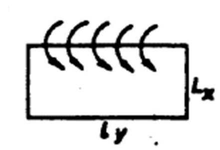
$M_{yf} = 6.39$ $M_{ys} = 8.59$ $M_{ys,adj} = 2.28$

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.55

ΔM_{ys} = Mys- M_ys,adj = 6.31





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 2.035

Mxf ,adj = ΔMxf + Mxf = **8.645**

ΔM_{yf} = CyΔMxs + CyΔMys = 2.529

Myf ,adj = ΔMyf + Myf = **8.919**

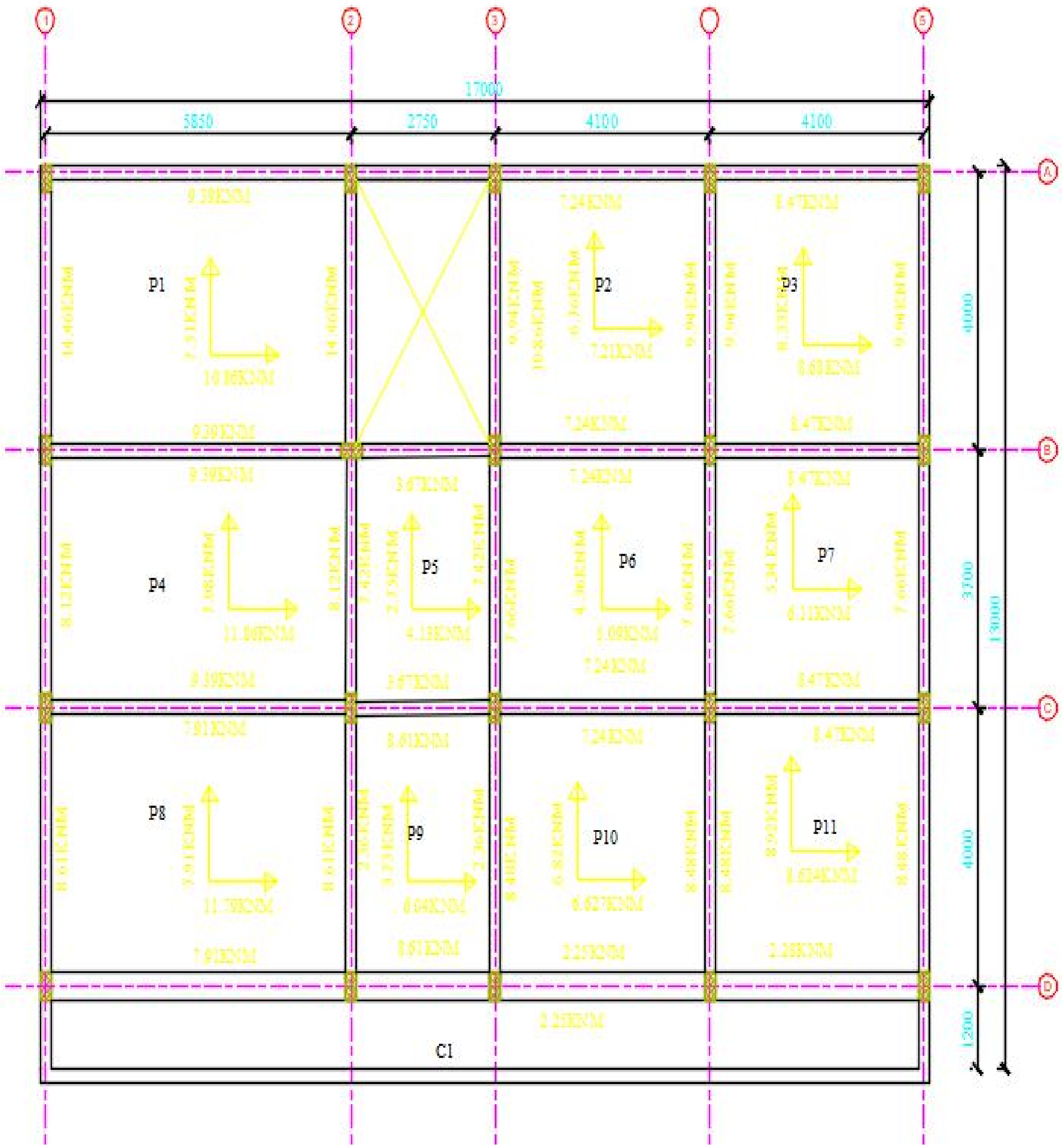


Figure 10- 5First floor slab adjusted moment

Appendix C7: Load transfer to beam

Table 27Load transfer to beam

panel	support condition	ly/lx	lx	pd	shear force coeficient Location value		shear force location	Shear force $V_i=\beta_{Vi} P_d$ Lx
P-1	Type 2	1.46	4.00	15.85	β_{vxc} =	0.46	V_{xc} =	29.164
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.36	V_{yc} =	22.824
					β_{vyd} =	0.24	V_{yd} =	15.216
P-2	Type 3	1.03	4.00	14.09	β_{vxc} =	0.372	V_{xc} =	20.966
					β_{vxd} =	0.249	V_{xd} =	14
					β_{vyc} =	0.36	V_{yc} =	20.290
					β_{vyd} =	0	V_{yd} =	0.000
P-3	Type 4	1.03	4.00	13.27	β_{vxc} =	0.412	V_{xc} =	21.869
					β_{vxd} =	0.269	V_{xd} =	14
					β_{vyc} =	0.4	V_{yc} =	21.232
					β_{vyd} =	0.26	V_{yd} =	13.801
P-4	Type 2	1.58	3.70	16.65	β_{vxc} =	0.48	V_{xc} =	29.570
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.36	V_{yc} =	22.178
					β_{vyd} =	0.24	V_{yd} =	14.785
P-5	Type 1	1.35	2.75	15.17	β_{vxc} =	0.42	V_{xc} =	17.521
					β_{vyc} =	0.33	V_{yc} =	13.767
P-6	Type 1	1.10	3.70	13.27	β_{vxc} =	0.43	V_{xc} =	21.113
					β_{vyc} =	0.4	V_{yc} =	19.640
P-7	Type 2	1.10	3.70	13.27	β_{vxc} =	0.38	V_{xc} =	18.658
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.36	V_{yc} =	17.676
					β_{vyd} =	0.24	V_{yd} =	12
P-8	Type 2	1.46	4.00	13.27	β_{vxc} =	0.49	V_{xc} =	26.009
					β_{vxd} =	0.32	V_{xd} =	16.986
					β_{vyc} =	0.4	V_{yc} =	21.232
					β_{vyd} =	0.24	V_{yd} =	12.739
P-9	Type 1	1.45	2.75	17.40	β_{vxc} =	0.49	V_{xc} =	23.447
					β_{vyc} =	0.4	V_{yc} =	19.140
P-10	Type 1	1.03	4.00	13.27	β_{vxc} =	0.49	V_{xc} =	26.009
					β_{vyc} =	0.4	V_{yc} =	21.232
P-11	Type 2	1.03	4.00	13.77	β_{vxc} =	0.49	V_{xc} =	26.989
					β_{vxd} =	0.32	V_{xd} =	17.626
					β_{vyc} =	0.4	V_{yc} =	22.032
					β_{vyd} =	0.24	V_{yd} =	13.219

Appendix C8: Shear capacity check

$$v_{Rd,c} = \left[0.12K(100\rho_1 f_{CK})^{\frac{1}{3}}\right] \text{bd} \geq V_{min}$$

$$K = 1 + \left(\frac{200}{d}\right)^{0.5} \leq 2$$

d= 140

k= 2.20 not < 2,so k=2

$$\rho_1 = \frac{A_{sl}}{bd} \leq 0.02$$

taking minimum reinforcment ø 10 c/c 300

$\rho_1 =$ 0.0019

$$V_{min} = [0.035K^{\frac{3}{2}} f_{ck}^{\frac{1}{2}}]bd$$

$V_{min} =$ 61.981

$v_{Rd,c} =$ 41.867 $\leq V_{min}$ so, $v_{Rd,c} =$ 61.981

maximum shear force transfererd to beams is Vi= Vi=34.59

Vi= 34.59 KN/m

61.981>37.21,slab thickness is sufficient to support design load

Appendix C9: First floor Reinforcement calculation

Table 28Reinforcement of first floor

panel	moment		Ast=Msd/Z*fyd	ø	spacing	spacing used	Reinforcement			
P-1	M_{xs} =	14.46	329.94	10	237.92	220	ø	10	c/c	220
	M_{xf} =	10.86	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	9.39	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.51	250.00	10	314.00	300	ø	10	c/c	300
P-2	M_{xs} =	9.94	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	7.21	250.00	10	314.00	240	ø	10	c/c	240
	M_{ys} =	7.24	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	6.76	250.00	10	314.00	300	ø	10	c/c	300
P-3	M_{xs} =	9.94	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	8.681	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	8.47	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	8.33	250.00	10	314.00	300	ø	10	c/c	300
P-4	M_{xs} =	8.12	250.00	10	314.00	250	ø	10	c/c	250
	M_{xf} =	11.68	266.50	10	294.55	280	ø	10	c/c	280
	M_{ys} =	9.39	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.08	250.00	10	314.00	300	ø	10	c/c	300
P-5	M_{xs} =	7.42	250.00	10	314.00	260	ø	10	c/c	260
	M_{xf} =	4.13	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	3.67	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	2.75	250.00	10	314.00	300	ø	10	c/c	300
P-6	M_{xs} =	7.66	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	5.09	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	7.24	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	4.36	250.00	10	314.00	300	ø	10	c/c	300
P-7	M_{xs} =	7.66	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	6.11	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	8.47	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	5.34	250.00	10	314.00	300	ø	10	c/c	300
P-8	M_{xs} =	8.61	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	11.79	269.01	10	291.81	220	ø	10	c/c	220
	M_{ys} =	2.6	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.91	250.00	10	314.00	300	ø	10	c/c	300
P-9	M_{xs} =	8.61	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	6.04	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	2.36	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	3.73	250.00	10	314.00	300	ø	10	c/c	300
P-10	M_{xs} =	8.48	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	6.627	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	2.25	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	6.82	250.00	10	314.00	300	ø	10	c/c	300
P-11	M_{xs} =	8.48	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	8.645	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	2.28	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	8.92	250.00	10	314.00	300	ø	10	c/c	300

C-1		2.6	330.00	10	237.88	200	ø	10	c/c	200
C-2		2.36	330.00	10	237.88	150	ø	10	c/c	150
C-3		2.25	330.00	10	237.88	200	ø	10	c/c	200
C-4		2.28	330.00	10	237.88	100	ø	10	c/c	100

Appendix C10: Reinforcement Detailing

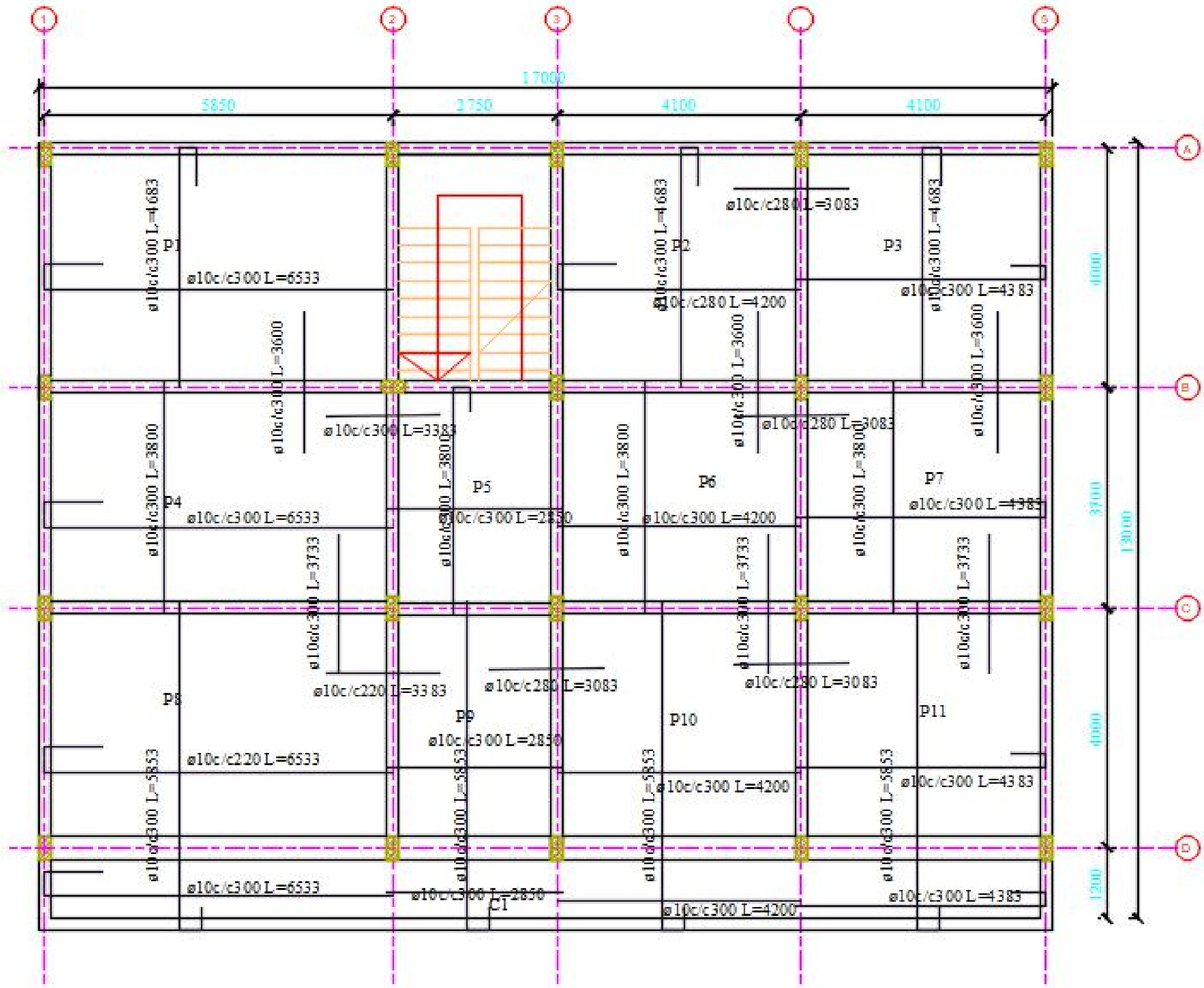


Figure 10- 6Reinforcement Detailing

Appendix D: Ground floor slab design

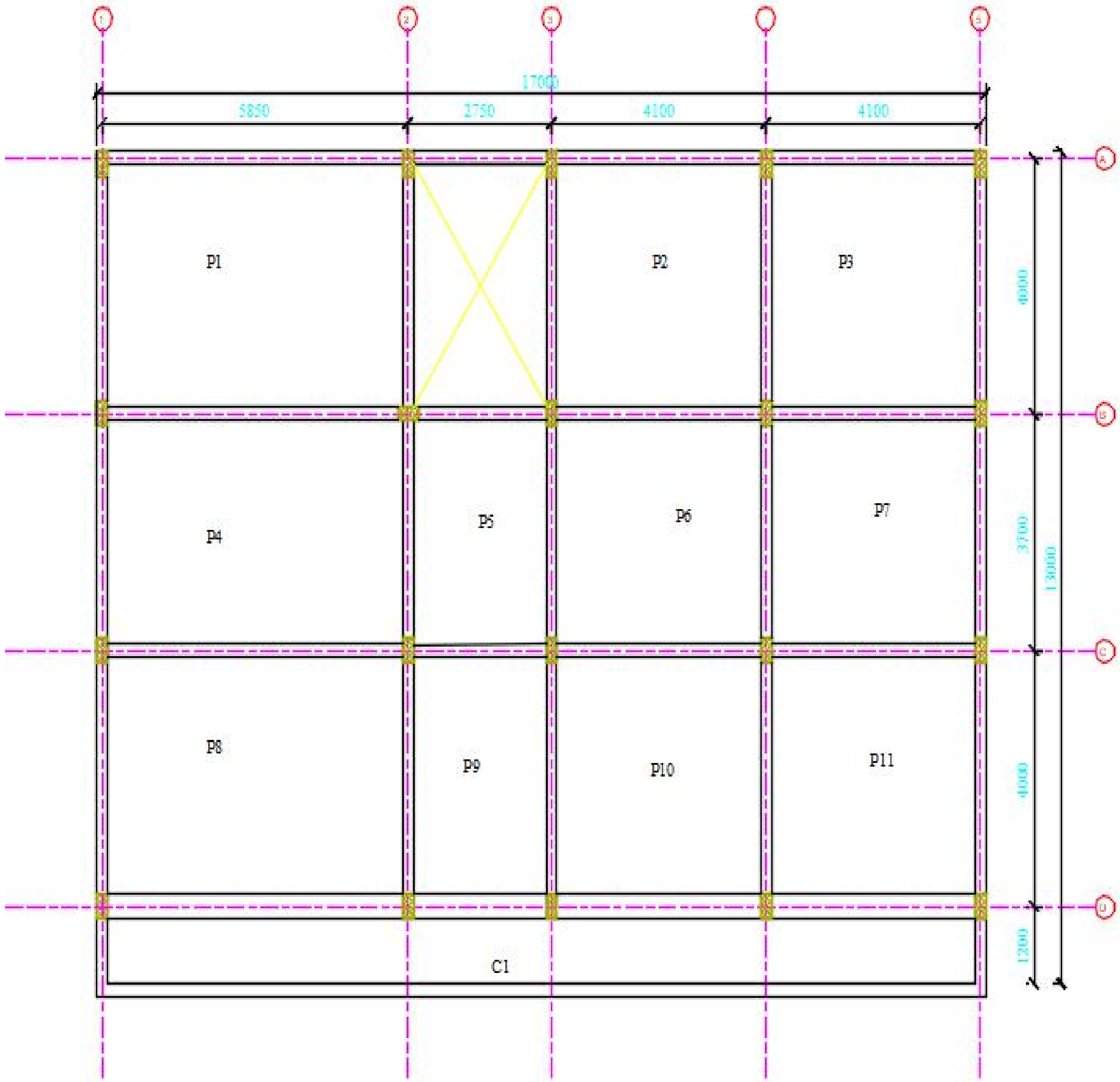


Figure 10- 7Ground floor slab lay out

Appendix D1: Ground floor slab depth calculation

Φbar= 10
Cnom= 20
ϒs = 1.15 fcd= $\frac{0.85*fck}{ϒc}$=11.33 fyd= $\frac{fyk}{ϒs}$= 347.83
ϒc = 1.5
fck = 20
fyk= 400 unit weight 25
F1= 1.25
F2= 1
F3= 1
d'= 25

panel	Ly	Lx	Ly/Lx	Type	support	N	K	Lx/d	d	D	D provided
p1	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	200
p2	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
p3	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
p4	5850	3700	1.58	two way	End	17.71	1.3	28.78	128.6	153.58	
p5	3700	2750	1.35	two way	Interior	17.71	1.5	33.20	82.8	107.82	
p6	4100	3700	1.11	two way	Interior	17.71	1.5	33.20	111.4	136.44	
p7	4100	3700	1.11	two way	End	17.71	1.3	28.78	128.6	153.58	
p8	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	
p9	4000	2750	1.45	two way	Interior	17.71	1.5	33.20	82.8	107.82	
p10	4100	4000	1.03	two way	Interior	17.71	1.5	33.20	120.5	145.47	
p11	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
C1	5850	1200	4.88	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C2	2750	1200	2.29	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C3	4100	1200	3.42	cantilever	End	17.71	0.4	8.85	135.5	160.53	
C4	4100	1200	3.42	cantilever	End	17.71	0.4	8.85	135.5	160.53	

Appendix D2: Ground floor slab load calculation

Table 29Ground floor slab load calculation

P1			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Ceramic	23	0.005	0.115
partition wall	load of pa	Area	HCB	14	0.15	3.022649573
	70.73	23.4				
LIVE LOAD Q_k (category A)				2	KN/m^2	
DEAD LOAD Gk				9.52	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				15.85	KN/m^2	

P2			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Ceramic	23	0.005	0.115
partition wall	load of pa	Area	HCB	14	0.15	4.579268293
	75.1	16.4				
LIVE LOAD Q_k (category A)				2	KN/m^2	
DEAD LOAD Gk				11.07	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				17.95	KN/m^2	

P3			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area	HCB	14	0.15	2.333536585
	38.27	16.4				
LIVE LOAD Q_k (category B1)				3	KN/m^2	
DEAD LOAD Gk				6.50	KN/m^2	

P4			Material	Unit weig	Dimensio	Load KN/m2
Slab			Reinforced concrete	25	0.200	5.00
cement screed			concrete	23	0.03	0.69
celling plaster			concrete	23	0.03	0.69
floor finish			Porcelain	23	0.005	0.115
partition wall	load of pa	Area				2.272517321
	49.2	21.65	HCB	14	0.15	
LIVE LOAD Q_k (category A1)				2	KN/m^2	
DEAD LOAD Gk				8.77	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$				14.84	KN/m^2	

P5 andP9	Material	Unit weig	Dimensio	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	Ceramic	23	0.005	0.115
LIVE LOAD Q_k (category C1)		3	KN/m^2	
DEAD LOAD Gk		6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		13.27	KN/m^2	

P6, P7, P10 & P11	Material	Unit weig	Dimensio	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	Porcelain	23	0.005	0.115
LIVE LOAD Q_k (category C2)		4	KN/m^2	
DEAD LOAD Gk		6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		14.77	KN/m^2	

P8	Material	Unit weig	Dimensio	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	Porcelain	23	0.005	0.115
LIVE LOAD Q_k (category A1)		2	KN/m^2	
DEAD LOAD Gk		6.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		11.77	KN/m^2	

C1	Material	Unit weig	Dimensio	Load KN/m2
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.03	0.69
celling plaster	concrete	23	0.03	0.69
floor finish	Ceramic	23	0.005	0.115
LIVE LOAD Q_k (category A1)		4	KN/m^2	
DEAD LOAD Gk		6.38	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$		14.61	KN/m^2	

Appendix D3: Moment analysis of ground floor slab

Table 30Moment analysis of ground floor slab

panel	support conditio	ly/lx	lx	lx^2	Pd	moment coeficient		moment	moment
						location	value	location	$Mi=\alpha_i P_d$
P-1	Type 2	1.46	4	16.00	15.85	$\beta_{sx,sup}$=	0.057	M_{xs} =	14.46
						$\beta_{sx,span}$=	0.042	M_{xf} =	10.65
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.89
						$\beta_{sy,span}$=	0.029	M_{yf} =	7.35
P-2	Type 3	1.03	4.00	16.00	17.92	$\beta_{sx,sup}$=	0.042	M_{xs} =	12.04
						$\beta_{sx,span}$=	0.032	M_{xf} =	9.18
						$\beta_{sy,sup}$=	0.039	M_{ys} =	11.18
						$\beta_{sy,span}$=	0.03	M_{yf} =	8.60
P-3	Type 4	1.03	4	16	6.50	$\beta_{sx,sup}$=	0.049	M_{xs} =	5.10
						$\beta_{sx,span}$=	0.038	M_{xf} =	3.95
						$\beta_{sy,sup}$=	0.047	M_{ys} =	4.89
						$\beta_{sy,span}$=	0.036	M_{yf} =	3.74
P-4	Type 2	1.58	3.7	13.69	14.84	$\beta_{sx,sup}$=	0.060	M_{xs} =	12.19
						$\beta_{sx,span}$=	0.044	M_{xf} =	8.94
						$\beta_{sy,sup}$=	0.039	M_{ys} =	7.92
						$\beta_{sy,span}$=	0.029	M_{yf} =	5.89
P-5	Type 1	1.35	2.75	7.56	13.27	$\beta_{sx,sup}$=	0.048	M_{xs} =	4.82
						$\beta_{sx,span}$=	0.036	M_{xf} =	3.61
						$\beta_{sy,sup}$=	0.032	M_{ys} =	3.21
						$\beta_{sy,span}$=	0.024	M_{yf} =	2.41
P-6	Type 1	1.10	3.7	13.69	14.77	$\beta_{sx,sup}$=	0.037	M_{xs} =	7.48
						$\beta_{sx,span}$=	0.028	M_{xf} =	5.66
						$\beta_{sy,sup}$=	0.032	M_{ys} =	6.47
						$\beta_{sy,span}$=	0.024	M_{yf} =	4.85
P-7	Type 2	1.10	3.7	13.69	14.77	$\beta_{sx,sup}$=	0.044	M_{xs} =	8.90
						$\beta_{sx,span}$=	0.033	M_{xf} =	6.67
						$\beta_{sy,sup}$=	0.039	M_{ys} =	7.89
						$\beta_{sy,span}$=	0.029	M_{yf} =	5.86
P-8	Type 2	1.46	4	16.00	11.77	$\beta_{sx,sup}$=	0.057	M_{xs} =	10.73
						$\beta_{sx,span}$=	0.042	M_{xf} =	7.91
						$\beta_{sy,sup}$=	0.039	M_{ys} =	7.34
						$\beta_{sy,span}$=	0.029	M_{yf} =	5.46
P-9	Type 1	1.50	2.75	7.56	13.27	$\beta_{sx,sup}$=	0.053	M_{xs} =	5.32
						$\beta_{sx,span}$=	0.04	M_{xf} =	4.01
						$\beta_{sy,sup}$=	0.032	M_{ys} =	3.21
						$\beta_{sy,span}$=	0.024	M_{yf} =	2.41
P-10	Type 1	1.03	4	16.00	14.77	$\beta_{sx,sup}$=	0.034	M_{xs} =	8.03
						$\beta_{sx,span}$=	0.025	M_{xf} =	5.91
						$\beta_{sy,sup}$=	0.032	M_{ys} =	7.56
						$\beta_{sy,span}$=	0.024	M_{yf} =	5.67
P-11	Type 2	1.03	4	16.00	14.77	$\beta_{sx,sup}$=	0.041	M_{xs} =	9.69
						$\beta_{sx,span}$=	0.03	M_{xf} =	7.09
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.22
						$\beta_{sy,span}$=	0.029	M_{yf} =	6.85

Appendix D5: Moment Adjustment

Support moment adjustment

For panel P1&P4 on axis-B

M_{ys}=9.89KNm, M_{ys}=7.92KNm

Δm_x=1.97KN.m

20% 9.89KNm=1.978 since Δm_x<1.978, the average of the moments is taken

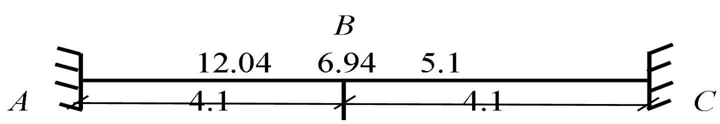
$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.89+7.92)/2 = 8.91KNm

For panel P2&P3 on axis-4

M_{xs}=12.04KNm, M_{xs}=5.1KNm, Δm_x=6.94KN.m

20% 12.04KNm=2.41 since Δm_x>2.41, Therefore we use moment distribution



Relative bending stiffness K

$K_{AB}=\frac{I}{L}=\frac{I}{4.1}=$ 0.2439 I

$K_{BC}=\frac{I}{L}=\frac{I}{4.1}=$ 0.24 I

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.500$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.500$$

Balanced support moments

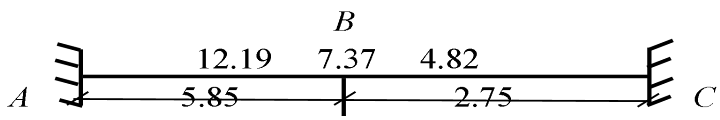
Mxs1=0.5(12.04-5.1) +5.1=8.57KN-m

Mxs2=12.04-0.5(12.04-5.1) =8.57KN-m

For panel P4&P5 on axis-2

M_{xs}=12.19KNm, M_{xs}=4.82KNm, Δm_x=7.37KN.m

20% 12.19KNm=2.438   since Δm_x>2.438, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{5.85}=0.17094\ I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{2.75}=0.36\ I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.320$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.680$$

Balanced support moments

Mxs1=0.32(12.19-4.82) +4.82=7.2KN-m

Mxs2=12.19-0.68(12.19-4.82) =7.2KN-m

For panel P5&P6 on axis-3

M_{xs}=7.2KNm, M_{xs}=7.48KNm, Δm_x=0.28KN.m

20% 7.48KNm=1.496   since Δm_x<1.496, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (7.48+7.2)/2 = 7.34KNm

For panel P6&P7 on axis-4

M_{xs}=7.34KNm, M_{xs}=8.9KNm, Δm_x=1.56KN.m

20% 8.9KNm=1.78, since Δm_x<1.78 , the average of the moments is taken

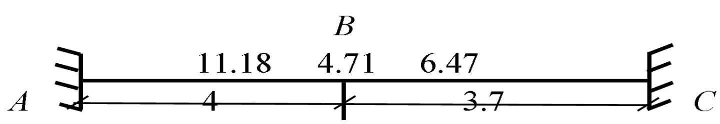
$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (7.34+78.9)/2 = 8.12KNm

For panel P2&P6 on axis-B

M_{ys}=11.18KNm, M_{ys}=6.47KNm, Δm_x=4.71KN.m

20% 11.18KNm=2.236   since Δm_x>2.236, Therefore we use moment distribution



Relative bending stiffness K

$K_{AB}=\frac{I}{L}=\frac{I}{4}=$ 0.25 I

$K_{BC}=\frac{I}{L}=\frac{I}{3.7}=$ 0.27 I

Distribution factor Df

$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=$ 0.480

$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=$ 0.520

Balanced support moments

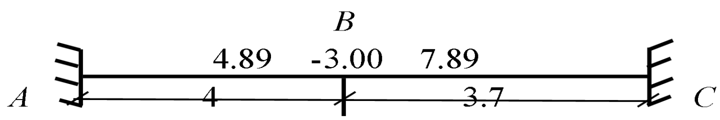
Mys1=0.48(11.18-6.47) +6.47=8.73KN-m

Mys2=11.18-0.52(11.18-6.47) = 8.73KN-m

For panel P3&P7 on axis-B

M_{ys}=4.89KNm, M_{ys}=7.89KNm, Δm_x=3KN.m

20% 7.89KNm=1.578 since Δm_x>1.578, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=0.25I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{3.7}=0.27I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.480$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.520$$

Balanced support moments

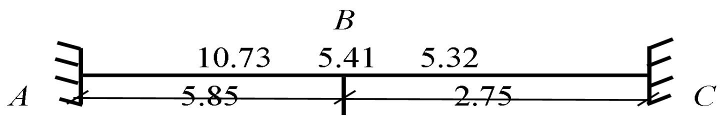
Mys1=0.48(7.89-4.89) +4.89=6.33KN-m

Mys2=7.89-0.52(7.89-4.89) =6.33KN-m

For panel P8&P9 on axis-2

M_{xs}=10.73KNm, M_{xs}=5.32KNm, Δm_x=5.41KN.m

20% 10.73KNm=2.15 since Δm_x>2.42, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{5.85}=0.17094\ I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{2.75}=0.36\ I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.320$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.680$$

Balanced support moments

Mxs1=0.32(10.73-5.32) +5.32=7.05KN-m

Mxs2=10.73-0.68(10.73-5.32) =7.05KN-m

For panel P9&P10 on axis-3

M_{xs}=7.05KNm, M_{xs}=8.03KNm, Δm_x=0.98KN.m

20% 8.03KNm=1.61 since Δm_x<1.61, The average moment is used

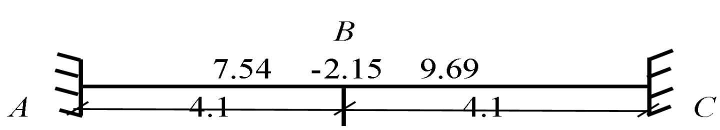
ΔM<20% use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (8.03+7.05)/2 = 7.54KNm

For panel P10&P11 on axis-4

M_{xs}=7.54KNm, M_{xs}=9.69KNm, Δm_x=2.15KN.m

20% 9.69KNm=1.938 since Δm_x>1.938, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB} = \frac{I}{L} = \frac{I}{4.1} = \underline{0.2439 \quad I}$$

$$K_{BC} = \frac{I}{L} = \frac{I}{4.1} = \underline{0.24 \quad I}$$

Distribution factor Df

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.500$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.500$$

Balanced support moments

Mys1=0.5(9.69-7.54) +7.54=8.62KN-m

Mys2=9.69-0.5(9.69-7.54) =8.62KN-m

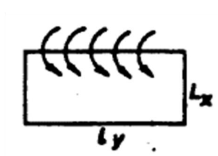
Field Moment adjustment

For Panel 1

M_{xf} = 10.65	M_{Xs} = 14.46	$M_{Xs,adj}$ = 14.46
M_{yf} = 7.35	M_{ys} = 9.89	$M_{ys,adj}$ = 8.91
Ly/Lx= 1.46		

ΔM_{xs} = Mxs- M_Xs,adj = 0.00 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.98



Cx = 0.31 Cx = 0.41

Cy = 0.1 Cy = 0.32

ΔMxf = CxΔMxs + CxΔMys = 0.402

Mxf ,adj = ΔMxf + Mxf = **11.052**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.314

Myf ,adj = ΔMyf + Myf = **7.664**

For Panel 2

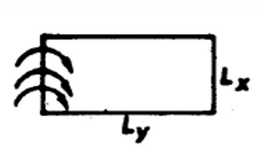
M_{xf} = 12.04 M_{Xs} = 9.18 $M_{Xs,adj}$ = 8.57

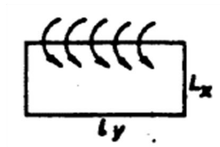
M_{yf} = 11.18 M_{ys} = 11.18 $M_{ys,adj}$ = 8.73

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.61

ΔM_{ys} = Mys- M_ys,adj = 2.45





Cx = 0.373   Cx = 0.29

Cy = 0.262   Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 0.938

Mxf ,adj = ΔMxf + Mxf = **12.978**

ΔM_{yf} = CyΔMxs + CyΔMys = 1.086

Myf ,adj = ΔMyf + Myf = **12.266**

For panel 3 no need of adjustment

For Panel 4

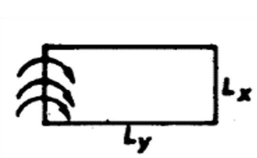
M_{xf} = 8.94 M_{Xs} = 12.19 $M_{Xs,adj}$ = 7.2

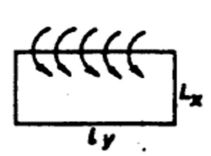
M_{yf} = 5.89 M_{ys} = 7.92 $M_{ys,adj}$ = 8.91

Ly/Lx= 1.58

ΔM_{xs} = Mxs- M_Xs,adj = 4.99

ΔM_{ys} = Mys- M_ys,adj = 0.00





Cx = 0.297   Cx = 0.416

Cy = 0.0852   Cy = 0.293

ΔMxf = CxΔMxs + CxΔMys = 1.482

Mxf ,adj = ΔMxf + Mxf = **10.422**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.425

Myf ,adj = ΔMyf + Myf = **6.315**

For panel 5 and panel 6 no need of adjustment

For Panel 7

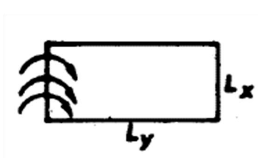
M_{xf} = 6.67 M_{Xs} = 8.9 $M_{Xs,adj}$ = 8.12

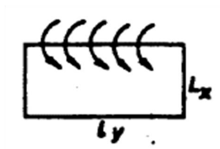
M_{yf} = 5.86 M_{ys} = 7.89 $M_{ys,adj}$ = 6.33

Ly/Lx= 1.10

ΔM_{xs} = Mxs- M_Xs,adj = 0.78

ΔM_{ys} = Mys- M_ys,adj = 1.56 neglect





Cx = 0.356 Cx = 0.314

Cy = 0.22 Cy = 0.374

ΔMxf = CxΔMxs + CxΔMys = 0.768

Mxf ,adj = ΔMxf + Mxf = **7.438**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.755

Myf ,adj = ΔMyf + Myf = **6.615**

For panel 9 and panel 10 no need of adjustment

For Panel 11

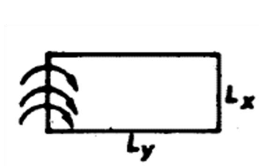
M_{xf} = 7.09 M_{Xs} = 9.69 $M_{Xs,adj}$ = 8.62

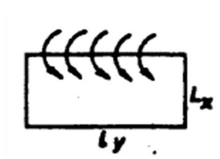
M_{yf} = 6.85 M_{ys} = 9.22 $M_{ys,adj}$ = 9.22

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 1.07

ΔM_{ys} = Mys- M_ys,adj = 0.00





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 0.399

Mxf ,adj = ΔMxf + Mxf = **7.489**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.280

Myf ,adj = ΔMyf + Myf = **7.130**

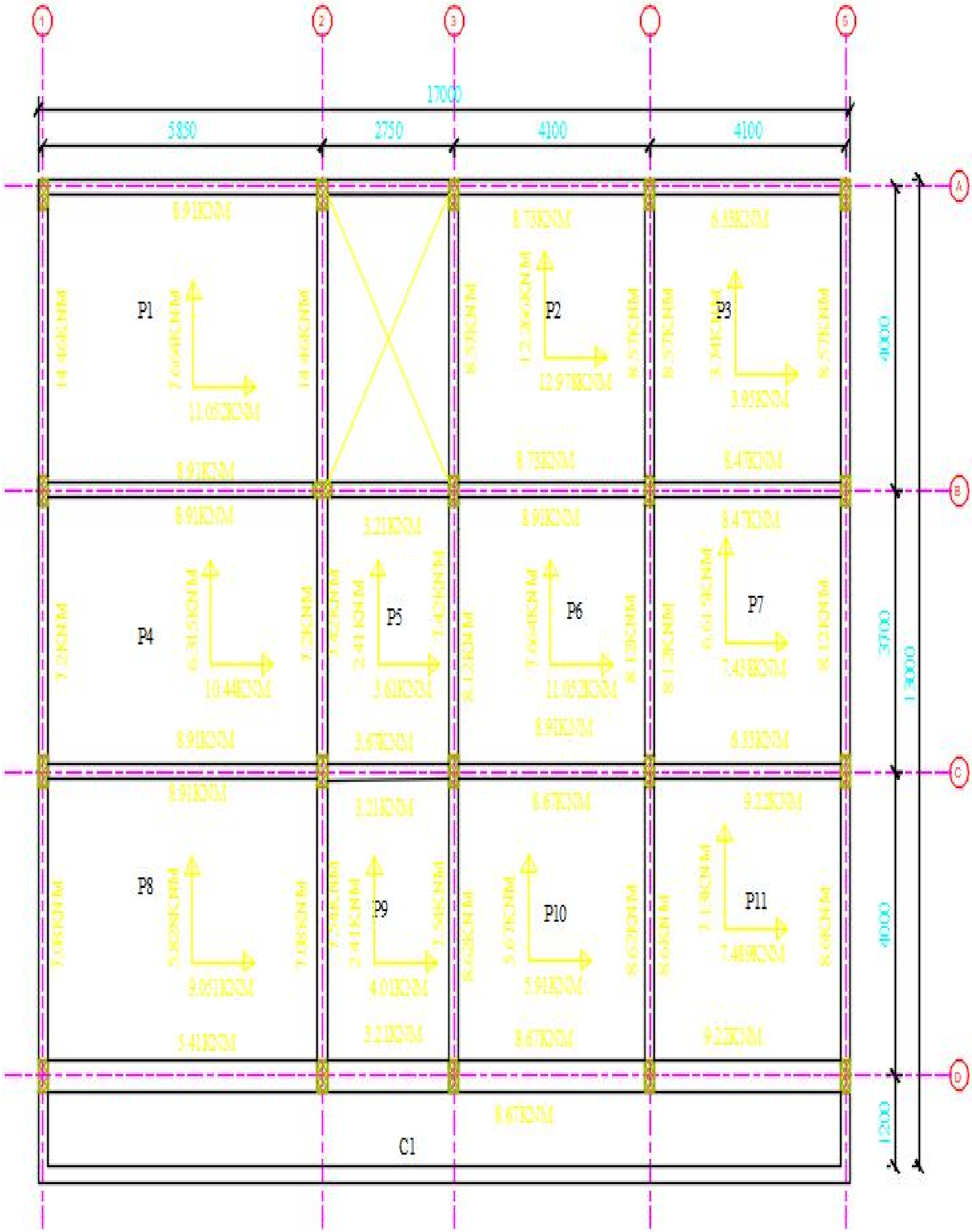


Figure 10- 8Ground floor slab adjusted moment

Appendix D6: Reinforcement calculation

Table 31Reinforcement calculation

panel	moment	Ast=Msd/Z*fyd		ø	spacing	spacing used	Reinforcement			
P-1	M_{xs} =	14.46	329.94	10	237.92	220	ø	10	c/c	220
	M_{xf} =	11.052	252.18	10	311.29	300	ø	10	c/c	300
	M_{ys} =	8.91	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.664	250.00	10	314.00	300	ø	10	c/c	300
P-2	M_{xs} =	8.57	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	12.978	296.12	10	265.09	240	ø	10	c/c	240
	M_{ys} =	8.73	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	12.266	279.88	10	280.48	300	ø	10	c/c	300
P-3	M_{xs} =	8.57	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	3.95	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	6.33	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	3.74	250.00	10	314.00	300	ø	10	c/c	300
P-4	M_{xs} =	7.2	250.00	10	314.00	250	ø	10	c/c	250
	M_{xf} =	10.422	250.00	10	314.00	280	ø	10	c/c	280
	M_{ys} =	8.91	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	6.315	250.00	10	314.00	300	ø	10	c/c	300
P-5	M_{xs} =	7.34	250.00	10	314.00	260	ø	10	c/c	260
	M_{xf} =	3.61	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	3.21	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	2.41	250.00	10	314.00	300	ø	10	c/c	300
P-6	M_{xs} =	8.12	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	11.052	252.18	10	311.29	300	ø	10	c/c	300
	M_{ys} =	8.91	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.664	250.00	10	314.00	300	ø	10	c/c	300
P-7	M_{xs} =	8.12	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	7.438	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	6.33	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	6.615	250.00	10	314.00	300	ø	10	c/c	300
P-8	M_{xs} =	7.05	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	9.051	250.00	10	314.00	220	ø	10	c/c	220
	M_{ys} =	5.46	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	5.828	250.00	10	314.00	300	ø	10	c/c	300
P-9	M_{xs} =	7.54	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	4.01	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	3.21	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	2.41	250.00	10	314.00	300	ø	10	c/c	300
P-10	M_{xs} =	8.62	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	5.91	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	8.67	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	5.67	250.00	10	314.00	300	ø	10	c/c	300
P-11	M_{xs} =	8.6	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	7.489	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	9.22	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.13	250.00	10	314.00	300	ø	10	c/c	300

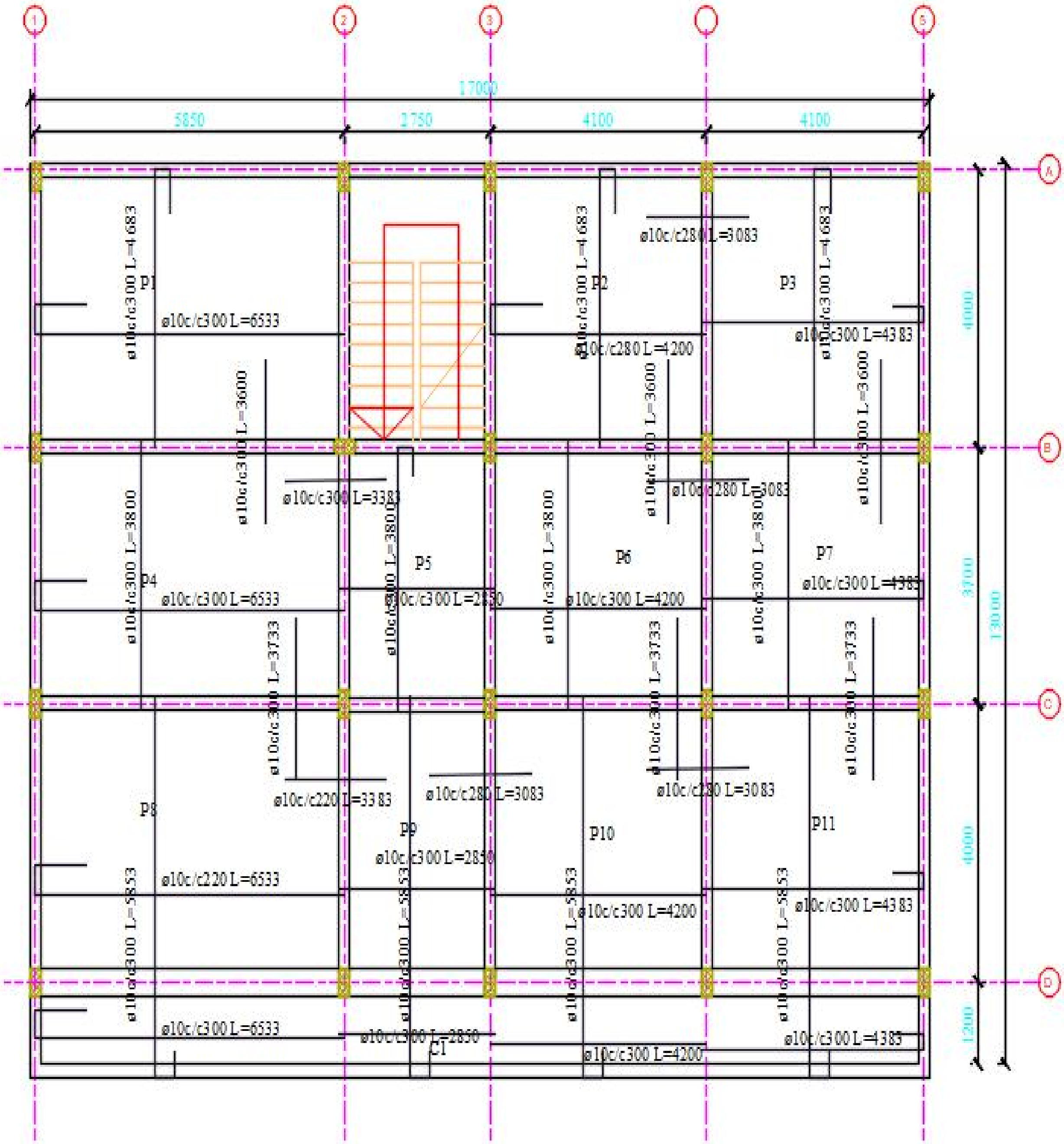


Figure 10- 9ground floor slab reinforcement detailing

Appendix E: roof slab design

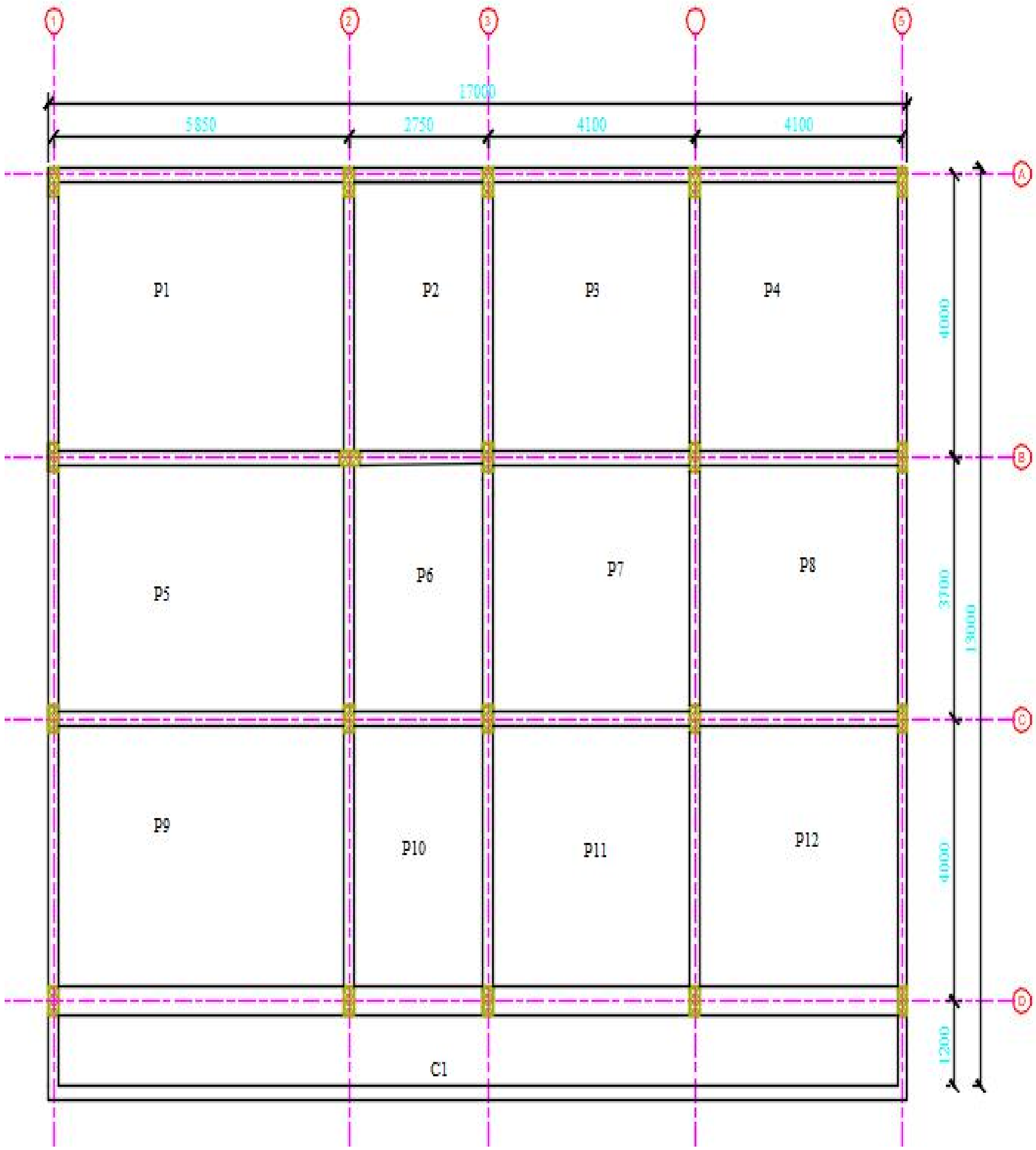


Fig 6.7 roof floor slab layout

Appendix E1: Depth determination

We use C30/37 concrete and S400 rebar

Φbar= 10
Cnom= 20
ϒs = 1.15
ϒc = 1.5
fck = 20
fyk= 400
F1= 1.25
F2= 1
F3= 1
d'= 25

fcd= $\frac{0.85*fck}{\Upsilon c}$=11.33

fyd= $\frac{fyk}{\Upsilon s}$= 347.83

unit weight 25

panel	Ly	Lx	Ly/Lx	Type	support	N	K	Lx/d	d	D	D provided
p1	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	200
P2	4000	2750	1.45	two way	End	17.71	1.3	28.78	95.6	120.57	
P3	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
P4	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
P5	5850	3700	1.58	two way	End	17.71	1.3	28.78	128.6	153.58	
P6	3700	2750	1.35	two way	Interior	17.71	1.5	33.20	82.8	107.82	
P7	4100	3700	1.11	two way	Interior	17.71	1.5	33.20	111.4	136.44	
P8	4100	3700	1.11	two way	End	17.71	1.3	28.78	128.6	153.58	
P9	5850	4000	1.46	two way	End	17.71	1.3	28.78	139.0	164.01	
P10	4000	2750	1.45	two way	Interior	17.71	1.5	33.20	82.8	107.82	
P11	4100	4000	1.03	two way	Interior	17.71	1.5	33.20	120.5	145.47	
P12	4100	4000	1.03	two way	End	17.71	1.3	28.78	139.0	164.01	
p12	17000	1200	14.17	cantilever	End	17.71	0.4	8.85	135.5	160.53	

Pd = 14.77 KN/m^2

Appendix E2: Moment analysis of roof slab

Table 32Moment analysis of roof slab

panel	support conditio	ly/lx	lx	lx^2	Pd	moment coeficient		moment	moment
						location	value	location	$Mi=\alpha_i P_d$
P-1	Type 2	1.46	4	16.00	14.77	$\beta_{sx,sup}$=	0.057	M_{xs} =	13.47
						$\beta_{sx,span}$=	0.042	M_{xf} =	9.93
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.22
						$\beta_{sy,span}$=	0.029	M_{yf} =	6.85
P-2	Type 2	1.50	2.75	7.56	14.77	$\beta_{sx,sup}$=	0.058	M_{xs} =	6.48
						$\beta_{sx,span}$=	0.043	M_{xf} =	4.80
						$\beta_{sy,sup}$=	0.039	M_{ys} =	4.36
						$\beta_{sy,span}$=	0.03	M_{yf} =	3.35
P-3	Type 3	1.03	4.00	16.00	14.77	$\beta_{sx,sup}$=	0.042	M_{xs} =	9.93
						$\beta_{sx,span}$=	0.032	M_{xf} =	7.56
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.22
						$\beta_{sy,span}$=	0.03	M_{yf} =	7.09
P-4	Type 4	1.03	4	16	14.77	$\beta_{sx,sup}$=	0.049	M_{xs} =	11.58
						$\beta_{sx,span}$=	0.038	M_{xf} =	8.98
						$\beta_{sy,sup}$=	0.047	M_{ys} =	11.11
						$\beta_{sy,span}$=	0.036	M_{yf} =	8.51
P-5	Type 2	1.58	3.7	13.69	14.77	$\beta_{sx,sup}$=	0.060	M_{xs} =	12.13
						$\beta_{sx,span}$=	0.044	M_{xf} =	8.90
						$\beta_{sy,sup}$=	0.039	M_{ys} =	7.89
						$\beta_{sy,span}$=	0.029	M_{yf} =	5.86
P-6	Type 1	1.35	2.75	7.56	14.77	$\beta_{sx,sup}$=	0.048	M_{xs} =	5.36
						$\beta_{sx,span}$=	0.036	M_{xf} =	4.02
						$\beta_{sy,sup}$=	0.032	M_{ys} =	3.57
						$\beta_{sy,span}$=	0.024	M_{yf} =	2.68
P-7	Type 1	1.10	3.7	13.69	14.77	$\beta_{sx,sup}$=	0.037	M_{xs} =	7.48
						$\beta_{sx,span}$=	0.028	M_{xf} =	5.66
						$\beta_{sy,sup}$=	0.032	M_{ys} =	6.47
						$\beta_{sy,span}$=	0.024	M_{yf} =	4.85
P-8	Type 2	1.10	3.7	13.69	14.77	$\beta_{sx,sup}$=	0.044	M_{xs} =	8.90
						$\beta_{sx,span}$=	0.033	M_{xf} =	6.67
						$\beta_{sy,sup}$=	0.039	M_{ys} =	7.89
						$\beta_{sy,span}$=	0.029	M_{yf} =	5.86
P-9	Type 2	1.46	4	16.00	14.77	$\beta_{sx,sup}$=	0.057	M_{xs} =	13.47
						$\beta_{sx,span}$=	0.042	M_{xf} =	9.93
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.22
						$\beta_{sy,span}$=	0.029	M_{yf} =	6.85
P-10	Type 1	1.50	2.75	7.56	14.77	$\beta_{sx,sup}$=	0.053	M_{xs} =	5.92
						$\beta_{sx,span}$=	0.04	M_{xf} =	4.47
						$\beta_{sy,sup}$=	0.032	M_{ys} =	3.57
						$\beta_{sy,span}$=	0.024	M_{yf} =	2.68
P-11	Type 1	1.03	4	16.00	14.77	$\beta_{sx,sup}$=	0.034	M_{xs} =	8.03
						$\beta_{sx,span}$=	0.025	M_{xf} =	5.91
						$\beta_{sy,sup}$=	0.032	M_{ys} =	7.56
						$\beta_{sy,span}$=	0.024	M_{yf} =	5.67
P-12	Type 2	1.03	4	16.00	14.77	$\beta_{sx,sup}$=	0.041	M_{xs} =	9.69
						$\beta_{sx,span}$=	0.03	M_{xf} =	7.09
						$\beta_{sy,sup}$=	0.039	M_{ys} =	9.22
						$\beta_{sy,span}$=	0.029	M_{yf} =	6.85

Appendix E3 moment adjustment roof slab

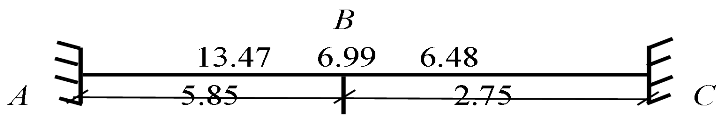
Support adjustment

For panel P1&P2 on axis-2

M_{xs}=13.47KNm, M_{xs}=6.48KNm

Δm_x=6.99KN.m

20% 13.47KNm=2.694 since Δm_x>2.694, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB} = \frac{I}{L} = \frac{I}{5.85} = 0.17094\ I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{2.75} = 0.36\ I$$

Distribution factor Df

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.320$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.680$$

Balanced support moments

Mxs1=0.32(13.47-6.48) +6.48=8.72KN-m

Mxs2=13.47-0.64(13.47-6.48) =8.72KN-m

For panel P2&P3 on axis-3

M_{xs}=8.72KNm, M_{xs}=9.93KNm, Δm_x=1.21KN.m

20% 9.93KNm=1.986 since Δm_x<1.986, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (8.72+9.93)/2 = 9.33KNm

For panel P3&P4 on axis-4

M_{xs}=9.33KNm, M_{xs}=11.58KNm, Δm_x=2.25KN.m

20% 11.58KNm=2.316 since Δm_x<2.316, the average of the moments is taken

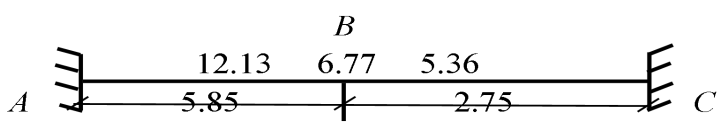
$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.33+11.58)/2 = 10.46KNm

For panel P5&P6 on axis-2

M_{xs}=12.13KNm, M_{xs}=5.36KNm, Δm_x=6.77KN.m

20% 12.13KNm=2.426 since Δm_x>2.426, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{5.85}=0.17094\ I$$

$$K_{BC}=\frac{I}{L}=\frac{I}{2.75}=0.36\ I$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.320$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=0.680$$

Balanced support moments

Mxs1=0.32(12.13-5.36) +5.36=7.5264KN-m

Mxs2=12.13-0.68(12.13-5.36) =7.5264KN-m

For panel P6&P7 on axis-3

M_{xs}=7.53KNm, M_{xs}=7.48KNm, Δm_x=0.05KN.m

20% 7.53KNm=1.506 since Δm_x<1.506, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (7.48+7.53)/2 = 7.51KNm

For panel P7&P8 on axis-4

M_{xs}=7.51KNm, M_{xs}=8.9KNm, Δm_x=1.39KN.m

20% 8.9KNm=1.78, since Δm_x<1.78, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (7.51+8.9)/2 = 8.21KNm

For panel P1&P5 on axis-B

M_{ys}=9.22KNm, M_{ys}=7.89KNm, Δm_x=1.33KN.m

20% 9.22KNm=1.844 since Δm_x<1.844, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (9.22+7.89)/2 = 8.56KNm

For panel P2&P6 on axis-B

M_{ys}=4.36KNm, M_{ys}=3.57KNm, Δm_x=0.79KN.m

20% 4.36KNm=0.872 since Δm_x<0.872, the average of the moments is taken

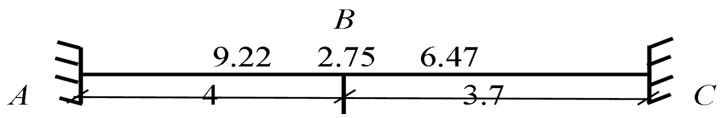
$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (4.36+3.57)/2 = 3.97KNm

For panel P3&P7 on axis-2

M_{xs}=9.22KNm, M_{xs}=6.47KNm, Δm_x=2.75KN.m

20% 9.22KNm=1.844 since Δm_x>1.844, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB}=\frac{I}{L}=\frac{I}{4}=\underline{0.25\quad I}$$

$$K_{BC}=\frac{I}{L}=\frac{I}{3.7}=\underline{0.27\quad I}$$

Distribution factor Df

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.770$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.230$$

Balanced support moments

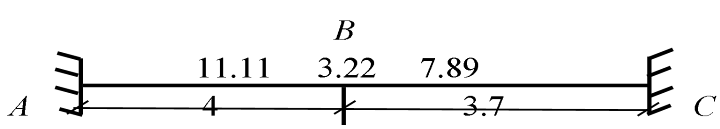
Mys1=0.77(9.22-6.47) +6.47=8.588KN-m

Mys2=9.22-0.23(9.22-6.47) =8.588KN-m

For panel P4&P8 on axis-3

M_{ys}=11.11KNm, M_{ys}=7.89KNm, Δm_x=3.22KN.m

20% KNm=2.22 since Δm_x>2.22, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB} = \frac{I}{L} = \frac{I}{4} = 0.25\ I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{3.7} = 0.27\ I$$

Distribution factor Df

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.770$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.230$$

Balanced support moments

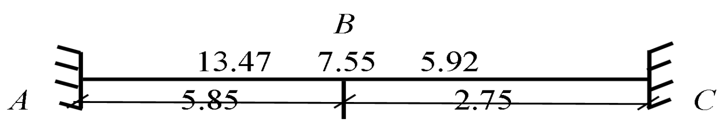
Mys1=0.77(11.11-7.89) +7.89=10.37KN-m

Mys2=11.11-0.23(11.11-7.89) =10.37KN-m

For panel P9&P10 on axis-2

M_{xs}=13.47KNm, M_{xs}=5.92KNm, Δm_x=7.55KN.m

20% 13.47KNm=2.694 since Δm_x>2.694, Therefore we use moment distribution



Relative bending stiffness K

$$K_{AB} = \frac{I}{L} = \frac{I}{5.85} = 0.17094\ I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{2.75} = 0.36\ I$$

Distribution factor Df

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.320$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.640$$

Balanced support moments

$Mxs1=0.32(13.47-5.92) +5.92=8.34KN-m$

$Mxs2=13.47-0.64(13.47-5.92) =8.34KN-m$

For panel P10&P11 on axis-3

M_{xs}=8.34KNm, M_{xs}=8.03KNm, Δm_x=0.31KN.m

20% 8.34KNm=1.668 since $\Delta m_x<1.668$, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (8.34+8.03)/2 = 8.2KNm

For panel P11&P12 on axis-4

M_{xs}=8.2KNm, M_{xs}=9.69KNm, Δm_x=1.49KN.m

20% 9.69KNm=1.938 since $\Delta m_x<1.938$, the average of the moments is taken

$\Delta M<20\%$ use $M=\left(\frac{M\text{large}+M\text{small}}{2}\right)$

M= (8.2+9.69)/2 = 8.95KNm

Field moment Adjustment

For Panel 1

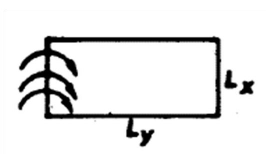
M_{xf} = 9.93 M_{Xs} = 13.47 $M_{Xs,adj}$ = 8.72

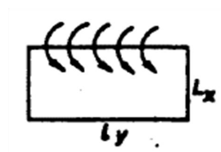
M_{yf} = 6.85 M_{ys} = 9.22 $M_{ys,adj}$ = 8.56

Ly/Lx= 1.46

ΔM_{xs} = Mxs- M_Xs,adj = 4.75

ΔM_{ys} = Mys- M_ys,adj = 0.66





Cx = 0.31 Cx = 0.41

Cy = 0.1 Cy = 0.32

ΔMxf = CxΔMxs + CxΔMys = 1.743

Mxf,adj = ΔMxf + Mxf= **11.673**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.686

Myf,adj = ΔMyf + Myf= **7.536**

For Panel 2

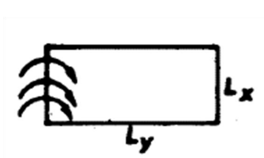
M_{xf} = 4.8 M_{Xs} = 6.48 $M_{Xs,adj}$ = 9.33

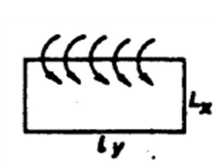
M_{yf} = 3.35 M_{ys} = 4.36 $M_{ys,adj}$ = 3.97

Ly/Lx= 1.50

ΔM_{xs} = Mxs- M_Xs,adj = 0.00 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.39





Cx = 0.305 Cx = 0.421

Cy = 0.094 Cy = 0.31

ΔMxf = CxΔMxs + CxΔMys = 0.119

Mxf,adj = ΔMxf + Mxf= **4.919**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.037

Myf,adj = ΔMyf + Myf= **3.387**

For Panel 3

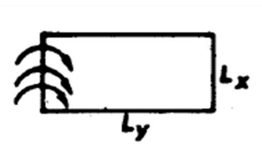
M_{xf} = 7.56 M_{Xs} = 9.93 $M_{Xs,adj}$ = 10.46

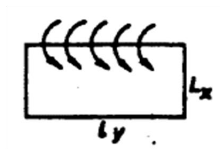
M_{yf} = 7.09 M_{ys} = 9.22 $M_{ys,adj}$ = 8.6

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.00 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.62





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 0.180

Mxf ,adj = ΔMxf + Mxf = **7.740**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.234

Myf ,adj = ΔMyf + Myf = **7.324**

For Panel 4

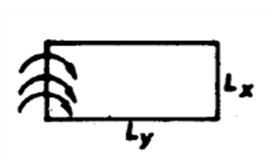
M_{xf} = 8.98 M_{Xs} = 11.58 $M_{Xs,adj}$ = 10.46

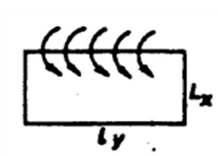
M_{yf} = 8.51 M_{ys} = 11.11 $M_{ys,adj}$ = 10.37

Ly/Lx= 1.27

ΔM_{xs} = Mxs- M_Xs,adj = 1.12

ΔM_{ys} = Mys- M_ys,adj = 0.74





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf = CxΔMxs + CxΔMys = 0.651

Mxf ,adj = ΔMxf + Mxf = **9.631**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.505

Myf ,adj = ΔMyf + Myf = **9.015**

For Panel 5

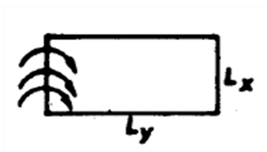
$M_{xf} = 8.9$ $M_{Xs} = 12.13$ $M_{Xs,adj} = 7.53$

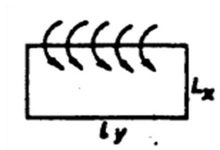
$M_{yf} = 5.86$ $M_{ys} = 7.89$ $M_{ys,adj} = 8.56$

Ly/Lx= 1.46

ΔM_{xs} = Mxs- M_Xs,adj = 4.60

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.31 Cx = 0.41

Cy = 0.1 Cy = 0.32

ΔMxf = CxΔMxs + CxΔMys = 1.886

Mxf ,adj = ΔMxf + Mxf = **10.786**

ΔM_{yf} = CyΔMxs + CyΔMys = 1.472

Myf ,adj = ΔMyf + Myf = **7.332**

For panel 6 and panel 7 no need of adjustment

For Panel 8

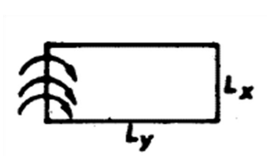
$M_{xf} = 6.67$ $M_{Xs} = 8.9$ $M_{Xs,adj} = 8.21$

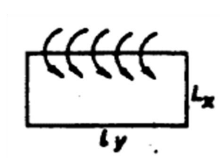
$M_{yf} = 5.86$ $M_{ys} = 7.89$ $M_{ys,adj} = 10.37$

Ly/Lx= 1.10

ΔM_{xs} = Mxs- M_Xs,adj = 0.69

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.356 Cx = 0.314

Cy = 0.22 Cy = 0.374

ΔMxf = CxΔMxs + CxΔMys = 0.246

Mxf ,adj = ΔMxf + Mxf = **6.916**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.152

Myf ,adj = ΔMyf + Myf = **6.012**

For Panel 9

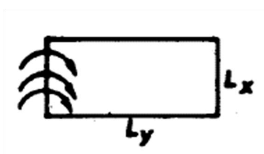
M_{xf} = 9.93 M_{Xs} = 13.47 $M_{Xs,adj}$ = 8.34

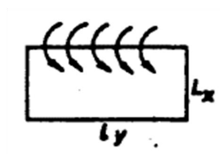
M_{yf} = 6.85 M_{ys} = 9.22 $M_{ys,adj}$ = 9.22

Ly/Lx= 1.46

ΔM_{xs} = Mxs- M_Xs,adj = 5.13

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.31 Cx = 0.41

Cy = 0.1 Cy = 0.32

ΔMxf = CxΔMxs + CxΔMys = 2.103

Mxf ,adj = ΔMxf + Mxf = **12.033**

ΔM_{yf} = CyΔMxs + CyΔMys = 1.642

Myf ,adj = ΔMyf + Myf = **8.492**

For panel 10 and panel 11 no need of adjustment

For Panel 12

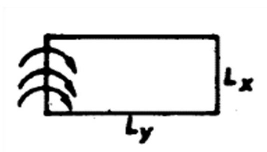
M_{xf} = 7.09 M_{Xs} = 9.69 $M_{Xs,adj}$ = 8.95

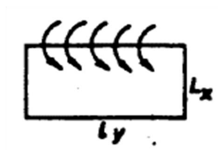
M_{yf} = 6.85 M_{ys} = 9.22 $M_{ys,adj}$ = 9.22

Ly/Lx= 1.03

ΔM_{xs} = Mxs- M_Xs,adj = 0.74

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.373 Cx = 0.29

Cy = 0.262 Cy = 0.378

ΔMxf = CxΔMxs + CxΔMys = 0.276

Mxf ,adj = ΔMxf + Mxf = **7.366**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.194

Myf ,adj = ΔMyf + Myf = **7.044**

Appendix E4: Load transfer to top tie beam

Table 33Load transfer to top tie beam

panel	support condition	ly/lx	lx	pd	shear force coeficient Location value		shear force location	Shear force $V_i=\beta_{Vi}P_d$ Lx
P-1	Type 4	1.46	4.00	14.77	$\beta_{vxc}=$	0.46	$V_{xc}=$	27.177
					$\beta_{vxd}=$	0	$V_{xd}=$	0.000
					$\beta_{vyc}=$	0.36	$V_{yc}=$	21.269
					$\beta_{vyd}=$	0.24	$V_{yd}=$	14.179
P-2	Type 2	1.50	2.75	14.77	$\beta_{vxc}=$	0.47	$V_{xc}=$	19.090
					$\beta_{vyc}=$	0.36	$V_{yc}=$	14.622
					$\beta_{vyd}=$	0.24	$V_{yd}=$	9.748
P-3	Type 3	1.03	4.00	14.77	$\beta_{vxc}=$	0.372	$V_{xc}=$	21.978
					$\beta_{vxd}=$	0.249	$V_{xd}=$	15
					$\beta_{vyc}=$	0.36	$V_{yc}=$	21.269
P-4	Type 4	1.03	4.00	14.77	$\beta_{vxc}=$	0.412	$V_{xc}=$	24.341
					$\beta_{vxd}=$	0.269	$V_{xd}=$	16
					$\beta_{vyc}=$	0.4	$V_{yc}=$	23.632
					$\beta_{vyd}=$	0.26	$V_{yd}=$	15.361
P-5	Type 2	1.58	3.70	14.77	$\beta_{vxc}=$	0.48	$V_{xc}=$	26.232
					$\beta_{vyc}=$	0.36	$V_{yc}=$	19.674
					$\beta_{vyd}=$	0.24	$V_{yd}=$	13.116
P-6	Type 1	1.35	2.75	14.77	$\beta_{vxc}=$	0.42	$V_{xc}=$	17.059
					$\beta_{vyc}=$	0.33	$V_{yc}=$	13.404
P-7	Type 1	1.10	3.70	14.77	$\beta_{vxc}=$	0.43	$V_{xc}=$	23.499
					$\beta_{vyc}=$	0.4	$V_{yc}=$	21.860
P-8	Type 2	1.10	3.70	14.77	$\beta_{vxc}=$	0.38	$V_{xc}=$	20.767
					$\beta_{vyc}=$	0.36	$V_{yc}=$	19.674
					$\beta_{vyd}=$	0.24	$V_{yd}=$	13
P-9	Type 2	1.46	4.00	14.77	$\beta_{vxc}=$	0.49	$V_{xc}=$	28.949
					$\beta_{vxd}=$	0.32	$V_{xd}=$	18.906
					$\beta_{vyc}=$	0.4	$V_{yc}=$	23.632
					$\beta_{vyd}=$	0.24	$V_{yd}=$	14.179
P-10	Type 1	1.45	2.75	14.77	$\beta_{vxc}=$	0.49	$V_{xc}=$	19.903
					$\beta_{vyc}=$	0.4	$V_{yc}=$	16.247
P-11	Type 1	1.03	4.00	14.77	$\beta_{vxc}=$	0.49	$V_{xc}=$	28.949
					$\beta_{vyc}=$	0.4	$V_{yc}=$	23.632
P-12	Type 2	1.03	4.00	14.77	$\beta_{vxc}=$	0.49	$V_{xc}=$	28.949
					$\beta_{vxd}=$	0.32	$V_{xd}=$	18.906
					$\beta_{vyc}=$	0.4	$V_{yc}=$	23.632
					$\beta_{vyd}=$	0.24	$V_{yd}=$	14.179

Appendix E5 shear capacity check

$$v_{Rd,c} = \left[0.12K(100\rho_1 f_{CK})^{\frac{1}{3}}\right] \mathrm{bd} \geq V_{min}$$

$$K = 1 + \left(\frac{200}{d}\right)^{0.5} \leq 2$$

d= 140

k= 2.20 not < 2,so k=2

$$\rho_1 = \frac{A_{sl}}{bd} \leq 0.02$$

taking minimum reinforcment ø 10 c/c 300

$\rho_1 =$ 0.0019

$$V_{min} = [0.035K^{\frac{3}{2}} f_{ck}{}^{\frac{1}{2}}]bd$$

$V_{min} =$ 61.981

$v_{Rd,c} =$ 41.867 $\leq V_{min}$ so, $v_{Rd,c} =$ 61.981

maximum shear force transfererd to beams is Vi= Vi=20.45

Vi= 20.45 KN/m

61.981>20.45,slab thickness is sufficient to support design load

Table 34roof slab reinforcement

panel	moment	Ast=Msd/Z*fyd		ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	8.72	250.00	10	314.00	220	ø	10	c/c	220
	M_{xf} =	11.673	266.35	10	294.73	300	ø	10	c/c	300
	M_{ys} =	8.56	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.536	250.00	10	314.00	300	ø	10	c/c	300
P-2	M_{xs} =	9.33	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	4.92	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	3.97	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	3.4	250.00	10	314.00	300	ø	10	c/c	300
P-3	M_{xs} =	10.46	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	7.74	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	8.6	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.324	250.00	10	314.00	300	ø	10	c/c	300
P-4	M_{xs} =	10.46	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	9.631	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	10.37	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	9.015	250.00	10	314.00	300	ø	10	c/c	300
P-5	M_{xs} =	7.53	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	10.786	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	8.56	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.332	250.00	10	314.00	300	ø	10	c/c	300
P-6	M_{xs} =	7.51	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	4.02	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	3.97	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	2.68	250.00	10	314.00	300	ø	10	c/c	300
P-7	M_{xs} =	8.21	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	5.66	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	8.6	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	4.85	250.00	10	314.00	300	ø	10	c/c	300
P-8	M_{xs} =	8.21	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	6.916	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	10.37	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	6.012	250.00	10	314.00	300	ø	10	c/c	300
P-9	M_{xs} =	8.34	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	12.033	274.56	10	285.91	280	ø	10	c/c	280
	M_{ys} =	9.22	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	2.41	250.00	10	314.00	300	ø	10	c/c	300
P-10	M_{xs} =	8.34	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	4.47	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	3.57	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	2.68	250.00	10	314.00	300	ø	10	c/c	300
P-11	M_{xs} =	8.95	250.00	10	314.00	300	ø	10	c/c	300
	M_{xf} =	5.91	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	7.56	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	5.67	250.00	10	314.00	300	ø	10	c/c	300

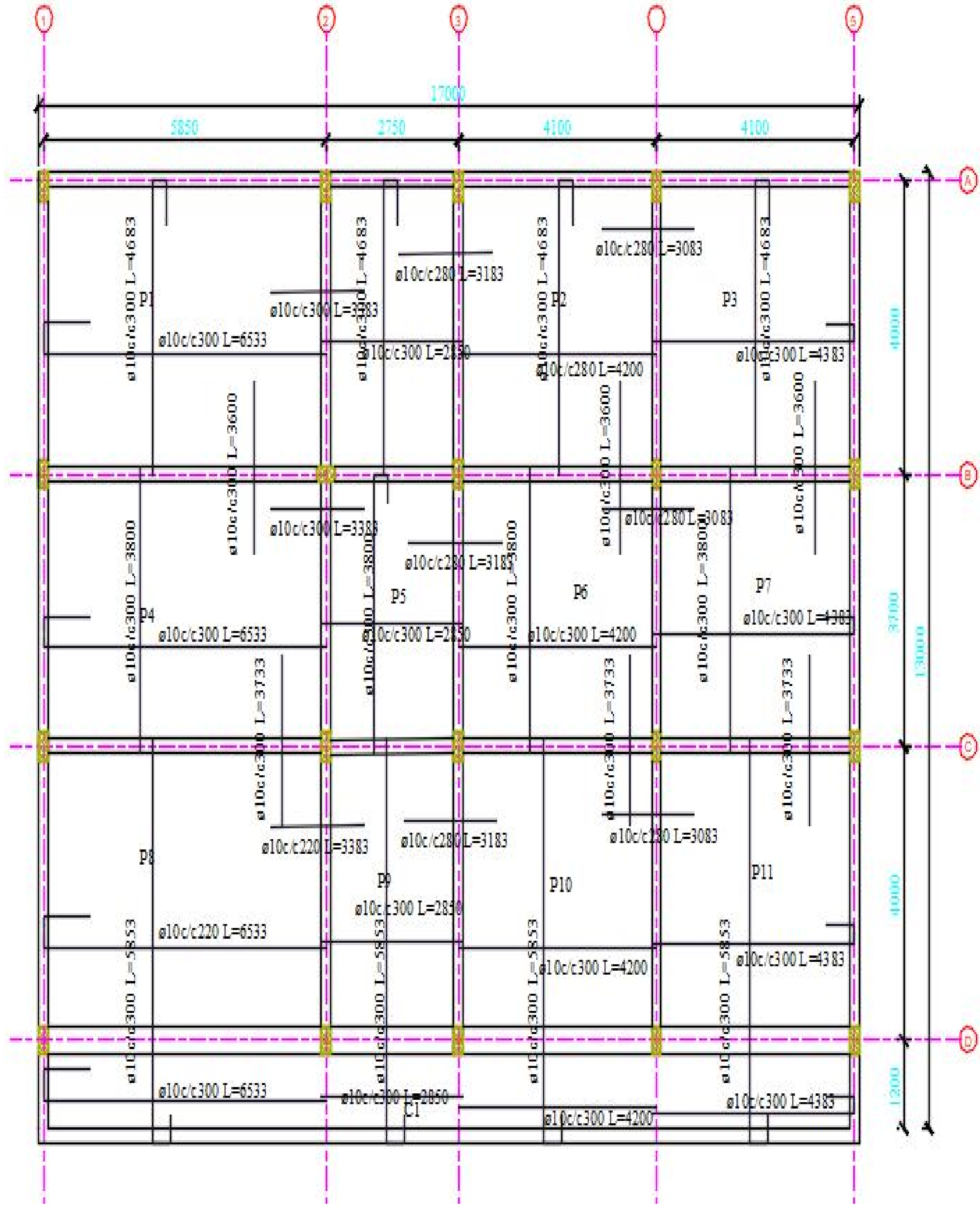


Figure 10- 10roof slab reinforcement detailing

Appendix F Beam design

Appendix F beam longitudinal reinforcement

Table 35beam longitudinal reinforcement

story- 3 Axis A

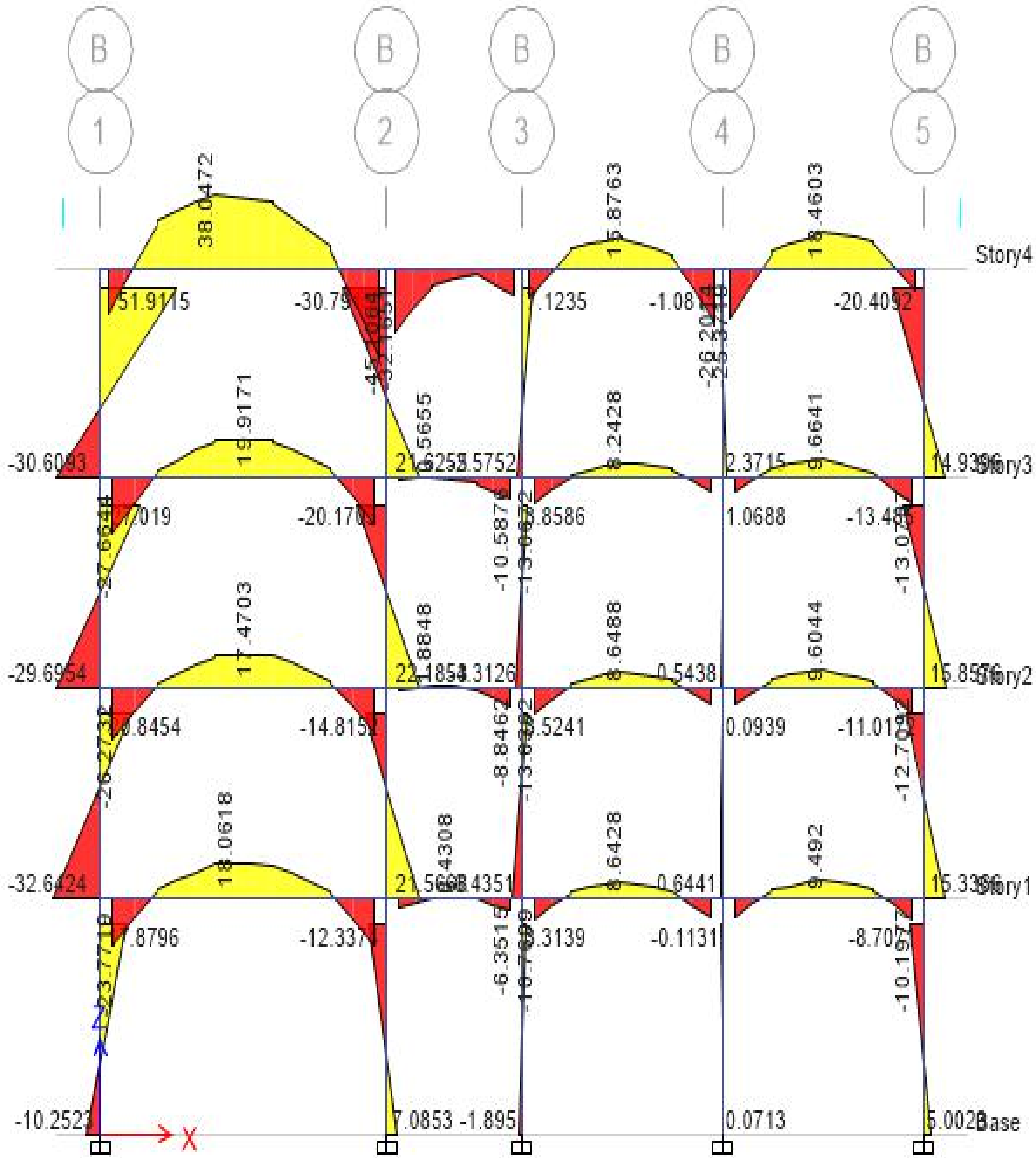
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	28.1	400	400	0.0220	392.0943	single	228.8	6400	206.041	228.8	0.73	2Ø20
span	1-2	18.34	400	400	0.0143	394.8765	single	228.8	6400	133.5291	228.8	0.73	2Ø20
support	2	19.71	400	400	0.0154	394.4884	single	228.8	6400	143.6449	228.8	0.73	2Ø20
span	2-3	11.24	400	400	0.0088	396.8758	single	228.8	6400	81.42345	228.8	0.73	3Ø20
support	3	13.94	400	400	0.0109	396.1179	single	228.8	6400	101.1757	228.8	0.73	2Ø20
span	3-4	8.668	400	400	0.0068	397.5951	single	228.8	6400	62.67809	228.8	0.73	2Ø20
support	4	2.49	400	400	0.0019	399.3121	single	228.8	6400	17.92771	228.8	0.73	2Ø20
span	4-5	10.38	400	400	0.0081	397.1166	single	228.8	6400	75.14795	228.8	0.73	2Ø20
support	5	14.21	400	400	0.0111	396.042	single	228.8	6400	103.1551	228.8	0.73	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	28.02	400	400	0.0219	392.1173	single	228.8	6400	205.4424	228.8	0.73	2Ø20
span	1-2	17.84	400	400	0.0139	395.018	single	228.8	6400	129.8422	228.8	0.73	2Ø20
support	2	20.55	400	400	0.0161	394.25	single	228.8	6400	149.8573	228.8	0.73	2Ø20
span	2-3	1.68	400	400	0.0013	399.5361	single	228.8	6400	12.08902	228.8	0.73	2Ø20
support	3	8.729	400	400	0.0068	397.578	single	228.8	6400	63.12188	228.8	0.73	2Ø20
span	3-4	8.678	400	400	0.0068	397.5923	single	228.8	6400	62.75084	228.8	0.73	2Ø20
support	4	0.8725	400	400	0.0007	399.7592	single	228.8	6400	6.274871	228.8	0.73	2Ø20
span	4-5	9.834	400	400	0.0077	397.2693	single	228.8	6400	71.16771	228.8	0.73	2Ø20
support	5	13.49	400	400	0.0105	396.2445	single	228.8	6400	97.87834	228.8	0.73	2Ø20

story- 1

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	22.47	400	400	0.0176	393.7041	single	228.8	6400	164.0858	228.8	0.73	2Ø20
span	1-2	18.21	400	400	0.0142	394.9133	single	228.8	6400	132.5702	228.8	0.73	2Ø20
support	2	21.36	400	400	0.0167	394.0199	single	228.8	6400	155.8551	228.8	0.73	2Ø20
span	2-3	0.69	400	400	0.0005	399.8096	single	228.8	6400	4.961737	228.8	0.73	3Ø20
support	3	11.24	400	400	0.0088	396.8758	single	228.8	6400	81.42345	228.8	0.73	2Ø20
span	3-4	8.75	400	400	0.0068	397.5722	single	228.8	6400	63.27467	228.8	0.73	2Ø20
support	4	0.62	400	400	0.0005	399.8289	single	228.8	6400	4.458157	228.8	0.73	2Ø20
span	4-5	9.806	400	400	0.0077	397.2772	single	228.8	6400	70.96368	228.8	0.73	2Ø20
support	5	1.05	400	400	0.0008	399.7102	single	228.8	6400	7.552346	228.8	0.73	2Ø20



Story4
Story3
Story2
Story1
Base
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51.9115
-30.6093
19.9171
-20.1709
-29.6954
17.4703
-14.8152
-32.6424
18.0618
-12.3374
-10.2523
7.0853
-1.895
0.0713
1.1235
8.2428
8.6488
8.6428
-8.7077
-20.4092
15.8763
18.4603
9.6641
9.6044
9.492
1.0688
0.0939
-13.488
-11.0172
-0.1131
7.8796
8.5241
8.3139
Z
X

Axis B

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	1	51.91	400	400	0.0406	385.1315	single	228.8	6400	387.5072	387.5072	1.23	2Ø20
span	1-2	38.04	400	400	0.0297	389.2187	single	228.8	6400	280.986	280.986	0.89	2Ø20
support	2	30.79	400	400	0.0241	391.3204	single	228.8	6400	226.2117	228.8	0.73	2Ø20
span	2-3	0.58	400	400	0.0005	399.84	single	228.8	6400	4.170418	228.8	0.73	3Ø20
support	3	1.1235	400	400	0.0009	399.6899	single	228.8	6400	8.081421	228.8	0.73	2Ø20
span	3-4	15.87	400	400	0.0124	395.5744	single	228.8	6400	115.3418	228.8	0.73	2Ø20
support	4	26.2	400	400	0.0205	392.6391	single	228.8	6400	191.8429	228.8	0.73	2Ø20
span	4-5	18.46	400	400	0.0144	394.8426	single	228.8	6400	134.4143	228.8	0.73	2Ø20
support	5	20.4	400	400	0.0159	394.2926	single	228.8	6400	148.7474	228.8	0.73	2Ø20

story- 3

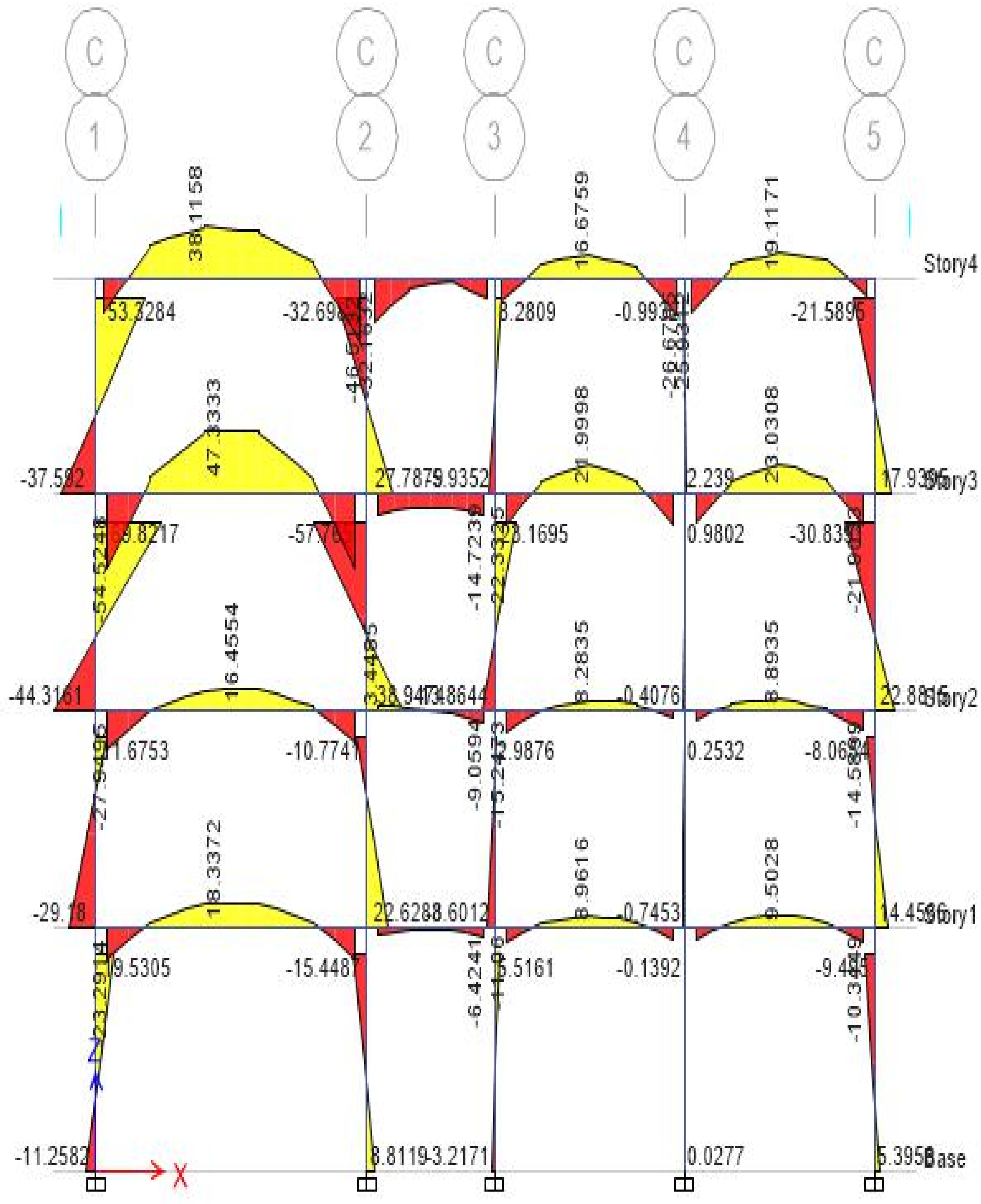
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	1	30.608	400	400	0.0239	391.3728	single	228.8	6400	224.8444	228.8	0.73	2Ø20
span	1-2	19.917	400	400	0.0156	394.4297	single	228.8	6400	145.1751	228.8	0.73	2Ø20
support	2	20.17	400	400	0.0158	394.3579	single	228.8	6400	147.046	228.8	0.73	2Ø20
span	2-3	0.565	400	400	0.0004	399.8441	single	228.8	6400	4.062521	228.8	0.73	3Ø20
support	3	10.587	400	400	0.0083	397.0587	single	228.8	6400	76.65775	228.8	0.73	2Ø20
span	3-4	8.242	400	400	0.0064	397.7139	single	228.8	6400	59.57988	228.8	0.73	2Ø20
support	4	2.378	400	400	0.0019	399.3431	single	228.8	6400	17.11999	228.8	0.73	2Ø20
span	4-5	9.664	400	400	0.0076	397.3169	single	228.8	6400	69.92908	228.8	0.73	2Ø20
support	5	14.94	400	400	0.0117	395.8365	single	228.8	6400	108.5107	228.8	0.73	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	29.69	400	400	0.0232	391.6372	single	228.8	6400	217.9536	228.8	0.73	2Ø20
span	1-2	17.47	400	400	0.0136	395.1226	single	228.8	6400	127.1156	228.8	0.73	2Ø20
support	2	14.81	400	400	0.0116	395.8731	single	228.8	6400	107.5566	228.8	0.73	2Ø20
span	2-3	1.884	400	400	0.0015	399.4798	single	228.8	6400	13.55888	228.8	0.73	3Ø20
support	3	8.84	400	400	0.0069	397.5471	single	228.8	6400	63.92954	228.8	0.73	2Ø20
span	3-4	8.6488	400	400	0.0068	397.6004	single	228.8	6400	62.53841	228.8	0.73	2Ø20
support	4	0.093	400	400	0.0001	399.9744	single	228.8	6400	0.66848	228.8	0.73	2Ø20
span	4-5	9.604	400	400	0.0075	397.3336	single	228.8	6400	69.49198	228.8	0.73	2Ø20
support	5	12.79	400	400	0.0100	396.4411	single	228.8	6400	92.75338	228.8	0.73	2Ø20

story- 1

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	32.642	400	400	0.0255	390.7857	single	228.8	6400	240.1463	240.1463	0.76	2Ø20
span	1-2	18.06	400	400	0.0141	394.9558	single	228.8	6400	131.4641	228.8	0.73	2Ø20
support	2	12.337	400	400	0.0096	396.5682	single	228.8	6400	89.43952	228.8	0.73	2Ø20
span	2-3	21.75	400	400	0.0170	393.909	single	228.8	6400	158.7454	228.8	0.73	3Ø20
support	3	0.43	400	400	0.0003	399.8814	single	228.8	6400	3.091542	228.8	0.73	2Ø20
span	3-4	8.64	400	400	0.0068	397.6029	single	228.8	6400	62.4744	228.8	0.73	2Ø20
support	4	0.644	400	400	0.0005	399.8223	single	228.8	6400	4.630807	228.8	0.73	2Ø20
span	4-5	9.49	400	400	0.0074	397.3655	single	228.8	6400	68.6616	228.8	0.73	2Ø20
support	5	10.19	400	400	0.0080	397.1698	single	228.8	6400	73.76254	228.8	0.73	2Ø20


C
1
2
3
4
5
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Story4
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-32.6988
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-21.5895
47.3333
21.9998
23.0308
-37.592
27.7875
2.239
59.8217
8.1695
0.9802
16.4554
8.2835
8.8935
-44.3161
-0.4076
Story2
11.6753
-10.7741
2.9876
0.2532
18.3372
8.9616
9.5028
-29.18
22.6288
-0.7453
Story1
9.5305
-15.4487
5.5161
-0.1392
-11.2582
0.0277
Base
Z
X

Axis C

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	1	53.32	400	400	0.0417	384.711	single	228.8	6400	398.468	398.468	1.27	2Ø20
span	1-2	38.11	400	400	0.0298	389.1983	single	228.8	6400	281.5178	281.5178	0.90	2Ø20
support	2	32.69	400	400	0.0255	390.7718	single	228.8	6400	240.508	240.508	0.77	2Ø20
span	2-3	0.58	400	400	0.0005	399.84	single	228.8	6400	4.170418	228.8	0.73	3Ø20
support	3	8.28	400	400	0.0065	397.7033	single	228.8	6400	59.85617	228.8	0.73	2Ø20
span	3-4	16.67	400	400	0.0130	395.3486	single	228.8	6400	121.2253	228.8	0.73	2Ø20
support	4	0.99	400	400	0.0008	399.7268	single	228.8	6400	7.120488	228.8	0.73	2Ø20
span	4-5	19.11	400	400	0.0149	394.6585	single	228.8	6400	139.2121	228.8	0.73	2Ø20
support	5	21.58	400	400	0.0169	393.9574	single	228.8	6400	157.4853	228.8	0.73	2Ø20

story- 3

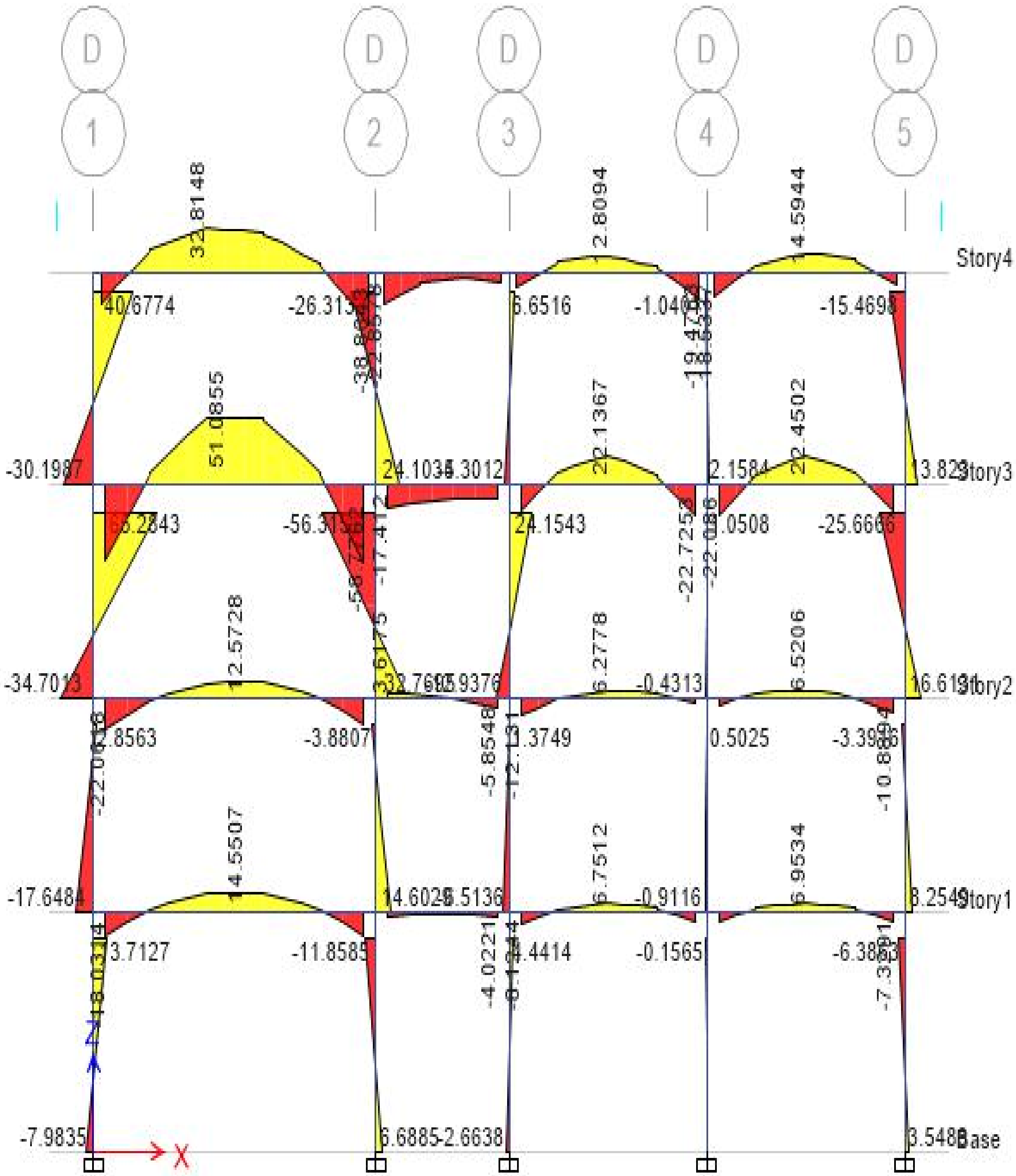
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	1	37.58	400	400	0.0294	389.3527	single	228.8	6400	277.4926	277.4926	0.88	2Ø20
span	1-2	47.33	400	400	0.0370	386.491	single	228.8	6400	352.0748	352.0748	1.12	2Ø20
support	2	27.78	400	400	0.0217	392.1862	single	228.8	6400	203.6469	228.8	0.73	2Ø20
span	2-3	9.93	400	400	0.0078	397.2425	single	228.8	6400	71.86731	228.8	0.73	3Ø20
support	3	14.72	400	400	0.0115	395.8984	single	228.8	6400	106.8961	228.8	0.73	2Ø20
span	3-4	21.99	400	400	0.0172	393.8407	single	228.8	6400	160.5249	228.8	0.73	2Ø20
support	4	2.239	400	400	0.0017	399.3816	single	228.8	6400	16.11773	228.8	0.73	2Ø20
span	4-5	23.03	400	400	0.0180	393.5446	single	228.8	6400	168.2433	228.8	0.73	2Ø20
support	5	17.93	400	400	0.0140	394.9926	single	228.8	6400	130.5056	228.8	0.73	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	1	44.31	400	400	0.0346	387.3821	single	228.8	6400	328.8517	328.8517	1.05	2Ø20
span	1-2	16.45	400	400	0.0129	395.4107	single	228.8	6400	119.6066	228.8	0.73	2Ø20
support	2	3.448	400	400	0.0027	399.0468	single	228.8	6400	24.8417	228.8	0.73	2Ø20
span	2-3	4.86	400	400	0.0038	398.6552	single	228.8	6400	35.04909	228.8	0.73	3Ø20
support	3	15.24	400	400	0.0119	395.752	single	228.8	6400	110.7133	228.8	0.73	2Ø20
span	3-4	8.28	400	400	0.0065	397.7033	single	228.8	6400	59.85617	228.8	0.73	2Ø20
support	4	4.58	400	400	0.0036	398.7329	single	228.8	6400	33.02336	228.8	0.73	2Ø20
span	4-5	8.89	400	400	0.0069	397.5331	single	228.8	6400	64.29339	228.8	0.73	2Ø20
support	5	14.58	400	400	0.0114	395.9379	single	228.8	6400	105.8689	228.8	0.73	2Ø20

story- 1

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	1	23.29	400	400	0.0182	393.4705	single	228.8	6400	170.1748	228.8	0.73	2Ø20
span	1-2	18.33	400	400	0.0143	394.8794	single	228.8	6400	133.4553	228.8	0.73	2Ø20
support	2	15.448	400	400	0.0121	395.6934	single	228.8	6400	112.241	228.8	0.73	2Ø20
span	2-3	8.6	400	400	0.0067	397.614	single	228.8	6400	62.18342	228.8	0.73	3Ø20
support	3	11.08	400	400	0.0087	396.9206	single	228.8	6400	80.25534	228.8	0.73	2Ø20
span	3-4	8.96	400	400	0.0070	397.5135	single	228.8	6400	64.80282	228.8	0.73	2Ø20
support	4	0.74	400	400	0.0006	399.7958	single	228.8	6400	5.321466	228.8	0.73	2Ø20
span	4-5	9.5	400	400	0.0074	397.3627	single	228.8	6400	68.73444	228.8	0.73	2Ø20
support	5	10.34	400	400	0.0081	397.1278	single	228.8	6400	74.85626	228.8	0.73	2Ø20


D
1
2
3
4
5
Story4
Story3
Story2
Story1
Base
32.8148
2.8094
40.6774
6.6516
-15.4698
51.0855
22.1367
22.4502
-30.1987
24.1543
-25.6666
12.5728
6.5206
-34.7013
-0.4313
2.8563
-3.8807
1.3749
0.5025
14.5507
6.7512
6.9534
-17.6484
-0.9116
3.7127
-11.8585
-0.1565
-7.9835
Z
X

Axis D

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	1	26.31	400	400	0.0206	392.6076	single	228.8	6400	192.6638	228.8	0.73	2Ø20
span	1-2	32.81	400	400	0.0256	390.7371	single	228.8	6400	241.4123	241.4123	0.77	2Ø20
support	2	22.85	400	400	0.0179	393.5959	single	228.8	6400	166.9066	228.8	0.73	2Ø20
span	2-3	6.58	400	400	0.0051	398.1771	single	228.8	6400	47.51027	228.8	0.73	3Ø20
support	3	6.65	400	400	0.0052	398.1576	single	228.8	6400	48.01805	228.8	0.73	2Ø20
span	3-4	12.8	400	400	0.0100	396.4383	single	228.8	6400	92.82655	228.8	0.73	2Ø20
support	4	19.47	400	400	0.0152	394.5565	single	228.8	6400	141.8713	228.8	0.73	2Ø20
span	4-5	14.59	400	400	0.0114	395.935	single	228.8	6400	105.9423	228.8	0.73	2Ø20
support	5	7.34	400	400	0.0057	397.9654	single	228.8	6400	53.02596	228.8	0.73	2Ø20

story- 3

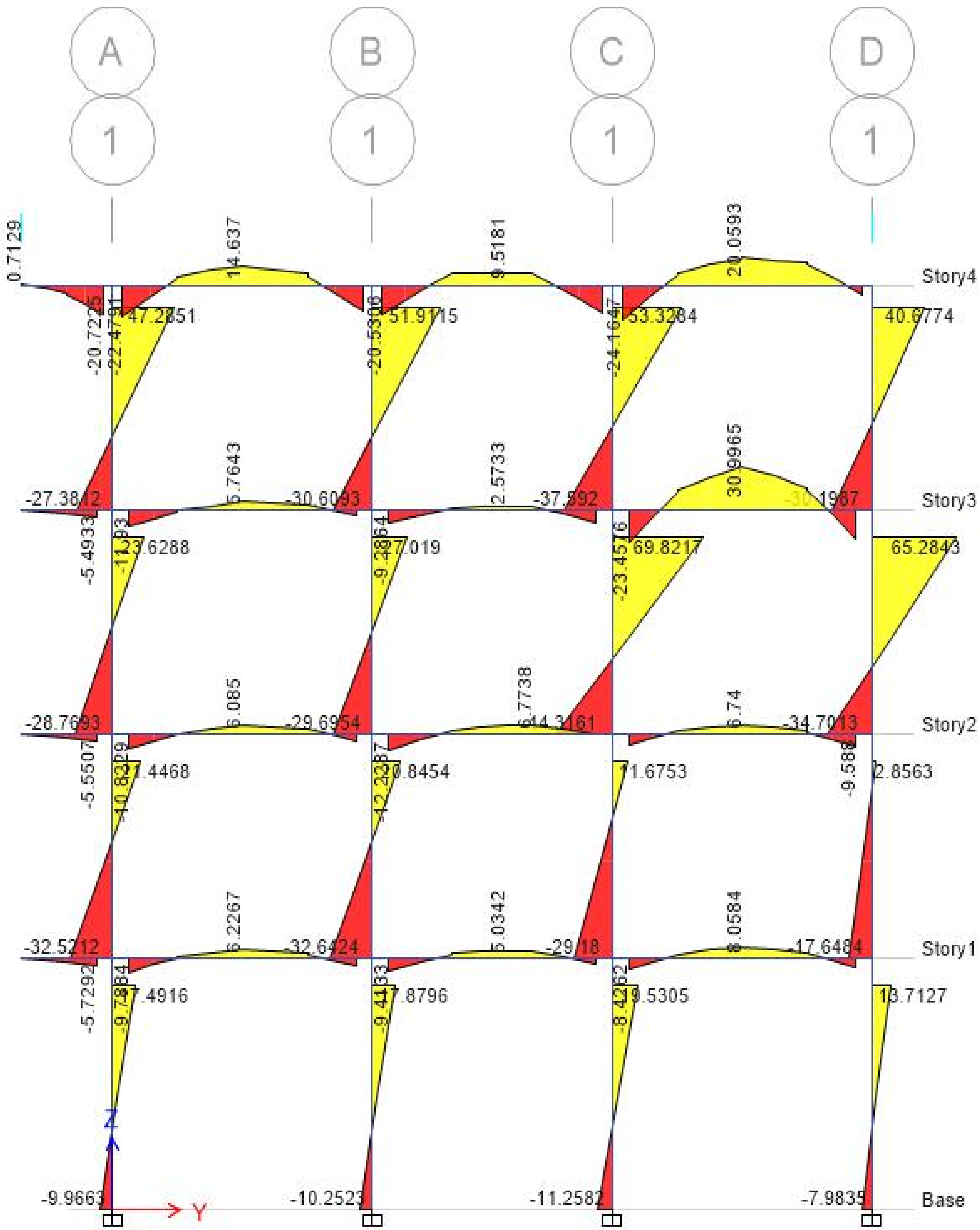
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	1	30.19	400	400	0.0236	391.4933	single	228.8	6400	221.7056	228.8	0.73	2Ø20
span	1-2	51.08	400	400	0.0399	385.3786	single	228.8	6400	381.0668	381.0668	1.21	2Ø20
support	2	58.77	400	400	0.0459	383.0763	single	228.8	6400	441.0707	441.0707	1.40	2Ø20
span	2-3	6.3	400	400	0.0049	398.255	single	228.8	6400	45.47966	228.8	0.73	3Ø20
support	3	22.72	400	400	0.0178	393.6329	single	228.8	6400	165.9414	228.8	0.73	2Ø20
span	3-4	22.13	400	400	0.0173	393.8009	single	228.8	6400	161.5632	228.8	0.73	2Ø20
support	4	22.08	400	400	0.0173	393.8151	single	228.8	6400	161.1924	228.8	0.73	2Ø20
span	4-5	22.45	400	400	0.0175	393.7098	single	228.8	6400	163.9374	228.8	0.73	2Ø20
support	5	25.66	400	400	0.0200	392.7936	single	228.8	6400	187.8149	228.8	0.73	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	1	22.06	400	400	0.0172	393.8208	single	228.8	6400	161.0441	228.8	0.73	2Ø20
span	1-2	12.75	400	400	0.0100	396.4523	single	228.8	6400	92.46068	228.8	0.73	2Ø20
support	2	3.617	400	400	0.0028	399	single	228.8	6400	26.06234	228.8	0.73	2Ø20
span	2-3	5.93	400	400	0.0046	398.3579	single	228.8	6400	42.79757	228.8	0.73	3Ø20
support	3	12.31	400	400	0.0096	396.5758	single	228.8	6400	89.24208	228.8	0.73	2Ø20
span	3-4	6.277	400	400	0.0049	398.2614	single	228.8	6400	45.31289	228.8	0.73	2Ø20
support	4	3.39	400	400	0.0026	399.0629	single	228.8	6400	24.42284	228.8	0.73	2Ø20
span	4-5	6.52	400	400	0.0051	398.1937	single	228.8	6400	47.07507	228.8	0.73	2Ø20
support	5	10.88	400	400	0.0085	396.9766	single	228.8	6400	78.79557	228.8	0.73	2Ø20

story- 1

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma:	As,cal	As,prov	No of	Remark
support	1	18.03	400	400	0.0141	394.9643	single	228.8	6400	131.2429	228.8	0.73	2Ø20
span	1-2	14.55	400	400	0.0114	395.9463	single	228.8	6400	105.6488	228.8	0.73	2Ø20
support	2	18.03	400	400	0.0141	394.9643	single	228.8	6400	131.2429	228.8	0.73	2Ø20
span	2-3	51	400	400	0.0398	385.4024	single	228.8	6400	380.4465	380.4465	1.21	3Ø20
support	3	8.12	400	400	0.0063	397.748	single	228.8	6400	58.69294	228.8	0.73	2Ø20
span	3-4	6.75	400	400	0.0053	398.1297	single	228.8	6400	48.74353	228.8	0.73	2Ø20
support	4	8.13	400	400	0.0064	397.7452	single	228.8	6400	58.76564	228.8	0.73	2Ø20
span	4-5	6.95	400	400	0.0054	398.074	single	228.8	6400	50.19481	228.8	0.73	2Ø20
support	5	7.39	400	400	0.0058	397.9515	single	228.8	6400	53.38905	228.8	0.73	2Ø20


A
B
C
D
1
Story4
Story3
Story2
Story1
Base
Z
Y
47.2851
51.9115
53.3284
40.6774
14.637
9.5181
20.0593
-27.3812
-30.6093
-37.592
-28.7693
-29.6954
-34.7013
-32.5212
-32.6424
-17.6484
-9.9663
-10.2523
-11.2582
-7.9835
65.2843
2.8563
13.7127

Axis 1

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	20.72	400	400	0.0162	394.2018	single	228.8	6400	151.1155	228.8	0.73	2Ø20
span	A-B	14.63	400	400	0.0114	395.9238	single	228.8	6400	106.2357	228.8	0.73	2Ø20
support	B	20.53	400	400	0.0160	394.2557	single	228.8	6400	149.7093	228.8	0.73	2Ø20
span	B-C	9.51	400	400	0.0074	397.3599	single	228.8	6400	68.80727	228.8	0.73	3Ø20
support	C	24.16	400	400	0.0189	393.2223	single	228.8	6400	176.6431	228.8	0.73	2Ø20
span	C-D	29.05	400	400	0.0227	391.8213	single	228.8	6400	213.1552	228.8	0.73	2Ø20
support	D	12.35	400	400	0.0096	396.5646	single	228.8	6400	89.53459	228.8	0.73	2Ø20

story- 3

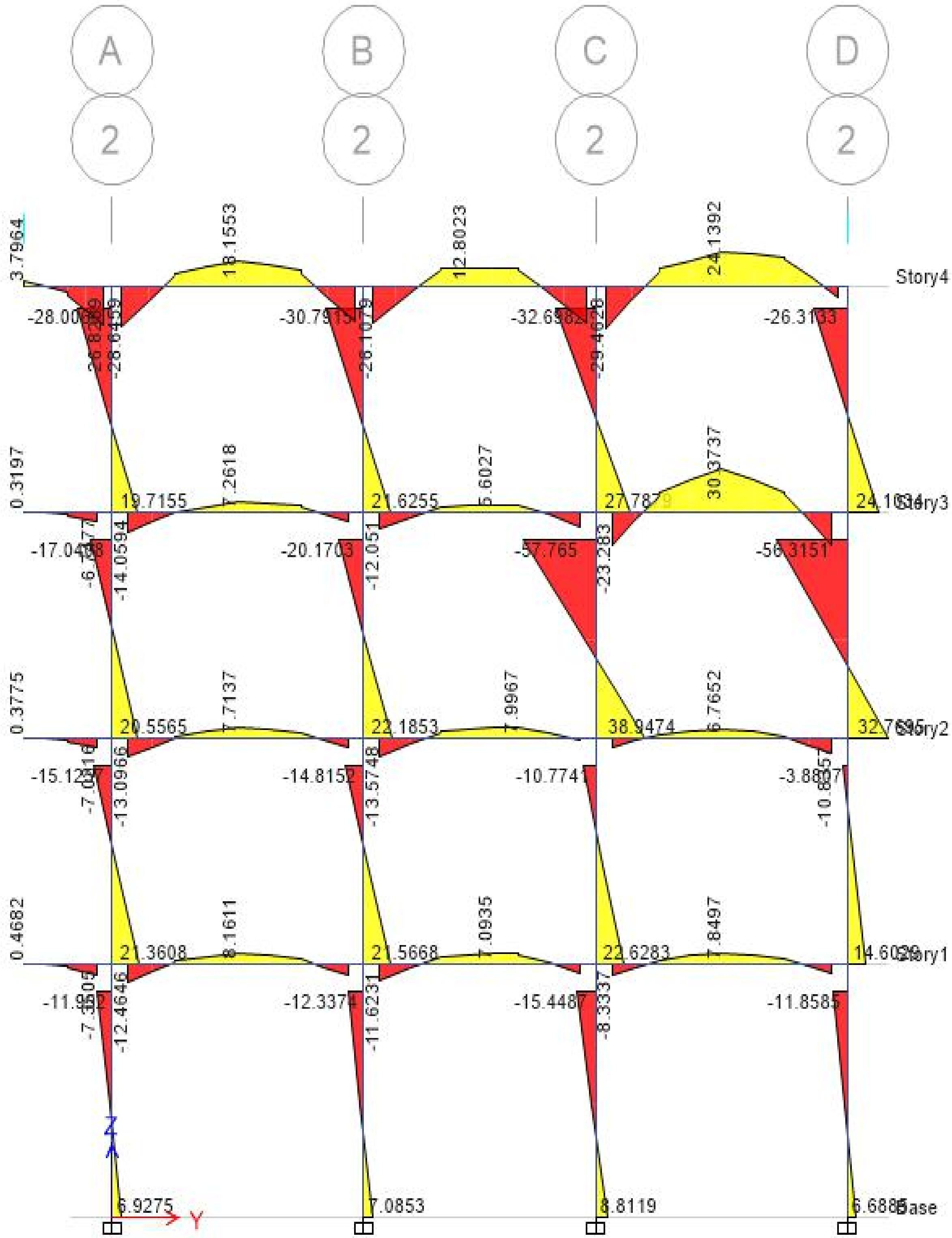
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	5.49	400	400	0.0043	398.4802	single	228.8	6400	39.60987	228.8	0.73	2Ø20
span	A-B	5.76	400	400	0.0045	398.4051	single	228.8	6400	41.56573	228.8	0.73	2Ø20
support	B	9.28	400	400	0.0073	397.4242	single	228.8	6400	67.1323	228.8	0.73	2Ø20
span	B-C	57	400	400	0.0445	383.6088	single	228.8	6400	427.193	427.193	1.36	3Ø20
support	C	23.45	400	400	0.0183	393.4248	single	228.8	6400	171.3637	228.8	0.73	2Ø20
span	C-D	30.99	400	400	0.0242	391.2627	single	228.8	6400	227.7147	228.8	0.73	2Ø20
support	D	19.87	400	400	0.0155	394.443	single	228.8	6400	144.8276	228.8	0.73	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	10.83	400	400	0.0085	396.9906	single	228.8	6400	78.43069	228.8	0.73	2Ø20
span	A-B	6.08	400	400	0.0048	398.3162	single	228.8	6400	43.88474	228.8	0.73	2Ø20
support	B	12.32	400	400	0.0096	396.573	single	228.8	6400	89.3152	228.8	0.73	2Ø20
span	B-C	6.77	400	400	0.0053	398.1242	single	228.8	6400	48.88864	228.8	0.73	3Ø20
support	C	4.52	400	400	0.0035	398.7496	single	228.8	6400	32.58938	228.8	0.73	2Ø20
span	C-D	6.74	400	400	0.0053	398.1325	single	228.8	6400	48.67098	228.8	0.73	2Ø20
support	D	9.58	400	400	0.0075	397.3403	single	228.8	6400	69.31715	228.8	0.73	2Ø20

story- 1

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	5.72	400	400	0.0045	398.4163	single	228.8	6400	41.27593	228.8	0.73	2Ø20
span	A-B	6.22	400	400	0.0049	398.2772	single	228.8	6400	44.89963	228.8	0.73	2Ø20
support	B	9.41	400	400	0.0074	397.3878	single	228.8	6400	68.07896	228.8	0.73	2Ø20
span	B-C	5.03	400	400	0.0039	398.608	single	228.8	6400	36.27938	228.8	0.73	3Ø20
support	C	8.42	400	400	0.0066	397.6643	single	228.8	6400	60.87421	228.8	0.73	2Ø20
span	C-D	8.05	400	400	0.0063	397.7675	single	228.8	6400	58.18411	228.8	0.73	2Ø20
support	D	8.42	400	400	0.0066	397.6643	single	228.8	6400	60.87421	228.8	0.73	2Ø20


A
B
C
D
2
2
2
2
Story4
Story3
Story2
Story1
Base
3.7964
18.1553
12.8023
24.1392
-28.0069
-30.7915
-32.6582
-26.3133
0.3197
7.2618
5.6027
30.3737
19.7155
21.6255
24.1034
-17.0408
-20.1703
-57.765
-56.3151
-14.0594
-12.051
-23.283
0.3775
7.7137
7.9967
6.7652
20.5565
22.1853
38.9474
-15.1237
-14.8152
-10.7741
-3.8807
-13.0966
-13.5748
0.4682
8.1611
7.0935
7.8497
21.3608
21.5668
22.6283
-11.9502
-12.3374
-15.4487
-11.8585
-12.4646
-11.6231
-8.3337
6.9275
7.0853
8.8119
6.6885
Z
Y

Axis 2

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	28.64	400	400	0.0224	391.9392	single	228.8	6400	210.0836	228.8	0.73	2Ø20
span	A-B	18.15	400	400	0.0142	394.9303	single	228.8	6400	132.1277	228.8	0.73	2Ø20
support	B	26.1	400	400	0.0204	392.6677	single	228.8	6400	191.0967	228.8	0.73	2Ø20
span	B-C	12.8	400	400	0.0100	396.4383	single	228.8	6400	92.82655	228.8	0.73	3Ø20
support	C	29.4	400	400	0.0230	391.7207	single	228.8	6400	215.7788	228.8	0.73	2Ø20
span	C-D	24.13	400	400	0.0189	393.2308	single	228.8	6400	176.4199	228.8	0.73	2Ø20
support	D	5.43	400	400	0.0042	398.4969	single	228.8	6400	39.17534	228.8	0.73	2Ø20

story- 3

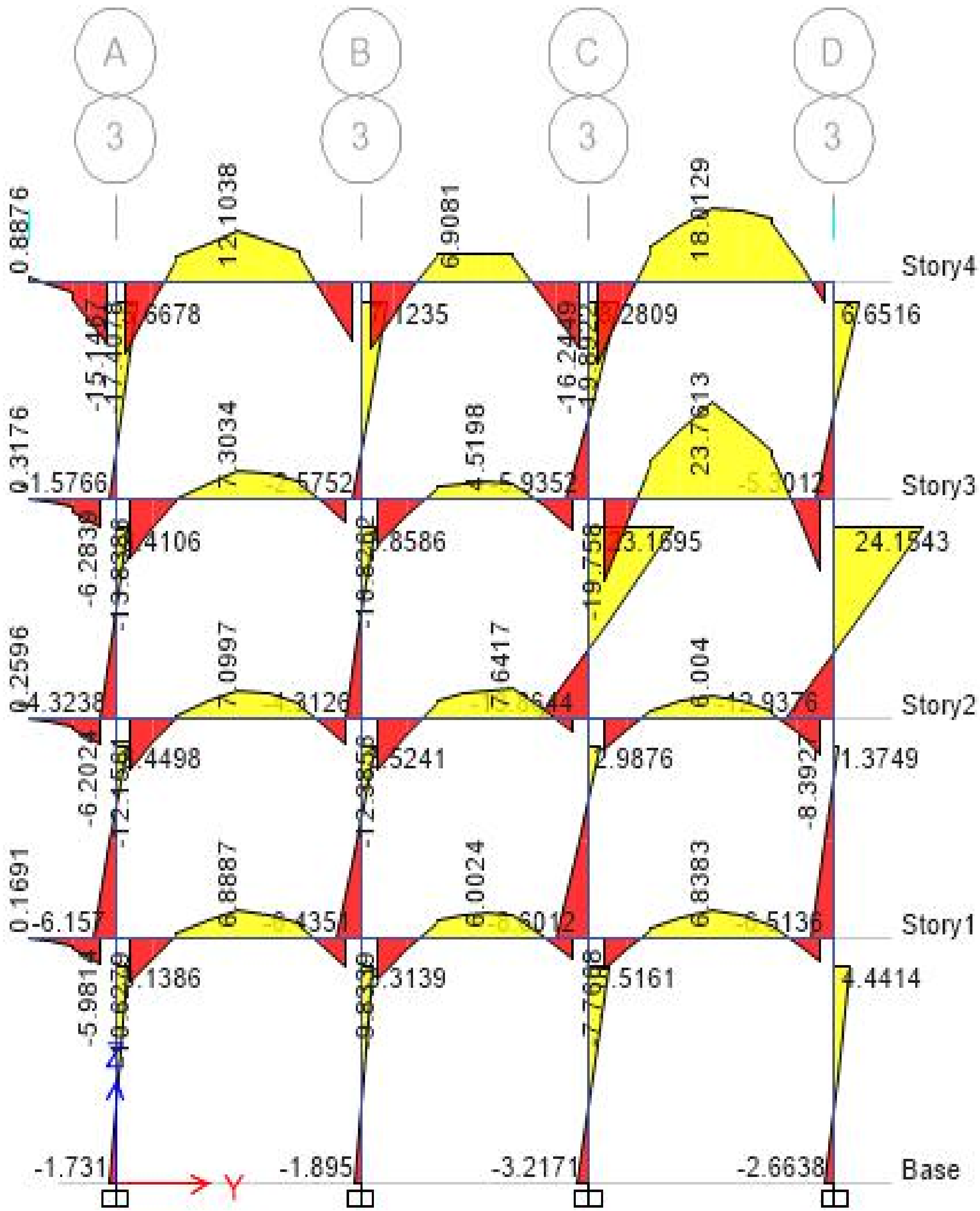
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	14.05	400	400	0.0110	396.087	single	228.8	6400	101.982	228.8	0.73	2Ø20
span	A-B	7.26	400	400	0.0057	397.9877	single	228.8	6400	52.44509	228.8	0.73	2Ø20
support	B	12.05	400	400	0.0094	396.6488	single	228.8	6400	87.34113	228.8	0.73	2Ø20
span	B-C	5.6	400	400	0.0044	398.4496	single	228.8	6400	40.40661	228.8	0.73	3Ø20
support	C	23.2	400	400	0.0181	393.4961	single	228.8	6400	169.5061	228.8	0.73	2Ø20
span	C-D	30.37	400	400	0.0237	391.4414	single	228.8	6400	223.057	228.8	0.73	2Ø20
support	D	23.28	400	400	0.0182	393.4733	single	228.8	6400	170.1005	228.8	0.73	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	13.09	400	400	0.0102	396.3568	single	228.8	6400	94.94916	228.8	0.73	2Ø20
span	A-B	7.71	400	400	0.0060	397.8623	single	228.8	6400	55.71337	228.8	0.73	2Ø20
support	B	13.57	400	400	0.0106	396.222	single	228.8	6400	98.46438	228.8	0.73	2Ø20
span	B-C	0.99	400	400	0.0008	399.7268	single	228.8	6400	7.120488	228.8	0.73	3Ø20
support	C	2.24	400	400	0.0018	399.3813	single	228.8	6400	16.12494	228.8	0.73	2Ø20
span	C-D	6.76	400	400	0.0053	398.1269	single	228.8	6400	48.81609	228.8	0.73	2Ø20
support	D	10.8	400	400	0.0084	396.999	single	228.8	6400	78.21177	228.8	0.73	2Ø20

story- 1

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	12.46	400	400	0.0097	396.5337	single	228.8	6400	90.3391	228.8	0.73	2Ø20
span	A-B	8.16	400	400	0.0064	397.7368	single	228.8	6400	58.98373	228.8	0.73	2Ø20
support	B	11.62	400	400	0.0091	396.7693	single	228.8	6400	84.1988	228.8	0.73	2Ø20
span	B-C	7.09	400	400	0.0055	398.0351	single	228.8	6400	51.21094	228.8	0.73	3Ø20
support	C	8.33	400	400	0.0065	397.6894	single	228.8	6400	60.21974	228.8	0.73	2Ø20
span	C-D	7.84	400	400	0.0061	397.8261	single	228.8	6400	56.65793	228.8	0.73	2Ø20
support	D	8.33	400	400	0.0065	397.6894	single	228.8	6400	60.21974	228.8	0.73	2Ø20


A
B
C
D
3
Story4
Story3
Story2
Story1
Base
12.1038
6.9081
18.8129
23.7613
24.1543
6.6516
1.3749
4.4414
-1.731
-1.895
-3.2171
-2.6638
Y

Axis 3

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	17.4	400	400	0.0136	395.1424	single	228.8	6400	126.5999	228.8	0.73	2Ø20
span	A-B	12.1	400	400	0.0095	396.6347	single	228.8	6400	87.70664	228.8	0.73	2Ø20
support	B	17.4	400	400	0.0136	395.1424	single	228.8	6400	126.5999	228.8	0.73	2Ø20
span	B-C	6.9	400	400	0.0054	398.088	single	228.8	6400	49.83195	228.8	0.73	3Ø20
support	C	19.89	400	400	0.0155	394.4374	single	228.8	6400	144.9755	228.8	0.73	2Ø20
span	C-D	18.01	400	400	0.0141	394.9699	single	228.8	6400	131.0954	228.8	0.73	2Ø20
support	D	2.45	400	400	0.0019	399.3232	single	228.8	6400	17.63922	228.8	0.73	2Ø20

story- 3

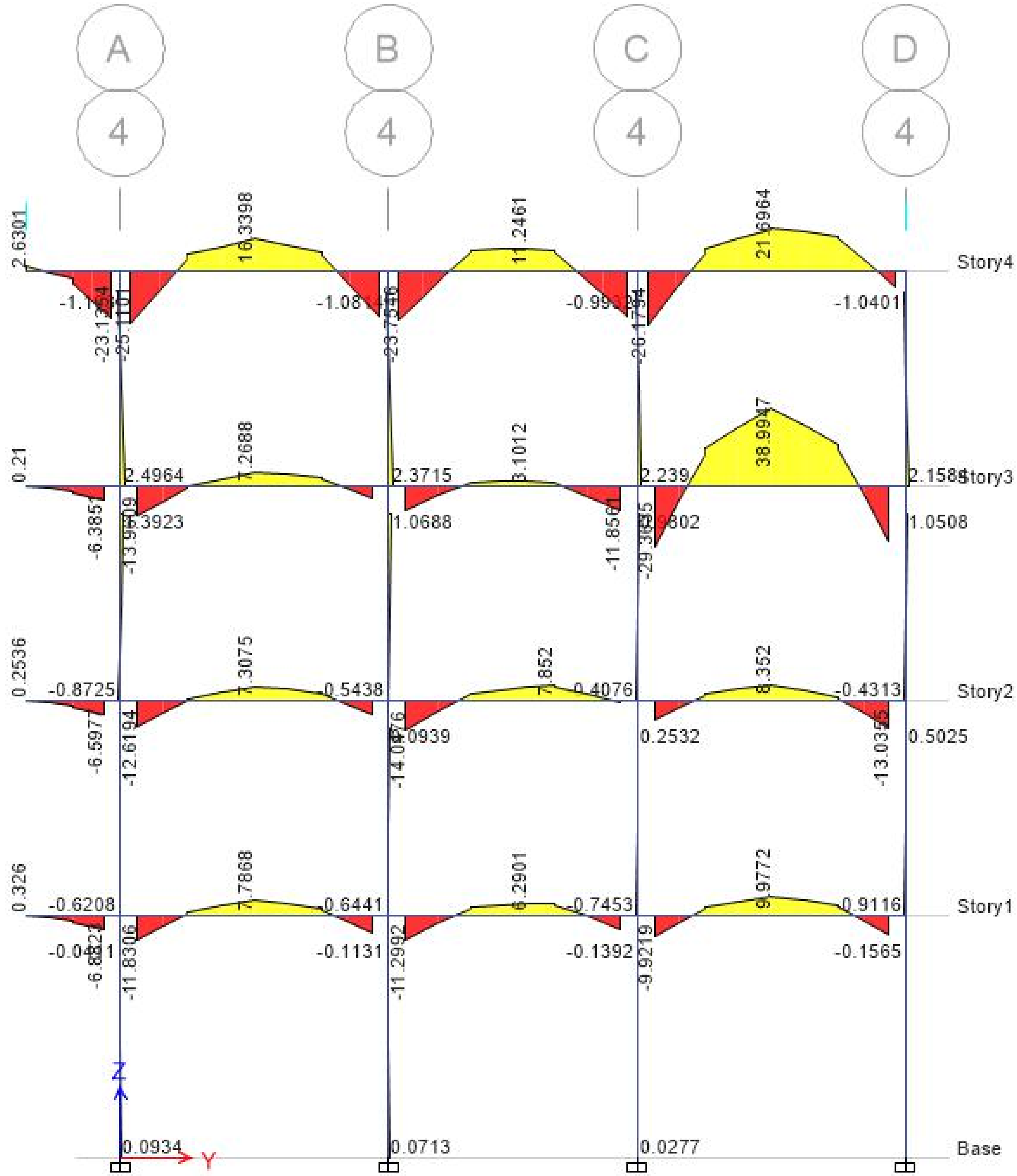
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	13.83	400	400	0.0108	396.1489	single	228.8	6400	100.3695	228.8	0.73	2Ø20
span	A-B	7.3	400	400	0.0057	397.9766	single	228.8	6400	52.73552	228.8	0.73	2Ø20
support	B	10.82	400	400	0.0085	396.9934	single	228.8	6400	78.35772	228.8	0.73	2Ø20
span	B-C	4.51	400	400	0.0035	398.7523	single	228.8	6400	32.51705	228.8	0.73	3Ø20
support	C	19.75	400	400	0.0154	394.4771	single	228.8	6400	143.9406	228.8	0.73	2Ø20
span	C-D	23.76	400	400	0.0186	393.3364	single	228.8	6400	173.6681	228.8	0.73	2Ø20
support	D	16.83	400	400	0.0131	395.3035	single	228.8	6400	122.4028	228.8	0.73	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	12.15	400	400	0.0095	396.6207	single	228.8	6400	88.07218	228.8	0.73	2Ø20
span	A-B	7.09	400	400	0.0055	398.0351	single	228.8	6400	51.21094	228.8	0.73	2Ø20
support	B	12.38	400	400	0.0097	396.5562	single	228.8	6400	89.75399	228.8	0.73	2Ø20
span	B-C	7.64	400	400	0.0060	397.8818	single	228.8	6400	55.20484	228.8	0.73	3Ø20
support	C	8.39	400	400	0.0066	397.6727	single	228.8	6400	60.65604	228.8	0.73	2Ø20
span	C-D	6	400	400	0.0047	398.3384	single	228.8	6400	43.30489	228.8	0.73	2Ø20
support	D	8.39	400	400	0.0066	397.6727	single	228.8	6400	60.65604	228.8	0.73	2Ø20

story- 1

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	10.62	400	400	0.0083	397.0494	single	228.8	6400	76.89848	228.8	0.73	2Ø20
span	A-B	6.88	400	400	0.0054	398.0935	single	228.8	6400	49.68681	228.8	0.73	2Ø20
support	B	9.83	400	400	0.0077	397.2704	single	228.8	6400	71.13857	228.8	0.73	2Ø20
span	B-C	6	400	400	0.0047	398.3384	single	228.8	6400	43.30489	228.8	0.73	3Ø20
support	C	7.76	400	400	0.0061	397.8484	single	228.8	6400	56.07664	228.8	0.73	2Ø20
span	C-D	6.83	400	400	0.0053	398.1075	single	228.8	6400	49.32399	228.8	0.73	2Ø20
support	D	7.76	400	400	0.0061	397.8484	single	228.8	6400	56.07664	228.8	0.73	2Ø20


A
B
C
D
4
Story4
Story3
Story2
Story1
Base
16.3398
11.2461
21.6964
38.9947
7.2688
3.1012
7.3075
7.852
8.352
7.7868
6.2901
9.9772
2.4964
2.3715
2.239
1.0688
1.0508
0.2532
0.5025
0.0934
0.0713
0.0277
-1.0401
-0.8725
-0.5438
-0.4076
-0.4313
-0.6208
-0.6441
-0.7453
-0.9116
-0.1131
-0.1392
-0.1565
-11.8306
-11.2992
-9.9219
-12.6194
-13.0325
-11.8561
-6.3857
-6.5977
2.6301
0.21
0.2536
0.326
Z
Y

Axis 4

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	25.11	400	400	0.0196	392.9509	single	228.8	6400	183.7157	228.8	0.73	2Ø20
span	A-B	16.33	400	400	0.0128	395.4446	single	228.8	6400	118.724	228.8	0.73	2Ø20
support	B	23.7	400	400	0.0185	393.3535	single	228.8	6400	173.222	228.8	0.73	2Ø20
span	B-C	11.24	400	400	0.0088	396.8758	single	228.8	6400	81.42345	228.8	0.73	3Ø20
support	C	26.17	400	400	0.0204	392.6477	single	228.8	6400	191.619	228.8	0.73	2Ø20
span	C-D	21.69	400	400	0.0169	393.9261	single	228.8	6400	158.3006	228.8	0.73	2Ø20
support	D	5.34	400	400	0.0042	398.5219	single	228.8	6400	38.52361	228.8	0.73	2Ø20

story- 3

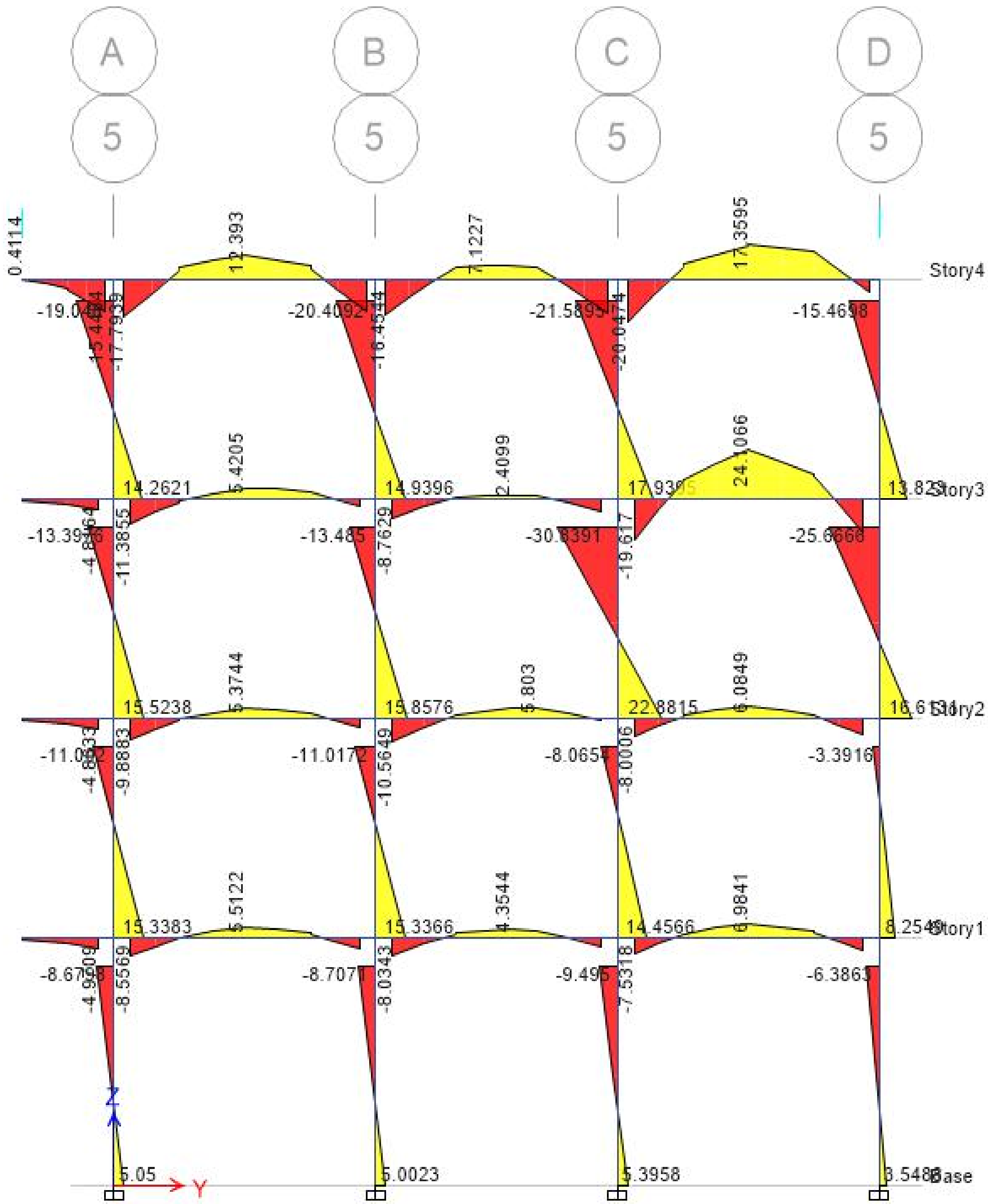
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	13.96	400	400	0.0109	396.1123	single	228.8	6400	101.3223	228.8	0.73	2Ø20
span	A-B	7.26	400	400	0.0057	397.9877	single	228.8	6400	52.44509	228.8	0.73	2Ø20
support	B	2.37	400	400	0.0019	399.3453	single	228.8	6400	17.0623	228.8	0.73	2Ø20
span	B-C	3.1	400	400	0.0024	399.1432	single	228.8	6400	22.32908	228.8	0.73	3Ø20
support	C	11.85	400	400	0.0093	396.7048	single	228.8	6400	85.87934	228.8	0.73	2Ø20
span	C-D	38.99	400	400	0.0305	388.9416	single	228.8	6400	288.2085	288.2085	0.92	2Ø20
support	D	29.39	400	400	0.0230	391.7235	single	228.8	6400	215.7038	228.8	0.73	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	12.61	400	400	0.0099	396.4916	single	228.8	6400	91.43636	228.8	0.73	2Ø20
span	A-B	7.3	400	400	0.0057	397.9766	single	228.8	6400	52.73552	228.8	0.73	2Ø20
support	B	14.04	400	400	0.0110	396.0898	single	228.8	6400	101.9087	228.8	0.73	2Ø20
span	B-C	7.85	400	400	0.0061	397.8233	single	228.8	6400	56.73059	228.8	0.73	3Ø20
support	C	9.85	400	400	0.0077	397.2649	single	228.8	6400	71.28431	228.8	0.73	2Ø20
span	C-D	8.35	400	400	0.0065	397.6838	single	228.8	6400	60.36517	228.8	0.73	2Ø20
support	D	13.03	400	400	0.0102	396.3737	single	228.8	6400	94.50993	228.8	0.73	2Ø20

story- 1

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	11.83	400	400	0.0092	396.7105	single	228.8	6400	85.73318	228.8	0.73	2Ø20
span	A-B	7.78	400	400	0.0061	397.8428	single	228.8	6400	56.22196	228.8	0.73	2Ø20
support	B	11.29	400	400	0.0088	396.8618	single	228.8	6400	81.78855	228.8	0.73	2Ø20
span	B-C	6.29	400	400	0.0049	398.2577	single	228.8	6400	45.40715	228.8	0.73	3Ø20
support	C	9.92	400	400	0.0078	397.2453	single	228.8	6400	71.79443	228.8	0.73	2Ø20
span	C-D	9.97	400	400	0.0078	397.2313	single	228.8	6400	72.15884	228.8	0.73	2Ø20
support	D	6.86	400	400	0.0054	398.0991	single	228.8	6400	49.54168	228.8	0.73	2Ø20


A
B
C
D
5
5
5
5
Story4
Story3
Story2
Story1
Base
12.393
7.1227
17.3595
-19.0
-20.4092
-21.5895
-15.4658
14.2621
5.4205
14.9396
2.4099
24.1066
-13.485
-30.8391
-25.6668
15.5238
5.3744
15.8576
5.803
22.8815
6.0849
-11.0172
-8.0654
-3.3916
15.3383
5.5122
15.3366
4.3544
14.4566
6.9841
-8.7071
-9.495
-6.3863
5.05
5.0023
5.3958
Z
Y

Axis 5

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	17.79	400	400	0.0139	395.0322	single	228.8	6400	129.4736	228.8	0.73	2Ø20
span	A-B	12.39	400	400	0.0097	396.5534	single	228.8	6400	89.82713	228.8	0.73	2Ø20
support	B	16.45	400	400	0.0129	395.4107	single	228.8	6400	119.6066	228.8	0.73	2Ø20
span	B-C	7.12	400	400	0.0056	398.0267	single	228.8	6400	51.42871	228.8	0.73	3Ø20
support	C	20.04	400	400	0.0157	394.3948	single	228.8	6400	146.0846	228.8	0.73	2Ø20
span	C-D	17.38	400	400	0.0136	395.1481	single	228.8	6400	126.4526	228.8	0.73	2Ø20
support	D	6.57	400	400	0.0051	398.1798	single	228.8	6400	47.43774	228.8	0.73	2Ø20

story- 3

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	11.38	400	400	0.0089	396.8366	single	228.8	6400	82.44577	228.8	0.73	2Ø20
span	A-B	5.42	400	400	0.0042	398.4996	single	228.8	6400	39.10292	228.8	0.73	2Ø20
support	B	8.76	400	400	0.0068	397.5694	single	228.8	6400	63.34743	228.8	0.73	2Ø20
span	B-C	4	400	400	0.0031	398.8938	single	228.8	6400	28.82973	228.8	0.73	3Ø20
support	C	19.61	400	400	0.0153	394.5168	single	228.8	6400	142.9058	228.8	0.73	2Ø20
span	C-D	24.1	400	400	0.0188	393.2394	single	228.8	6400	176.1967	228.8	0.73	2Ø20
support	D	19.61	400	400	0.0153	394.5168	single	228.8	6400	142.9058	228.8	0.73	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	9.88	400	400	0.0077	397.2565	single	228.8	6400	71.50293	228.8	0.73	2Ø20
span	A-B	7	400	400	0.0055	398.0601	single	228.8	6400	50.55769	228.8	0.73	2Ø20
support	B	10.56	400	400	0.0083	397.0662	single	228.8	6400	76.4608	228.8	0.73	2Ø20
span	B-C	5.8	400	400	0.0045	398.394	single	228.8	6400	41.85555	228.8	0.73	3Ø20
support	C	8	400	400	0.0063	397.7814	single	228.8	6400	57.8207	228.8	0.73	2Ø20
span	C-D	6.08	400	400	0.0048	398.3162	single	228.8	6400	43.88474	228.8	0.73	2Ø20
support	D	3.39	400	400	0.0026	399.0629	single	228.8	6400	24.42284	228.8	0.73	2Ø20

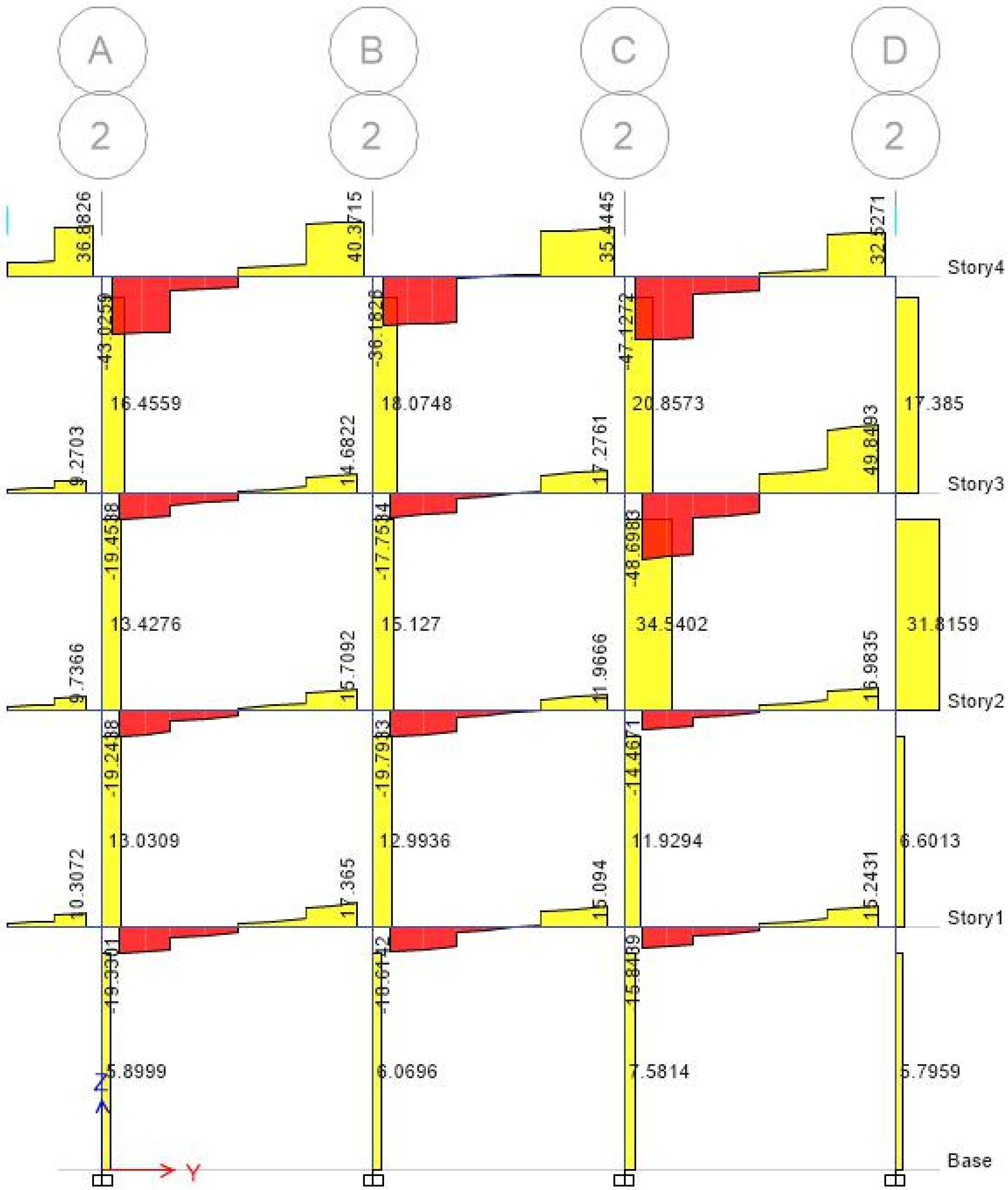
story- 1

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,prov	No of	Remark
support	A	8.55	400	400	0.0067	397.628	single	228.8	6400	61.81972	228.8	0.73	2Ø20
span	A-B	5.51	400	400	0.0043	398.4746	single	228.8	6400	39.75473	228.8	0.73	2Ø20
support	B	8.03	400	400	0.0063	397.7731	single	228.8	6400	58.03874	228.8	0.73	2Ø20
span	B-C	4.35	400	400	0.0034	398.7967	single	228.8	6400	31.35996	228.8	0.73	3Ø20
support	C	7.53	400	400	0.0059	397.9125	single	228.8	6400	54.40581	228.8	0.73	2Ø20
span	C-D	6.98	400	400	0.0055	398.0657	single	228.8	6400	50.41253	228.8	0.73	2Ø20
support	D	6.86	400	400	0.0054	398.0991	single	228.8	6400	49.54168	228.8	0.73	2Ø20

Shear

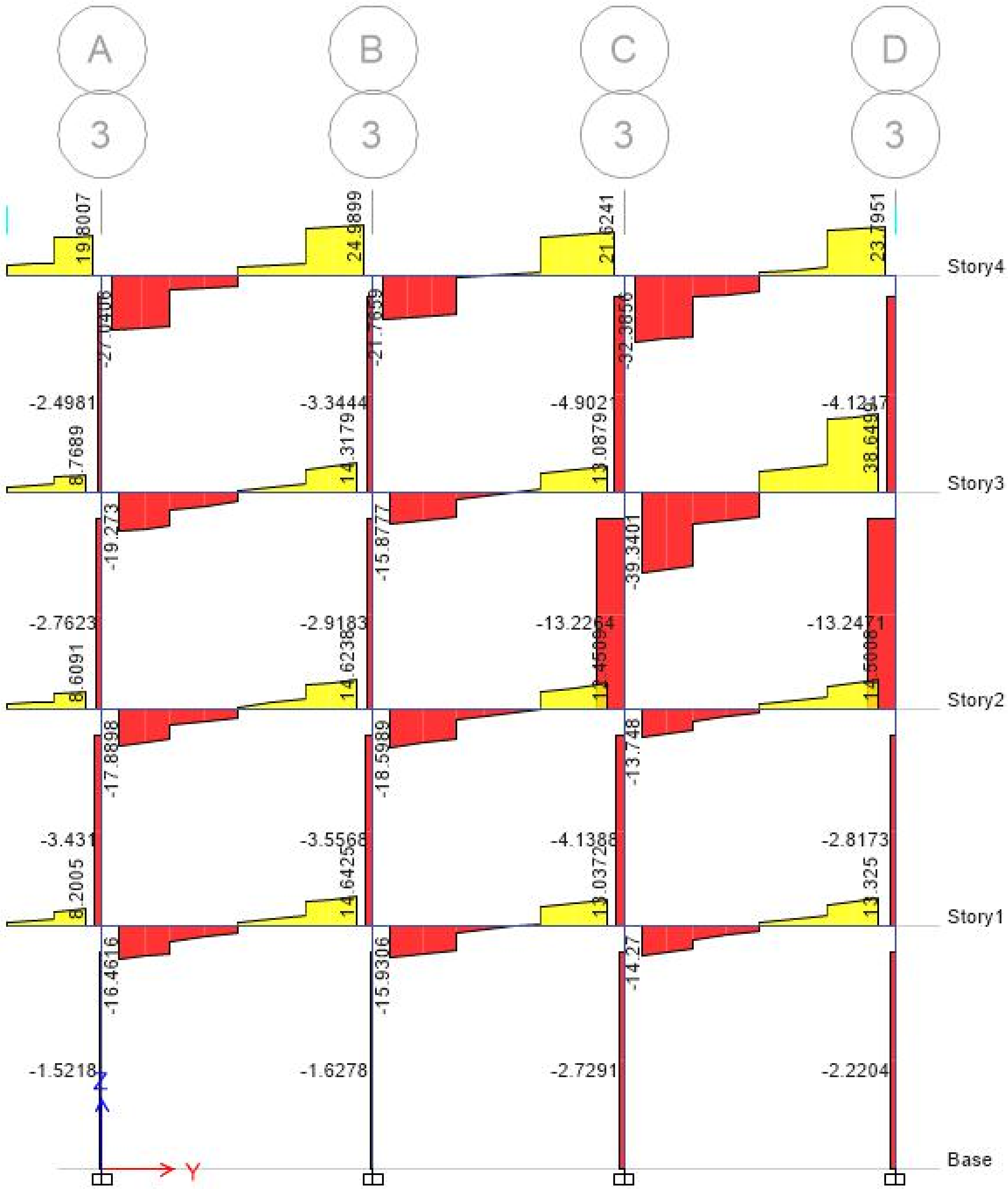
Table 37beam stirrup calculation

axis 1							
Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	33.51	31.11	225	174.5	188.2	174.48	200
B-C	28.18	26.72	225	203.1	188.2	203.15	200
C-D	38.37	27.24	225	199.3	188.2	199.27	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	16	10.87	225	499.4	188.2	225.00	200
B-C	12.54	12.43	225	436.7	188.2	225.00	200
C-D	49.11	48.86	225	111.1	188.2	111.09	200
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	15.54	12.06	225	450.1	188.2	225.00	200
B-C	16.77	8.32	225	3322.8	188.2	225.00	200
C-D	16.18	15.32	225	1804.5	188.2	225.00	200
Story -1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	14.99	12.85	225	422.4	188.2	225.00	200
B-C	14.48	11.02	225	2508.7	188.2	225.00	200
C-D	16.02	14.9	225	1855.4	188.2	225.00	200


A
B
C
D
2
2
2
2
36.8826
40.3715
35.4445
32.5271
Story4
16.4559
18.0748
20.8573
17.385
9.2703
14.6822
17.2761
49.8493
Story3
13.4276
15.127
34.5402
31.8159
9.7366
15.7092
11.9666
16.9835
Story2
13.0309
12.9936
11.9294
6.6013
10.3072
17.365
15.094
15.2431
Story1
5.8999
6.0696
7.5814
5.7959
Base
Z
Y

axis 2

Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	43.02	40.37	225	134.5	188.2	134.46	200
B-C	36.18	35.44	225	153.2	188.2	153.16	200
C-D	47.12	32.52	225	166.9	188.2	166.92	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	19.45	14.68	225	369.8	188.2	225.00	200
B-C	17.75	17.27	225	314.3	188.2	225.00	200
C-D	49.84	48.69	225	111.5	188.2	111.09	200
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	19.24	15.7	225	345.7	188.2	225.00	200
B-C	19.79	11.96	225	453.9	188.2	225.00	200
C-D	16.98	14.46	225	375.4	188.2	225.00	200
Story -1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	19.33	17.36	225	312.7	188.2	225.00	200
B-C	15.09	10.61	225	511.6	188.2	225.00	200
C-D	15.84	15.24	225	356.2	188.2	225.00	200


A
B
C
D
3
Story4
Story3
Story2
Story1
Base
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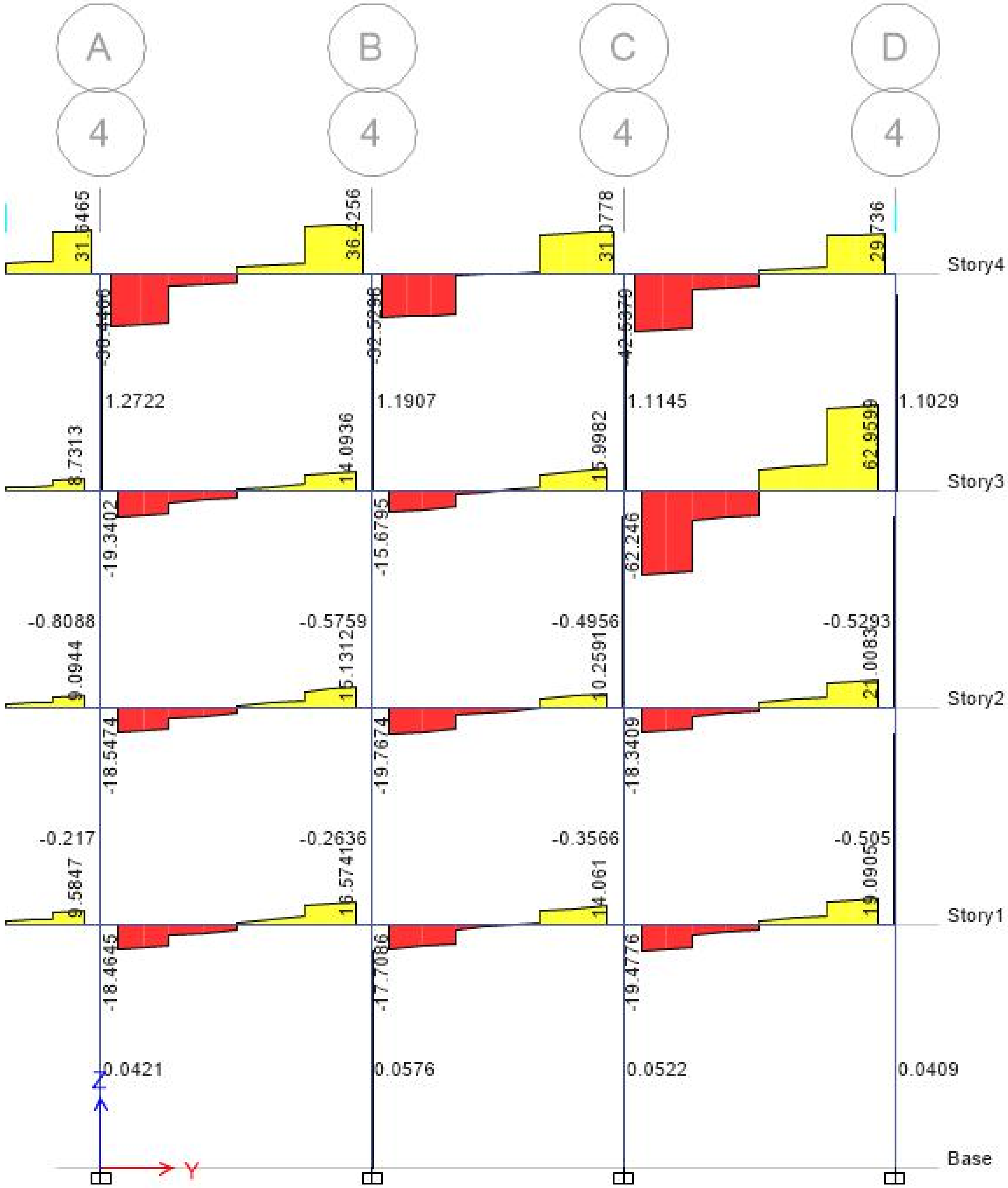
-2.7623
-2.9183
-13.2264
-13.2471
-3.431
-3.5568
-4.1388
-2.8173
-1.5218
-1.6278
-2.7291
-2.2204
-19.273
-15.8777
-39.3401
-17.8898
-18.5989
-13.748
-16.4616
-15.9306
-14.27
8.7689
14.3179
13.0879
38.6495
8.6091
14.6238
8.2005
13.325
Y
Z

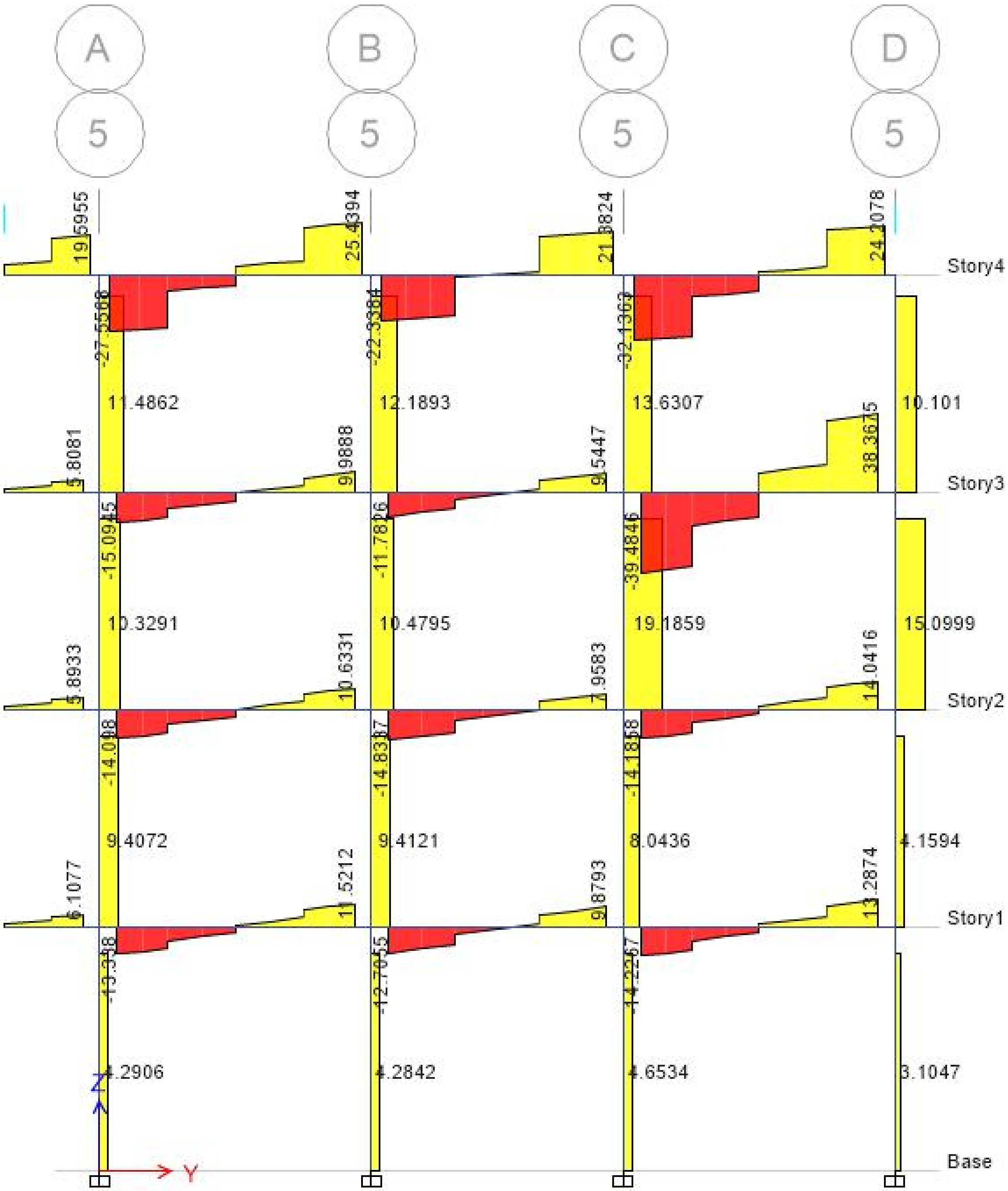
axis 3

Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	27.04	24.98	225	217.3	188.2	217.30	200
B-C	21.76	21.6	225	251.3	188.2	225.00	200
C-D	32.38	23.79	225	228.2	188.2	225.00	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	19.27	14.31	225	379.3	188.2	225.00	200
B-C	15.87	13.08	225	415.0	188.2	225.00	200
C-D	39.34	38.64	225	140.5	188.2	111.09	200
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	17.88	14.62	225	371.3	188.2	225.00	200
B-C	18.59	12.45	225	436.0	188.2	225.00	200
C-D	14.5	13.74	225	395.1	188.2	225.00	200
Story -1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	16.46	14.64	225	370.8	188.2	225.00	200
B-C	15.93	13.03	225	416.6	188.2	225.00	200
C-D	14.27	13.32	225	407.5	188.2	225.00	200


A
B
C
D
4
4
4
4
Story4
Story3
Story2
Story1
Base
1.2722
1.1907
1.1145
1.1029
-0.8088
-0.5759
-0.4956
-0.5293
-0.217
-0.2636
-0.3566
-0.505
0.0421
0.0576
0.0522
0.0409
8.7313
14.0936
15.9982
62.9599
-19.3402
-15.6795
-62.246
9.0944
15.1312
10.2591
21.0083
-18.5474
-19.7674
-18.3409
9.5847
16.5741
14.061
19.0905
-18.4645
-17.7086
-19.4776
Z
Y

axis 4

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
Story -4							
A-B	36.42	30.44	225	178.3	188.2	178.32	200
B-C	31.07	32.52	225	166.9	188.2	166.92	200
C-D	42.53	29.73	225	182.6	188.2	182.58	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	19.34	14.09	225	385.2	188.2	225.00	200
B-C	15.99	15.67	225	346.4	188.2	225.00	200
C-D	62.95	62.24	225	87.2	188.2	111.09	200
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	18.54	15.13	225	358.8	188.2	225.00	200
B-C	19.76	10.26	225	529.1	188.2	225.00	200
C-D	21	18.34	225	296.0	188.2	225.00	200
Story -1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	18.46	16.57	225	327.6	188.2	225.00	200
B-C	17.71	14.06	225	386.1	188.2	225.00	200
C-D	19.47	19.09	225	284.3	188.2	225.00	200


A
B
C
D
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5
5
5
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25.4394
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24.2078
Story4
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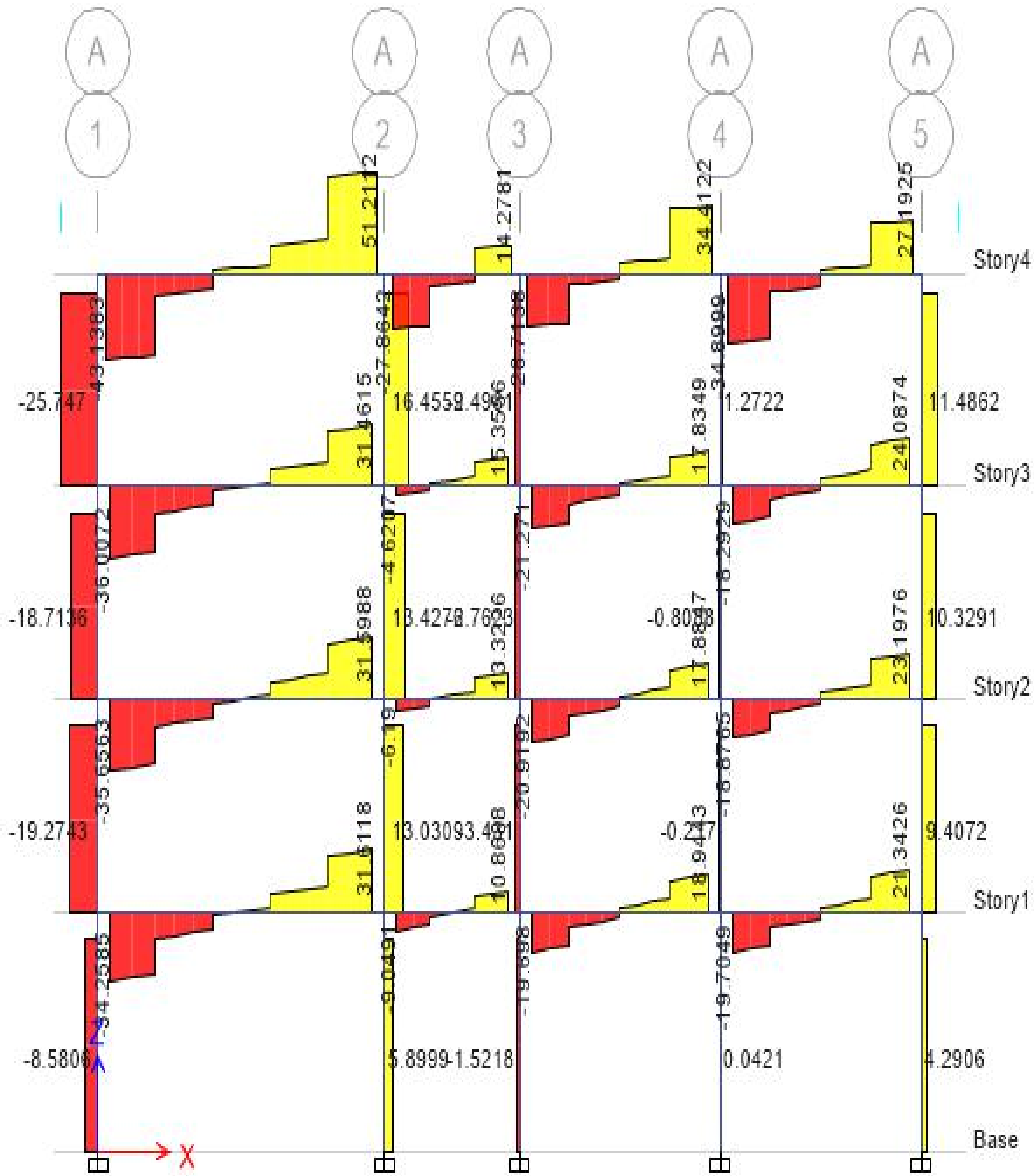
-22.3384
-32.1363
11.4862
12.1893
13.6307
10.101
5.8081
9.9888
9.5447
38.3675
Story3
-15.0945
-11.7806
-39.4846
10.3291
10.4795
19.1859
15.0999
5.8933
10.6331
7.9583
14.0416
Story2
-14.008
-14.8387
-14.1858
9.4072
9.4121
8.0436
4.1594
6.1077
11.5212
9.8793
13.2874
Story1
-13.398
-12.7055
-14.2267
4.2906
4.2842
4.6534
3.1047
Z
Y
Base

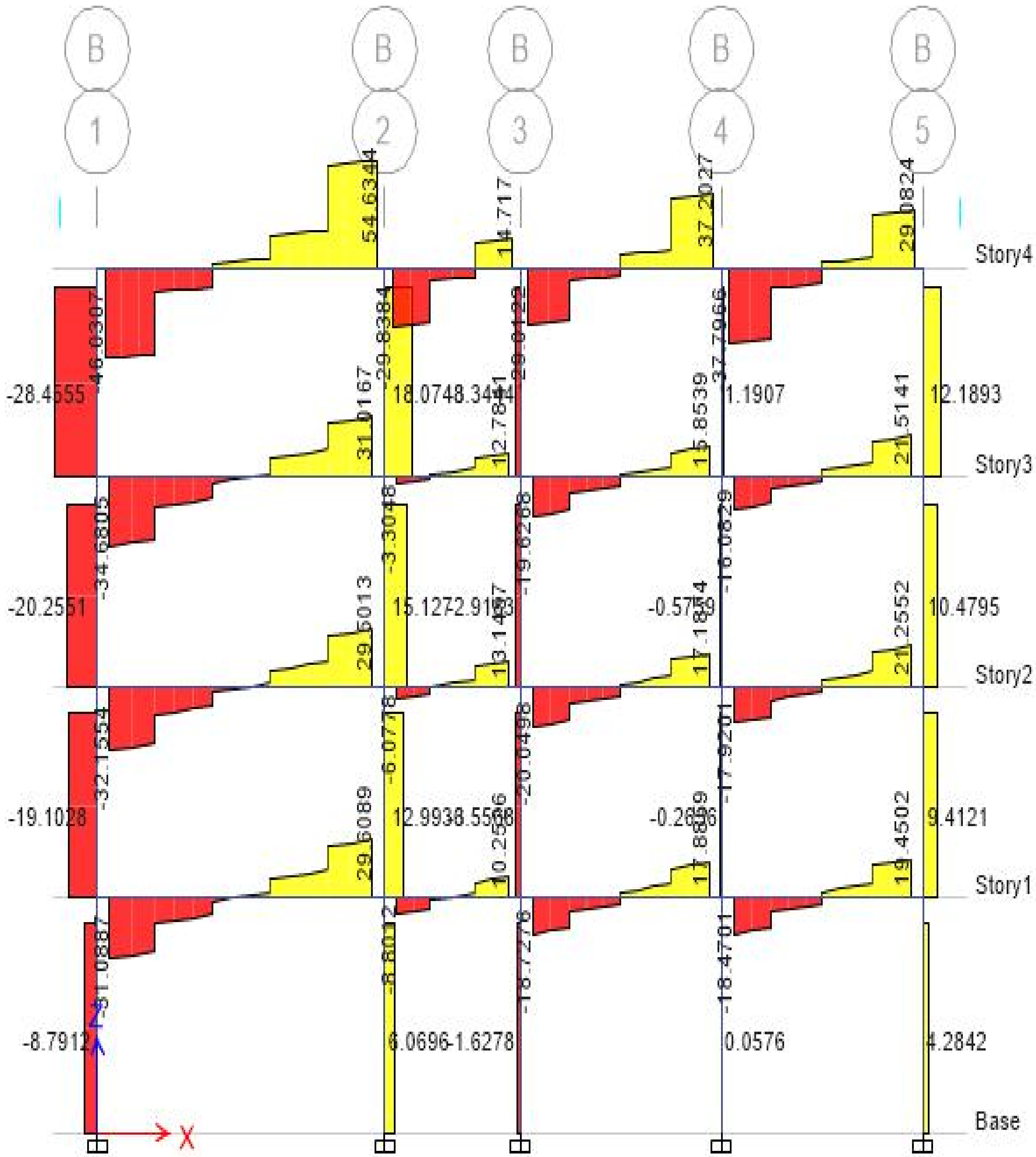
axis 5

Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	27.55	25.43	225	213.5	188.2	213.45	200
B-C	22.33	21.88	225	248.1	188.2	225.00	200
C-D	13.63	24.2	225	224.3	188.2	224.30	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	15.09	9.98	225	543.9	188.2	225.00	200
B-C	11.78	9.54	225	569.0	188.2	225.00	200
C-D	39.48	38.36	225	141.5	188.2	111.09	200
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	14.09	10.63	225	510.6	188.2	225.00	200
B-C	14.83	7.95	225	682.8	188.2	225.00	200
C-D	14.18	14.04	225	386.6	188.2	225.00	200
Story -1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
A-B	11.52	10.08	225	538.5	188.2	225.00	200
B-C	12.7	9.87	225	550.0	188.2	225.00	200
C-D	14.22	13.28	225	408.7	188.2	225.00	200


A
1
2
3
4
5
Story4
Story3
Story2
Story1
Base
-25.747
-18.7136
-19.2743
-8.5806
11.4862
10.3291
9.4072
4.2906
1.2722
0.0421
5.8999-1.5218
X

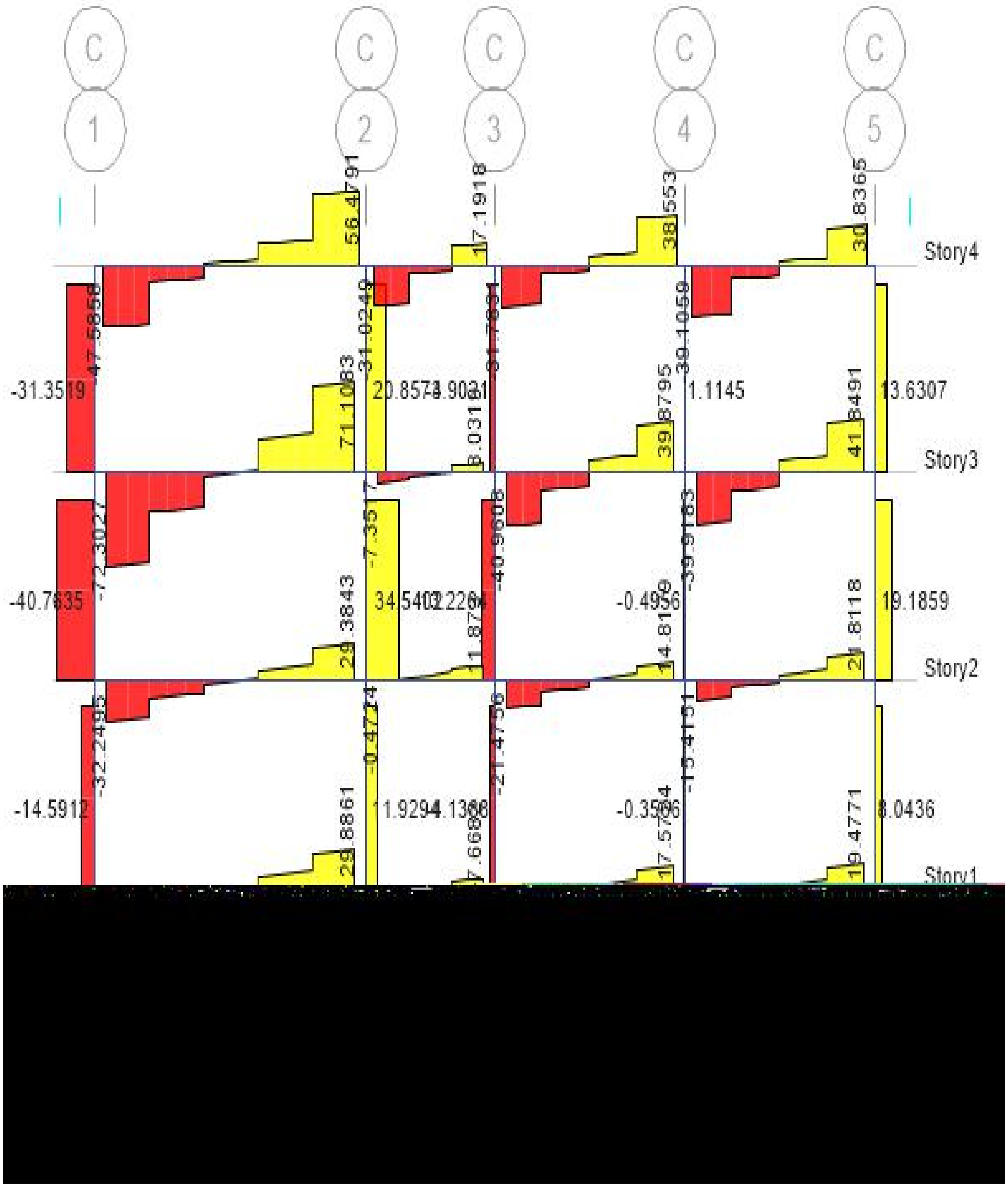
axis A

Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	51.21	43.13	225	125.9	188.2	125.85	200
2-3	27.86	14.27	225	380.4	188.2	225.00	200
3-4	34.41	20.71	225	262.1	188.2	225.00	200
4-5	34.89	27.19	225	199.6	188.2	199.64	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	36	31.46	225	172.5	188.2	172.54	200
2-3	15.35	4.62	225	1174.9	188.2	225.00	200
3-4	21.27	17.83	225	304.4	188.2	225.00	200
4-5	24.08	18.29	225	296.8	188.2	225.00	200
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	35.65	31.59	225	171.8	188.2	171.83	200
2-3	13.34	6.19	225	876.9	188.2	225.00	200
3-4	20.19	17.88	225	303.6	188.2	225.00	200
4-5	23.19	18.87	225	287.7	188.2	225.00	200
Story -1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	34.25	31.51	225	172.3	188.2	172.27	200
2-3	10.86	9.04	225	600.5	188.2	225.00	200
3-4	19.69	18.94	225	286.6	188.2	225.00	200
4-5	21.34	19.7	225	275.5	188.2	225.00	200


B
1
B
2
B
3
B
4
B
5
Story4
Story3
Story2
Story1
Base
-28.4555
12.1893
-20.2551
10.4795
-19.1028
9.4121
-8.7912
4.2842
1.1907
0.0576
X

axis B

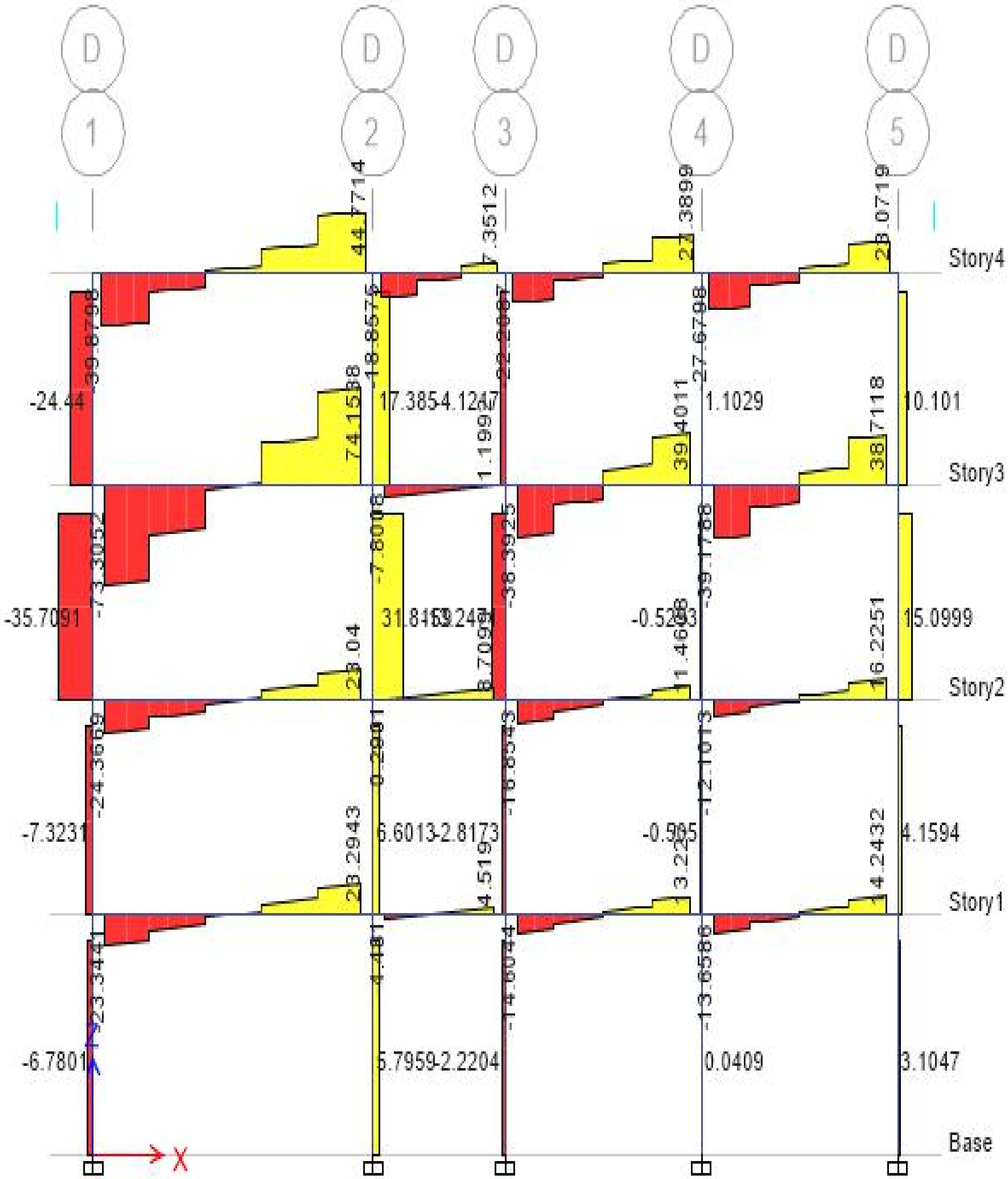
Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	63	46.03	225	117.9	188.2	117.92	200
2-3	29.83	14.71	225	369.0	188.2	225.00	200
3-4	37.2	20.01	225	271.3	188.2	225.00	200
4-5	37.7	29.08	225	186.7	188.2	186.66	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	34.68	31.01	225	175.0	188.2	175.04	200
2-3	12.78	3.3	225	1644.9	188.2	225.00	200
3-4	19.82	15.85	225	342.5	188.2	225.00	200
4-5	21.51	10.08	225	538.5	188.2	225.00	200
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	32.15	29.5	225	184.0	188.2	184.00	200
2-3	13.14	6.07	225	894.2	188.2	225.00	200
3-4	20.05	17.18	225	316.0	188.2	225.00	200
4-5	21.25	17.92	225	302.9	188.2	225.00	200
Story -1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	31.08	29.5	225	184.0	188.2	184.00	200
2-3	10.25	8.8	225	616.8	188.2	225.00	200
3-4	18.72	17.88	225	303.6	188.2	225.00	200
4-5	19.45	18.47	225	293.9	188.2	225.00	200



C
1
2
3
4
5
Story4
Story3
Story2
Story1
-31.3519
1.1145
13.6307
-40.7635
-0.4956
19.1859
-14.5912
-0.3506
8.0436

axis C

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	56.47	47.58	225	114.1	188.2	114.08	200
2-3	31.02	17.19	225	315.8	188.2	225.00	200
3-4	38.53	31.78	225	170.8	188.2	170.80	200
4-5	39.1	30.83	225	176.1	188.2	176.06	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	72.3	71.1	225	76.3	188.2	76.34	200
2-3	8.03	7.35	225	738.5	188.2	225.00	200
3-4	40.98	39.87	225	136.1	188.2	136.14	200
4-5	41.84	39.91	225	136.0	188.2	136.01	200
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	32.24	29.38	225	184.8	188.2	184.75	200
2-3	11.87	0.47	225	11549.1	188.2	225.00	200
3-4	21.47	14.81	225	366.5	188.2	225.00	200
4-5	21.81	13.41	225	404.8	188.2	225.00	200
Story -1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	30.69	29.88	225	181.7	188.2	181.66	200
2-3	7.66	6.07	225	894.2	188.2	225.00	200
3-4	19.34	17.57	225	308.9	188.2	225.00	200
4-5	19.47	18.04	225	300.9	188.2	225.00	200


D
1
2
3
4
5
Story4
Story3
Story2
Story1
Base
X
-24.44
17.385
1.1029
10.101
-35.7091
15.0999
-7.3231
4.1594
-6.7801
0.0409
3.1047

axis D

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	44.77	39.87	225	136.1	188.2	136.14	200
2-3	18.85	7.35	225	738.5	188.2	225.00	200
3-4	27.38	22.2	225	244.5	188.2	225.00	200
4-5	28.07	27.67	225	196.2	188.2	196.17	200
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	74.15	73.3	225	74.1	188.2	74.05	200
2-3	7.8	1.2	225	4523.4	188.2	225.00	200
3-4	39.4	38.39	225	141.4	188.2	141.39	200
4-5	39.17	38.71	225	140.2	188.2	140.22	200
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	24.36	233.04	225	23.3	188.2	23.29	200
2-3	8.7	0.29	225	18717.5	188.2	225.00	200
3-4	16.85	11.46	225	473.7	188.2	225.00	200
4-5	16.22	12.1	225	448.6	188.2	225.00	200
Story -1							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	23.34	23.29	225	233.1	188.2	225.00	200
2-3	4.51	1.48	225	3667.6	188.2	225.00	200
3-4	14.6	13.22	225	410.6	188.2	225.00	200
4-5	14.24	13.65	225	397.7	188.2	225.00	200

Appendix G: Column design parameter

Table 36column design parameter checks

location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
Roof	C1	300	300	300	300	2800	0	5850	4000	0	2.09	2.09	1.43	1.43	2551.90	2464.64
Roof	C2	300	300	300	300	2800	5850	2750	4000	0	0.67	0.67	1.43	1.43	2236.54	2464.64
Roof	C3	300	300	300	300	2800	2750	4100	4000	0	0.59	0.59	1.43	1.43	2192.98	2464.64
Roof	C4	300	300	300	300	2800	4100	4100	4000	0	0.73	0.73	1.43	1.43	2267.07	2464.64
Roof	C5	300	300	300	300	2800	4100	0	4000	0	1.46	1.46	1.43	1.43	2470.90	2464.64
Roof	C6	300	300	300	300	2800	4100	0	3700	4000	1.46	1.46	0.69	0.69	2470.90	2245.65
Roof	C7	300	300	300	300	2800	4100	0	4000	3700	1.46	1.46	0.69	0.69	2470.90	2245.65
Roof	C8	300	300	300	300	2800	4100	0	1200	4000	1.46	1.46	0.33	0.33	2470.90	1991.97
Roof	C9	300	300	300	300	2800	4100	4100	1200	4000	0.73	0.73	0.33	0.33	2267.07	1991.97
Roof	C11	300	300	300	300	2800	2750	4100	1200	4000	0.59	0.59	0.33	0.33	2192.98	1991.97
Roof	C12	300	300	300	300	2800	2750	4100	4000	3700	0.59	0.59	0.69	0.69	2192.98	2245.65
Roof	C13	300	300	300	300	2800	2750	4100	3700	4000	0.59	0.59	0.69	0.69	2192.98	2245.65
Roof	C14	300	300	300	300	2800	5850	2750	3700	4000	0.67	0.67	0.69	0.69	2236.54	2245.65
Roof	C15	300	300	300	300	2800	5850	2750	4000	3700	0.67	0.67	0.69	0.69	2236.54	2245.65
Roof	C16	300	300	300	300	2800	0	5850	4000	3700	2.09	2.09	0.69	0.69	2551.90	2245.65
Roof	C17	300	300	300	300	2800	0	5850	3700	4000	2.09	2.09	0.69	0.69	2551.90	2245.65
Roof	C18	300	300	300	300	2800	0	5850	1200	4000	2.09	2.09	0.33	0.33	2551.90	1991.97
Roof	C19	300	300	300	300	2800	5850	2750	1200	4000	0.67	0.67	0.33	0.33	2236.54	1991.97
Roof	C20	300	300	300	300	2800	4100	4100	4000	3700	0.73	0.73	0.69	0.69	2267.07	2245.65
Roof	C21	300	300	300	300	2800	4100	4100	3700	4000	0.73	0.73	0.69	0.69	2267.07	2245.65

locatio	colum	Mxtop	Mxbot	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slende	slende
Roof	C1	7.666	22.84	15.44	13.27	257	12.81	18.41	27.98	20.58	28.13	28.13	38.11	24.71	short	slender
Roof	C2	7.742	65.79	7.304	10.09	381.8	15.38	14.94	73.42	17.72	26.08	28.13	37.51	21.57	short	slender
Roof	C3	7.861	71.8	4.815	18.88	214.7	12.15	9.11	76.09	23.18	25.78	28.13	51.69	43.87	short	short
Roof	C4	7.157	50.2	7.846	35.14	336.3	13.88	14.57	56.93	41.86	27.71	28.13	39.05	36.25	short	short
Roof	C5	1.878	11.88	6.236	46.17	196.7	5.81	10.17	15.81	50.10	27.71	26.32	46.72	52.49	short	short
Roof	C6	0.911	10.27	1.591	16.87	213.5	5.18	5.86	14.54	21.14	21.71	21.92	60.30	63.85	short	short
Roof	C7	1.42	19.76	6.518	9.318	231.8	6.06	11.15	24.40	13.95	21.82	21.92	62.52	38.79	short	short
Roof	C8	3.18	36.13	17	32.76	106	5.30	19.12	38.25	34.88	28.13	26.32	74.58	55.01	short	short
Roof	C9	7.158	6.86	13.39	40.25	174.5	10.35	16.88	10.65	43.74	28.13	26.46	27.10	48.92	slender	short
Roof	C11	9.037	9.105	1.593	14.68	113	11.30	3.85	11.37	16.94	21.71	21.97	43.55	90.83	short	short
Roof	C12	9.957	6.256	3.364	45.42	255.9	11.37	8.48	15.07	50.54	27.71	26.46	29.07	47.10	short	short
Roof	C13	15.33	25.82	4.328	34.86	234.8	20.03	9.02	30.52	39.56	27.71	28.31	33.50	47.24	short	short
Roof	C14	12.21	33.36	3.903	15.3	404.5	20.30	11.99	41.45	23.39	26.40	28.31	29.59	29.03	short	short
Roof	C15	12.27	34.35	5.431	2.049	426.2	20.79	10.57	42.88	13.95	26.40	28.31	28.95	22.45	short	slender
Roof	C16	12.83	26.65	12.61	28.7	297.3	18.78	18.55	32.59	34.65	28.36	28.31	32.06	33.22	short	short
Roof	C17	12.44	51.77	23	14.79	278.4	18.00	20.36	57.34	28.57	27.71	26.46	40.85	29.10	short	short
Roof	C18	42.43	94.28	12.01	8.979	139.1	45.21	11.76	97.06	14.79	27.71	28.31	51.47	37.74	short	short
Roof	C19	43.04	89.68	13.37	38.02	196.4	46.97	17.30	93.61	41.95	26.40	28.31	42.05	45.19	short	short
Roof	C20	24.91	44.66	24.19	48.75	380.4	32.52	31.79	52.27	56.36	26.40	28.31	27.18	28.64	short	short
Roof	C21	0.866	28.42	30.4	69.96	362.7	8.12	37.65	35.68	77.21	28.36	28.31	38.02	31.30	short	short

colmn	location	MED:	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal	shear reinforcement	
Roof	C1	23.28	15.88	0.0	0.05	0.08	0.2	689.83	180	3600	689.828	20	2.20	4ø20	ø 10 c/c	300
Roof	C2	66.24	10.54	0.0	0.03	0.22	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
Roof	C3	72.24	19.32	0.0	0.06	0.24	0.8	2759.3	180	3600	2759.31	20	8.79	10ø20	ø 10 c/c	300
Roof	C4	50.63	35.57	0.0	0.12	0.17	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
Roof	C5	12.34	46.63	0.0	0.15	0.04	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C6	11.05	17.65	0.0	0.02	0.02	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
Roof	C7	20.57	10.13	0.0	0.01	0.03	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
Roof	C8	36.68	33.31	0.0	0.11	0.12	0.5	1724.6	180	3600	1724.57	20	5.49	6ø20	ø 10 c/c	300
Roof	C9	7.62	40.71	0.0	0.13	0.02	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C10	10.65	10.23	0.0	0.01	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
Roof	C11	9.89	15.47	0.0	0.02	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
Roof	C12	10.38	45.85	0.0	0.15	0.03	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C13	26.28	35.33	0.0	0.12	0.09	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C14	33.79	15.73	0.0	0.05	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C15	34.79	5.87	0.0	0.02	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C16	27.07	29.12	0.0	0.10	0.09	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300
Roof	C17	54.27	25.50	0.1	0.08	0.18	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C18	99.27	17.01	0.2	0.06	0.32	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C19	94.78	43.12	0.2	0.14	0.31	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C20	47.65	51.74	0.1	0.17	0.16	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
Roof	C21	33.54	75.07	0.3	0.25	0.11	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300

location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
G+3	C1	500	500	400	400	2800	0	5850	4000	0	5.10	5.10	3.49	3.49	2686.50	2640.01
G+3	C2	500	500	400	400	2800	5850	2750	4000	0	1.63	1.63	3.49	3.49	2497.27	2640.01
G+3	C3	500	500	400	400	2800	2750	4100	4000	0	1.44	1.44	3.49	3.49	2465.82	2640.01
G+3	C4	500	500	400	400	2800	4100	4100	4000	0	1.79	1.79	3.49	3.49	2518.43	2640.01
G+3	C5	500	500	400	400	2800	4100	0	4000	0	3.57	3.57	3.49	3.49	2643.48	2640.01
G+3	C6	500	500	400	400	2800	4100	0	3700	4000	3.57	3.57	1.68	1.68	2643.48	2503.66
G+3	C7	500	500	400	400	2800	4100	0	4000	3700	3.57	3.57	1.68	1.68	2643.48	2503.66
G+3	C8	500	500	400	400	2800	4100	0	1200	4000	3.57	3.57	0.80	0.80	2643.48	2297.95
G+3	C9	500	500	400	400	2800	4100	4100	1200	4000	1.79	1.79	0.80	0.80	2518.43	2297.95
G+3	C11	500	500	400	400	2800	2750	4100	1200	4000	1.44	1.44	0.80	0.80	2465.82	2297.95
G+3	C12	500	500	400	400	2800	2750	4100	4000	3700	1.44	1.44	1.68	1.68	2465.82	2503.66
G+3	C13	500	500	400	400	2800	2750	4100	3700	4000	1.44	1.44	1.68	1.68	2465.82	2503.66
G+3	C14	500	500	400	400	2800	5850	2750	3700	4000	1.63	1.63	1.68	1.68	2497.27	2503.66
G+3	C15	500	500	400	400	2800	5850	2750	4000	3700	1.63	1.63	1.68	1.68	2497.27	2503.66
G+3	C16	500	500	400	400	2800	0	5850	4000	3700	5.10	5.10	1.68	1.68	2686.50	2503.66
G+3	C17	500	500	400	400	2800	0	5850	3700	4000	5.10	5.10	1.68	1.68	2686.50	2503.66
G+3	C18	500	500	400	400	2800	0	5850	1200	4000	5.10	5.10	0.80	0.80	2686.50	2297.95
G+3	C19	500	500	400	400	2800	5850	2750	1200	4000	1.63	1.63	0.80	0.80	2497.27	2297.95
G+3	C20	500	500	400	400	2800	4100	4100	4000	3700	1.79	1.79	1.68	1.68	2518.43	2503.66
G+3	C21	500	500	400	400	2800	4100	4100	3700	4000	1.79	1.79	1.68	1.68	2518.43	2503.66

locatio	**colum**	**Mxtop**	**Mxbot**	**Mytop**	**Mybot**	**NED**	**M01x**	**M01y**	**M02x**	**M02y**	**λx**	**λy**	**λxlim**	**λylim**	**slende**	**slende**
G+3	C1	8.9	33.2	15.44	13.27	257	14.04	18.41	38.34	20.58	28.13	28.13	40.91	24.71	short	slender
G+3	C2	21.3	18.3	7.304	10.09	381.8	25.94	14.94	28.94	17.72	26.08	28.13	20.23	21.57	slender	slender
G+3	C3	23.3	43.2	4.815	18.88	214.7	27.59	9.11	47.49	23.18	25.78	28.13	37.56	43.87	short	short
G+3	C4	25.6	45	7.846	35.14	336.3	32.33	14.57	51.73	41.86	27.71	28.13	28.83	36.25	short	short
G+3	C5	1.878	3	6.236	46.17	196.7	5.81	10.17	6.93	50.10	27.71	26.32	30.22	52.49	short	short
G+3	C6	0.911	10.27	1.591	16.87	213.5	5.18	5.86	14.54	21.14	21.71	21.92	60.30	63.85	short	short
G+3	C7	1.42	19.76	6.518	9.318	231.8	6.06	11.15	24.40	13.95	21.82	21.92	62.52	38.79	short	short
G+3	C8	3.18	36.13	17	32.76	106	5.30	19.12	38.25	34.88	28.13	26.32	74.58	55.01	short	short
G+3	C9	7.158	6.86	13.39	40.25	174.5	10.35	16.88	10.65	43.74	28.13	26.46	27.10	48.92	slender	short
G+3	C11	9.037	9.105	1.593	14.68	113	11.30	3.85	11.37	16.94	21.71	21.97	43.55	90.83	short	short
G+3	C12	9.957	6.256	3.364	45.42	255.9	11.37	8.48	15.07	50.54	27.71	26.46	29.07	47.10	short	short
G+3	C13	15.33	25.82	4.328	34.86	234.8	20.03	9.02	30.52	39.56	27.71	28.31	33.50	47.24	short	short
G+3	C14	12.21	33.36	3.903	15.3	404.5	20.30	11.99	41.45	23.39	26.40	28.31	29.59	29.03	short	short
G+3	C15	12.27	34.35	5.431	2.049	426.2	20.79	10.57	42.88	13.95	26.40	28.31	28.95	22.45	short	slender
G+3	C16	12.83	26.65	12.61	28.7	297.3	18.78	18.55	32.59	34.65	28.36	28.31	32.06	33.22	short	short
G+3	C17	12.44	51.77	23	14.79	278.4	18.00	20.36	57.34	28.57	27.71	26.46	40.85	29.10	short	short
G+3	C18	42.43	94.28	12.01	8.979	139.1	45.21	11.76	97.06	14.79	27.71	28.31	51.47	37.74	short	short
G+3	C19	43.04	89.68	13.37	38.02	196.4	46.97	17.30	93.61	41.95	26.40	28.31	42.05	45.19	short	short
G+3	C20	24.91	44.66	24.19	48.75	380.4	32.52	31.79	52.27	56.36	26.40	28.31	27.18	28.64	short	short
G+3	C21	0.866	28.42	30.4	69.96	362.7	8.12	37.65	35.68	77.21	28.36	28.31	38.02	31.30	short	short

colmn	location	MEDx	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal	shear reinforcement	
G+3	C1	34.20	15.88	0.0	0.05	0.11	0.2	689.83	180	3600	689.828	20	2.20	4ø20	ø 10 c/c	300
G+3	C2	32.10	10.54	0.0	0.03	0.10	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
G+3	C3	13.20	19.32	0.0	0.06	0.04	0.8	2759.3	180	3600	2759.31	20	8.79	10ø20	ø 10 c/c	300
G+3	C4	50.63	35.57	0.0	0.12	0.17	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
G+3	C5	12.34	46.63	0.0	0.15	0.04	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C6	11.05	17.65	0.0	0.02	0.02	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+3	C7	20.57	10.13	0.0	0.01	0.03	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+3	C8	36.68	33.31	0.0	0.11	0.12	0.5	1724.6	180	3600	1724.57	20	5.49	6ø20	ø 10 c/c	300
G+3	C9	7.62	40.71	0.0	0.13	0.02	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C10	10.65	10.23	0.0	0.01	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C11	9.89	15.47	0.0	0.02	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C12	10.38	45.85	0.0	0.15	0.03	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C13	26.28	35.33	0.0	0.12	0.09	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C14	33.79	15.73	0.0	0.05	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C15	34.79	5.87	0.0	0.02	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C16	27.07	29.12	0.0	0.10	0.09	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300
G+3	C17	54.27	25.50	0.1	0.08	0.18	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C18	99.27	17.01	0.2	0.06	0.32	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C19	94.78	43.12	0.2	0.14	0.31	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C20	47.65	51.74	0.1	0.17	0.16	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+3	C21	33.54	75.07	0.3	0.25	0.11	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300

location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
G+2	C1	500	500	400	400	2800	0	5850	4000	0	5.10	5.10	3.49	3.49	2686.50	2640.01
G+2	C2	500	500	400	400	2800	5850	2750	4000	0	1.63	1.63	3.49	3.49	2497.27	2640.01
G+2	C3	500	500	400	400	2800	2750	4100	4000	0	1.44	1.44	3.49	3.49	2465.82	2640.01
G+2	C4	500	500	400	400	2800	4100	4100	4000	0	1.79	1.79	3.49	3.49	2518.43	2640.01
G+2	C5	500	500	400	400	2800	4100	0	4000	0	3.57	3.57	3.49	3.49	2643.48	2640.01
G+2	C6	500	500	400	400	2800	4100	0	3700	4000	3.57	3.57	1.68	1.68	2643.48	2503.66
G+2	C7	500	500	400	400	2800	4100	0	4000	3700	3.57	3.57	1.68	1.68	2643.48	2503.66
G+2	C8	500	500	400	400	2800	4100	0	1200	4000	3.57	3.57	0.80	0.80	2643.48	2297.95
G+2	C9	500	500	400	400	2800	4100	4100	1200	4000	1.79	1.79	0.80	0.80	2518.43	2297.95
G+2	C11	500	500	400	400	2800	2750	4100	1200	4000	1.44	1.44	0.80	0.80	2465.82	2297.95
G+2	C12	500	500	400	400	2800	2750	4100	4000	3700	1.44	1.44	1.68	1.68	2465.82	2503.66
G+2	C13	500	500	400	400	2800	2750	4100	3700	4000	1.44	1.44	1.68	1.68	2465.82	2503.66
G+2	C14	500	500	400	400	2800	5850	2750	3700	4000	1.63	1.63	1.68	1.68	2497.27	2503.66
G+2	C15	500	500	400	400	2800	5850	2750	4000	3700	1.63	1.63	1.68	1.68	2497.27	2503.66
G+2	C16	500	500	400	400	2800	0	5850	4000	3700	5.10	5.10	1.68	1.68	2686.50	2503.66
G+2	C17	500	500	400	400	2800	0	5850	3700	4000	5.10	5.10	1.68	1.68	2686.50	2503.66
G+2	C18	500	500	400	400	2800	0	5850	1200	4000	5.10	5.10	0.80	0.80	2686.50	2297.95
G+2	C19	500	500	400	400	2800	5850	2750	1200	4000	1.63	1.63	0.80	0.80	2497.27	2297.95
G+2	C20	500	500	400	400	2800	4100	4100	4000	3700	1.79	1.79	1.68	1.68	2518.43	2503.66
G+2	C21	500	500	400	400	2800	4100	4100	3700	4000	1.79	1.79	1.68	1.68	2518.43	2503.66

locatio	colum	Mxtop	Mxbot	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slende	slende
G+2	C1	10.8	35.4	15.44	13.27	358.2	17.96	20.43	42.56	22.61	28.13	28.13	33.20	20.69	short	slender
G+2	C2	13.2	8.93	7.304	10.09	506.6	19.06	17.44	23.33	20.22	26.08	28.13	19.29	18.30	slender	slender
G+2	C3	26.3	23.4	4.815	18.88	335.5	30.11	11.53	33.01	25.60	25.78	28.13	21.15	33.55	slender	short
G+2	C4	25.8	45.3	7.846	35.14	458.3	34.97	17.01	54.47	44.30	27.71	28.13	24.30	30.23	slender	short
G+2	C5	3.58	11.88	6.236	46.17	277	9.12	11.78	17.42	51.71	27.71	26.32	34.76	43.50	short	short
G+2	C6	4.58	10.27	1.591	16.87	291.2	10.40	7.41	16.09	22.70	21.71	21.92	40.48	52.77	short	short
G+2	C7	4.32	19.76	6.518	9.318	375.5	11.83	14.03	27.27	16.83	21.82	21.92	42.85	29.32	short	short
G+2	C8	3.18	36.13	17	32.76	219.1	7.56	21.38	40.51	37.14	28.13	26.32	50.28	37.35	short	short
G+2	C9	7.158	6.86	13.39	40.25	388	14.62	21.15	14.92	48.01	28.13	26.46	17.97	31.44	slender	short
G+2	C11	9.037	9.105	1.593	14.68	255	14.14	6.69	14.21	19.78	21.71	21.97	28.94	55.91	short	short
G+2	C12	9.957	6.256	3.364	45.42	427.3	14.80	11.91	18.50	53.97	27.71	26.46	21.41	35.19	slender	short
G+2	C13	15.33	25.82	4.328	34.86	349.8	22.33	11.32	32.82	41.86	27.71	28.31	26.81	37.59	slender	short
G+2	C14	12.21	33.36	3.903	15.3	526.5	22.74	14.43	43.89	25.83	26.40	28.31	25.33	24.46	slender	slender
G+2	C15	12.27	34.35	5.431	2.049	669	25.65	15.43	47.73	18.81	26.40	28.31	22.11	16.73	slender	slender
G+2	C16	12.83	26.65	12.61	28.7	501	22.85	22.62	36.67	38.72	28.36	28.31	23.66	24.51	slender	slender
G+2	C17	12.44	51.77	23	14.79	375.9	19.95	22.31	59.29	30.52	27.71	26.46	34.58	24.58	short	slender
G+2	C18	42.43	94.28	12.01	8.979	306.3	48.55	15.10	100.40	18.14	27.71	28.31	34.18	24.36	short	slender
G+2	C19	43.04	89.68	13.37	38.02	390.5	50.85	21.18	97.49	45.83	26.40	28.31	29.32	30.81	short	short
G+2	C20	24.91	44.66	24.19	48.75	644	37.79	37.07	57.54	61.63	26.40	28.31	20.22	21.29	slender	slender
G+2	C21	0.866	28.42	30.4	69.96	477	10.41	39.94	37.96	79.50	28.36	28.31	32.11	26.96	short	slender

colmn	location	MED:	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal	shear reinforcment	
G+2	C1	23.28	15.88	0.0	0.05	0.08	0.2	689.83	180	3600	689.828	20	2.20	4ø20	ø 10 c/c	300
G+2	C2	66.24	10.54	0.0	0.03	0.22	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
G+2	C3	72.24	19.32	0.0	0.06	0.24	0.8	2759.3	180	3600	2759.31	20	8.79	10ø20	ø 10 c/c	300
G+2	C4	50.63	35.57	0.0	0.12	0.17	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
G+2	C5	12.34	46.63	0.0	0.15	0.04	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C6	11.05	17.65	0.0	0.02	0.02	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+2	C7	20.57	10.13	0.0	0.01	0.03	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+2	C8	36.68	33.31	0.0	0.11	0.12	0.5	1724.6	180	3600	1724.57	20	5.49	6ø20	ø 10 c/c	300
G+2	C9	7.62	40.71	0.0	0.13	0.02	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C10	10.65	10.23	0.0	0.01	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C11	9.89	15.47	0.0	0.02	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C12	10.38	45.85	0.0	0.15	0.03	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C13	26.28	35.33	0.0	0.12	0.09	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C14	33.79	15.73	0.0	0.05	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C15	34.79	5.87	0.0	0.02	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C16	27.07	29.12	0.0	0.10	0.09	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300
G+2	C17	54.27	25.50	0.1	0.08	0.18	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C18	99.27	17.01	0.2	0.06	0.32	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C19	94.78	43.12	0.2	0.14	0.31	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C20	47.65	51.74	0.1	0.17	0.16	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+2	C21	33.54	75.07	0.3	0.25	0.11	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300

location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
G+1	C1	500	500	400	400	2800	0	5850	4000	0	5.10	5.10	3.49	3.49	2686.50	2640.01
G+1	C2	500	500	400	400	2800	5850	2750	4000	0	1.63	1.63	3.49	3.49	2497.27	2640.01
G+1	C3	500	500	400	400	2800	2750	4100	4000	0	1.44	1.44	3.49	3.49	2465.82	2640.01
G+1	C4	500	500	400	400	2800	4100	4100	4000	0	1.79	1.79	3.49	3.49	2518.43	2640.01
G+1	C5	500	500	400	400	2800	4100	0	4000	0	3.57	3.57	3.49	3.49	2643.48	2640.01
G+1	C6	500	500	400	400	2800	4100	0	3700	4000	3.57	3.57	1.68	1.68	2643.48	2503.66
G+1	C7	500	500	400	400	2800	4100	0	4000	3700	3.57	3.57	1.68	1.68	2643.48	2503.66
G+1	C8	500	500	400	400	2800	4100	0	1200	4000	3.57	3.57	0.80	0.80	2643.48	2297.95
G+1	C9	500	500	400	400	2800	4100	4100	1200	4000	1.79	1.79	0.80	0.80	2518.43	2297.95
G+1	C11	500	500	400	400	2800	2750	4100	1200	4000	1.44	1.44	0.80	0.80	2465.82	2297.95
G+1	C12	500	500	400	400	2800	2750	4100	4000	3700	1.44	1.44	1.68	1.68	2465.82	2503.66
G+1	C13	500	500	400	400	2800	2750	4100	3700	4000	1.44	1.44	1.68	1.68	2465.82	2503.66
G+1	C14	500	500	400	400	2800	5850	2750	3700	4000	1.63	1.63	1.68	1.68	2497.27	2503.66
G+1	C15	500	500	400	400	2800	5850	2750	4000	3700	1.63	1.63	1.68	1.68	2497.27	2503.66
G+1	C16	500	500	400	400	2800	0	5850	4000	3700	5.10	5.10	1.68	1.68	2686.50	2503.66
G+1	C17	500	500	400	400	2800	0	5850	3700	4000	5.10	5.10	1.68	1.68	2686.50	2503.66
G+1	C18	500	500	400	400	2800	0	5850	1200	4000	5.10	5.10	0.80	0.80	2686.50	2297.95
G+1	C19	500	500	400	400	2800	5850	2750	1200	4000	1.63	1.63	0.80	0.80	2497.27	2297.95
G+1	C20	500	500	400	400	2800	4100	4100	4000	3700	1.79	1.79	1.68	1.68	2518.43	2503.66
G+1	C21	500	500	400	400	2800	4100	4100	3700	4000	1.79	1.79	1.68	1.68	2518.43	2503.66

locatio	colum	Mxtop	Mxbot	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slende	slende
G+1	C1	3.3	25.6	15.44	13.27	458.2	12.46	22.43	34.76	24.61	28.13	28.13	30.82	18.11	short	slender
G+1	C2	21.3	32.1	7.304	10.09	634.9	34.00	20.00	44.80	22.78	26.08	28.13	18.37	16.05	slender	slender
G+1	C3	24.5	4.2	4.815	18.88	452.4	13.25	13.86	33.55	27.93	25.78	28.13	30.17	27.83	short	slender
G+1	C4	43.2	31	7.846	35.14	580.2	42.60	19.45	54.80	46.74	27.71	28.13	18.84	26.21	slender	slender
G+1	C5	2.8	11.88	6.236	46.17	355	9.90	13.34	18.98	53.27	27.71	26.32	30.75	37.84	short	short
G+1	C6	4.88	10.27	1.591	16.87	373.1	12.34	9.05	17.73	24.34	21.71	21.92	34.08	45.08	short	short
G+1	C7	8.9	19.76	6.518	9.318	453.1	17.96	15.58	28.82	18.38	21.82	21.92	33.17	26.25	short	short
G+1	C8	3.18	36.13	17	270.6	106	5.30	19.12	38.25	272.72	28.13	26.32	74.58	77.85	short	short
G+1	C9	7.158	6.86	13.39	40.25	340.6	13.67	20.21	13.97	47.06	28.13	26.46	19.22	33.86	slender	short
G+1	C11	9.037	9.105	1.593	14.68	582.3	20.68	13.24	20.75	26.33	21.71	21.97	19.11	32.53	slender	short
G+1	C12	9.957	6.256	3.364	45.42	582.2	17.90	15.01	21.60	57.06	27.71	26.46	17.76	29.29	slender	short
G+1	C13	15.33	25.82	4.328	34.86	470.1	24.74	13.73	35.22	44.26	27.71	28.31	22.63	31.52	slender	short
G+1	C14	12.21	33.36	3.903	15.3	656.7	25.34	17.04	46.49	28.43	26.40	28.31	22.16	21.12	slender	slender
G+1	C15	12.27	34.35	5.431	2.049	775.7	27.78	17.56	49.87	20.95	26.40	28.31	20.18	15.21	slender	slender
G+1	C16	12.83	26.65	12.61	28.7	598.3	24.80	24.57	38.61	40.67	28.36	28.31	21.27	22.03	slender	slender
G+1	C17	12.44	51.77	23	14.79	479.1	22.02	24.38	61.35	32.58	27.71	26.46	30.13	21.38	short	slender
G+1	C18	42.43	94.28	12.01	8.979	375.5	49.94	16.49	101.79	19.52	27.71	28.31	30.69	21.70	short	slender
G+1	C19	43.04	89.68	13.37	38.02	479.2	52.63	22.95	99.27	47.61	26.40	28.31	26.28	27.36	slender	slender
G+1	C20	24.91	44.66	24.19	48.75	754.8	40.01	39.28	59.76	63.85	26.40	28.31	18.45	19.42	slender	slender
G+1	C21	0.866	28.42	30.4	69.96	603.9	12.94	42.48	40.50	82.03	28.36	28.31	27.62	23.66	slender	slender

colmn	location	MED:	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal	shear reinforcment	
G+1	C1	23.28	15.88	0.0	0.05	0.08	0.2	689.83	180	3600	689.828	20	2.20	4ø20	ø 10 c/c	300
G+1	C2	66.24	10.54	0.0	0.03	0.22	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
G+1	C3	72.24	19.32	0.0	0.06	0.24	0.8	2759.3	180	3600	2759.31	20	8.79	10ø20	ø 10 c/c	300
G+1	C4	50.63	35.57	0.0	0.12	0.17	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
G+1	C5	12.34	46.63	0.0	0.15	0.04	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C6	11.05	17.65	0.0	0.02	0.02	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+1	C7	20.57	10.13	0.0	0.01	0.03	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+1	C8	36.68	33.31	0.0	0.11	0.12	0.5	1724.6	180	3600	1724.57	20	5.49	6ø20	ø 10 c/c	300
G+1	C9	7.62	40.71	0.0	0.13	0.02	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C10	10.65	10.23	0.0	0.01	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+1	C11	9.89	15.47	0.0	0.02	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+1	C12	10.38	45.85	0.0	0.15	0.03	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C13	26.28	35.33	0.0	0.12	0.09	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C14	33.79	15.73	0.0	0.05	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C15	34.79	5.87	0.0	0.02	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C16	27.07	29.12	0.0	0.10	0.09	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300
G+1	C17	54.27	25.50	0.1	0.08	0.18	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C18	99.27	17.01	0.2	0.06	0.32	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C19	94.78	43.12	0.2	0.14	0.31	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C20	47.65	51.74	0.1	0.17	0.16	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+1	C21	33.54	75.07	0.3	0.25	0.11	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300

location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
G+0	C1	500	500	400	400	2800	0	5850	4000	0	5.10	5.10	3.49	3.49	2686.50	2640.01
G+0	C2	500	500	400	400	2800	5850	2750	4000	0	1.63	1.63	3.49	3.49	2497.27	2640.01
G+0	C3	500	500	400	400	2800	2750	4100	4000	0	1.44	1.44	3.49	3.49	2465.82	2640.01
G+0	C4	500	500	400	400	2800	4100	4100	4000	0	1.79	1.79	3.49	3.49	2518.43	2640.01
G+0	C5	500	500	400	400	2800	4100	0	4000	0	3.57	3.57	3.49	3.49	2643.48	2640.01
G+0	C6	500	500	400	400	2800	4100	0	3700	4000	3.57	3.57	1.68	1.68	2643.48	2503.66
G+0	C7	500	500	400	400	2800	4100	0	4000	3700	3.57	3.57	1.68	1.68	2643.48	2503.66
G+0	C8	500	500	400	400	2800	4100	0	1200	4000	3.57	3.57	0.80	0.80	2643.48	2297.95
G+0	C9	500	500	400	400	2800	4100	4100	1200	4000	1.79	1.79	0.80	0.80	2518.43	2297.95
G+0	C11	500	500	400	400	2800	2750	4100	1200	4000	1.44	1.44	0.80	0.80	2465.82	2297.95
G+0	C12	500	500	400	400	2800	2750	4100	4000	3700	1.44	1.44	1.68	1.68	2465.82	2503.66
G+0	C13	500	500	400	400	2800	2750	4100	3700	4000	1.44	1.44	1.68	1.68	2465.82	2503.66
G+0	C14	500	500	400	400	2800	5850	2750	3700	4000	1.63	1.63	1.68	1.68	2497.27	2503.66
G+0	C15	500	500	400	400	2800	5850	2750	4000	3700	1.63	1.63	1.68	1.68	2497.27	2503.66
G+0	C16	500	500	400	400	2800	0	5850	4000	3700	5.10	5.10	1.68	1.68	2686.50	2503.66
G+0	C17	500	500	400	400	2800	0	5850	3700	4000	5.10	5.10	1.68	1.68	2686.50	2503.66
G+0	C18	500	500	400	400	2800	0	5850	1200	4000	5.10	5.10	0.80	0.80	2686.50	2297.95
G+0	C19	500	500	400	400	2800	5850	2750	1200	4000	1.63	1.63	0.80	0.80	2497.27	2297.95
G+0	C20	500	500	400	400	2800	4100	4100	4000	3700	1.79	1.79	1.68	1.68	2518.43	2503.66
G+0	C21	500	500	400	400	2800	4100	4100	3700	4000	1.79	1.79	1.68	1.68	2518.43	2503.66

locatio	colum	Mxtop	Mxbot	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slende	slende
G+0	C1	13.4	9.2	15.44	13.27	558.8	20.38	24.44	24.58	26.62	28.13	28.13	18.12	16.26	slender	slender
G+0	C2	34.2	24.8	7.304	10.09	770.7	40.21	22.72	49.61	25.50	26.08	28.13	15.76	14.33	slender	slender
G+0	C3	34.8	18.9	4.815	18.88	562.1	30.14	16.06	46.04	30.13	25.78	28.13	21.68	24.21	slender	slender
G+0	C4	7.157	28.2	7.846	35.14	708.9	21.34	22.02	42.38	49.31	27.71	28.13	22.10	23.15	slender	slender
G+0	6.78	42.3	11.88	6.236	46.17	432.6	20.53	14.89	50.95	54.82	27.71	26.32	30.67	33.77	short	short
G+0	C6	0.911	10.27	1.591	16.87	453.9	9.99	10.67	19.35	25.95	21.71	21.92	36.43	39.67	short	short
G+0	C7	1.42	19.76	6.518	9.318	533.9	12.10	17.20	30.44	20.00	21.82	21.92	36.96	23.84	short	short
G+0	C8	3.18	36.13	17	32.76	330.3	9.79	23.61	42.73	39.37	28.13	26.32	39.80	29.77	short	short
G+0	C9	7.158	6.86	13.39	40.25	555.6	17.97	24.51	18.27	51.36	28.13	26.46	14.94	25.51	slender	slender
G+0	C11	9.037	9.105	1.593	14.68	420.2	17.44	10.00	17.51	23.08	21.71	21.97	22.51	40.52	short	short
G+0	C12	9.957	6.256	3.364	45.42	728	20.82	17.92	24.52	59.98	27.71	26.46	15.51	25.54	slender	slender
G+0	C13	15.33	25.82	4.328	34.86	583.6	27.01	16.00	37.49	46.53	27.71	28.31	19.94	27.61	slender	slender
G+0	C14	12.21	33.36	3.903	15.3	793.4	28.08	19.77	49.23	31.16	26.40	28.31	19.72	18.60	slender	slender
G+0	C15	12.27	34.35	5.431	2.049	905.2	30.37	20.15	52.46	23.54	26.40	28.31	18.32	13.79	slender	slender
G+0	C16	12.83	26.65	12.61	28.7	699.5	26.82	26.60	40.64	42.69	28.36	28.31	19.34	20.03	slender	slender
G+0	C17	12.44	51.77	23	14.79	582.1	24.08	26.44	63.41	34.64	27.71	26.46	26.91	19.10	slender	slender
G+0	C18	42.43	94.28	12.01	8.979	444.4	51.32	17.87	103.17	20.90	27.71	28.31	28.05	19.71	short	slender
G+0	C19	43.04	89.68	13.37	38.02	572.7	54.50	24.82	101.14	49.48	26.40	28.31	23.86	24.62	slender	slender
G+0	C20	24.91	44.66	24.19	48.75	882.2	42.55	41.83	62.31	66.40	26.40	28.31	16.84	17.72	slender	slender
G+0	C21	0.866	28.42	30.4	69.96	734.1	15.55	45.08	43.10	84.64	28.36	28.31	24.31	21.19	slender	slender

colmn	location	MEDx	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal	shear reinforcment	
G+0	C1	23.28	15.88	0.0	0.05	0.08	0.2	689.83	180	3600	689.828	20	2.20	4ø20	ø 10 c/c	300
G+0	C2	66.24	10.54	0.0	0.03	0.22	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
G+0	C3	72.24	19.32	0.0	0.06	0.24	0.8	2759.3	180	3600	2759.31	20	8.79	10ø20	ø 10 c/c	300
G+0	C4	50.63	35.57	0.0	0.12	0.17	0.7	2414.4	180	3600	2414.4	20	7.69	8ø20	ø 10 c/c	300
G+0	C5	12.34	46.63	0.0	0.15	0.04	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C6	11.05	17.65	0.0	0.02	0.02	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+0	C7	20.57	10.13	0.0	0.01	0.03	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+0	C8	36.68	33.31	0.0	0.11	0.12	0.5	1724.6	180	3600	1724.57	20	5.49	6ø20	ø 10 c/c	300
G+0	C9	7.62	40.71	0.0	0.13	0.02	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C10	10.65	10.23	0.0	0.01	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+0	C11	9.89	15.47	0.0	0.02	0.01	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+0	C12	10.38	45.85	0.0	0.15	0.03	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C13	26.28	35.33	0.0	0.12	0.09	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C14	33.79	15.73	0.0	0.05	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C15	34.79	5.87	0.0	0.02	0.11	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C16	27.07	29.12	0.0	0.10	0.09	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300
G+0	C17	54.27	25.50	0.1	0.08	0.18	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C18	99.27	17.01	0.2	0.06	0.32	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C19	94.78	43.12	0.2	0.14	0.31	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C20	47.65	51.74	0.1	0.17	0.16	0.4	1379.7	180	3600	1379.66	20	4.39	6ø20	ø 10 c/c	300
G+0	C21	33.54	75.07	0.3	0.25	0.11	0.45	1552.1	180	3600	1552.11	20	4.94	6ø20	ø 10 c/c	300

Appendix H: Exposure classes related to environmental conditions in accordance with ESEN 206-1

Table 37Exposure classes related to environmental conditions in accordance with ESEN 206-1

2. Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2

Appendix I: Minimum cover, cmin,b,requirements with regard to bond

Table 38Minimum cover, cmin,b,requirements with regard to bond

Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n) (see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $C_{min,b}$, should be increased by 5 mm.	

Appendix J: Values of minimum cover, cmin,dur, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

Table 39Values of minimum cover, cmin,dur, requirements

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Appendix K : K factor for design of beam (EBCS 2, 2015)

Table 40K factor for design of beam (EBCS 2, 2015)

Structural System	K	Concrete highly stressed $\rho = 1.5\%$	Concrete lightly stressed $\rho = 0.5\%$
Simply supported beam, one – or two-way spanning simply supported slab	11.0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	11.3	18	26
Interior span of beam or one-way or two-way spanning slab	11.5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	11.2	17	24
Cantilever	0.4	6	8
Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible. **Note 2:** For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken. **Note 3:** The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory			

Appendix L: partial factors for material for ultimate limit state (*ES EN 1992-1-1*, 2015)(2.4.2.4)

Table 41partial factors for material for ultimate limit state

Design situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressingsteel
Persistent & Transient	1.5	1.15	1.15
Accidental	1.2	1.0	1.0