Civil Engineering Department
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Structural design of B+G+4 mixed use building using EBCS 2015

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As members of the examining board of the final B.Sc. open defense, we verify that we have read and evaluated the final BSc thesis/project prepared by **Endris Setegn, Megersa Alemu, Genet Shibru, Mastewal Worku,Deriba Bayissa and Medhanit Yacob** entitled **Structural design of B+G+4 mixed use building using EBCS 2015**, and recommended for acceptance as a fulfillment of the requirement of **B.Sc. in Civil Engineering**

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Internal Examiner	Signature	Date

Executive Summary

This project is about the structural design and analysis of B+G+4 building considering all the effects the building receives in its service year. Structural design involves proportioning of structural members of the building and providing reinforcement which results economical.

In addition to the structural design and analysis, it also consists of cost estimation of the building with the help of preparing take off sheet. In short, the main structural designs included the paper is as follows.

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 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 gratitude acknowledge our advisers Mr.Abdeta and Mr. Gezachew 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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Finally, but not ends, we would like to appreciate all the group member for the unity and patience

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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
$A_{\mathrm{s,min}}$	minimum cross sectional area of reinforcement
A_{sw}	Cross sectional area of shear reinforcement
$E_{\mathrm{c,eff}}$	Effective modulus of elasticity of concrete
E_{cd}	Design value of modulus of elasticity of concrete
E_{cm}	Secant modulus of elasticity of concrete
$E_{\mathrm{c}}(t)$	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 B+G+4 it is 8 stories mixed used building that is around 286 m2 area its basement floor used as parking and store, Ground floor used for shop and office (8 shops and 3 offices), first floor and ground floor have the same functions but it’s not typical because the first floor have cantilever corridors in addition to ground floor, second floor used for dining hall and rooms to prepare dining hall like kitchen(used the dining hall is assumed 30 families in one time)and the third and fourth floor typical floor used for bed rooms(have 5 bed rooms each and rooms used parallel to bed room like showers, toilets ……) and also our project roof type is RC solid slab.

This project deals about the structural analysis and design of B+G+4 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.

Finally, 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 objectives are design safe, durable and economical structural design.

1.3.2. Sub-objectives

our selected project has the following objectives

- ✓ To help students to understand and develop the theoretical course in relation to their practical application.
- ✓ To develop the habit of working together.
- ✓ To work and practice the general concepts of designing methodology
- ✓ To design heavy loads and how to design this load
- ✓ To design seismic (wind and earth quake) impact on multi-story building

- ✓ Knowing design code and design criteria
- ✓ To consider the load consideration and calculation
- ✓ 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,
    - d. 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.(Kassimali, 2011)

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

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

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

fctm=2.2

b) Steel

$\Upsilon s=1.15$

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

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, 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

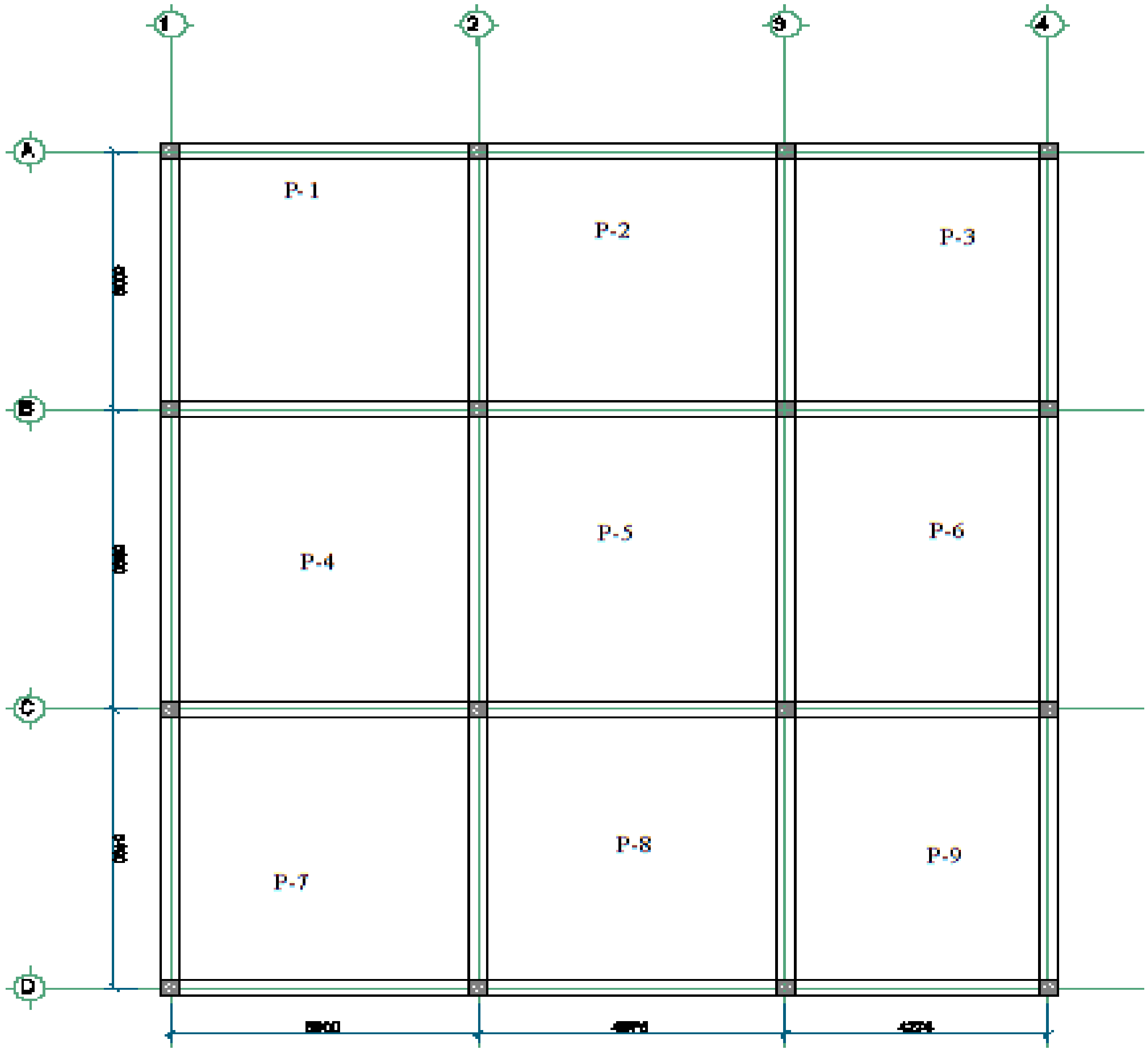


Figure 1-1. 3rd and 4th floor slab layout

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.

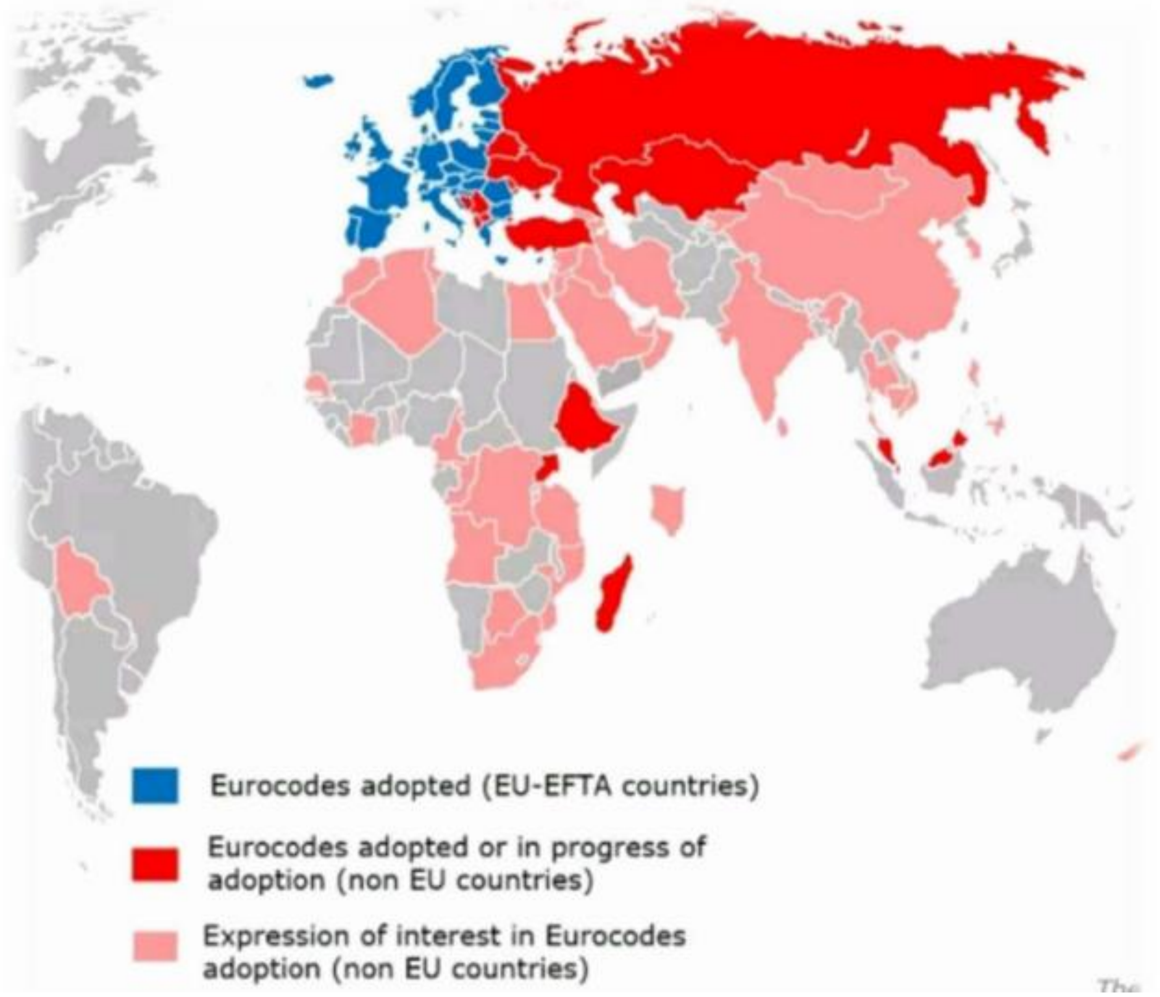
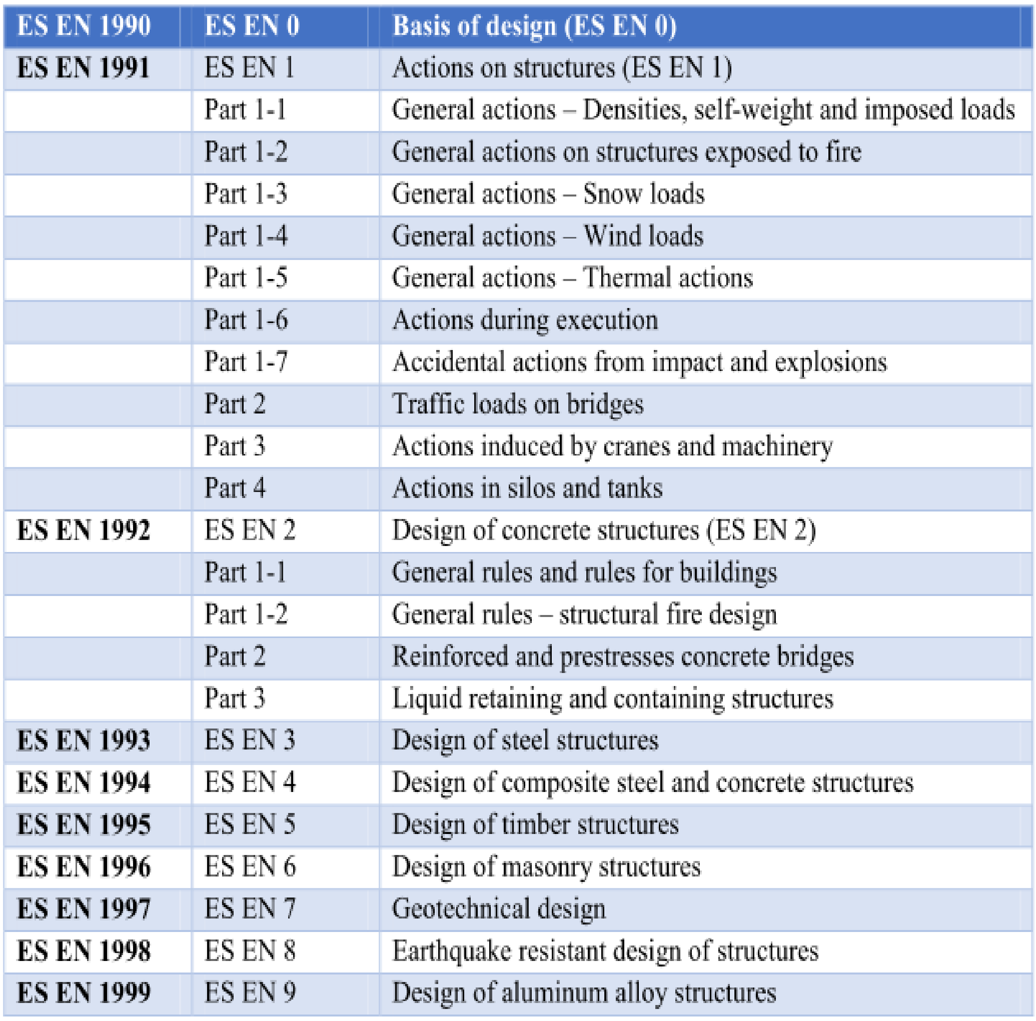


Figure 1-2-euro code practice around the world

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 B+G+4building for mixed use purpose located in wolkite.

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

Table 1-necessary data of wind load analysis

Roof type	**Flat roof**
Location	**Wolkite**
Building height	**15m**

Terrain category	**Category III**: - for area with regular cover of vegetation or buildings with obstacles with separations of maximum 20 obstacle heights(such as villages, sub urban terrain, permanent forest)
Directional factor	**1:** recommended value
Seasonal factor	**1**: recommended value
Basic Wind velocity	**22m/s**: for Ethiopia
Z_0	**0.3m**: for terrain category III from table
$Z_{0.\mathbf{II}}$	**0.05m:** for (terrain category II)
Minimum height	**5m:** for terrain category III from table
maximum height	**200m**
orography factor	**1:** recommended
Turbulence factor	**1:** recommended
Air density	**1.25 kg/m^3:** recommended
Reference height	**15m:** 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 $C_r(Z)$

$C_r(Z) = \begin{cases} K_r \ln\left(\frac{Z}{Z_0}\right) & for\ Z_{min} \leq Z \leq Z_{max} \\ C_r(Z_{min}) & for\ Z \leq Z_{min} \end{cases}$ (*Wind Actions,ES EN 1991:2015*, 2018, p. 8)

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

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

$Z_{min} = minimum\ height$

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

$$K_r = 0.19\left(\frac{0.3}{0.05}\right)^{0.07} = \mathbf{0.215}$$

$C_r(Z) = K_r \ln\left(\frac{Z}{Z_0}\right) \qquad for\ 5 \leq 15 \leq 200$

$= 0.215 ln\left(\frac{15}{0.3}\right)$

$$C_r(Z) = \mathbf{0.841}$$

c) Mean wind velocity $V_m(Z)$

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

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

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

$V_b = basic\ wind\ velocity$

$V_m(Z) = 0.841 * 1 * 22\ m/s$ = **18.50 m/s**

d) Wind turbulence $l_V(Z)$

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

Where $K_l = turbulence\ factor$

$C_0(Z) = orography\ factor$

$$l_V(Z) = \frac{1}{1 * ln\left(\frac{15}{0.3}\right)} = \mathbf{0.257}$$

e) Peak velocity pressure $q_p(Z)$

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

$where\ \rho = air\ density$

$$q_p(Z) = [1 + 7 * 0.257] * \tfrac{1}{2} * 1.25 * 18.50^2 = \mathbf{598.72\ N = 0.598KN}$$

f) Wind pressure

- External pressure

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

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

$Z_e\ reference\ height$

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

✓ External pressure coefficient

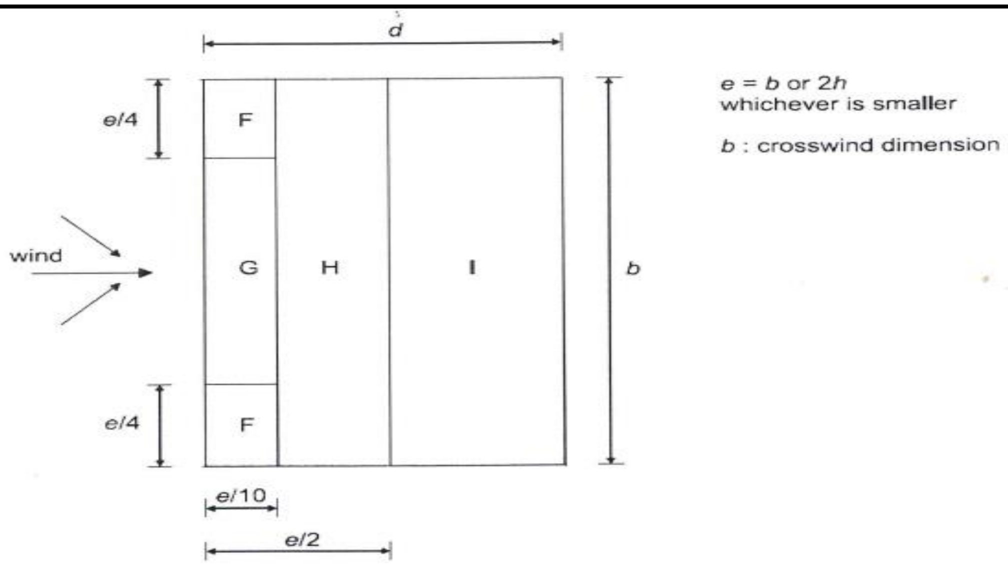
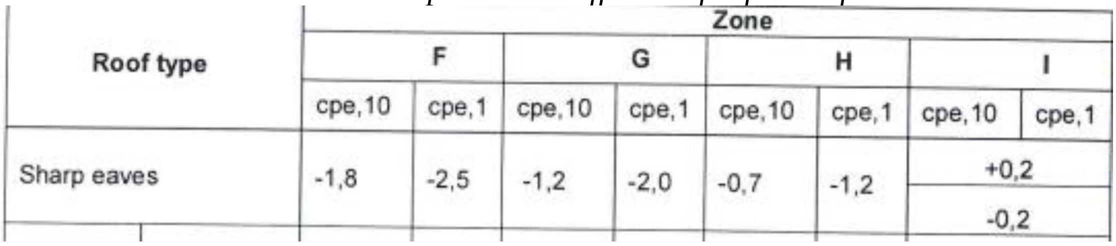
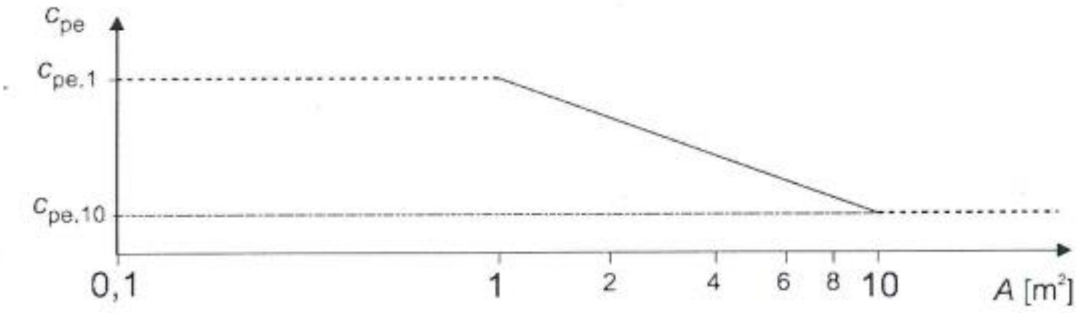


Figure 2-1.key for flat roof

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	

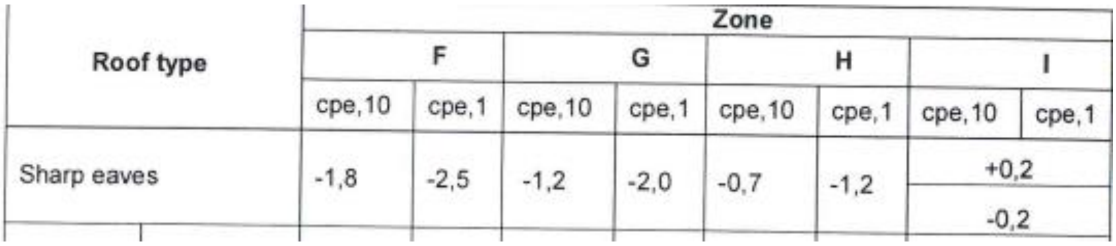


The figure is based on the following:

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 3.External pressure coefficients for flat roofs



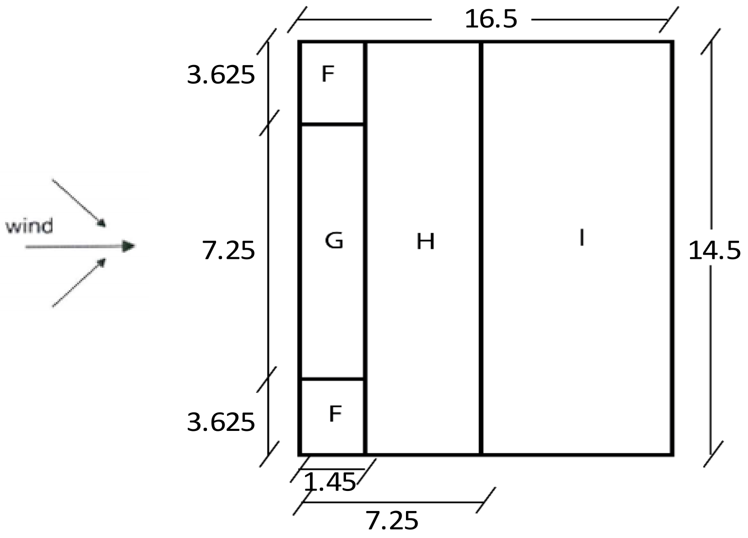
Roof type	Zone							
	F		G		H		I	
	cpe,10	cpe,1	cpe,10	cpe,1	cpe,10	cpe,1	cpe,10	cpe,1
Sharp eaves	-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	+0,2	
							-0,2	

Case -1 Ө=90^{o}

b = 14.5, d = 16.5.07m, h=15

2h= 30

14.5 < 30 so, e =14.5 m



zone	Cpe values		cpe	w_e
F	cpe,10	-1.8	-1.30	-0.776
	cpe,1	-2.5		
G	cpe,10	-1.2	-1.20	-0.718
	cpe,1	-2		
H	cpe,10	-0.7	-0.70	-0.419
	cpe,1	-1.2		
I	cpe,10	0.2	0.20	0.120
	cpe,1	-0.2		
	cpe,10	-0.2	-0.20	-0.120
	cpe,1	0.2		

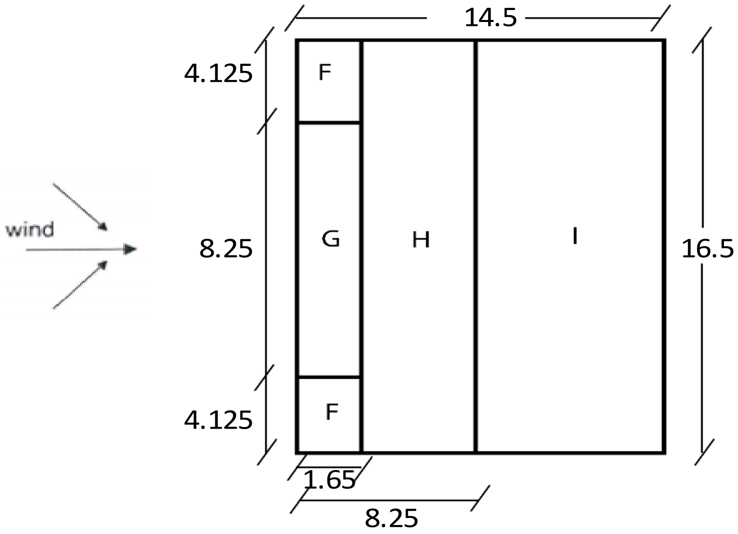
Critical external pressures: - pressure = **0.120**and suction = **-0.776**

Case -2 ϴ=0^o

b = 16.5m, d = 14.5m, h=15

2h= 30

30 < 16.5 so, e =16.5m



zone	Cpe values		cpe	w_e
F	cpe,10	-1.8	-1.50	-0.89654
	cpe,1	-2.5		
G	cpe,10	-1.2	-1.20	-0.7176
	cpe,1	-2		
H	cpe,10	-0.7	-0.70	-0.4186
	cpe,1	-1.2		
I	cpe,10	0.2	0.20	0.1196
	cpe,1	-0.2		
	cpe,10	-0.2	-0.20	-0.1196
	cpe,1	0.2		

Critical external pressures: - pressure = **0.1196**and suction = **-0.89654**

- Internal pressure

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

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

$Z_i\ reference\ height$

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

Internal Wind Pressure Coefficient

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

$$W_i = 0.598 * \ 0.8 = 0.4784$$

$$= 0.598 * -0.5 = -0.299$$

g) Net pressure = $W_e - W_i$

We	Wi	Wnet
0.12	0.4784	-0.3584
-0.776	-0.299	-0.477
0.1196	0.4784	-0.3588
-0.8965	-0.299	-0.5975

Hence, the critical net wind loads on the roof are

Critical Press.are (KN/m2)

Wnet min= -0.3584

Wnet max= -0.5975

2.3. Concrete cover design for roof slab

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

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*, 2015a)

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*, 2015a). 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*, 2015b, 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≤ 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 $fcd = \alpha cc * \frac{\text{fck}}{\gamma c}$ =0.85 $*\frac{25}{1.5}$= 11.33MPa

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

d) Steel

ϒs=1.15

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

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

Where;

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

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)

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

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

∆Cdur,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 ∆Cdur,γ , ∆Cdur,st , and ∆Cdur,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 Δcdev 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

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

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

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 todesign 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 excess ive 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)

ρo=$10^{-3}\sqrt{fck}$=$10^{-3}\sqrt{20}$=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 spanig 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

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=5000 mm

d=5000/28.78=173.76mm

Similarly for the rest panels d will be calculated as follows

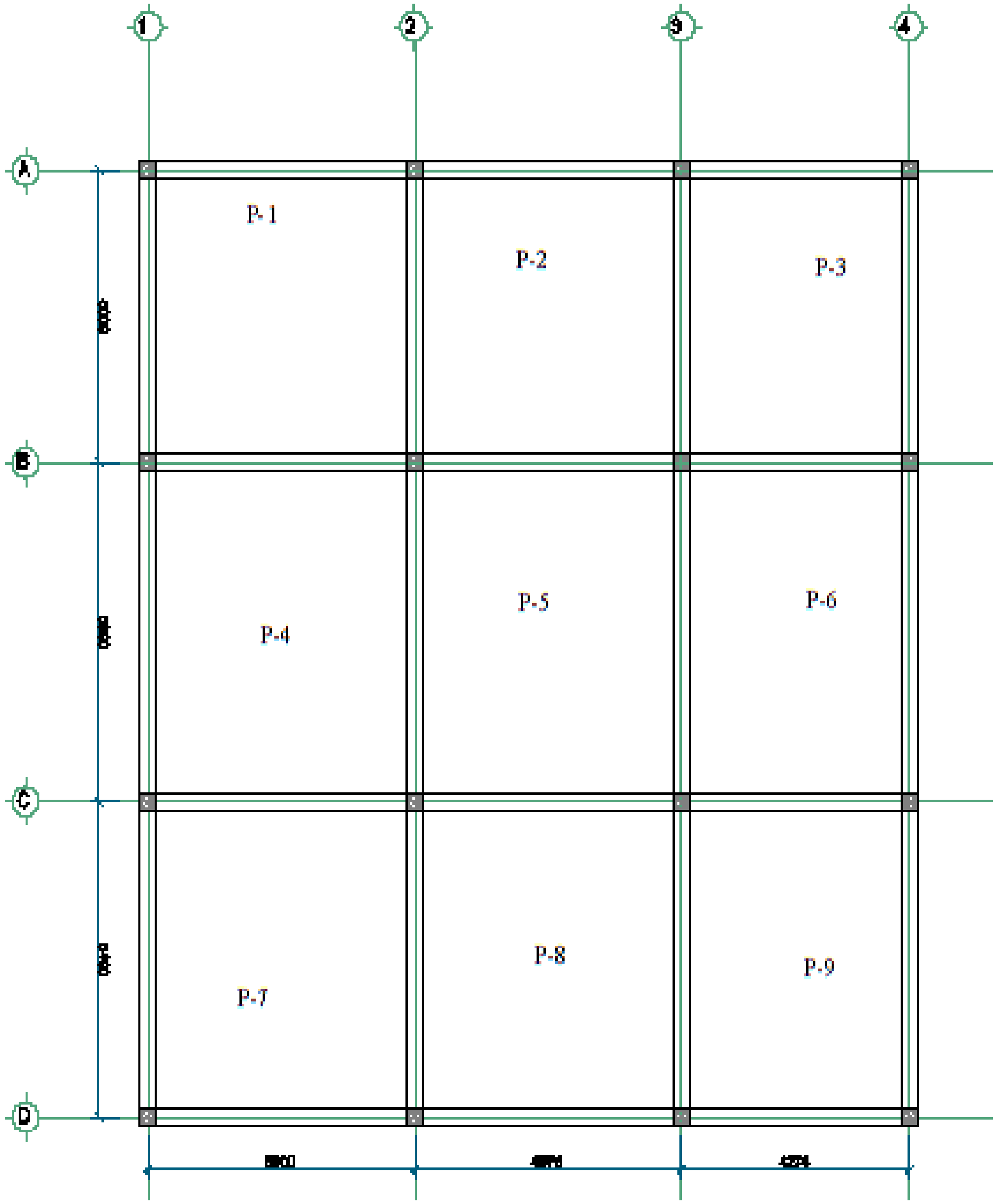


Figure 3-1. 3rd and 4th floor slab layout

3.1.3. Depth determination of 3rd and 4th floor slab

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

Table 4. slab depth determination

panel	Ly	Lx	Ly/Lx	Type	support	N	K	Lx/d	d	D	D provided
p1	5000	5000	1.00	two way	End	17.71	1.3	28.78	173.8	198.76	200
p2	5000	4250	1.18	two way	End	17.71	1.3	28.78	147.7	172.69	
p3	5800	5000	1.16	two way	Interior	17.71	1.5	33.20	150.6	175.59	
p4	5800	5000	1.16	two way	Interior	17.71	1.5	33.20	150.6	175.59	
p5	5800	4250	1.36	two way	Interior	17.71	1.5	33.20	128.0	153.00	
p6	5400	5000	1.08	two way	End	17.71	1.3	28.78	173.8	198.76	
p7	5400	5000	1.08	two way	End	17.71	1.3	28.78	173.8	198.76	
p8	5400	4250	1.27	two way	End	17.71	1.3	28.78	147.7	172.69	

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.

$P_d = 1.35DL + 1.5LL$

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*, 2015a)

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

Floor finish -on this panel there are two types of floor finishes calculate each based on covered area

For ceramic= 0.005m * 23KN/m^3 = **0.11KN/m^2**

For carpet= 0.015m * 23KN/m^3 = **0.35KN/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. Steps to distribute the partition load described below

1. Calculate area of slab

   A=L*W= 5m*5m = 25m^2

2. Calculate volume of partition

   V=L*W*H=9.976*0.2*3= 5.98m^3

3. Calculate load

   DL= unit weight of HCB * A/V =14 KN/m*25/5.98m^3 = **3.35 KN/m^2**

Total DL=**5KN/m^2 + 0.69KN/m^2 + 0.35KN/m^2 + 0.11KN/m^2 +3.35KN/m^2 = 10.19 KN/m^2**

b) Live load

live load is selected with respect to functional use of panel

Others are done in tabular form below and another panels in appendix A1

Table 5.Design load calculation

P1				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.005	0.11
	6	25	0.24				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.0048				
	A_{finish}	A_{total}	A_f/A_t	Carpet	23	0.015	0.35
	19	25	0.76				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.0152				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.239	3.35
	25	9.976*0.2*3	0.239424				
		5.9856					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					10.19	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					19.76	KN/m^2	

3.1.5 Moment analysis of 3rd and 4th 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,

$mi=\beta siPdLx^2$

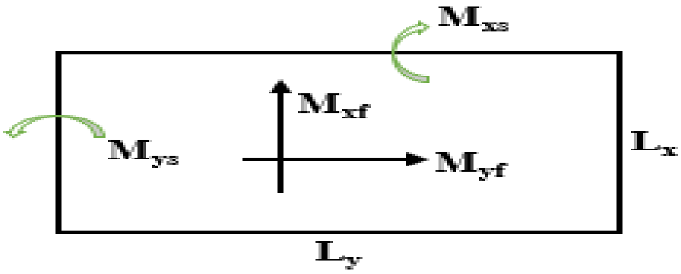


Figure 3-2. Support and span moment notations

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 then the β is calculated below

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

$\beta_{sy,sup}=0.037$ $\beta_{sy,span}=0.028$

$M_{xs=}\beta_{sx,sup}*pd*Lx^2=0.039*19.78*5^2=19.29$

$M_{xf}=\beta_{sx,span}*pd*Lx2=0.029*19.78*5^2=14.34$

$M_{ys}=\beta_{sy,sup}*pd*Lx^2=0.037*19.78*5^2=18.30$

$M_{yf} = \beta_{sy,span} * pd * Lx^2 = 0.028 * 19.78 * 5^2 = 13.85$

Then the rest of panels calculated in tabular below

b) Strip method moment analysis

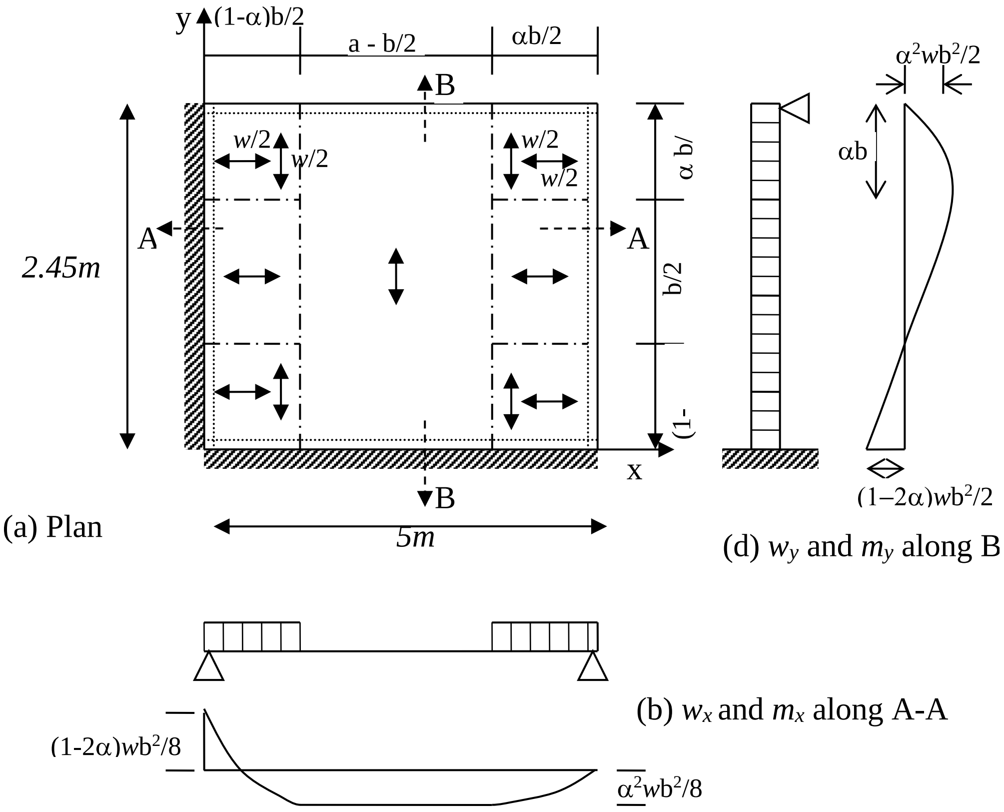
Strip method is a lower bound approach, based on the satisfaction of equilibrium requirements everywhere in the slab. By the strip method, a moment field is first determined that fulfills equilibrium requirements, after which the reinforcement of the slab at each point is designed for this moment field. (Kassimali, 2011)

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 by strip method



(a) Plan

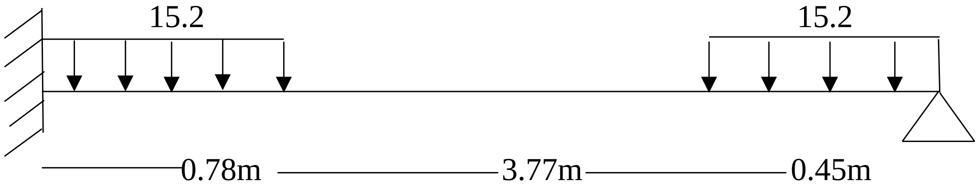
(d) w_y and m_y along B

(b) w_x and m_x along A-A

Figure 3-3. slab strip method partition

For middle strip x-direction

Section A-A



$$Mx\sigma = (1-2\alpha)\frac{\mathrm{wb2}}{8}$$

$$Mxf = 2\alpha\frac{\mathrm{wb2}}{8}$$

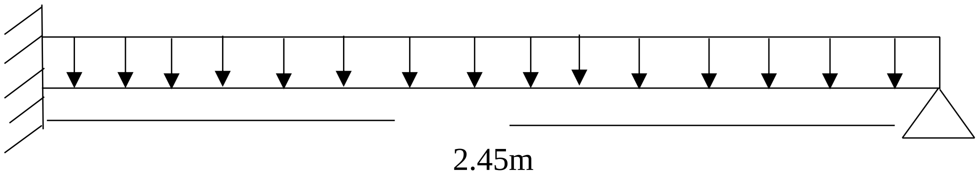
$$Mx\sigma = (1 - 2 * 0.368)(\frac{15.2 * 2.45 * 2.45}{8})$$

$$Mx\sigma = 3.01$$

$$Mxf = 0.368 * 0.368 * \frac{15.2 * 2.45 * 2.45}{8})$$

$$Mxf = 4.2$$

Y-direction section B-B



$$Mx\sigma = (1 - 2\alpha)\frac{\text{wb2}}{2}$$

$$Mxf = 2\alpha\frac{\text{wb2}}{2}$$

$$Mx\sigma = (1 - 2 * 0.368)(\frac{15.2 * 2.45 * 2.45}{2})$$

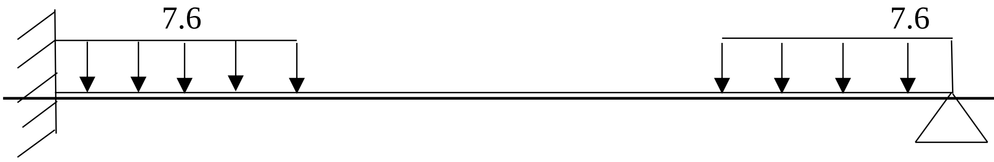
$$Mx\sigma = 12.04$$

$$Mxf = (0.368 * 0.368 * \frac{15.2 * 2.45 * 2.45}{2})$$

$$Mxf = 6.18$$

Edge strip

Y-direction section C-C

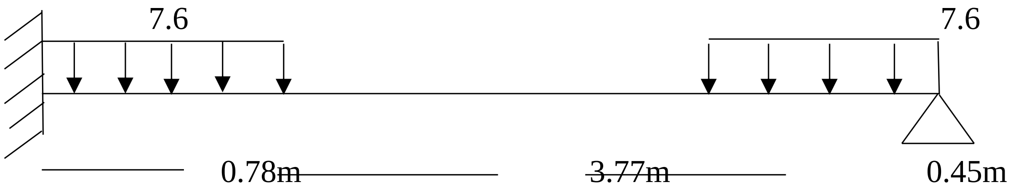


0.78m 3.77m 0.45m

$Mx\sigma = \left(\frac{w*\alpha*b*b}{16}\right)$ $Mx\sigma = \left(\frac{7.6*0.368*2.45*2.45}{16}\right) = 0.75$

$Mxf = \frac{w\alpha b2}{16}$ $Mxf = \frac{7.6*0.368*2.4*2.4}{16}$ =1.05

X-direction section D-D



$Mx\sigma = \left(\frac{w*\alpha*b*b}{16}\right)$ $Mx\sigma = \left(\frac{7.6*0.368*2.45*2.45}{16}\right) = 0.75$

$Mxf = \frac{w\alpha b2}{16}$ $Mxf = \frac{7.6*0.368*2.4*2.4}{16}$ =1.05

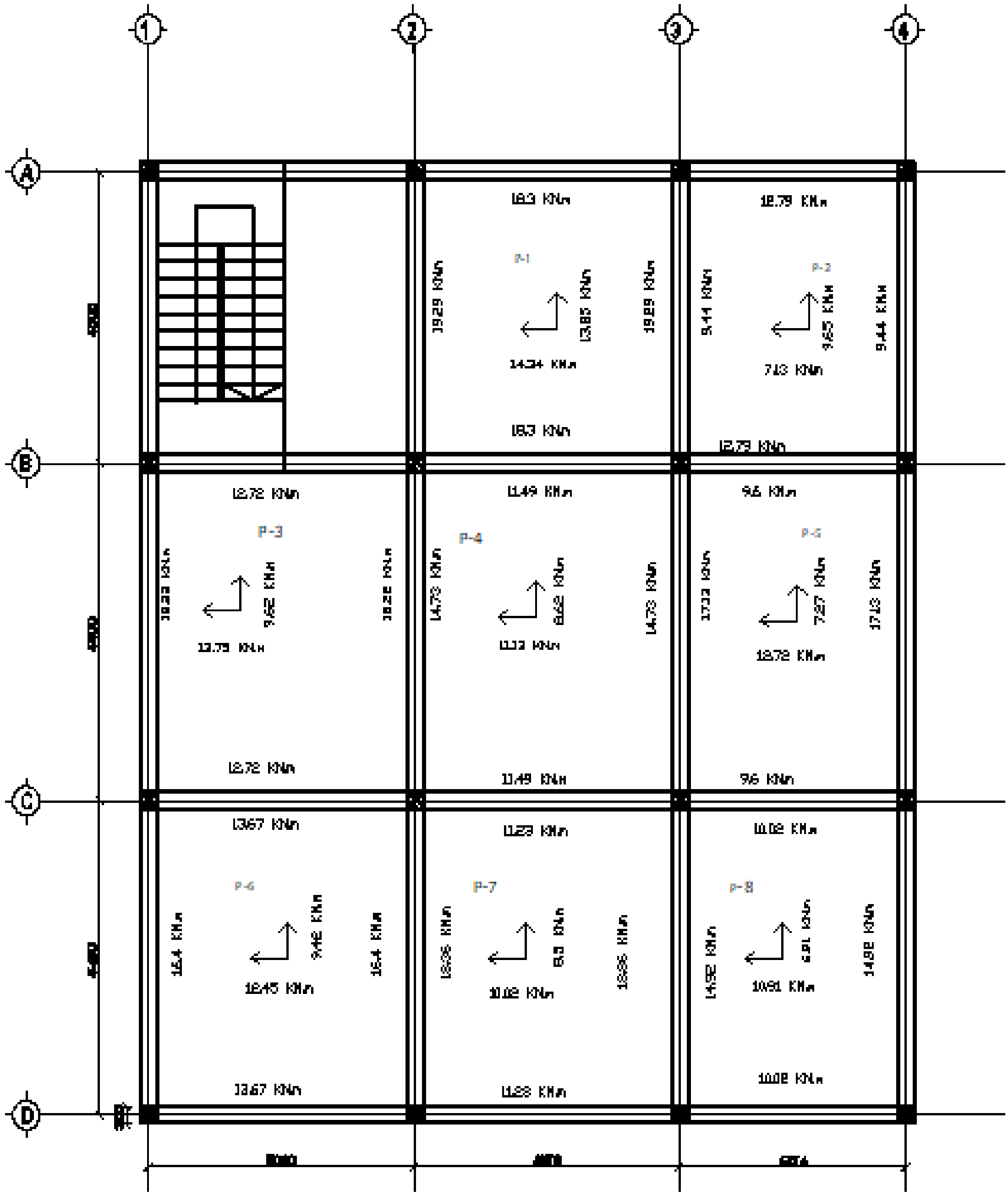


Figure 3-4. 3rd and 4th floor slab unadjusted moment

3.1.6 Moment adjustment of 3rd and 4th floor slab

a) Support moment adjustment

Compute % change M=(M_{large}-M_{small}/ M_{large})*100% on each support

When % of change M is less than 20% 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

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

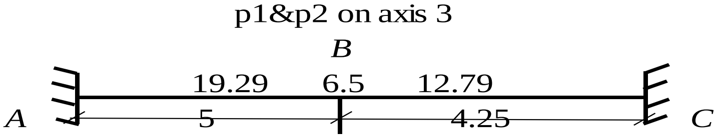
Distribution factor Df

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

For example: - P1&P2 on axis 3

$$\Delta M = \left(\frac{M\text{large} - M\text{small}}{M\text{large}}\right) * 100\% = \left(\frac{19.29 - 12.79}{19.29}\right) * 100\% = 33.7\%$$

So ΔM>20% use moment distribution required



Relative bending stiffness K

$$K_{AB} = \frac{I}{L} = \frac{I}{5} = 0.2\ I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{4.74} = 0.24\ I$$

Distribution factor Df

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.459$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.541$$

For example: - P3&P4 on axis 2

$\Delta M=\left(\frac{M\text{large}-M\text{small}}{M\text{large}}\right)*100\%=\left(\frac{18.22-14.73}{18.22}\right)*100\%=19.15\%$

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

P3&P4 –on axis 2 adjustment is required.

$\Delta M_s=$ 18.22 - 14.73 = 3.49

$0.2M_{s,max}$ 0.2 x 18.22 = 3.644

$\Delta M_s=$ 3.49 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 18.22 + 14.73 $)=$ 16.48

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.

$(Mxf)d=Mxf+Cx\Delta Mxs+Cx\Delta Mys$

$(Myf)d=Myf+Cy\Delta Mxs+Cy\Delta Mys$

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

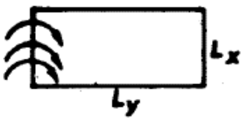
$M_{xf} = 14.34$ $M_{Xs} = 19.29$ $M_{Xs,adj} = 16.3$

$M_{yf} = 13.85$ $M_{ys} = 18.3$ $M_{ys,adj} = 15.13$

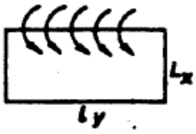
Ly/Lx= 1

ΔM_{xs} = Mxs- M_Xs,adj = 2.99 neglect

ΔM_{ys} = Mys- M_ys,adj = 3.17 neglect



Cx = 0.38
Cy = 0.28



Cx = 0.38
Cy = 0.28

ΔMxf = CxΔMxs + CxΔMys = 2.341

Mxf ,adj = ΔMxf + Mxf = **16.681**

ΔM_{yf} = CyΔMxs + CyΔMys = 1.725

Myf ,adj = ΔMyf + Myf = **15.575**

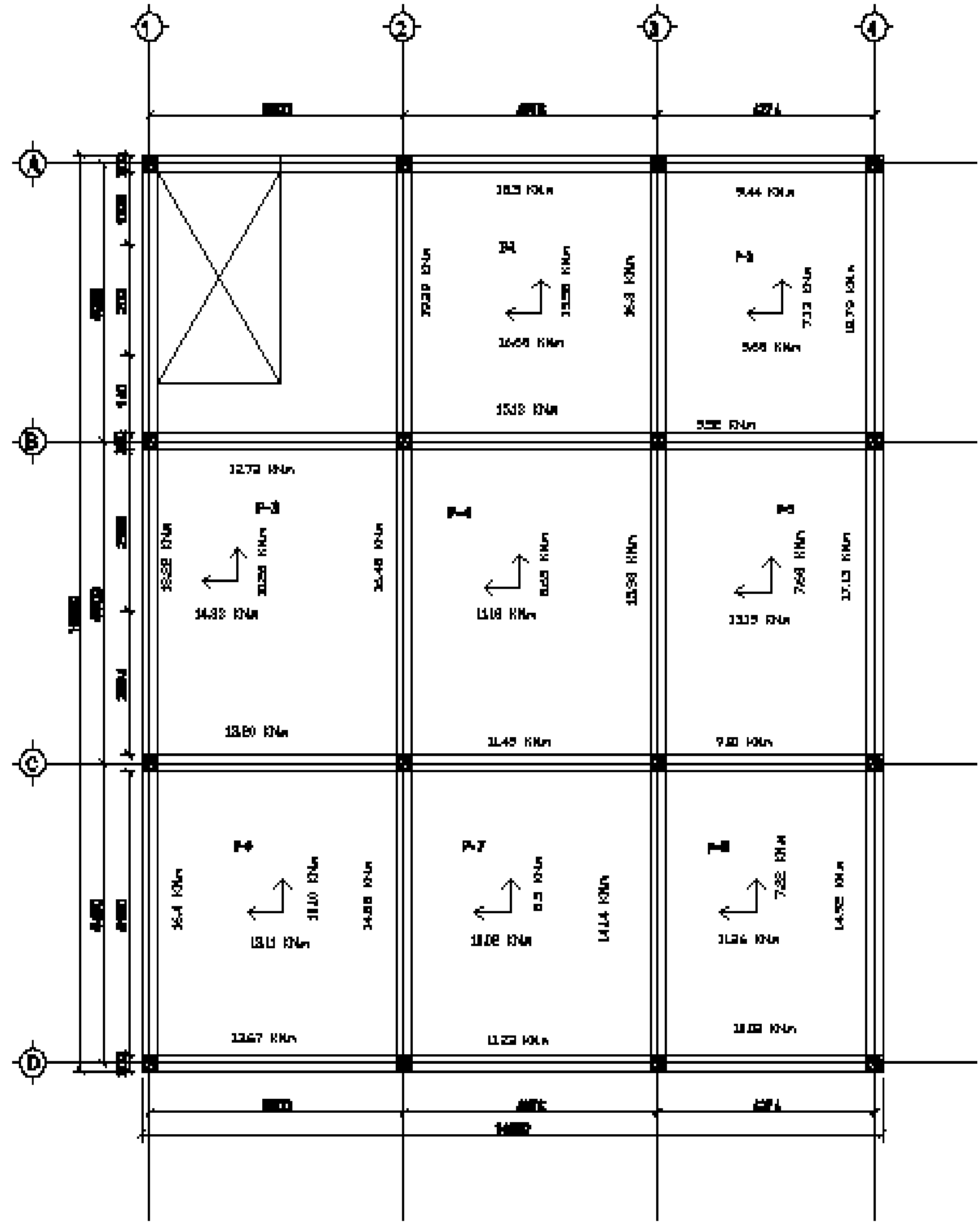


Figure 3-5. 3rd and 4th floor slab adjusted moment

3.1.7 Load Transfer to beam

Two-way slab shear

The shear force for each panel calculated by coefficient method

$V_x = \beta_{vx} P_d L_x$

$Vy=\beta vxPdLy$

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

β_{vxc} =0.36 β_{vxd} =0(in type 2 β_{vxd} is zero from table)

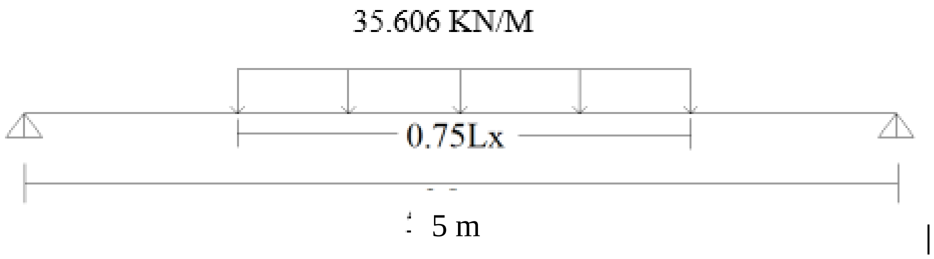
β_{vyc} =0.36 β_{vyd} =0.24

$Vx= \beta_{vxc}\ PdLx=0.36*19.78*5=35.606$

$V_{yc}= \beta_{vyc}\ PdLy=0.36*19.78*5=35.606$

$V_{yd} = \beta_{vyd}\ PdLy=0.24*19.78*5=23.373$

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= 175

k= 2.07 not < 2,so k=2

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

taking minimum reinforcment ø 10 c/c 300

$\rho_1 =$ 0.0015

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

$V_{min} = 77.476$

$v_{Rd,c} =$ 41.867 $\leq V_{min}$ so, $v_{Rd,c} =$ 77.476

maximum shear force transfererd to beams is V_i= Vi=35.506

V_i= 35.506 KN/m

77.476>35.605,slab thickness is sufficient to support design load (*ES EN 2 Part 1-1*, n.d.)

3.1.9 Reinforcement calculation of 3rd and 4th floor slab

- Area of steel

$$A_{st},\text{min} = \max \begin{cases} 0.26\frac{f_{ctm}}{fyk}bd = 0.26\frac{2.2}{400}1000 * 174 = 249\ mm^2 \\ 0.0013bd = 0.0013 * 1000 * 174 = 226.2\ mm^2 \end{cases}$$

A_{st},min = 250 mm^2

A_{st},max=0.04bd=0.04*1000*174=6960mm^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*174 mm = 156.6 mm

Example for panel 1 support moment = 16.3 KN-m

$$Ast = \frac{m_{sd}}{Z*fyd} = \frac{16.3*1000*1000N-mm}{156.6mm*347.83N/mm^2} = 299.25\ mm^2 > 250 \text{ mm}^2 \text{ OK!!}$$

a) Spacing

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

S_{max} = 600 mm

S_{min}=300mm on ES EN 2015

Let the ø of reinforcement be 10mm

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

Spacing of bars =S = $\frac{ba_s}{A_s} = \frac{1000*78.54}{299.25} = 262.46$ mm

$262.46\ mm$ ≈ 250 mm used < 630mm OK!!

Table 6. slab reinforcement

panel	moment	Ast=Msd/Z*fy		ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	19.29	354.14	10	221.66	220	ø	10	c/c	220
	M_{xf} =	16.68	306.22	10	256.35	250	ø	10	c/c	250
	M_{ys} =	18.3	335.96	10	233.66	230	ø	10	c/c	230
	M_{yf} =	15.58	286.03	10	274.45	250	ø	10	c/c	250
P-2	M_{xs} =	16.3	299.25	10	262.33	250	ø	10	c/c	250
	M_{xf} =	9.65	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	9.44	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
P-3	M_{xs} =	18.22	334.49	10	234.68	230	ø	10	c/c	230
	M_{xf} =	14.33	263.08	10	298.39	290	ø	10	c/c	290
	M_{ys} =	12.72	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	10.26	250.00	10	314.00	300	ø	10	c/c	300
P-4	M_{xs} =	16.48	302.55	10	259.46	250	ø	10	c/c	250
	M_{xf} =	11.18	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	15.13	277.77	10	282.61	280	ø	10	c/c	280
	M_{yf} =	8.65	250.00	10	314.00	300	ø	10	c/c	300
P-5	M_{xs} =	15.93	292.45	10	268.42	260	ø	10	c/c	260
	M_{xf} =	13.19	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	9.52	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.68	250.00	10	314.00	300	ø	10	c/c	300
P-6	M_{xs} =	16.4	301.08	10	260.73	260	ø	10	c/c	260
	M_{xf} =	13.11	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	13.2	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	10.26	250.00	10	314.00	300	ø	10	c/c	300
P-7	M_{xs} =	14.88	273.18	10	287.36	280	ø	10	c/c	280
	M_{xf} =	10.02	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	11.36	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	8.5	250.00	10	314.00	300	ø	10	c/c	300
P-8	M_{xs} =	14.14	259.59	10	302.40	300	ø	10	c/c	300
	M_{xf} =	11.26	250.00	10	314.00	300	ø	10	c/c	300
	M_{ys} =	9.81	250.00	10	314.00	300	ø	10	c/c	300
	M_{yf} =	7.27	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

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

$$l_{o,min} \geq max\begin{cases} 0.3\alpha_6 l_{b,rqd} \\ 15\emptyset \\ 200mm \end{cases} \quad (\textit{ES EN 1992-1-1}, \text{2015a})$$

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

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

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

$f_{ctk} = 1.mpa$,(*ES EN 1992-1-1*, 2015a) $from\ ESEN$ 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

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

$f_{ctd} = 0.85$

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

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

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

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

$\alpha_{1,}\ \alpha_2$ & $\alpha_3 = 1$, $\alpha_4 = 0.7$, 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(*ES EN 2 Part 1-1*, n.d.)

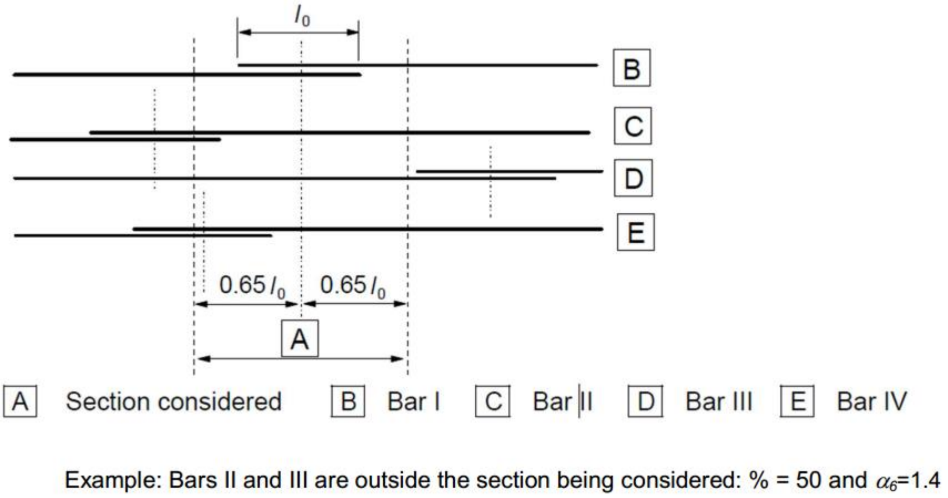


Figure 3-6. percentage of lapped bars in one lapped section

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

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

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

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

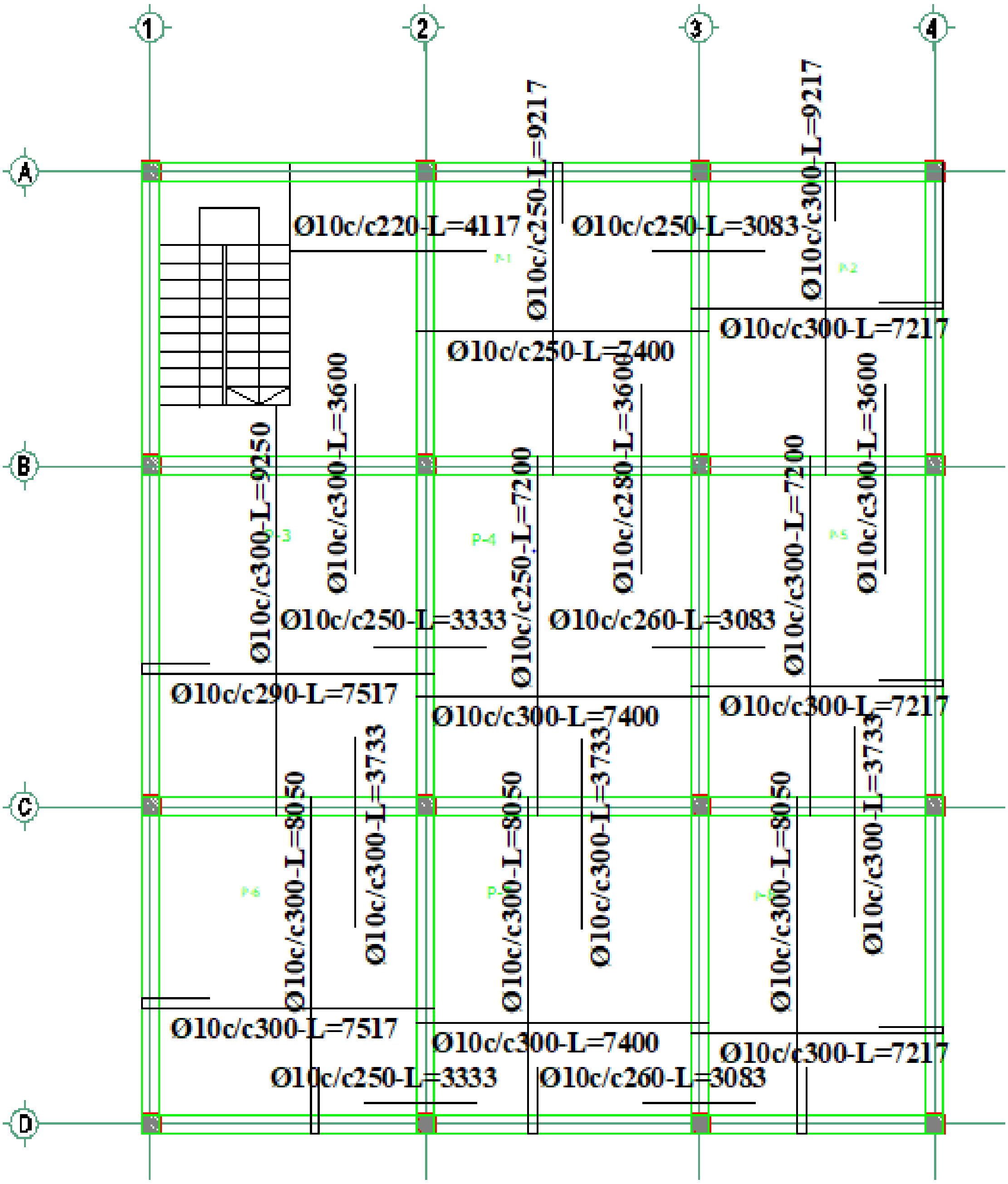


Figure 3-7. slab reinforcement detailing

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

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

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

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

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

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

$$N = 33.1412$$

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

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

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

$$d = 41.3002\ \text{mm}$$

d ≥ 41.300 it safe(*ES EN 2 Part 1-1*, n.d.)

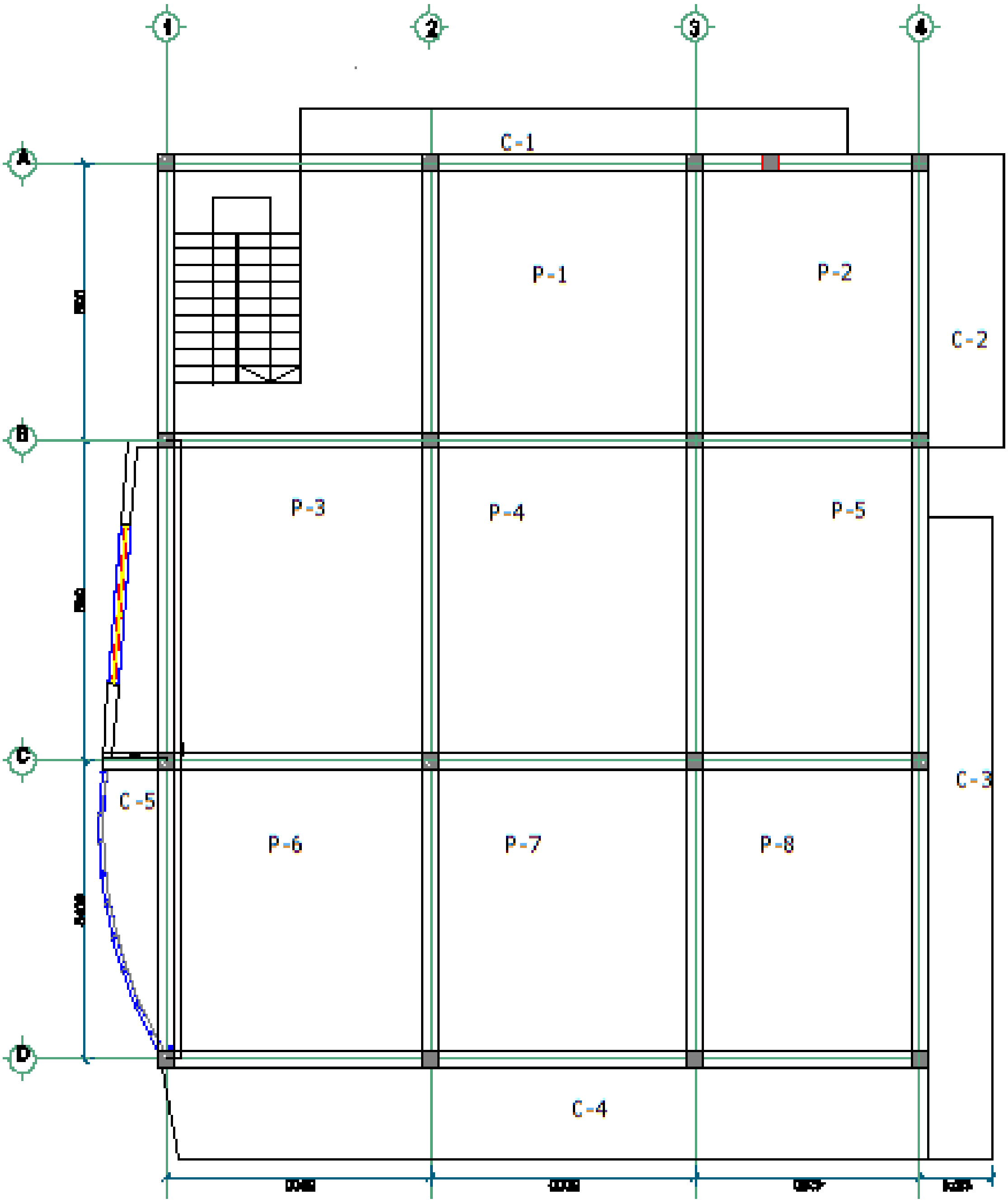
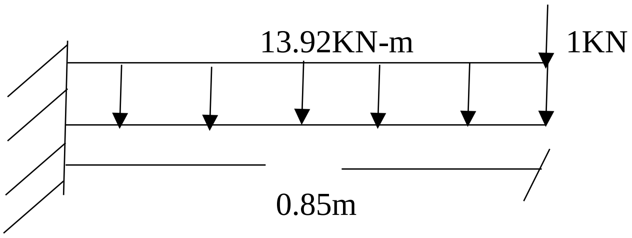


Figure 3-8. second floor slab lay out

The same procedure with 3rd and 4th floor design but in this floor, there are cantilever slabs

3.1.11. Second floor cantilever moment calculation

C-1

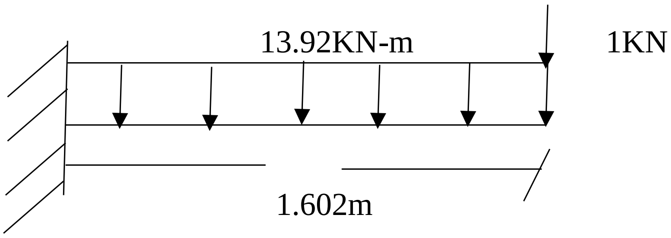


$M = \frac{wL2}{2}$+1KN*L

M= $\frac{13.92*0.85m*0.85m}{2}$+1KN*0.85m

M=5.88KNm

C-2

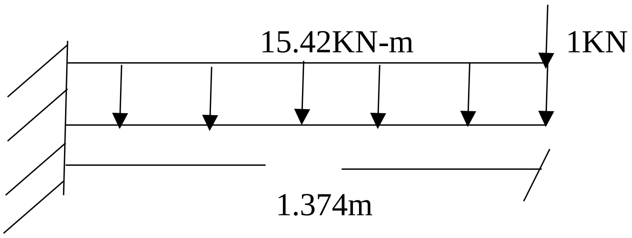


$M = \frac{wL2}{2}$+1KN*L

M= $\frac{13.92*1.602m*1.602m}{2}$+1KN*1.602m

M=19.46KNm

C-3

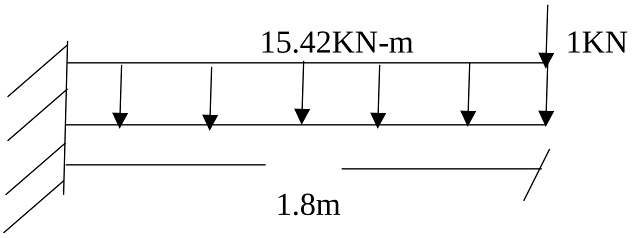


$M = \frac{wL2}{2}$+1KN*L

M= $\frac{15.42*1.374m*1.374m}{2}$+1KN*1.374m

M=15.93KNm

C-4

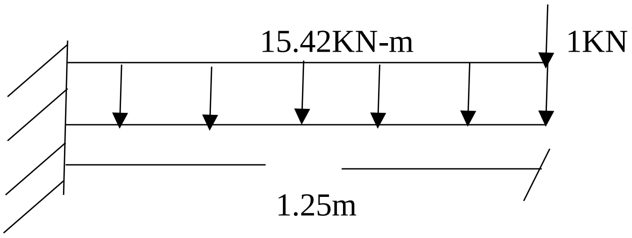


$M = \frac{wL2}{2}$+1KN*L

M= $\frac{15.42*1.8m*1.8m}{2}$+1KN*1.8m

M=26.78KNm

C-5



$M = \frac{wL2}{2}$+1KN*L

M= 15.42+1KN*1.25m

M=13.30KNm

4. STAIRCASE

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.

Guidelines 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 staircase. Following are some useful guidelines 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

$$\frac{l}{d} = \mathrm{k}\left[11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho} + 3.2\sqrt{fck}(\frac{\rho o}{\rho})^{\frac{3}{2}}\right] \quad \text{if } \rho \leq \rho o(\textit{ES EN 1992-1-1,} \text{ 2015a})$$

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

Table 7. basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural System	K	Concrete highly stressed ρ = 1.5%	Concrete lightly stressed ρ = 0.5%

Simply supported beam, one – or twoway spanning simply supported slab	11	14	20
End span of continuous beam or oneway continuous slab or two-way spanning slab continuous over one long side	11.3	18	26
Interior span of beam or one-way or two-way spanning slab	11.5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	11.2	17	24
Cantilever	0.4	6	8

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

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

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

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 8. 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/m2 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

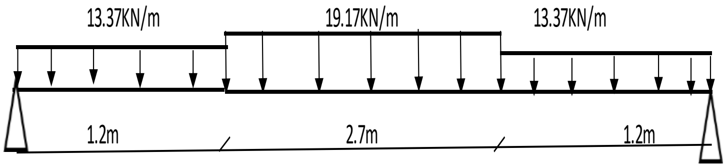
= (0.03x1x27) + (0.03x23x1) + (0.14x1x25) + (0.02x1x23) = 5.46KN/m

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

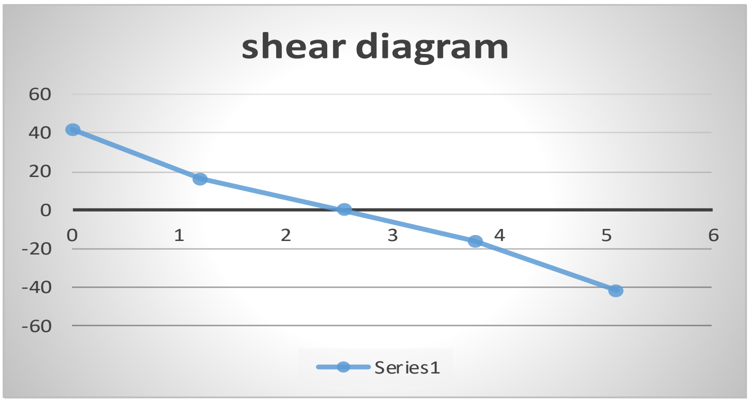
Pd=1.35DD+1.5LL=1.35x5.46+1.5x4 = 13.37 KN/m

STEP 4

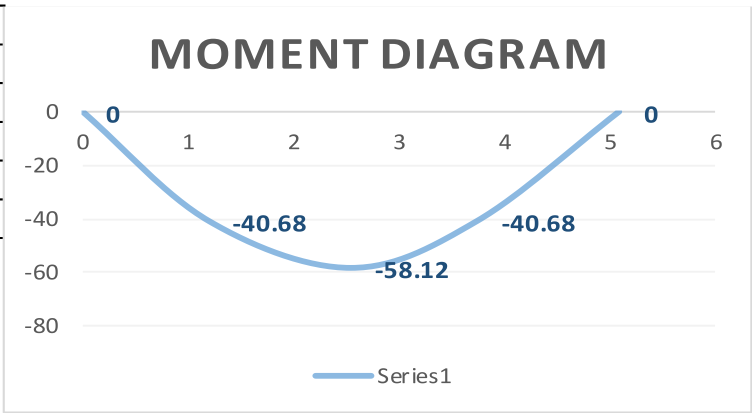
4.2.3. Moment Analysis



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

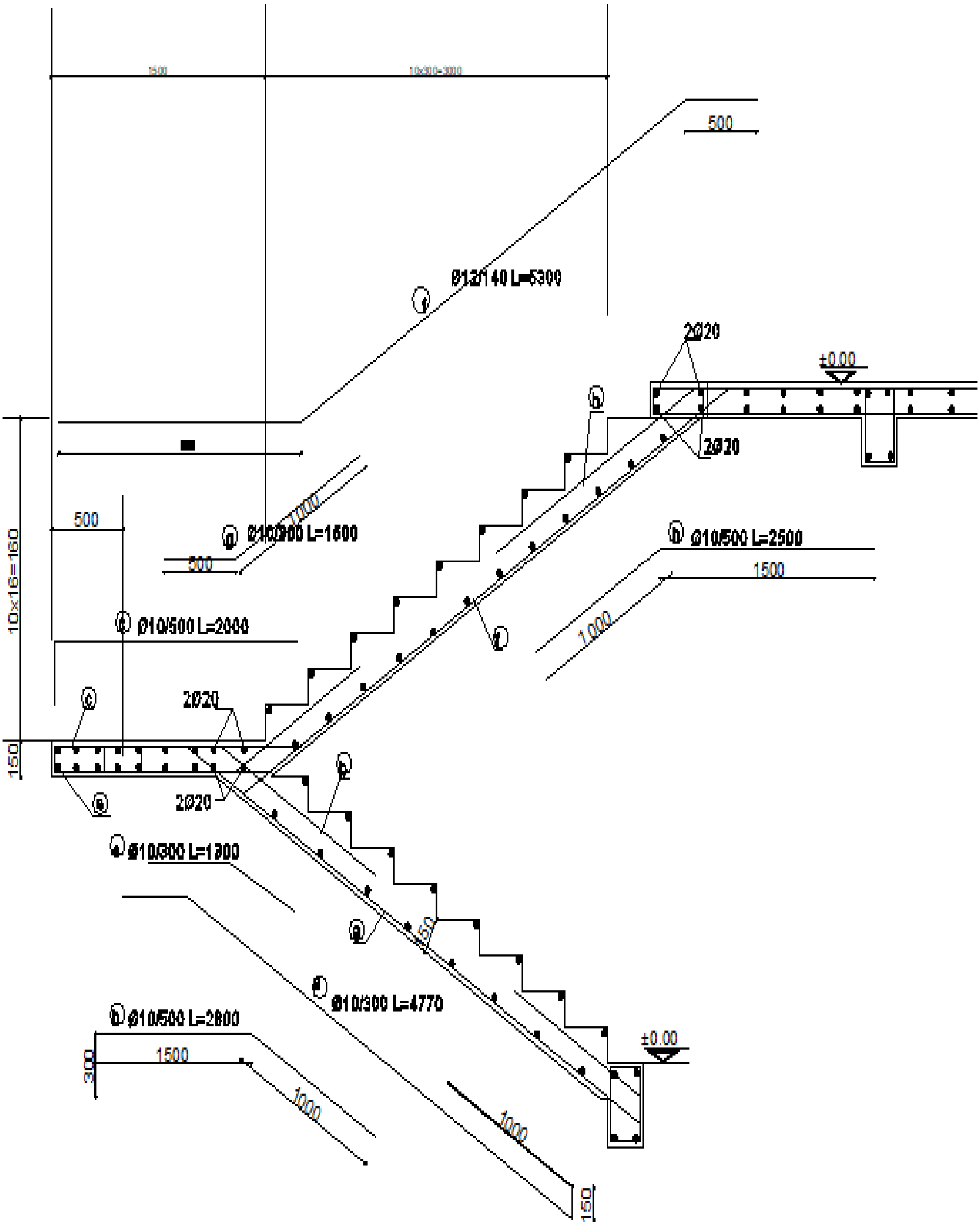


Figure 4-1. Staircase detailing

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 D)

Seismic zone: (Wolkite, 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 = 3, 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 WLX according to EUROCODE1 2005, as calculated by ETABS.

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = III

Wind Direction = 0.90 degrees

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

Top Story = Roof

Bottom Story = G+0

Include Parapet = No

Factors and Coefficients

Structural Factor, cs<c$_{d}$> [EC 6.2(1)]	$c_s c_d = 0.85$
Elevation, z_0	$z_0 = 0.3$
Minimum Elevation, z_{min}	$z_{min} = 5$

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]	$k_r = 0.19(^{\frac{z_0}{0.05}}$	$k_r = 0.215389$
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

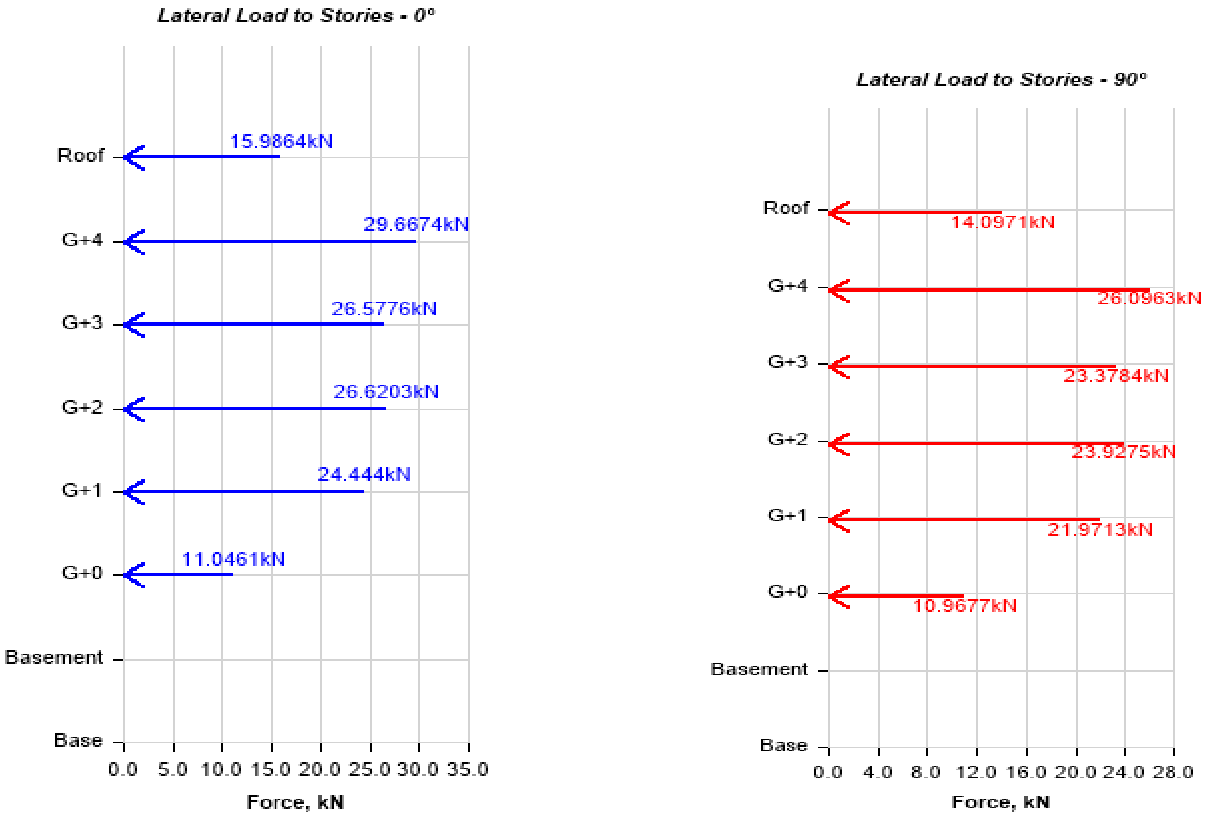


Figure 5-1. wind load along y-axis

Table 9.wind story forces

Story	**Elevation**	**X-Dir**	**Y-Dir**
	M	**kN**	**kN**
Roof	21	15.9864	0
G+4	18	29.6674	0
G+3	15	26.5776	0
G+2	12	26.6203	0
G+1	9	24.444	0
G+0	6	11.0461	0
Basement	3	0	0
Base	0	0	0

Story	**Elevation**	**X-Dir**	**Y-Dir**
	m	**kN**	**kN**
Roof	21	0	14.0971
G+4	18	0	26.0963
G+3	15	0	23.3784
G+2	12	0	23.9275
G+1	9	0	21.9713
G+0	6	0	10.9677
Basement	3	0	0
Base	0	0	0

b) Wind y direction

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

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = III

Wind Direction = 0 degrees

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

Top Story = Roof

Bottom Story = G+0

Include Parapet = No

Factors and Coefficients

Structural Factor, cs<$c_{d>}$ [EC 6.2(1)]		$c_s c_d = 0.85$
Elevation, z_0		$z_0 = 0.3$
Minimum Elevation, z_{min}		$z_{min} = 5$
Maximum Elevation, z_{max}		$z_{max} = 200$
Turbulence Factor, k_l [EC 4.4(1)]		$k_l = 1$
Orography Factor, c_o [EC 4.3.3]		$c_o = 1$
Turbulence Intensity, I_v [EC 4.4(1)]	$I_v = \frac{k_l}{c_o(z)\ln(\frac{z}{z_0})} \text{ for} z_{min} \leq z \leq z_{max}$ $= I_v(z_{min}) \text{ for } zz_{min}$	
Terrain Factor, k_r [EC 4.3.2(1) Eq. 4.5]	$k_r = 0.19(^{\frac{z_0}{0.05}}$	$k_r = 0.215389$
Roughness Factor, <$c_{r>(z)}$ [EC 4.3.2(1) Eq. 4.4]	$c_r(z) = k_r \ln(\frac{z}{z_0}) \text{ for} z_{min} \leq z \leq z_{max}$	

$= c_r(z_{min})$ 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

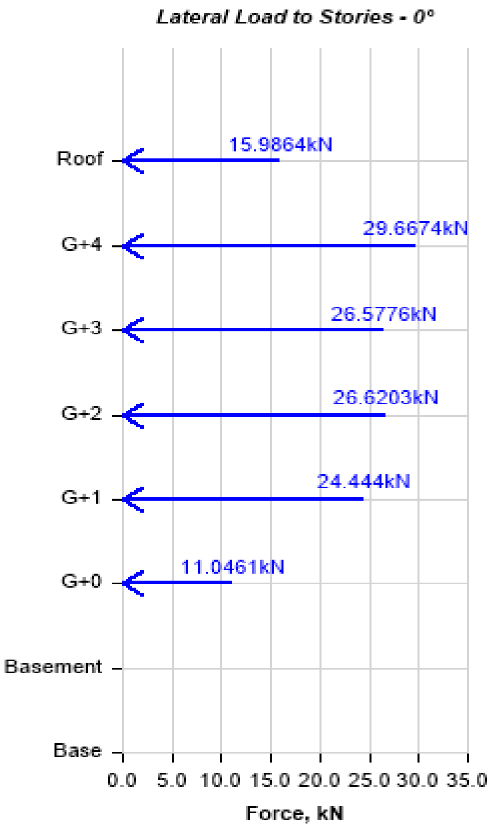


Figure 5-2. wind load along x-axis

Story	**Elevation**	**X-Dir**	**Y-Dir**
	m	**kN**	**kN**
Roof	21	15.9864	0
G+4	18	29.6674	0
G+3	15	26.5776	0
G+2	12	26.6203	0
G+1	9	24.444	0
G+0	6	11.0461	0
Basement	3	0	0
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 = D, 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 = 1.35 , for spectrum type 2 & ground

- Spectrum period, for spectrum type 2 & ground type D
    - Tb = 0.2
    - Tc = 0.8
    - 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)
- 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:

$TT=C_t \cdot H^{3/4}$

where

C_t	is 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures.
H	is the height of the building, in m, from the foundation or from the top of a rigid basement.

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

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

$T1\leq$ min (4Tc(s) =4*0.8=3.2sec, 2sec)

T1=3.2sec < 1sec…. not ok

Take T1=1sec

Table 10. 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:

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \tag{4.11}$$

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

$q=q_okw\geq1.5=0.8*3*1.3*1=3.12$

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

Sd(T) =0.2638

Table 11.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	18991.48	18991.48	7.3871	7.9429	18991.48	18991.48	7.3871	7.9429
G+4	D1	33104.15	33104.15	7.1179	8.1647	52095.63	52095.63	7.216	8.0838

Story	Diaphragm	Mass X kg	Mass Y Kg	XCM m	YCM m	Cumulative X kg	Cumulative Y kg	XCCM m	YCCM m
G+3	D1	33301.47	33301.47	7.0796	8.1921	85397.11	85397.11	7.1628	8.1261
G+2	D1	35938	35938	7.1336	8.595	121335.11	121335.11	7.1542	8.2649
G+1	D1	38140.59	38140.59	7.173	8.5282	159475.69	159475.69	7.1587	8.3279
G+0	D1	40700.98	40700.98	7.1814	8.5347	200176.67	200176.67	7.1633	8.3699
Basement	D1	37755.78	37755.78	7.2159	8.0575	237932.45	237932.45	7.1716	8.3204

$F_b = S_d(T_1) \cdot W_{tot} = 0.2638 * 237932.45 = 62766.58KN$

Ft=0.07T1Fb=0.07*1*62766.58=4393.66KN

a) Earth quake x direction

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

Direction and Eccentricity

Direction = X + Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075m$
Structure Height Above Base, H	$H = 21$ m

Factors and Coefficients

Country = Other

Design Ground Acceleration, a_g	$a_g = 0.07g$

Ground Type [EC Table 3.1] = D

Soil Factor, S [EC Table 3.3]	$S = 1.35$
Constant Acceleration Period Limit, T_B [EC Table 3.3]	$T_B = 0.15$ sec
Constant Acceleration Period Limit, T_C [EC Table 3.3]	$T_C = 0.5$ sec
Constant Displacement Period Limit, T_D [EC Table 3.3]	$T_D = 2$ sec
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0.2$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 4.9$

Seismic Response

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

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

for $T \leq T_B$

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

$$= a_g S \frac{2.5}{q}\left[\frac{T_C}{T}\right] \geq \beta a_g$$

for$T_C \leq T \leq T_D$

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

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	0.44	3816.4731	184.0085

Applied Story Forces

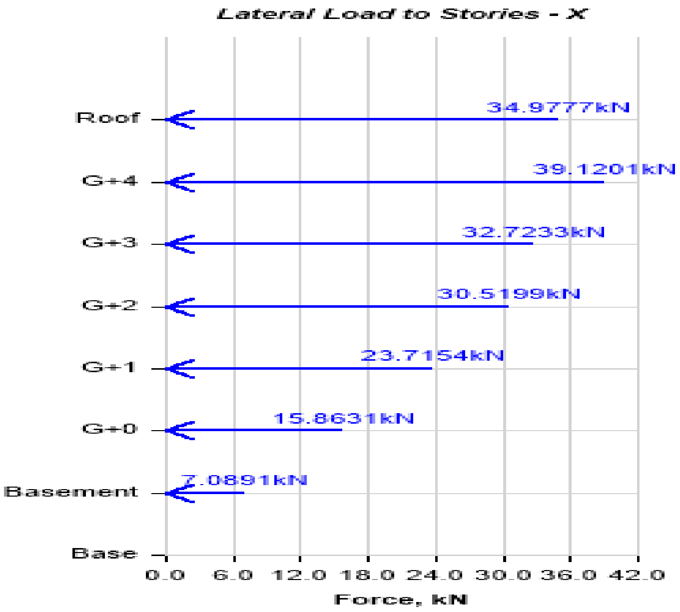


Figure 5-3. lateral load to stories x

Table 12. lateral load to stories x

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof	21	34.9777	0
G+4	18	39.1201	0
G+3	15	32.7233	0
G+2	12	30.5199	0
G+1	9	23.7154	0
G+0	6	15.8631	0
Basement	3	7.0891	0
Base	0	0	0

b) Earth quake y direction

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

Direction and Eccentricity

Direction = Y - Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2]	$C_t = 0.075\mathrm{m}$
Structure Height Above Base, H	$H = 21\ \mathrm{m}$

Factors and Coefficients

Country = Other

Design Ground Acceleration, a_g	$a_g = 0.07g$

Ground Type [EC Table 3.1] = D

Soil Factor, S [EC Table 3.3]	$S = 1.35$
Constant Acceleration Period Limit, T_B [EC Table 3.3]	$T_B = 0.15\ \mathrm{sec}$
Constant Acceleration Period Limit, T_C [EC Table 3.3]	$T_C = 0.5\ \mathrm{sec}$
Constant Displacement Period Limit, T_D [EC Table 3.3]	$T_D = 2\ \mathrm{sec}$
Lower Bound Factor, β [EC 3.2.2.5(4)]	$\beta_0 = 0.2$
Behavior Factor, q [EC 3.2.2.5(3)]	$q = 4.9$

Seismic Response

Spectral Response Acceleration, Sd(T_1) [EC 3.2.2.5(4) Eq. 3.13]	$S_d(T_1) = a_g S\left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{q} - \frac{2}{3}\right)\right]$ for $T \leq T_B$ $= a_g S\frac{2.5}{q}$ for $T_B \leq T \leq T_C$

$$= a_g S \frac{2.5}{q}\left[\frac{T_C}{T}\right] \geq \beta a_g$$

$\text{for} T_C \leq T \leq T_D$

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

$\text{for} T_D \leq T$

Equivalent Lateral Forces

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

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
Y - Ecc. X	0.44	3816.4731	184.0085

Applied Story Forces

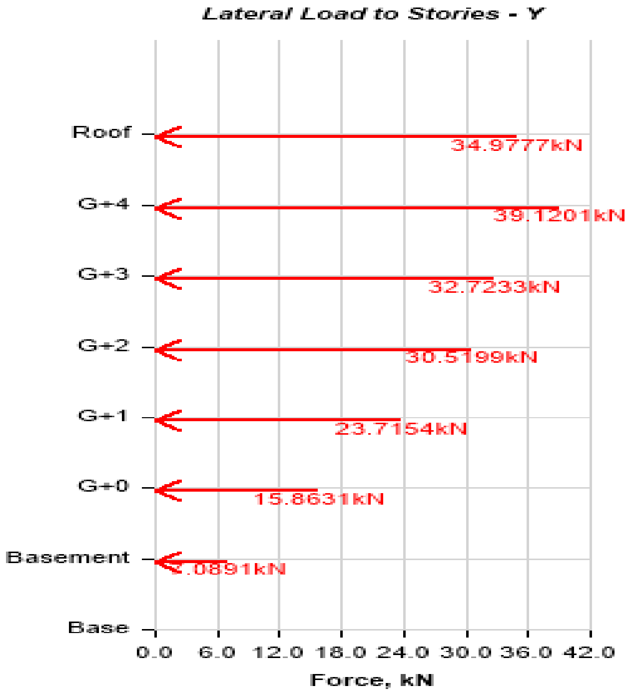


Figure 5-4.lateral load on story y

Table 13.lateral load on story y

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Roof	21	0	34.9777
G+4	18	0	39.1201
G+3	15	0	32.7233
G+2	12	0	30.5199
G+1	9	0	23.7154
G+0	6	0	15.8631
Basement	3	0	7.0891
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.(Kassimali, 2011)

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.

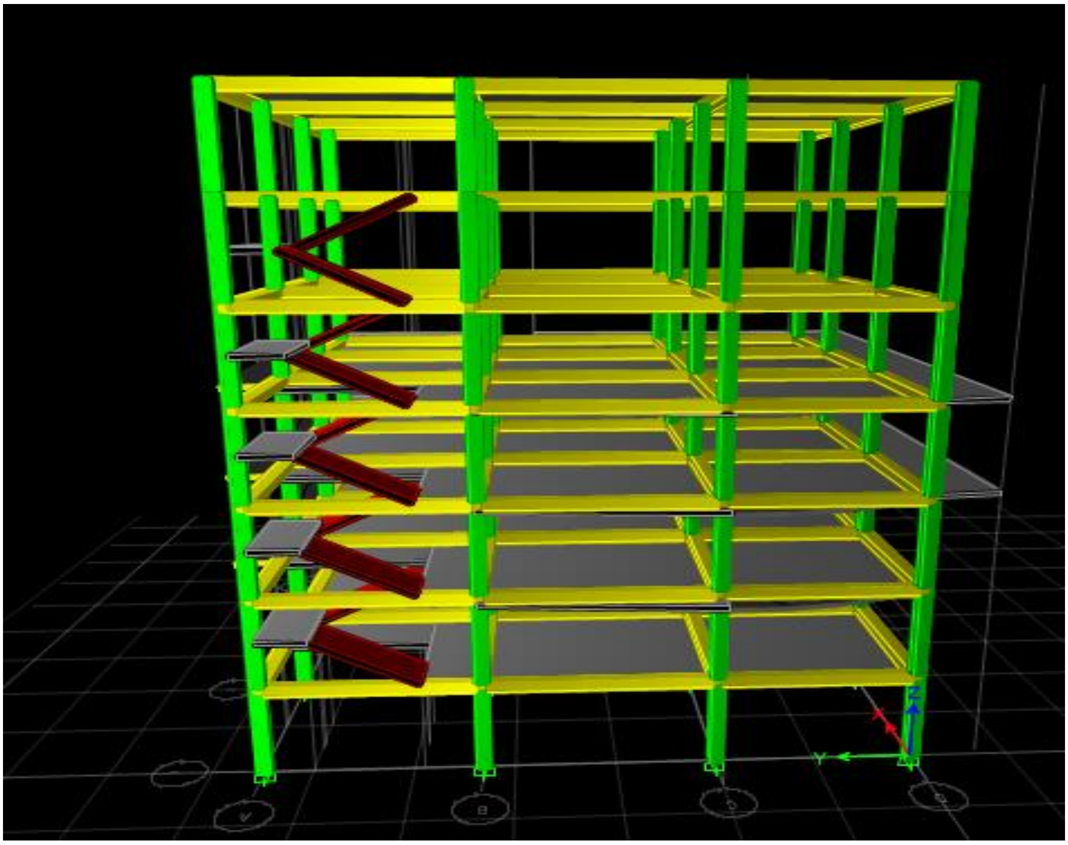


Figure 6-1.3D of the building

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*, 2015a)

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

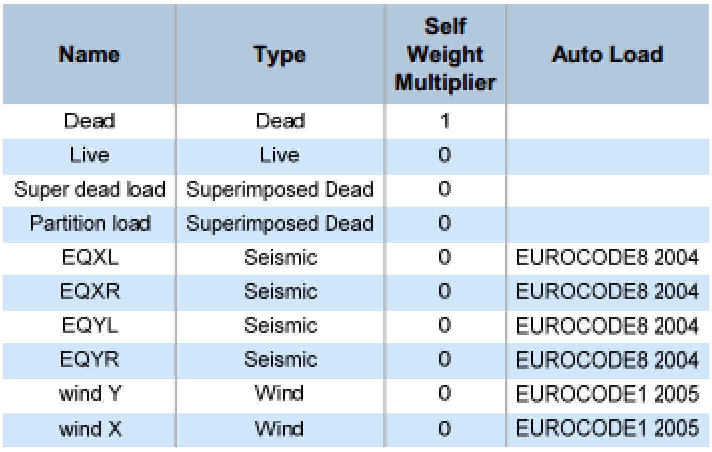
Coefficient of Thermal expansion=0.0000117

5. Define Section properties: - for initial we define column, beam and slab sections.
    a. Beam: - Cross section 300x500 and top tie beam 300x300
    b. Column: - Cross section 500x500 and 400x400
    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

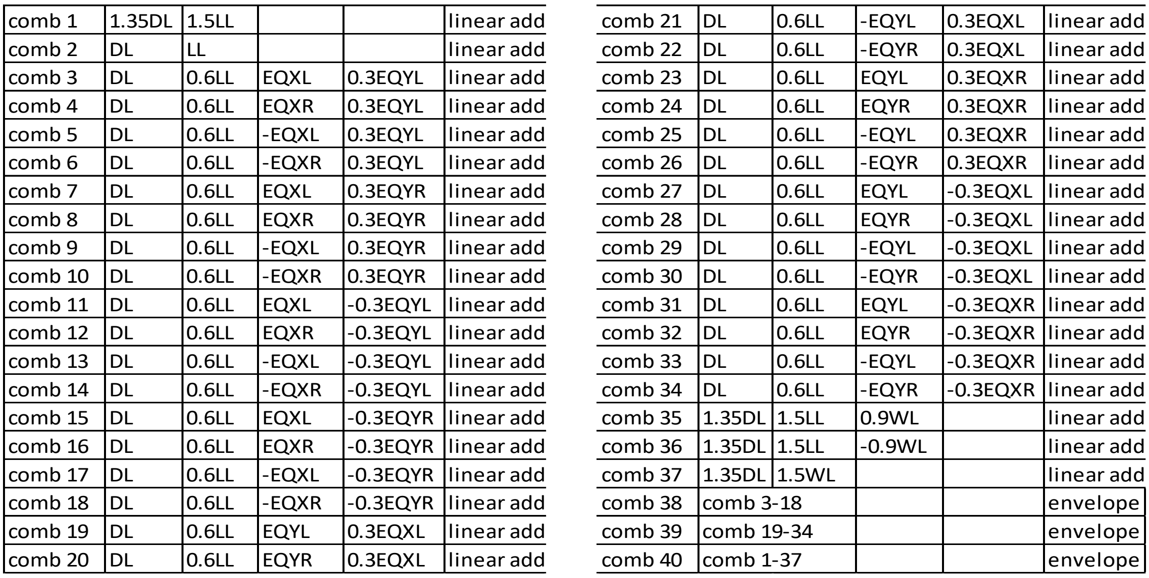
Table 14.load patterns



Name	Type	Self Weight Multiplier	Auto Load
Dead	Dead	1	
Live	Live	0	
Super dead load	Superimposed Dead	0	
Partition load	Superimposed Dead	0	
EQXL	Seismic	0	EUROCODE8 2004
EQXR	Seismic	0	EUROCODE8 2004
EQYL	Seismic	0	EUROCODE8 2004
EQYR	Seismic	0	EUROCODE8 2004
wind Y	Wind	0	EUROCODE1 2005
wind X	Wind	0	EUROCODE1 2005

8. Define load combinations.

Table 15.load combination



comb 1	1.35DL	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 1-37				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

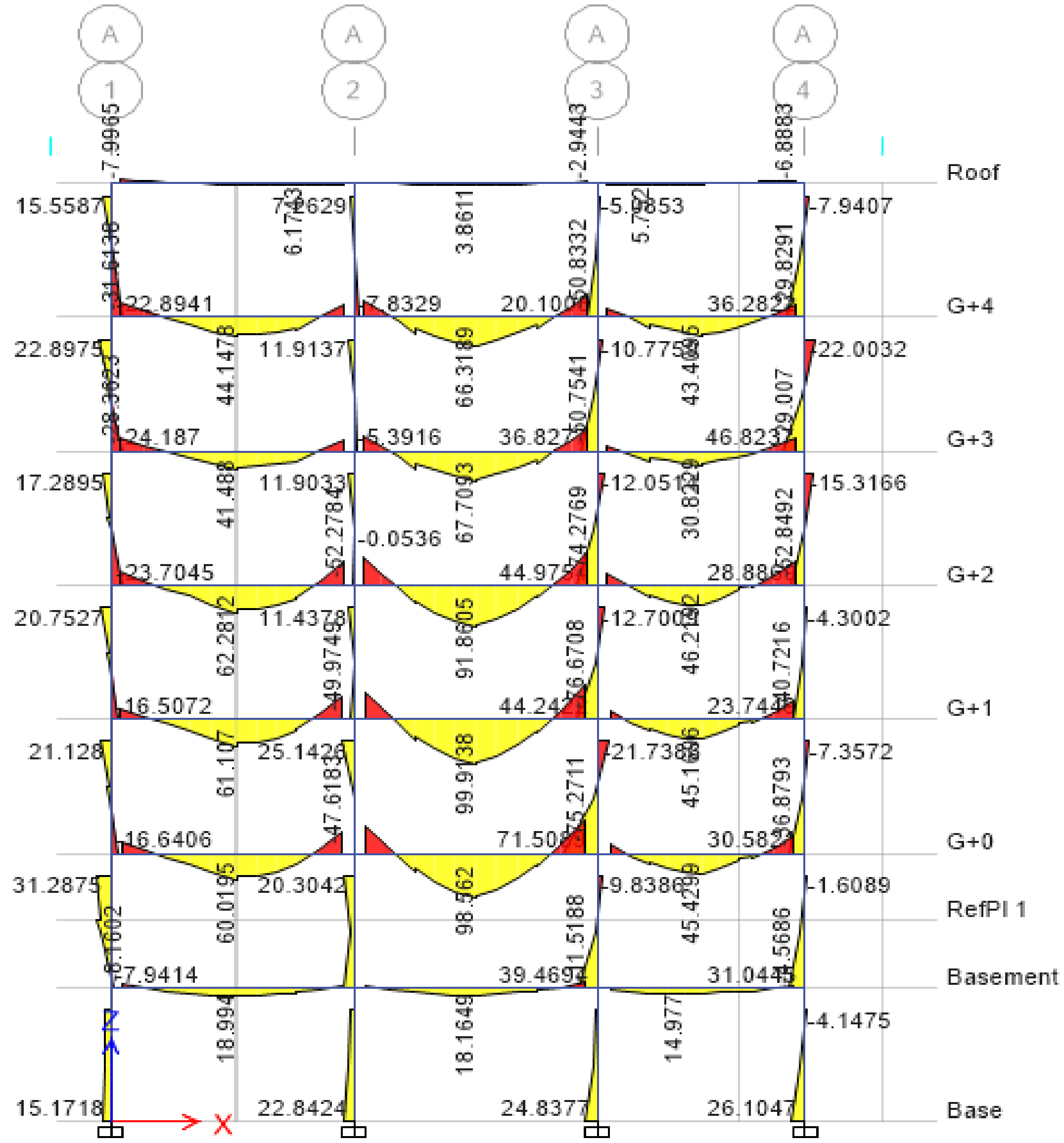


Figure 6-2bending moment diagram

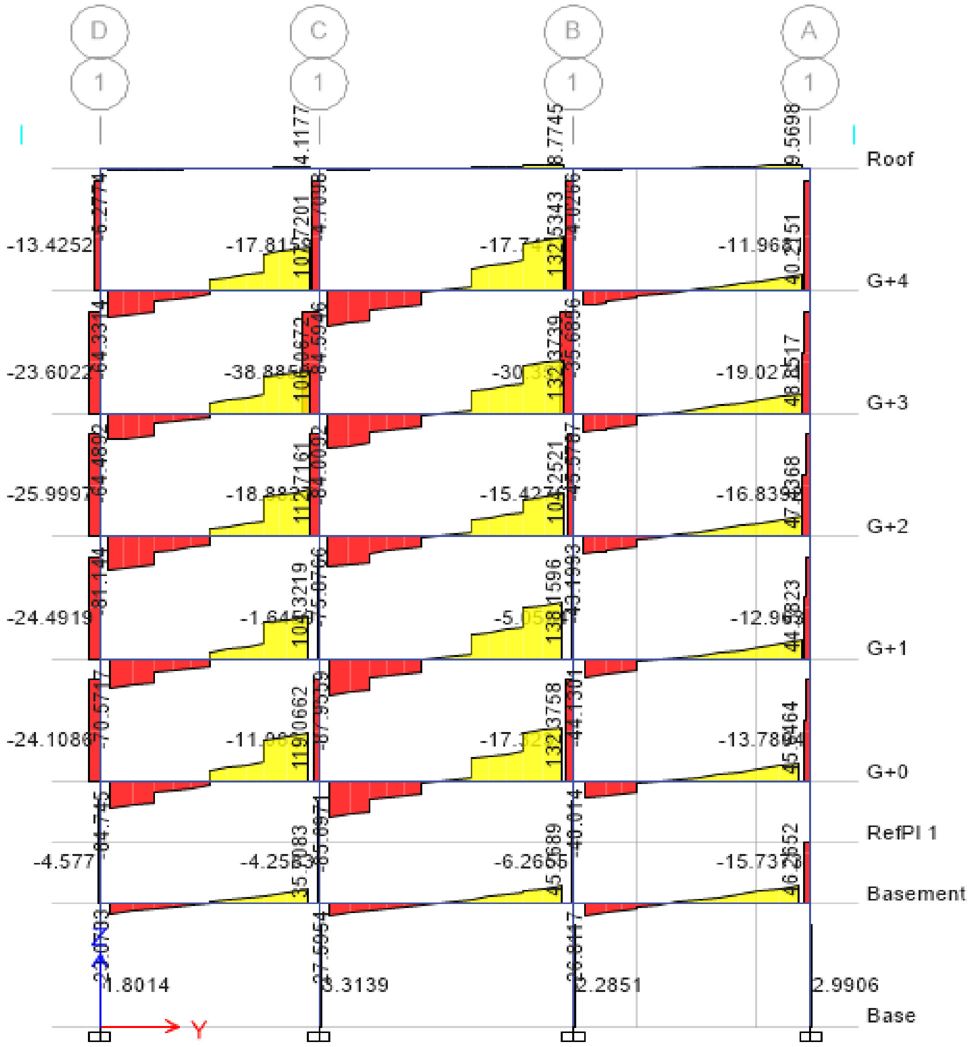


Figure 6-3shear force diagram

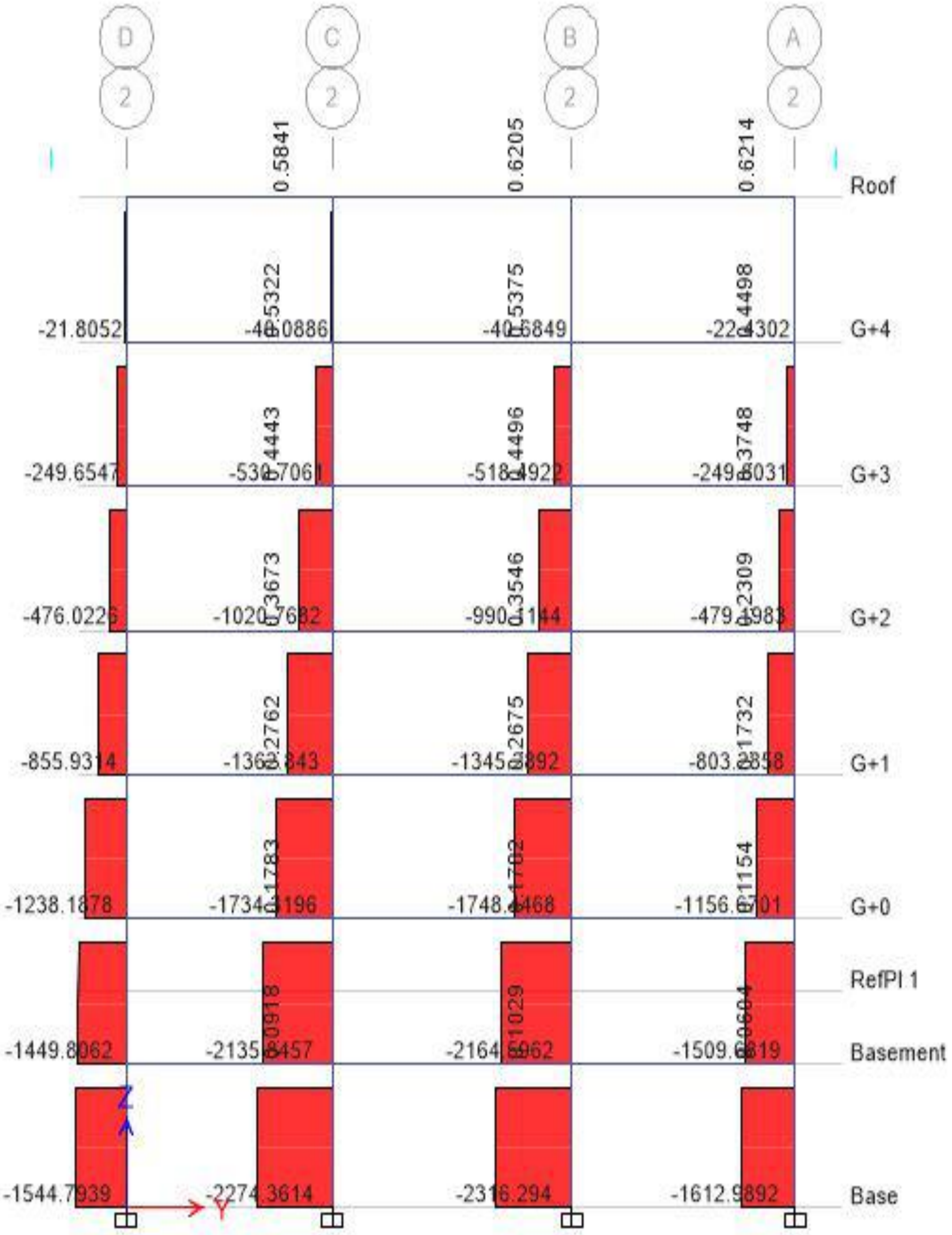


Figure 6-4Axial load diagram

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. (Kassimali, 2011)

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

B -Steel S-400

- ✓ Steel tensile strength, 400MPA
- ✓ Steel tensile design strength, $fyd = \frac{\text{fyk}}{\gamma s} = \frac{400}{1.15}$ = 347.83MPa
- ✓ 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(*ES EN 2 Part 1-1*, n.d.)

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.

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

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$\begin{cases} Cmin, b \\ Cminb, dur + \Delta Cdur, \Upsilon - \Delta Cdur, st - \Delta Cdur, add \\ 100 \end{cases}$

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

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

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

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

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

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

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*, 2015a)

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

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

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

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

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

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

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

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

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

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

$f_{ctd} = 0.85$

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

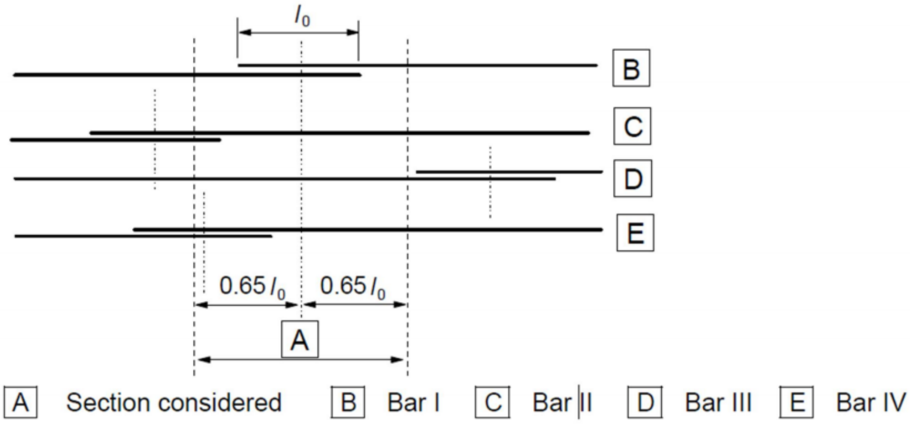
$\sigma sd = \frac{fyk}{\gamma_s} = \frac{400}{\mathbf{1.15}} = \mathbf{347.83}$

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

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

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



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

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

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

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

α_1 α_2 α_3 α_5 coefficients

l_{bd} = α_1 α_2 α_3 α_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} = α_1 α_2 α_3 α_5 $l_{b,rqd} \geq l_{b,min}$

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

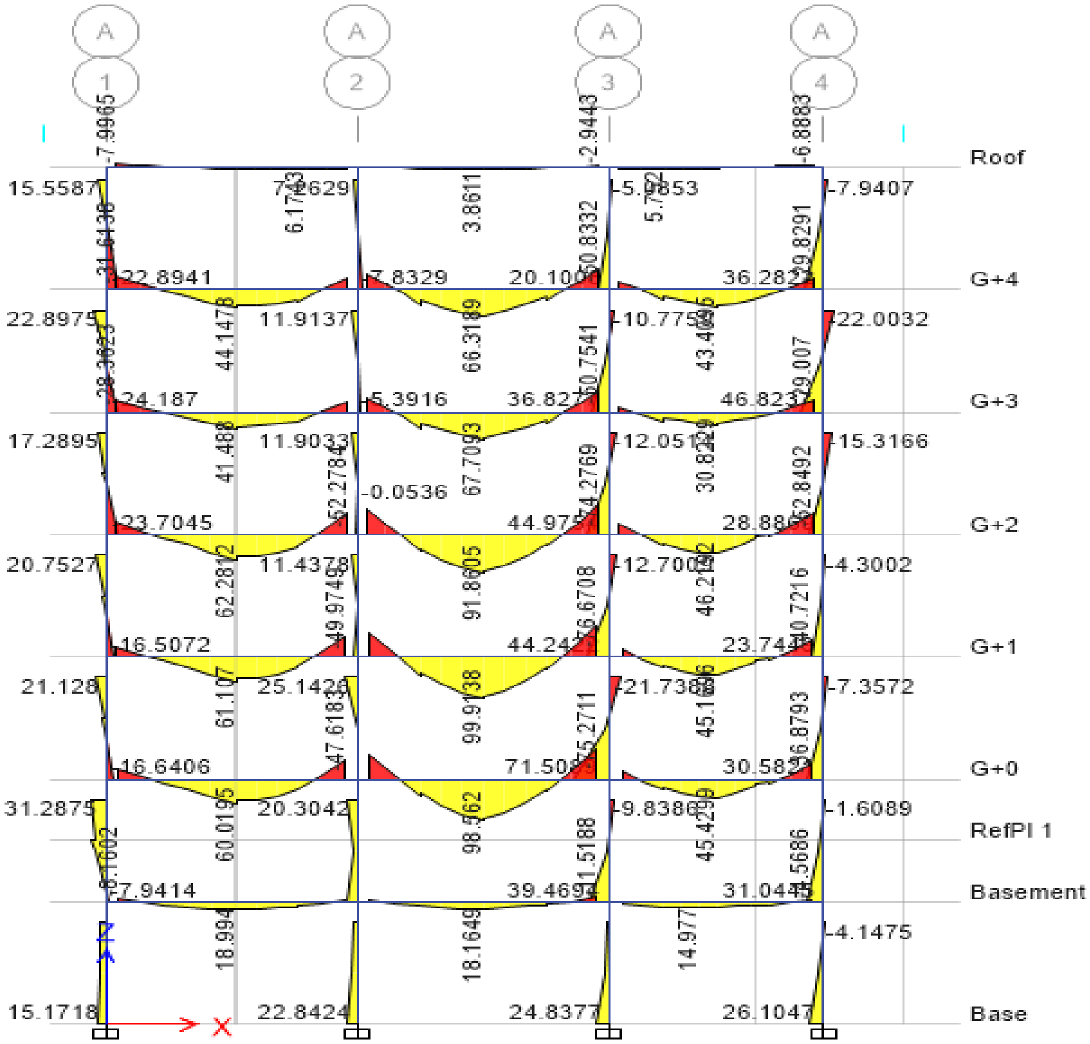


Figure 7-1.moment 3-3 diagram

Table 16.beam reinforcement

Axis A

story- 7

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	31.61	300	450	0.0260	439.4194	single	193.05	5400	206.8155	206.8155	0.66	2Ø20
span	1-2	44.15	300	450	0.0363	435.0744	single	193.05	5400	291.7461	291.7461	0.93	2Ø20
support	2	39.07	300	450	0.0322	436.8454	single	193.05	5400	257.1305	257.1305	0.82	2Ø20
span	2-3	66.32	300	450	0.0546	427.1644	single	193.05	5400	446.3621	446.3621	1.42	2Ø20
support	3	50.83	300	450	0.0418	432.7228	single	193.05	5400	337.7133	337.7133	1.08	2Ø20
span	3-4	43.41	300	450	0.0357	435.3333	single	193.05	5400	286.6855	286.6855	0.91	2Ø20
support	4	29.83	300	450	0.0246	440.0291	single	193.05	5400	194.8991	194.8991	0.62	2Ø20

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

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

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 tanα_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

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

Where

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

$A_{\boldsymbol{sw}}$ 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:

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

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

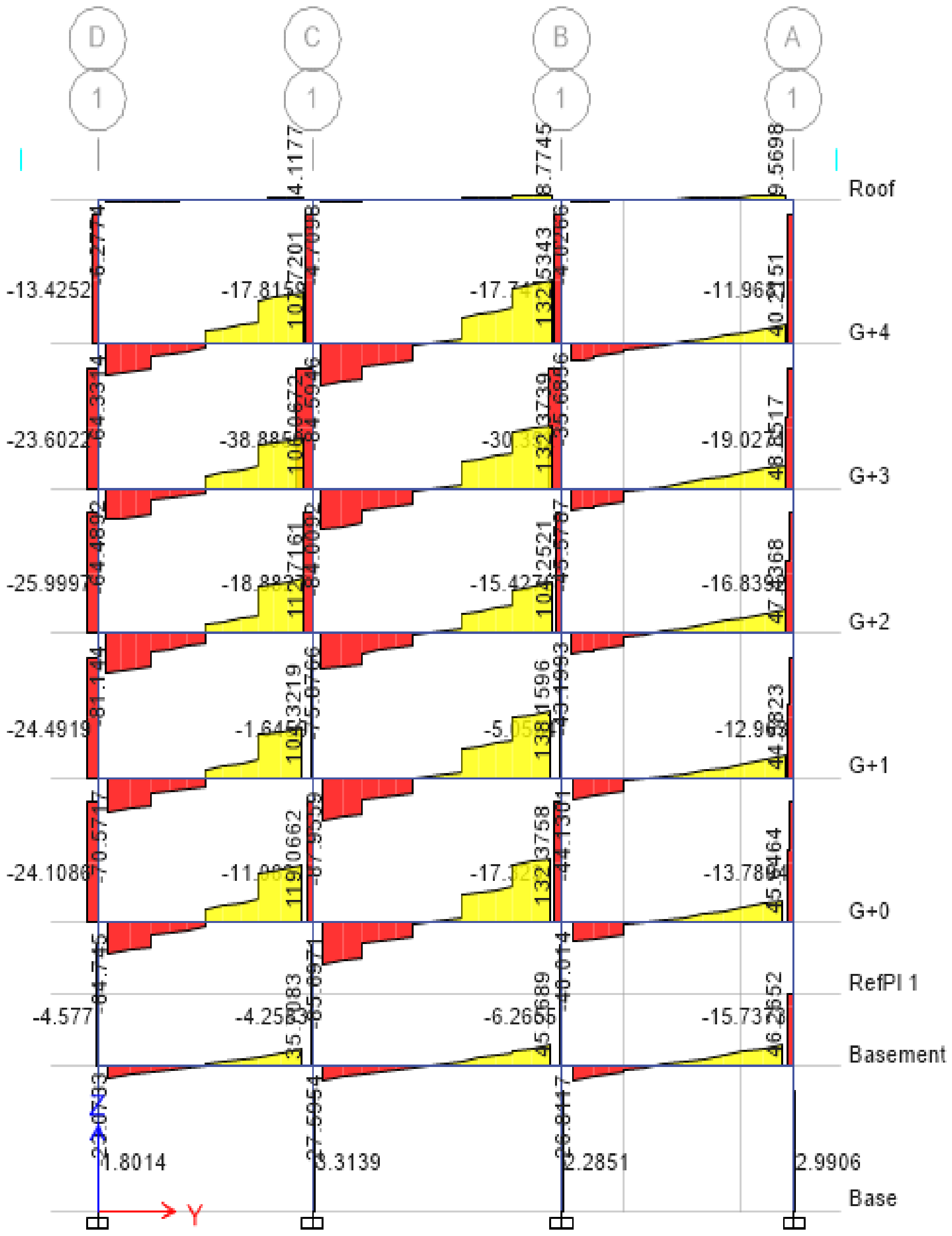
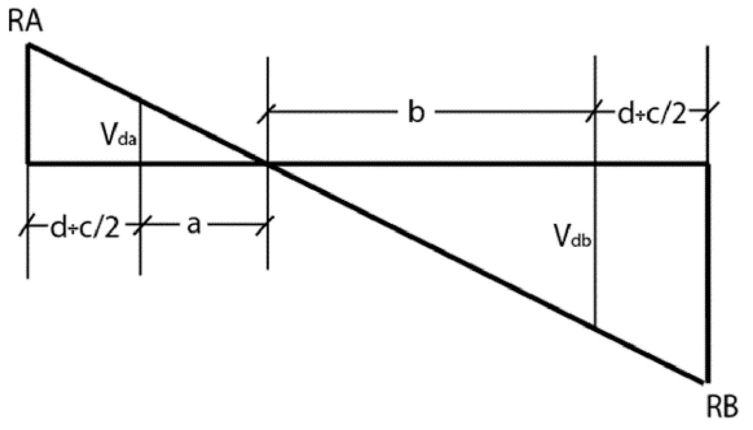


Figure 7-2.shear 2-2 diagram

RA=	192.46	KN
RB	123.96	KN
L=	5.8	m
d=	450	mm
C=	300	mm
D+c/2	0.6	m
a=	2.927805	m
b=	1.672195	m
VEda=	159.7269	KN
VEdb=	91.2269	KN
VEd=	159.7269	KN



Concrete 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= 450
b= 300
Asl= 314
k= 1.67 0

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

taking minimum reinforcment ø 8
@ 260

$\rho_1 =$ 0.0023
$v_{Rd,c} =$ 45.072
$v_{Rd,c} =$ VRdc <= VEd, shear reinforcment design required

longtudinal bar diam.= 16
minimum spacing = 240

diagonal compression check

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

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

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

V1= 0.552
fcd= 11.33
$z = 0.9d$
Z= 405
$cot\theta = 2.5$
$\tan\theta = 0.4$
$V_{Rd,max} =$ 262.0277
VRd max>VEd, so this section does not requre shear reinforcment for diagc

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 $S_{b,max}$

$S_{b,max} = 0.75d(1 + cot\alpha) = 0.6 * 367(1 + cot90)$ =337.5mm

$$S_{min} = \frac{A_{sw}}{\rho_w b_w sin\alpha} = \frac{78.5}{0.00089x250x1} = 292.6$$

Required shear reinforcement

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

$S_{calc} = \frac{A_{sw}0.78df_{yk}cot\theta}{V_{ED}} = \frac{50.24*0.9*367*400}{13.44} = 176.1$mm

$$\boldsymbol{use\ s = 175mm}$$

Then they remain beam spacing in tabular form below and appendix

Table 17.shear reinforcement

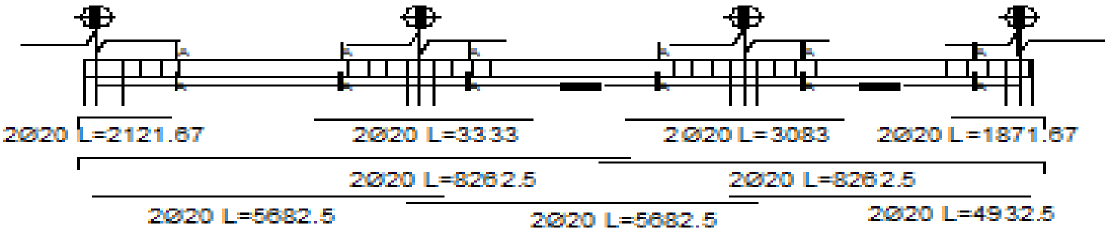
axis 1

Story -8

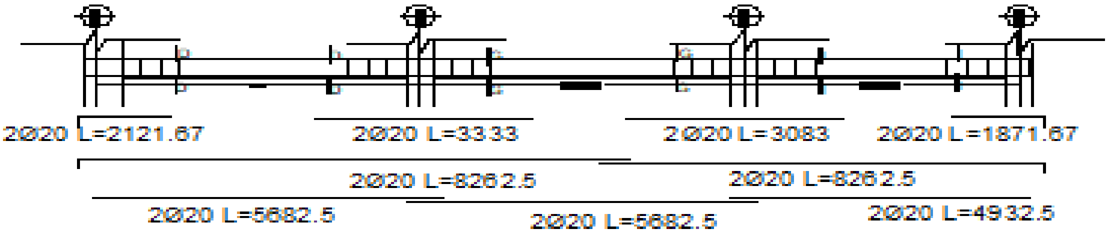
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	6.3	5.47	337.5	5054.0	292.6	337.50	330
B-C	8.8	7.87	337.5	3512.8	292.6	337.50	330
A-B	9.6	8.59	337.5	3218.4	292.6	337.50	300
Story -7							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	107.7	88.59	337.5	312.1	292.6	312.06	300
B-C	132.53	110.07	337.5	251.2	292.6	251.16	250
A-B	40.22	31.11	337.5	888.6	292.6	337.50	330

7.5. Beam reinforcement detailing

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



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



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

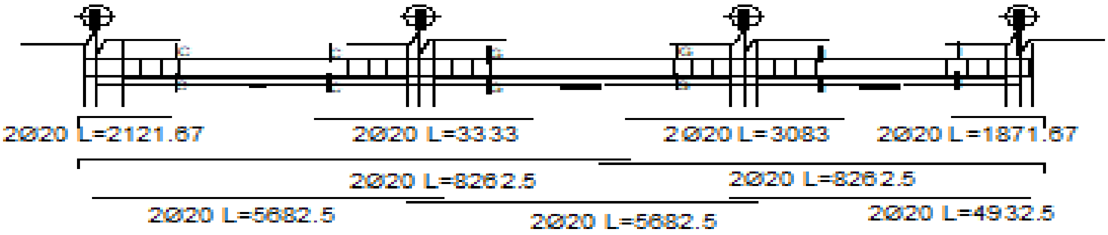
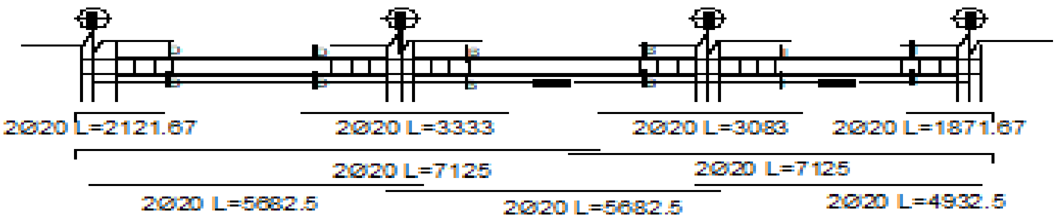
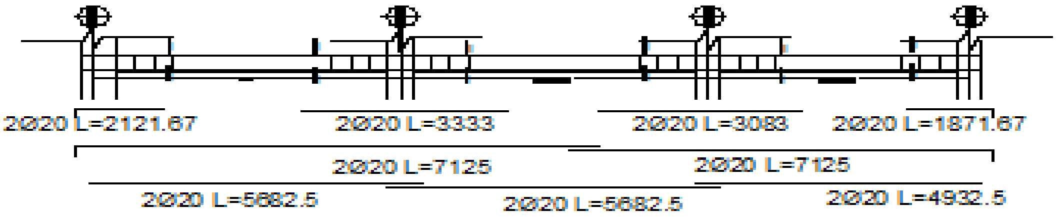


Figure 7-3.longitudinal reinforcement detailing

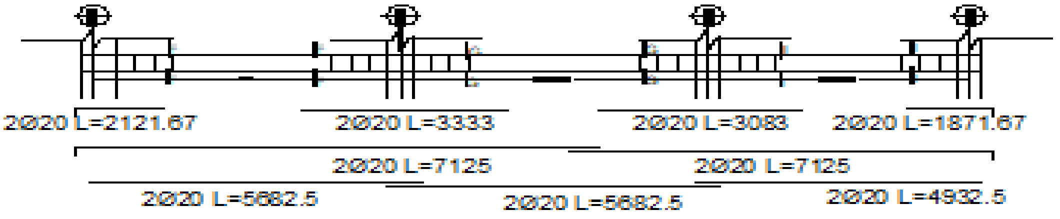
Second floor beam A4-B4-C3 -C4 & D5 on axis 1



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



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



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

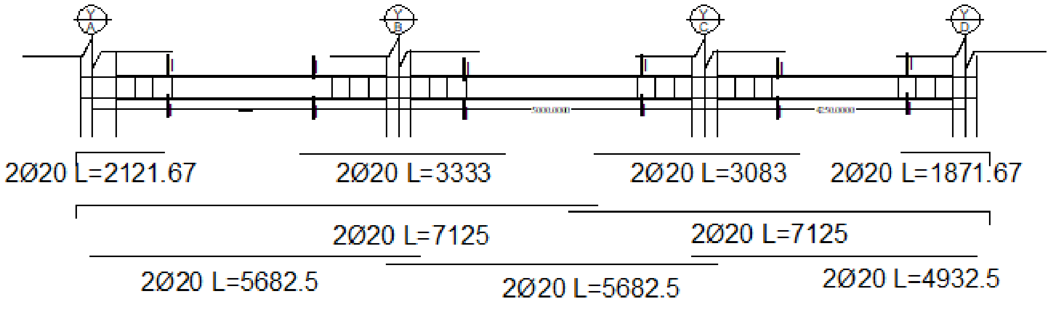


Figure 7-4.longtiuidinal reinforcement detailing

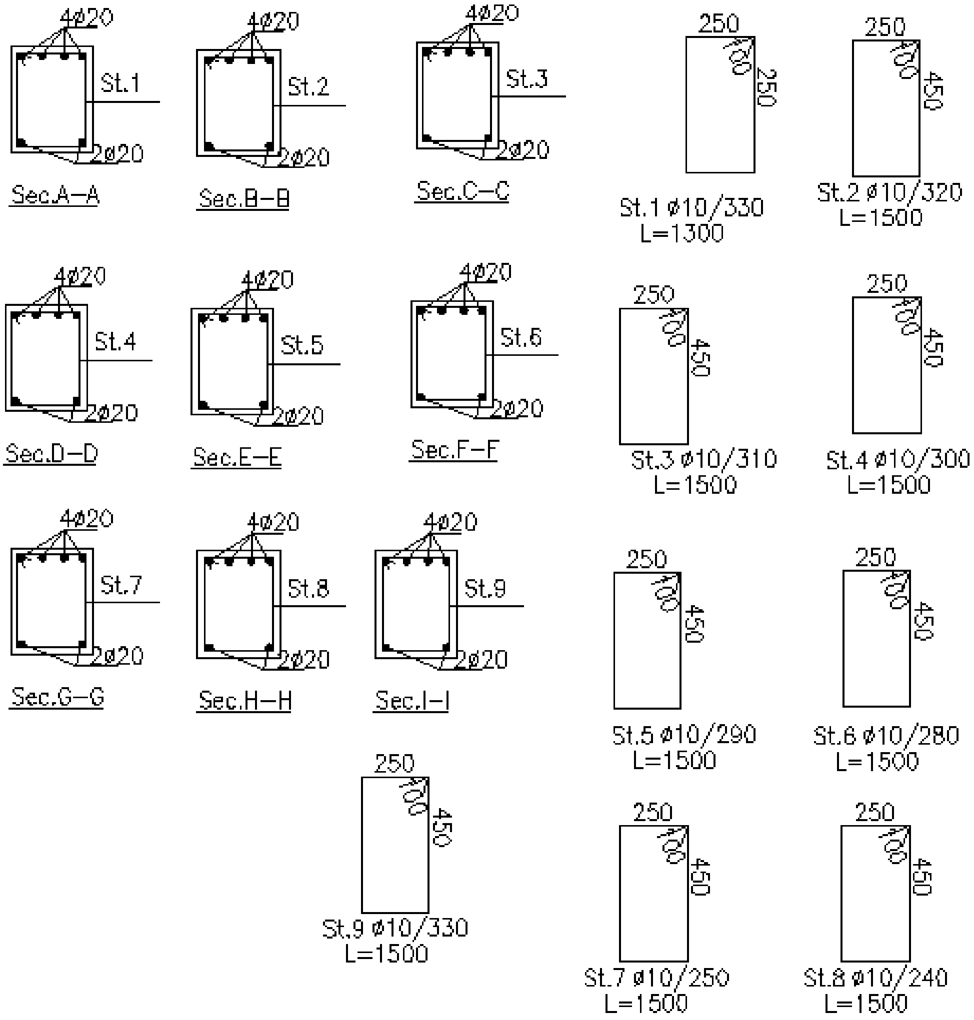


Figure 7-5.stirrup reinforcement detailing

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

- ✓ Based on lateral reinforcement, tied columns, spiral columns
- ✓ Based on manner by which lateral stability is provided to the structure as a whole, braced and unbraced columns
- ✓ Based on sensitivity to second order effect due to lateral displacements; sway and non-sway columns
- ✓ Based on degree of slenderness, short and slender columns
- ✓ Based on 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*, 2015a)

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*, 2015a)

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*, 2015a) 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,* 2015a) Art 5.8.3.1

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

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

Accidental eccentricity

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

In the both direction

8.2.4. Calculate reinforcement

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

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

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

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

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

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

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

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

For shear reinforcement

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

Sample calculation

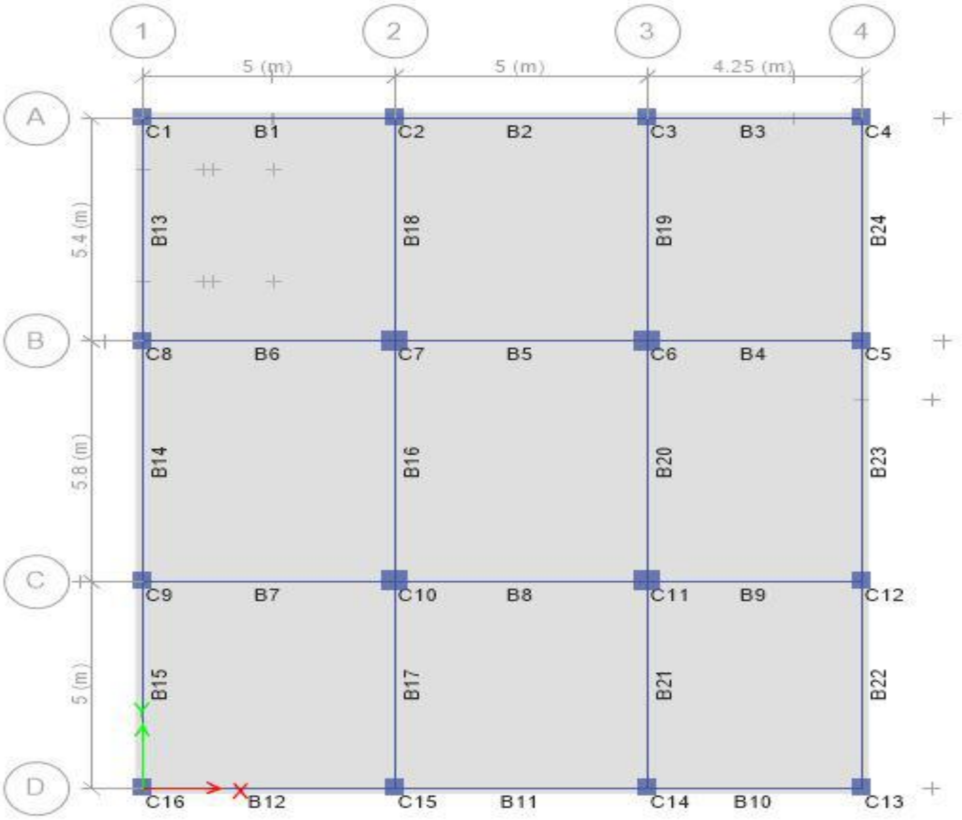


Figure 8-1. naming of column and beams

Effective length determination

Let calculate for C-6 on roof elevation

Given data

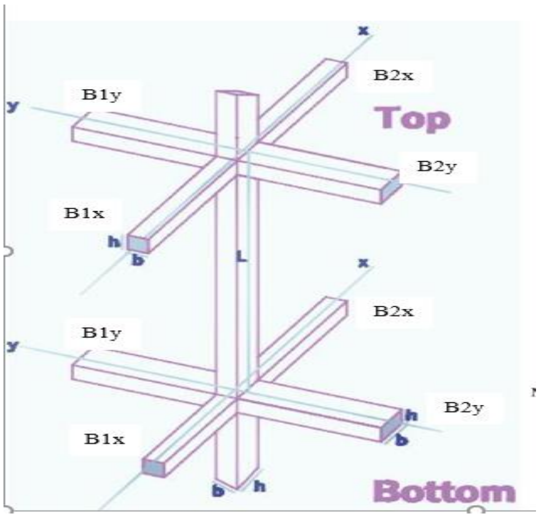


Figure 8-2. center column cross-section

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)	500	300	5000
B2 (x-x) (continous)	500	300	5000
B1 (y-y) (continous)	500	300	5000
B2 (y-y) (continous)	500	300	5800
Bott beam			
B1 (x-x) (continous)	500	300	5000
B2 (x-x) (continous)	500	300	5000
B1 (y-y) (continous)	500	300	5000
B2 (y-y) (continous)	500	300	5800

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

$$K_{1x} = \frac{\left(\frac{\frac{500*500^3}{12}}{3000}\right)}{\left(\frac{\frac{300*500^3}{12}}{5000}\right)+\left(\frac{\frac{300*500^3}{12}}{5000}\right)} = \frac{1736111.11}{625000+625000} = \mathbf{1.389}$$

Same for $K_{2x} = \mathbf{1.389}$

$$K_{1y} = \frac{\left(\frac{\frac{500*500^3}{12}}{3000}\right)}{\left(\frac{\frac{300*500^3}{12}}{5000}\right)+\left(\frac{\frac{300*500^3}{12}}{5800}\right)} = \frac{1736111.11}{1163793.1} = \mathbf{1.492}$$

Same for K_{2y}=**1.492**

$l_{ox} = 0.5*3000\sqrt{\left(1+\frac{1.389}{0.45+1.389}\right)\cdot\left(1+\frac{1.389}{0.45+1.389}\right)}$=**2632.93m**

$l_{oy} = 0.5*2800\sqrt{\left(1+\frac{1.492}{0.45+1.492}\right)\cdot\left(1+\frac{1.492}{0.45+1.492}\right)}$=**2652.38m**

Check slenderness limit

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

$$i = \sqrt{\frac{I_c}{A}} = \sqrt{\frac{\frac{300*500^3}{12}}{300*500}} = 144.34$$

$$\lambda_x = \frac{l_{ox}}{i} = \frac{2632.93}{144.34} = 18.24$$

$$\lambda_y = \frac{l_{oy}}{i} = \frac{2652.38}{144.34} = 18.38$$

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

$$e_{ix} = max\begin{Bmatrix} \frac{2632.93}{400} = 6.58 \\ \frac{500}{30} = 16.67 \\ 20mm \end{Bmatrix} = \mathbf{20\ mm}$$

$$e_{iy} = max\begin{Bmatrix} \frac{2652.38}{400} = 6.63 \\ \frac{500}{30} = 16.67 \\ 20mm \end{Bmatrix} = \mathbf{20\ mm}$$

$M01x = min\{|17.83|, |-4.05|\} + 0.02 * 1337.8$= 30.806Knm

$M02x = max\{|17.83|, |-4.05|\} + 0.02 * 1337.8$= 44.58Knm

$M01y = min\{|20.90|, |4.51|\} + 0.02 * 1337.8$= 31.27Knm

$M02y = min\{|20.90|, |4.51|\} + 0.02 * 1337.8$= 47.66Knm

$$n = \frac{NED}{A_c f_{cd}} = \frac{1337.8}{500 * 500 * 11.33} = 0.472$$

$$\text{λxlim} = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{30.806}{44.58})}{\sqrt{0.472}}$$

$\text{λxlim} = \mathbf{22.62}$

$$\text{λylim} = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{31.27}{47.66})}{\sqrt{0.472}}$$

$\text{λylim} = \mathbf{22.71}$

18.24 < 22.62, column is not slender

18.38 < 22.71 column is not slender

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

$M_{EDx} = max\{44.58; 0.02 * 1337.8 = 26.76\}$

$M_{EDy} = max\{47.66; 0.02 * 1337.8 = 26.76\}$

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*, 2015a)

$$V_{sd} = \frac{N_{sd}}{A_c f_{cd}} = \frac{1337.8 * 1000}{500 * 500 * 11.33} = 0.472$$

$\mu_{sd,x} = \frac{M_{EDx}}{f_{cd} * A_c * h} = \frac{44.58 * 1000000}{11.33 * 500 * 500 * 500} = 0.03$

$\mu_{sd,y} = \frac{M_{EDy}}{f_{cd} * A_c * h} = \frac{47.66 * 1000000}{11.33 * 500 * 500 * 500} = 0.03$

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

Minimum and maximum area of reinforcement

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

$$A_{s,min} = max\begin{Bmatrix} \left(0.1 * \frac{1337.8}{\frac{400}{1.15}} * 1000\right) = 291 \\ 0.002 * 500 * 500 = 500 \end{Bmatrix}$$

$A_{s,min} = 500mm2$

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

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

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

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

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

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

$\eta_1 = 1, our$ 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 **α_6 = 1.4** from the table value for $\rho_1 > 50\ \%$,

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

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

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**

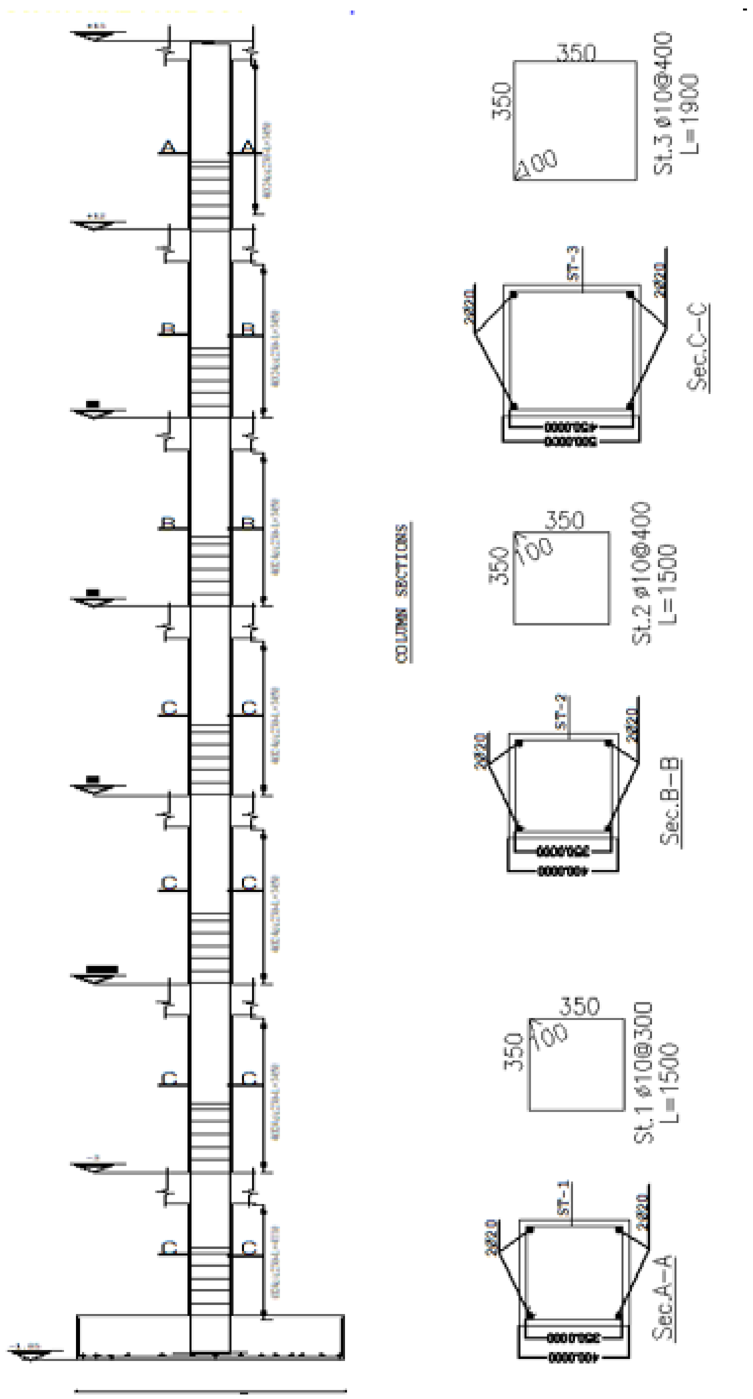


Figure 8-3.column reinforcement detailing

9. DESIGN OF SHEAR WALL

9.1. Introduction

A reinforced concrete wall is a vertical loadbearing member whose greatest lateral dimension is more than four times its least lateral dimension, and in which the reinforcement is taken in to account when considering its strength, A reinforced wall shall be considered as either short or slender and as either braced or unbraced as follows:

Short or Slender Walls: A wall may be considered short when the ratio of its effective height to its thickness does not exceed 7. It shall otherwise be considered slender.

Braced or Unbraced Walls: A wall may be considered as braced if, at right Angles to the plane of the wall, lateral stability to the structure as a whole is provided by walls or Other suitable bracing designed to resist all lateral forces in that direction. It shall otherwise be considered as unbraced.

Following the approximate method, the wall shall be design for uniaxial bending with an Equivalent eccentricity of load along the axis parallel to the larger relative eccentricity

$\boldsymbol{eeq}=\boldsymbol{etot}*(\mathbf{1}+\boldsymbol{k\alpha})$

Where:

$\boldsymbol{etot}$= total eccentricity in the direction of the larger relative eccentricity

k=relative eccentricity ratio

α=is obtained as a function of the relative normal force, =($\boldsymbol{fcd}*\boldsymbol{Ac}$)

The lateral load due to seismic action and the vertical loads from self-weight of the elevator car, top slab & from live load can be determine

9.2.Basement shear wall design

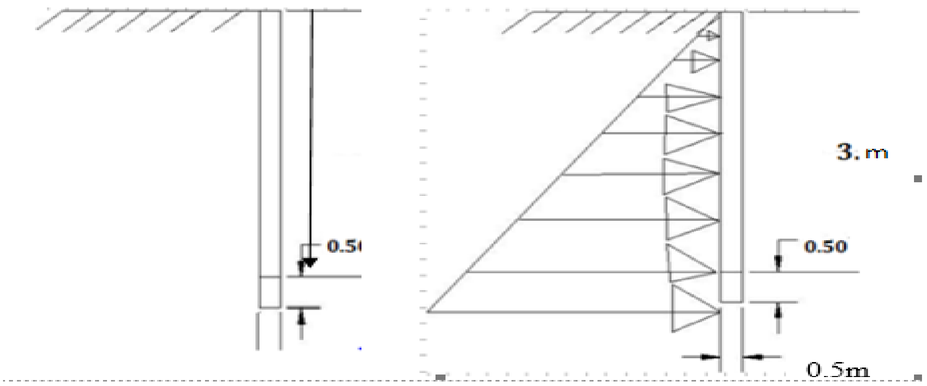


Figure 9-1.basement shear wall

Wall is a vertical load bearing member whose length exceeds four times its thickness.

Unbraced wall: is designed to carry lateral loads (horizontal loads) in additional to vertical loads. Mainly constructed by reinforced concrete.

Braced wall: does not carry any lateral loads (horizontal loads). All horizontal loads are carried by principal structural bracings or lateral supports. This type of wall is used mainly in steel structures.

Wall also subdivided in to two based on its length effect.

Stocky wall: where the effective height (Hc) divided by the thickness (t) doesn't exceed 15 for a braced wall and 10 for an unbraced wall.

- ✓ Hct≤15, unbraced wall
- ✓ Hct≤10, OR unbraced wall

Slender wall: is a wall other than stocky wall.

Our wall is unbraced wall that means it can resist horizontal loads in addition to vertical loads.

9.2.1. Concrete Cover Determination

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

$Cnom = Cmin + \Delta Cdev$ ($ES\ EN$ 1992: 2015) 4.4.12(1)

$$Cmin = Max\begin{cases} Cmin.b \\ Cmin,dur + \Delta Cdur,\gamma - \Delta Cdur,st - \Delta Cdur,and \\ 10mm \end{cases}$$

Where;

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

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

ΔCdur,γ- additive safety element, see ES EN Art 4.4.1.2 (6)

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

ΔCdur,add -reduction of minimum cover for use of additional protection, see ES EN Art 4.4.1 .2 (8)

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

a) Cover Design for Bond

In order to transmit bond forces safely and to ensure adequate compaction of the Concrete, the minimum cover should not be less than Cmin, b given in Table 4.2. (EBCS EN 2)

Assume Φ10 longitudinal bar and Φ20 nominal maximum aggregate size; Therefore; Cmin, b=10mm.

b) Cover Design for Corrosion/Durability

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

But based on the table Exposure class according to Table 4.1 the exposure class is Reduce by 1 and the structural class would be S3.

Therefore, the value of minimum cover required for durability of reinforcement steel is determined using

Cmin,dur=20mm

$$Cmin = Max\begin{cases} Cmin.b = 10mm \\ Cmin,dur = 20mm \\ 10mm \end{cases}$$

$Therefore$; $Cmin$= 20mm

c) ΔCdev (allowance in Design for Variation

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

$Then$; $Cnom$= $Cmin$+ $\Delta Cdev$= 10mm + 20mm = 30mm

9.2.2. Determination of depth for deflection

the limit state of deformation may be checked by either: by limiting the span/depth ratio, according to 7.4.2 or by comparing a calculated deflection, according to 7.4.3, with a limit value(*ES EN 1998-1*, 2015)

$$\frac{l}{d} = \mathrm{k}\left[11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho} + 3.2\sqrt{fck}\left(\frac{\rho o}{\rho}\right)^{\frac{3}{2}} * F1 * F2 * F3\right] \quad \text{if } \rho \leq \rho o$$

$$\frac{l}{d} = \mathrm{k}\left[11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho} + \frac{1}{12}\sqrt{fck} * \sqrt{\frac{\rho}{\rho}} * F1 * F2 * F3\right] \quad \text{if } \rho > \rho o$$

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/$\boldsymbol{\sigma s}$ =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.

K = 1.3 for end span

Assumption: - Slab is lightly reinforced (ρ=0.5%) based on Euro code 2 Part 1,1 - pr EN 1992-1-1-2002 Section 7.4.2.

P$_0$=$\sqrt{fck * 10^{-3}}$=$\sqrt{20 * 10^{-3}}$=0.00447

For end span of one-way continuous slab

$\frac{l}{d}$= 1.3 ∗ 11 + 1.5 ∗ 4.47 ∗ 0.89 = 20.27

K = 1.3, $\frac{l}{d}$=20.27 because we used S400 multiply the value by $\frac{500}{fyk} = 1.25$

$\frac{l}{d}$=20.27*1.25=25.34 =l=lx= 3000mm   d= 118.39mm

Using   ϕ10 and cover 34mm H =118.39+ 30 +10/2= 153.39Use D = 200mm

d = 200-30-5 =165mm

Design of reinforcement

D = 200mm

d = 200 – 30 -10/ 2= 165mm

A-Geometry of the wall:

Height of the wall (H) =3.00m

Thickness of the wall (t) =200mm

Strap beam depth (D) =500mm

Strap beam width (B) =200mm

Total height of wall= 3+0.5=3.5m

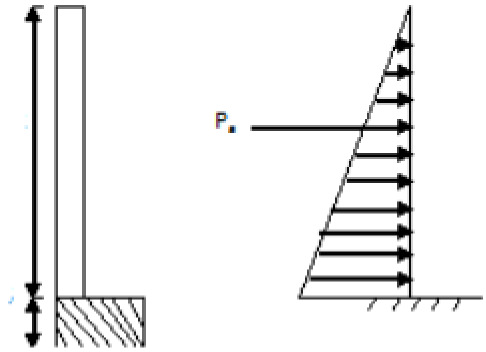
Length of the wall (L) = 47.5m

Total height of the wall = 3.5m

Check for slenderness: Hc/t=3.56/0.2=17.8≥7 the wall is slender wall retain only selected soil at the back of the building that have active earth

pressure on wall and at the front side there is a soil which gives support to the wall and act as passive pressure, but in design of wall we consider only the active soil pressure on the out-plan face of wall.

Shear wall design for basement



9.2.3. Load calculation

Assume γsoil = 18.5KN/m^3, φ = 30^0

Ka = (1- sin30)/ (1+ sin30) = 0.33

Kp = 1/Ka = 3.03

Active earth pressure

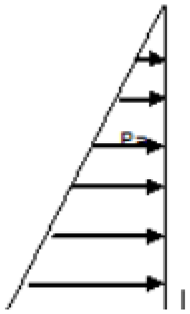
Pa = 1/2Kaγh^2= 27.47KN

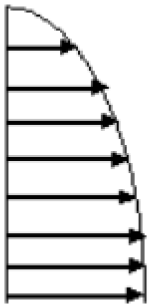
Factored Pa = 1.35Pa = 37.085KN

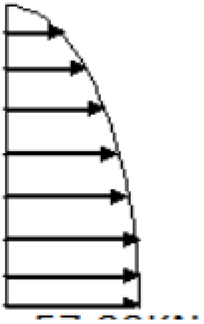
Determination of design shear and moment

V = 1/2pa*h= ½*37.085*3= 55.63KN

M = 1/6*pa* h2= 1/6*37.085*3*3=55.63KNm







37.085KN/m 55.63 KN 55.63KNm

Self-weight of wall, wt=t*H* $\gamma * 1m$=0.2m*3*1m*25KN/m3=15KN.

Factored weight of wall, Nsd=1.35*15KN=20.25KN.

Since the wall is slender wall it will have eccentricity in both direction and we designed to the longest direction.

Etot = eo+e2+ ea

1-Accidental (additional) eccentricity due to various imperfection:

ea=Le/300≥20mm

Where, Le is effective buckling length of the wall

Assuming non-sway mode Le=0.7*L=0.7*3=2.1m.

ea=2100/300=7mm<20mm.

Therefore, take ea=20mm.

9.2.4. Determination of design eccentricity in out-plan direction

2-First order eccentricity:

eo=Md/Nsd=55.63KNm/20.25KN=2.75m.

3-second order eccentricity:

$e2=0.4 * h *(\frac{le}{10*h})^2$ where, h=length of the wall=47.5m.

= 0.4*47.5*(2.75/(10*47.5))2=0.64

Therefore, total eccentricity, etot=2100mm+0.64mm+20mm=2120.64mm

Relative eccentricity: According to section 6.2.2.1. The relative eccentricity is the ratio of total eccentricity to the column width of the same direction.

erel= etot/h=2.1206/47.5=0.045

Determination of design eccentricity in-plan direction:

1-First order eccentricity:

No moment is carried out in this direction so:

eo=Md/Nsd=0

The second order and accidental eccentricities are the same as in out-plan direction.

Therefore: etot=0.64mm+20mm=20.64mm.

From this erel=20.64/200=0.1032 since D=200mm in this direction

Relative eccentricity ratio,k:

K=$\frac{small\ relative\ eccentricity}{large\ relative\ eccentricity}$=$\frac{0.06}{0.1}$ $= 0.1$

Equivalent eccentricity, eqq= etot max*(1+K*α)

Where, etotmax=maximum of the total eccentricities. α may be obtained from table

V	*0*	*0.2*	*0.4*	*0.6*	*0.8*	*≥10*
α	0.6	0.8	0.9	0.7	0.6	0.5

Vsd=Nsd/fcdA=(24.03*10^3N)/(11.33N/mm^2*200mm*3560mm)=0.003

Then, interpolating from the above for Vsd=0.003, α =0.63.

Substituting in to the above equation eeqq=2.1*(1+0.6*0.63) =3.35m

9.2.5. Check Depth for Flexure

$Mrd = 0.8kx\ (1 - 0.4kx)\ d * b * d^2$=0.8*0.448(1-0.448*0.4)*11.33*1000*$(160)^2$

Mrd= 84.18KNm/m>55.63KNm/m……………..Ok

9.2.6. Reinforcement calculation

Longitudinal Reinforcement

At the Support

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

Ast, min=max$\begin{cases} \frac{0.26*m*b*d}{fyk} \\ 0.0013bd \end{cases}$

Ast, min=max$\begin{cases} \frac{0.26*2.2*1000*165}{400} = 235.95 \\ 0.0013 * 1000 * 165 = 214.5 \end{cases}$

Ast, min=235.95mm2/m

Ast, max = 0.04Ac……………………(*ES EN 2 Part 1-1*, n.d.)

Ast, max = 0.04*1000*200=8000mm2/m Spacing between main bars

Smax, slab<$min\begin{cases} 3D \\ 400mm \end{cases}$

Smax, slab≤ $min\begin{cases} 3 * 200 = 600mm \\ 400mm \end{cases}$

Smax=400mm

Msd=55.63KNm/m

μ_{sd}=$\frac{Msd}{fcd*bd^2}$ =$\frac{55.63x10^6}{11.33*1000x165^2}$=0.200<µus=0.295 (for 0% moment redistribution) design as singly reinforced section

From General Design chart No 1,

Kz =0.884

As,cal=$\frac{Msd}{fydxdxKz}$=$\frac{55.63x10^6}{347.82x165x0.884}$

=1096.52mm^2/m it is between As, min and As,max……………OK

takeAs, cal=1096.52mm^2

Now determine the spacing of Φ10mm reinforcement bars by taking

As=$\frac{\pi*d^2}{4}$=$\frac{\pi*10x10}{4}$=78.5mm^2/m

S cal=$\frac{bxas}{As\,cal}$ =$\frac{1000x78.5}{1096.52}$=71.59mm<400mm (maximum spacing) ………OK

Provide ϕ10c/c spacing 70mm

On the Span

Msd= 55.63KNm/m

$\mu_{sd}=\frac{Msd}{fcd*bd^2}=\frac{55.63x10^6}{11.33*1000x165^2}$ =0.2<μsd,s=0.295 (for 0% moment redistribution) design as singly reinforced section.

From General Design Chart No 1,

Kz =0.951

As,cal=$\frac{Msd}{fydxdxKz}=\frac{55.63x10^6}{347.82x165x0.951}$ =1019.27mm^2/m

it is between As,min and As,max…………..OK

Then, take As, cal=1019.27mm^2/m

Now determine the spacing of Φ10mm reinforcement bars by taking

As=$\frac{\pi*d^2}{4}=\frac{\pi*10x10}{4}$=78.5mm^2/m

S cal=$\frac{bxas}{As\ cal}=\frac{1000x78.5}{1019.27}$=77.02mm<400mm (maximum spacing) ………. OK

Provide ϕ10c/c spacing 70mm

Transverse reinforcement

Transverse reinforcement is not less than 20% of the principal reinforcement should be provided. Spacing for the secondary reinforcement,3h≤ 450 mm.

Smax=min$\begin{cases}3D = 3X145 = 435 \\ 450\end{cases}$

Smax=450mm

At the Support

As,t=0.2As=0.2*1096.52=219.304mm^2/m

Now determine the spacing of Φ8 mm reinforcement bars by taking

As=$\frac{\pi*d^2}{4}=\frac{\pi*8x8}{4}$=50.24mm^2/m

S cal=$\frac{bxas}{As\ cal}$ =$\frac{1000x50.24}{219.304}$=229.09 mm<400mm (maximum spacing) ………. OK

Provide ϕ8 c/c spacing 220mm

On the Span

As,t=0.2As=0.2*1019.27=203.85mm^2/m

Now determine the spacing of Φ8 mm reinforcement bars by taking

As=$\frac{\pi * d^2}{4}$=$\frac{\pi * 8x8}{4}$=50.24mm^2/m

S cal=$\frac{bxas}{As\ cal}$ =$\frac{1000x50.24}{203.85}$=246.46 mm< 450mm (minimum spacing) ………. OK

Provide ϕ8c/c spacing 240mm

10.FOUNDATION DESIGN

10.1. 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.

10.2. Design procedure

10.2.1. Concrete cover design

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

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

Minimum strength class = C20/25, for exposure class XC1[Appendix H, 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 I, 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

10.2.2. Determination of footing size

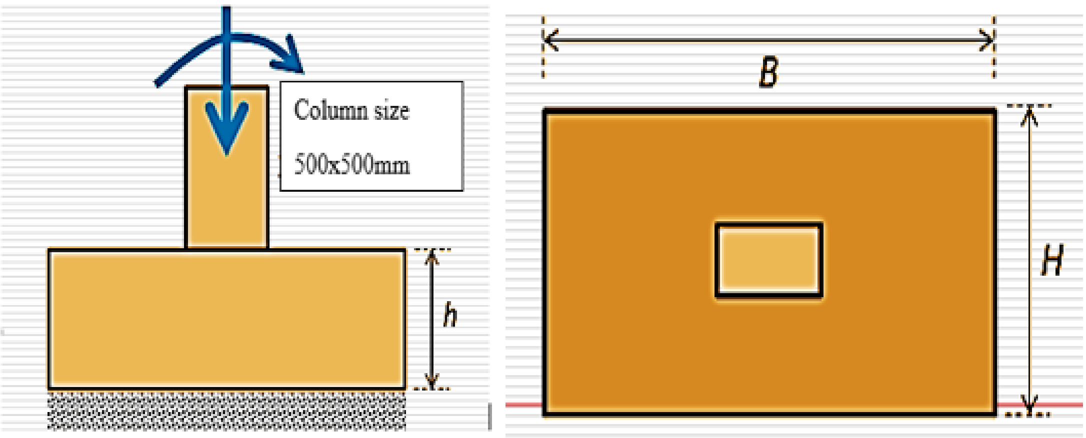


Figure 10-1. footing size

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

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

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.

10.2.3. Eccentricity

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

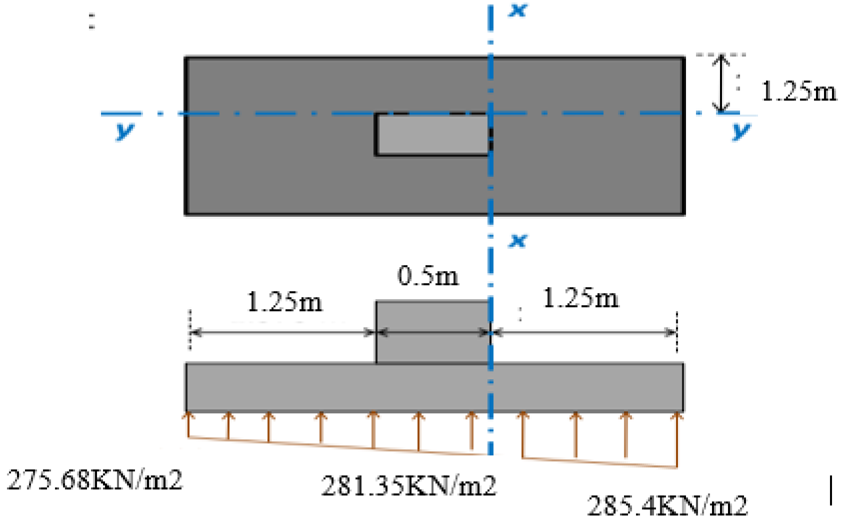
$e_y = M_x / p = 2.8/2550 = 0.0011\text{m}$

Contact pressure ρ max

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

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

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



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

$$\text{Myy} = \frac{(275.68 + 285.4)}{2} * \left(\frac{(1.25 * 1.25)}{2}\right) = 220kNm$$

Effective depth

$$\text{Dx} = \text{h} - \text{c} - 0.5\text{Øbar} = 700 - 30 - 0.5 * 24 = 658mm$$

$$\text{Dy} = \text{h} - \text{c} - 1.5\text{Øbar} = 700 - 30 - 1.5 * 24 = 634mm$$

10.2.4. 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

$$Z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53K}\right] \leq 0.95d\ = \frac{d}{2}\left[1 + \sqrt{1 - 3.53 * 0.167}\right] = 0.82d \leq 0.95d$$

$$z = 0.95 * 700 = 665$$

$$As,req = \frac{Mxx}{fydZ} = \frac{255}{347.82 * 665} = 1102.5mm2$$

$$As,min = \frac{0.26 * fctm * bd}{fyk} = \frac{0.26 * 2.2 * 3 * 0.7}{400} = 3003mm2$$

$$\geq 0.0013bd = 0.0013 * 3 * 0.7 = 2730mm2 where\ fck\ \geq 25$$

$$As,max = 0.04Ac = 0.04 * bh = 0.04 * 3000 * 700 = 84000mm2$$

$$\text{Since As, req } < \text{As, min use As, min} = 3003\text{mm2}$$

$$as = \frac{\Pi d2}{4} = \frac{3.14 * 24 * 24}{4} = 452.16mm2$$

$$no\ of\ bar = \frac{As,req}{as} = \frac{3003}{452.16} = 6.6$$

$$use\ 7Ø24$$

10.2.5. 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

$$Z = \frac{d}{2}\left\lfloor 1 + \sqrt{1 - 3.53K}\right\rfloor \leq 0.95d\ = \frac{d}{2}\left\lfloor 1 + \sqrt{1 - 3.53 * 0.167}\right\rfloor = 0.82d \leq 0.95d$$

$$z = 0.95 * 700 = 665$$

$$As,req = \frac{Mxx}{fydZ} = \frac{255}{347.82 * 665} = 1102.5mm2$$

$$As,min = \frac{0.26 * fctm * bd}{fyk} = \frac{0.26 * 2.2 * 3 * 0.7}{400} = 3003mm2$$

$$as = \frac{\Pi d2}{4} = \frac{3.14 * 24 * 24}{4} = 452.16mm2$$

$$no\ of\ bar = \frac{As,req}{as} = \frac{3003}{452.16} = 6.6$$

$$use\ 7Ø24$$

$$\geq 0.0013bd = 0.0013 * 3 * 0.7 = 2730mm2 where\ fck\ \geq 25$$

$$As, max = 0.04Ac = 0.04 * bh = 0.04 * 3000 * 700 = 84000mm2$$

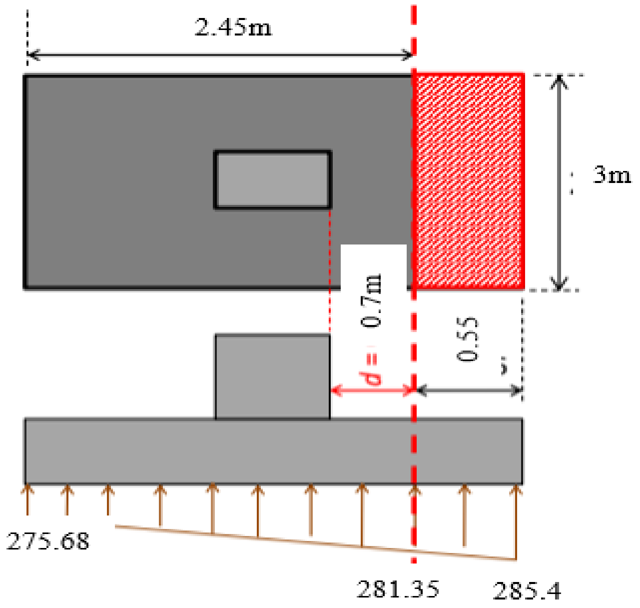
$$\text{Since As, req } < \text{As, min use As, min} = 3003\text{mm2}$$

10.2.6. 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, $\text{VEd} = 283.68 * 0.55 * 3 = 468.072KN$

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=$\lfloor 0.035 * K^{3/2}\sqrt{fck}\rfloor bd = \lfloor 0.035 * 1.535^{3/2}\sqrt{20}\rfloor 3000 * 700 = 625KN$

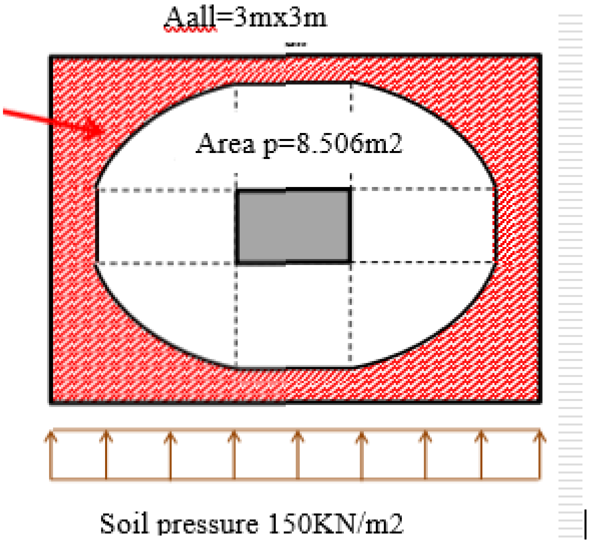
VEd<Vmin---ok!

10.2.7. 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

$U = 2(500 + 500) + (2\Pi * 1334) = 10377.52$

Area with in perimeter

$$A = (0.5 * 0.5) + (2 * 0.5 * 1.334) + (2 * 0.5 * 1.334) + (3.14 * 1.334 * 1.334)$$
$$= 8.506\text{m2}$$

Average punching shear force at control perimeter

$$VEd = 150 * ((3 * 3) - 8.506) = 74.1KN$$

Punching shear stress

$vEd = \frac{\beta VEd}{ud}$

Where

$$\beta = 1 + k(\frac{MEd}{VEd})(\frac{u1}{w1})$$

$$\mathrm{w1} = 0.5{C_1}^2 + C_1C_2 + 4C2_2\mathrm{d} = 16\mathrm{d2} + 2\,\Pi\mathrm{d}C_1$$

$$\mathrm{w1} = 0.5 * 500^2 + 500 * 500 + 4 * 500 * 667 + 16 * 667 * 667 + 2\,\Pi * 667 * 500$$

$$\mathrm{w1} = 10921604\mathrm{mm4}$$

$$\beta = 1 + 0.6\left(\frac{6.38 * 1000}{74.1}\right)\left(\frac{10377.52}{10921604}\right) = 1.05$$

$$vEd = \frac{\beta VEd}{ud} = \frac{1.05 * 74.1 * 1000}{10377 * 700} = 0.011N/mm2$$

Punching shear resistance

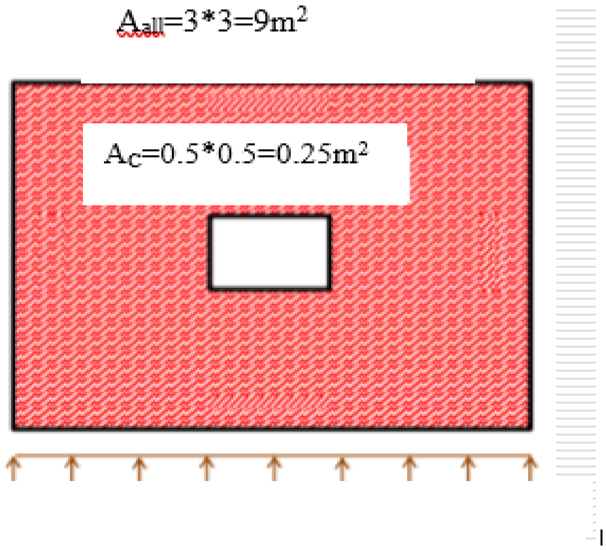
$K = 1 + \sqrt{\frac{200}{d}}$=1 + $\sqrt{\frac{200}{700}}$=1.535<2......................................ok

$$V_{Rd,c} = \mathrm{Vmin} = \left\lfloor 0.035 * \mathrm{K3/2}\sqrt{fck}\right\rfloor$$

$V_{Rd,c} = \left\lfloor 0.035 * 1.535^{3/2}\sqrt{20}\right\rfloor$=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

$$V_{Ed} = \frac{\beta VEd}{ud}$$

$W_1=0.5{C_1}^2+C_1C_2=0.5*500^2+500*500=0.375*10^6mm^2$

$$\beta = 1 + 0.6 * \left(\frac{6.38 * 1000}{2550 * 10^3}\right)\left(\frac{2000}{0.375 * 106}\right) = 1$$

$$V_{Ed} = \frac{1 * 2550 * 10^3}{2000 * 667} = 1.91N/mm2$$

Maximum shear resistance

$V_{Ed,max}=0.5\left[0.6\left(1-\frac{f_{ck}}{500}\right)\right]*\frac{f_{ck}}{1.5}$

$V_{Ed,max}=0.5\left[0.6\left(1-\frac{20}{500}\right)\right] * \frac{20}{1.5} = 3.84\text{N/mm2} >$ V_{Ed}................................ok

10.2.8. Reinforcement Detailing

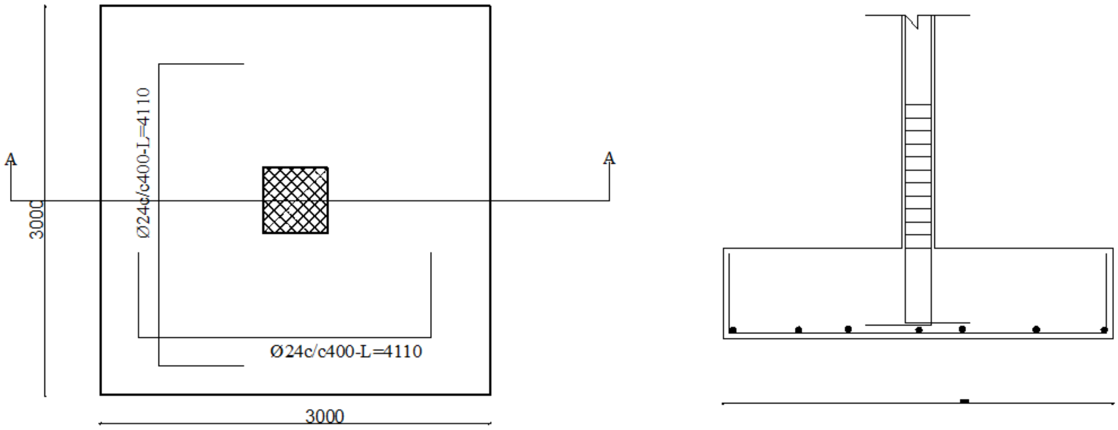


Figure 10-2. footing detailing

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Appendix

Appendix A1: determination of loading 3rd & 4th typical floor

Table 18. 3rd and 4th floor design load

P2				**Material**	**Unit weight**	**Dimension**	**Load KN/m2**
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.005	0.11
	5.1	21.25	0.24				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.0048				
	A_{finish}	A_{total}	A_f/A_t	Carpet	23	0.015	0.35
	16.15	21.25	0.76				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.0152				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.141	1.98
	21.25	$\frac{(5*0.2*3)}{3}$	0.141176				
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					8.82	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					17.90224	KN/m^2	

P3				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.010	0.23
	14.5	29	0.5				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.01				
	A_{finish}	A_{total}	A_f/A_t	Carpet	23	0.009	0.21
	12.97	29	0.447241				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.008945				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.176	2.46
	29	$\frac{(8.5*0.2*3)}{5.1}$	0.175862				
LIVE LOAD Q_k					5	KN/m^2	
DEAD LOAD Gk					9.28	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					20.03	KN/m^2	

P4				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.007	0.16
	10	29	0.344828				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.006897				
	A_{finish}	A_{total}	A_f/A_t	Carpet	23	0.013	0.30
	19	29	0.655172				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.013103				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.206	2.88
	29	.952*0.2*3	0.205903				
		5.9712					
LIVE LOAD Q_k					5	KN/m^2	
DEAD LOAD Gk					9.72	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					20.63	KN/m^2	

P5				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.007	0.16
	8.5	24.65	0.344828				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.006897				
	A_{finish}	A_{total}	A_f/A_t	Carpet	23	0.013	0.30
	16.15	24.65	0.655172				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.013103				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.208	2.91
	24.65	8.55*0.2*3	0.208114				
		5.13					
LIVE LOAD Q_k					5	KN/m^2	
DEAD LOAD Gk					9.75	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					20.67	KN/m^2	

P6				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.004	0.10
	6	27	0.222222				
	$\frac{A_{finish}}{A_{total}}*t_{finish}$=		0.004444				
	A_{finish}	A_{total}	A_f/A_t	Carpet	23	0.016	0.36
	21	27	0.777778				
	$\frac{A_{finish}}{A_{total}}*t_{finish}$=		0.015556				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.169	2.36
	27	(7.6*0.2*3)	0.168889				
		4.56					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					9.20	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					18.43	KN/m^2	

P7				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.004	0.10
	6	27	0.222222				
	$\frac{A_{finish}}{A_{total}}*t_{finish}$=		0.004444				
	A_{finish}	A_{total}	A_f/A_t	Carpet	23	0.016	0.36
	21	27	0.777778				
	$\frac{A_{finish}}{A_{total}}*t_{finish}$=		0.015556				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.168	2.35
	27	7.55*0.2*3	0.167778				
		4.53					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					9.19	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					18.41	KN/m^2	

P8				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.004	0.10
	5.1	22.95	0.222222				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.004444				
	A_{finish}	A_{total}	A_f/A_t	Carpet	23	0.016	0.36
	17.850	22.95	0.777778				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.015556				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.180	2.51
	22.95	6.87*0.2*3	0.179608				
		4.122					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					9.35	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					18.63	KN/m^2	

Appendix A2: moment analysis of 3rd & 4th typical floor

Table 19. moment analysis of 3rd & 4th typical floor

panel	**support conditio**	**ly/lx**	**lx**	**lx^2**	**Pd**	**moment coeficient**		**moment**	**moment**
						location	**value**	**location**	**$Mi=\alpha_iP_d$**
P-1	Type 2	1.00	5	25.00	19.78	$\beta_{sx,sup}=$	0.039	$M_{xs}=$	19.29
						$\beta_{sx,span}=$	0.029	$M_{xf}=$	14.34
						$\beta_{sy,sup}=$	0.037	$M_{ys}=$	18.30
						$\beta_{sy,span}=$	0.028	$M_{yf}=$	13.85
P-2	Type 4	1.18	4.25	18.06	11.61	$\beta_{sx,sup}=$	0.061	$M_{xs}=$	12.79
						$\beta_{sx,span}=$	0.046	$M_{xf}=$	9.65
						$\beta_{sy,sup}=$	0.045	$M_{ys}=$	9.44
						$\beta_{sy,span}=$	0.034	$M_{yf}=$	7.13
P-3	Type 3	1.16	5	25	13.75	$\beta_{sx,sup}=$	0.053	$M_{xs}=$	18.22
						$\beta_{sx,span}=$	0.040	$M_{xf}=$	13.75
						$\beta_{sy,sup}=$	0.037	$M_{ys}=$	12.72
						$\beta_{sy,span}=$	0.028	$M_{yf}=$	9.62
P-4	Type 1	1.16	5	25.00	14.37	$\beta_{sx,sup}=$	0.041	$M_{xs}=$	14.73
						$\beta_{sx,span}=$	0.031	$M_{xf}=$	11.13
						$\beta_{sy,sup}=$	0.032	$M_{ys}=$	11.49
						$\beta_{sy,span}=$	0.024	$M_{yf}=$	8.62
P-5	Type 3	1.36	4.25	18.06	14.37	$\beta_{sx,sup}=$	0.066	$M_{xs}=$	17.13
						$\beta_{sx,span}=$	0.049	$M_{xf}=$	12.72
						$\beta_{sy,sup}=$	0.037	$M_{ys}=$	9.60
						$\beta_{sy,span}=$	0.028	$M_{yf}=$	7.27
P-6	Type 4	1.08	5	25	12.15	$\beta_{sx,sup}=$	0.054	$M_{xs}=$	16.40
						$\beta_{sx,span}=$	0.041	$M_{xf}=$	12.45
						$\beta_{sy,sup}=$	0.045	$M_{ys}=$	13.67
						$\beta_{sy,span}=$	0.031	$M_{yf}=$	9.42
P-7	Type 2	1.08	5	25.00	12.14	$\beta_{sx,sup}=$	0.044	$M_{xs}=$	13.36
						$\beta_{sx,span}=$	0.033	$M_{xf}=$	10.02
						$\beta_{sy,sup}=$	0.037	$M_{ys}=$	11.23
						$\beta_{sy,span}=$	0.028	$M_{yf}=$	8.50
P-8	Type 4	1.27	4.25	18.06	12.33	$\beta_{sx,sup}=$	0.067	$M_{xs}=$	14.92
						$\beta_{sx,span}=$	0.049	$M_{xf}=$	10.91
						$\beta_{sy,sup}=$	0.045	$M_{ys}=$	10.02
						$\beta_{sy,span}=$	0.031	$M_{yf}=$	6.91

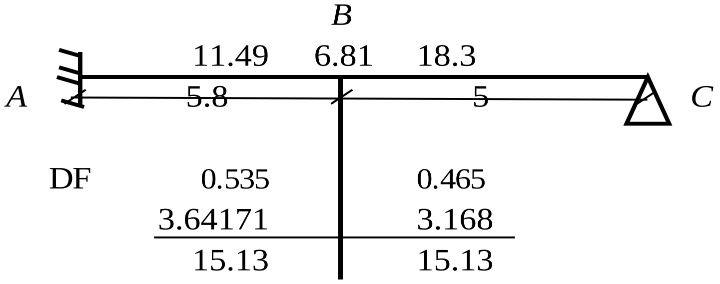
Appendix A3: support moment adjustment

P4&P5 –on axis 3 adjustment is required.
$\Delta M_s=$ 17.13 - 14.73 = 2.4
$0.2M_{s,max}$ 0.2 x 17.13 = 3.426
$\Delta M_s=$ 2.4 < $0.2M_{s,max}$ TRUE
Therefore, average of the moments can be taken.
$M_{s,avg}=\frac{1}{2}($ 17.13 + 14.73 $)=$ 15.93

P6&P7 –on axis 2 adjustment is required.
$\Delta M_s=$ 16.4 - 13.36 = 3.04
$0.2M_{s,max}$ 0.2 x 16.4 = 3.28
$\Delta M_s=$ 3.04 < $0.2M_{s,max}$ TRUE
Therefore, average of the moments can be taken.
$M_{s,avg}=\frac{1}{2}($ 16.4 + 13.36 $)=$ 14.88

P7&P8 –on axis 2 adjustment is required.
$\Delta M_s=$ 14.92 - 13.36 = 1.56
$0.2M_{s,max}$ 0.2 x 14.92 = 2.984
$\Delta M_s=$ 1.56 < $0.2M_{s,max}$ TRUE
Therefore, average of the moments can be taken.
$M_{s,avg}=\frac{1}{2}($ 14.92 + 13.36 $)=$ 14.14

P1&P4-on axis B



$$K_{AB} = \frac{I}{L} = \frac{I}{5.8} = \underline{0.17241\ I}$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4*4.274} = \underline{0.15\ \ I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} == 0.535$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} == 0.465$$

P2&P5 –on axis B adjustment is required.

$\Delta M_s =$ 9.6 - 9.44 = 0.16

$0.2M_{s,max}$ 0.2 x 9.6 = 1.92

$\Delta M_s =$ 0.16 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}($ 9.6 + 9.44 $) =$ 9.52

P3&P6 –on axis C adjustment is required.

$\Delta M_s =$ 13.67 - 12.72 = 0.95

$0.2M_{s,max}$ 0.2 x 13.67 = 2.734

$\Delta M_s =$ 0.95 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}($ 13.67 + 12.72 $) =$ 13.20

P4&P7 –on axis C adjustment is required.

$\Delta M_s =$ 11.49 - 11.23 = 0.26

$0.2M_{s,max}$ 0.2 x 11.49 = 2.298

$\Delta M_s =$ 0.26 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}($ 11.49 + 11.23 $) =$ 11.36

P5&P8 –on axis C adjustment is required.

ΔM_S= 10.02 - 9.6 = 0.42

$0.2M_{s,max}$ 0.2 x 10.02 = 2.004

ΔM_S= 0.42 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 10.02 + 9.6 $)=$ 9.81

Appendix A4: Field moment adjustment

For Panel 2

No need of adjustment

For Panel 3

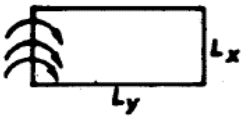
M_{xf} = 13.75 M_{Xs} = 18.22 $M_{Xs,adj}$ = 16.48

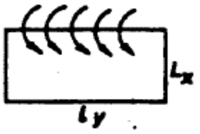
M_{yf} = 9.62 M_{ys} = 12.72 $M_{ys,adj}$ = 13.2

Ly/Lx= 1.16

ΔM_{xs} = Mxs- M_Xs,adj = 1.74 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.345 Cx = 0.332

Cy = 0.191 Cy = 0.368

ΔMxf = CxΔMxs + CxΔMys = 0.578

Mxf ,adj = ΔMxf + Mxf = **14.328**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.640

Myf ,adj = ΔMyf + Myf = **10.260**

For Panel 4

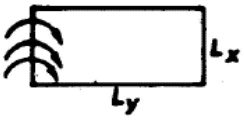
M_{xf} = 11.13 M_{Xs} = 14.73 $M_{Xs,adj}$ = 15.93

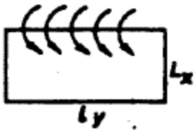
M_{yf} = 8.62 M_{ys} = 11.49 $M_{ys,adj}$ = 11.36

Ly/Lx= 1.16

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

ΔM_{ys} = Mys- M_ys,adj = 0.13 neglect





Cx = 0.345 Cx = 0.332

Cy = 0.191 Cy = 0.368

ΔMxf = CxΔMxs + CxΔMys = 0.045

Mxf,adj = ΔMxf + Mxf = **11.175**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.025

Myf,adj = ΔMyf + Myf = **8.645**

For Panel 5

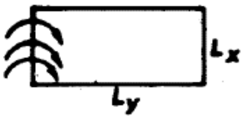
M_{xf} = 12.72 M_{Xs} = 17.13 $M_{Xs,adj}$ = 15.93

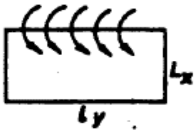
M_{yf} = 7.27 M_{ys} = 9.6 $M_{ys,adj}$ = 9.81

Ly/Lx= 1.36

ΔM_{xs} = Mxs- M_Xs,adj = 1.20 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.319 Cx = 0.388

Cy = 0.12 Cy = 0.339

ΔMxf = CxΔMxs + CxΔMys = 0.466

Mxf,adj = ΔMxf + Mxf = **13.186**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.407

Myf,adj = ΔMyf + Myf = **7.677**

For Panel 6

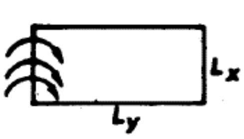
M_{xf} = 12.45 M_{Xs} = 16.4 $M_{Xs,adj}$ = 14.88

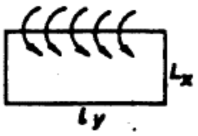
M_{yf} = 9.42 M_{ys} = 13.67 $M_{ys,adj}$ = 13.2

Ly/Lx= 1.08

ΔM_{xs} = Mxs- M_Xs,adj = 1.52 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.47 neglect





Cx = 0.361 Cx = 0.323

Cy = 0.232 Cy = 0.375

ΔMxf= CxΔMxs + CxΔMys = 0.661

Mxf,adj = ΔMxf+ Mxf= **13.111**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.679

Myf,adj = ΔMyf+ Myf= **10.099**

Panel 7

No need of adjustment

For Panel 8

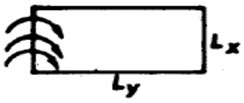
M_{xf} = 10.91 M_{Xs} = 14.92 $M_{Xs,adj}$ = 14.14

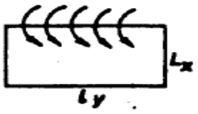
M_{yf} = 6.91 M_{ys} = 10.02 $M_{ys,adj}$ = 9.81

Ly/Lx= 1.27

ΔM_{xs} = Mxs- M_Xs,adj = 0.78 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.21 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 0.353

Mxf,adj = ΔMxf+ Mxf= **11.263**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.307

Myf,adj = ΔMyf+ Myf= **7.217**

Appendix A4: 3rd and 4th load transfer to beam

Table 20. 3rd and 4th 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	1.00	5.00	19.78	$\beta_{vxc} =$	0.36	$V_{xc} =$	35.606
					$\beta_{vxd} =$	0	$V_{xd} =$	0.000
					$\beta_{vyc} =$	0.36	$V_{yc} =$	35.606
					$\beta_{vyd} =$	0.24	$V_{yd} =$	23.737
P-2	Type 4	1.18	4.25	11.61	$\beta_{vxc} =$	0.46	$V_{xc} =$	22.697
					$\beta_{vxd} =$	0.31	$V_{xd} =$	15
					$\beta_{vyc} =$	0.4	$V_{yc} =$	19.737
					$\beta_{vyd} =$	0.24	$V_{yd} =$	11.842
P-3	Type 3	1.16	5.00	13.75	$\beta_{vxc} =$	0.42	$V_{xc} =$	28.870
					$\beta_{vxd} =$	0.28	$V_{xd} =$	19
					$\beta_{vyc} =$	0.36	$V_{yc} =$	24.746
					$\beta_{vyd} =$	0	$V_{yd} =$	0.000
P-4	Type 1	1.16	5.00	14.37	$\beta_{vxc} =$	0.38	$V_{xc} =$	27.297
					$\beta_{vxd} =$	0	$V_{xd} =$	0.000
					$\beta_{vyc} =$	0.33	$V_{yc} =$	23.705
					$\beta_{vyd} =$	0	$V_{yd} =$	0.000
P-5	Type 3	1.36	4.25	14.37	$\beta_{vxc} =$	0.48	$V_{xc} =$	29.310
					$\beta_{vxd} =$	0.32	$V_{xd} =$	19.540
					$\beta_{vyc} =$	0.36	$V_{yc} =$	21.983
					$\beta_{vyd} =$	0	$V_{yd} =$	0.000
P-6	Type 4	1.08	5.00	12.15	$\beta_{vxc} =$	0.43	$V_{xc} =$	26.119
					$\beta_{vxd} =$	0.28	$V_{xd} =$	17
					$\beta_{vyc} =$	0.4	$V_{yc} =$	24.297
					$\beta_{vyd} =$	0.24	$V_{yd} =$	14.578
P-7	Type 2	1.08	5.00	12.14	$\beta_{vxc} =$	0.38	$V_{xc} =$	23.071
					$\beta_{vxd} =$	0	$V_{xd} =$	0
					$\beta_{vyc} =$	0.36	$V_{yc} =$	21.857
					$\beta_{vyd} =$	0.24	$V_{yd} =$	15
P-8	Type 4	1.27	4.25	12.33	$\beta_{vxc} =$	0.49	$V_{xc} =$	25.681
					$\beta_{vxd} =$	0.32	$V_{xd} =$	16.772
					$\beta_{vyc} =$	0.4	$V_{yc} =$	20.964
					$\beta_{vyd} =$	0.24	$V_{yd} =$	12.579

Appendix B1: Second floor Depth determination

Table 21.second floor depth determination

Φbar= 10
Cnom= 20
ϒs = 1.15 fcd= $\frac{0.85*fck}{ϒc}$=11.33 fyd= $\frac{fyk}{ϒs}$= 347.8
ϒc = 1.5
fck = 20
fyk= 400 unit weigh 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	5000	5000	1.00	two way	Interior	17.71	1.5	33.20	150.59	175.59	
p2	5000	4250	1.18	two way	Interior	17.71	1.5	33.20	128.00	153.00	200
p3	5800	5000	1.16	two way	Interior	17.71	1.5	33.20	150.59	175.59	
p4	5800	5000	1.16	two way	Interior	17.71	1.5	33.20	150.59	175.59	
p5	5800	4250	1.36	two way	Interior	17.71	1.5	33.20	128.00	153.00	
p6	5400	5000	1.08	two way	Interior	17.71	1.5	33.20	150.59	175.59	
p7	5400	5000	1.08	two way	Interior	17.71	1.5	33.20	150.59	175.59	
p8	5400	4250	1.27	two way	Interior	17.71	1.5	33.20	128.00	153.00	
c1	10350	850	12.18	one way	cantilever	17.71	0.4	8.85	96.00	121.00	250
c2	5306	1602	3.31	one way	cantilever	17.71	0.4	8.85	180.93	205.93	
c3	11594	1374	8.44	one way	cantilever	17.71	0.4	8.85	155.18	180.18	
c4	14250	1800	7.92	one way	cantilever	17.71	0.4	8.85	203.30	228.30	
c5	11200	1212	9.24	one way	cantilever	17.71	0.4	8.85	136.89	161.89	

Appendix B2: Load calculation of 2nd floor slab

Table 22. load calculation of 2nd floor slab

P1				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	25	25	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					6.84	KN/m^2	
DESIGN LOAD $P_d = 1.35 G_k + 1.5 Q_k$					15.234	KN/m^2	

P2				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	21.25	21.25	1				
	$\frac{A_{finish}}{A_{total}} * t_{finish} =$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.261	3.66
	21.25	9.25*0.2*3	0.261176				
		5.55					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					5.85	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					13.89274	KN/m^2	

P3				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	29	29	1				
	$\frac{A_{finish}}{A_{total}} * t_{finish} =$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.120	1.68
	29	(5.8*0.2*3)	0.12				
		3.48					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					3.87	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					11.2245	KN/m^2	

P4				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	29	29	1				
	$\frac{A_{finish}}{A_{total}} * t_{finish} =$		0.02				
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					6.84	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					15.234	KN/m^2	

P5				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	24.65	24.65	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					6.84	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					15.234	KN/m^2	

P6				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	27	27	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					6.84	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					15.234	KN/m^2	

P7				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	27	27	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					6.84	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					15.234	KN/m^2	

P8				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	22.95	22.95	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					6.84	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					15.234	KN/m^2	

c1				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.016	0.37
	7.16125	8.7975	0.81401				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.01628				
	A_{finish}	A_{total}	A_f/A_t	Carpet	23	0.004	0.09
	1.636	8.7975	0.18599				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.00372				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.822	11.51
	8.7975	12.05*0.2* 7.23	0.821824				
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					19.60	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					32.45398	KN/m^2	

c2				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	8.500212	8.500212	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.601	8.41
	8.500212	8.51*0.2*3 5.106	0.600691				
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					16.50	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					28.27456	KN/m^2	

c3				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	15.93016	15.93016	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.540	7.56
	15.93016	4.342*0.2*	0.540183				
		8.6052					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					15.65	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					27.13096	KN/m^2	

c4				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	25.65	25.65	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.418	5.85
	25.65	7.85*0.2*	0.417544				
		10.71					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					13.94	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					24.81308	KN/m^2	

c5				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	13.5744	13.5744	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.251	3.51
	13.552	.662*0.2*	0.250679				
		3.3972					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					11.60	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					21.65933	KN/m^2	

Appendix B3: Moment analysis 2nd floor slab

Table 23.Moment analysis 2nd floor slab

panel	**support conditio**	**ly/lx**	**lx**	**lx^2**	**Pd**	**moment coeficient**		**moment**	**moment**
						location	**value**	**location**	**$M_i=\alpha_i P_d$**
P-1	Type 1	1.00	5.00	25.00	12.23	$\beta_{sx,sup=}$	0.034	M_{xs} =	10.40
						$\beta_{sx,span=}$	0.024	M_{xf} =	7.34
						$\beta_{sy,sup=}$	0.032	M_{ys} =	9.79
						$\beta_{sy,span=}$	0.024	M_{yf} =	7.34
P-2	Type 1	1.18	4.25	18.06	10.89	$\beta_{sx,sup=}$	0.041	M_{xs} =	8.07
						$\beta_{sx,span=}$	0.031	M_{xf} =	6.10
						$\beta_{sy,sup=}$	0.032	M_{ys} =	6.30
						$\beta_{sy,span=}$	0.024	M_{yf} =	4.72
P-3	Type 1	1.16	5.00	25.00	16.00	$\beta_{sx,sup=}$	0.04	M_{xs} =	16.00
						$\beta_{sx,span=}$	0.03	M_{xf} =	12.00
						$\beta_{sy,sup=}$	0.032	M_{ys} =	12.80
						$\beta_{sy,span=}$	0.024	M_{yf} =	9.60
P-4	Type 1	1.16	5.00	25.00	13.73	$\beta_{sx,sup=}$	0.04	M_{xs} =	13.73
						$\beta_{sx,span=}$	0.03	M_{xf} =	10.30
						$\beta_{sy,sup=}$	0.032	M_{ys} =	10.99
						$\beta_{sy,span=}$	0.024	M_{yf} =	8.24
P-5	Type 1	1.36	4.25	18.06	13.73	$\beta_{sx,sup=}$	0.048	M_{xs} =	11.91
						$\beta_{sx,span=}$	0.036	M_{xf} =	8.93
						$\beta_{sy,sup=}$	0.032	M_{ys} =	7.94
						$\beta_{sy,span=}$	0.024	M_{yf} =	5.95
P-6	Type 1	1.08	5.00	25.00	13.73	$\beta_{sx,sup=}$	0.036	M_{xs} =	12.36
						$\beta_{sx,span=}$	0.028	M_{xf} =	9.61
						$\beta_{sy,sup=}$	0.032	M_{ys} =	10.99
						$\beta_{sy,span=}$	0.024	M_{yf} =	8.24
P-7	Type 1	1.08	5.00	25.00	13.73	$\beta_{sx,sup=}$	0.036	M_{xs} =	12.36
						$\beta_{sx,span=}$	0.028	M_{xf} =	9.61
						$\beta_{sy,sup=}$	0.032	M_{ys} =	10.99
						$\beta_{sy,span=}$	0.024	M_{yf} =	8.24
P-8	Type 1	1.27	4.25	18.06	13.73	$\beta_{sx,sup=}$	0.045	M_{xs} =	11.16
						$\beta_{sx,span=}$	0.034	M_{xf} =	8.43
						$\beta_{sy,sup=}$	0.032	M_{ys} =	7.94
						$\beta_{sy,span=}$	0.024	M_{yf} =	5.95

Appendix B4: Moment adjustment of 2nd floor slab

P1&P2 –on axis 3 adjustment is required.

ΔM_s= 10.04 - 8.07 = 1.97

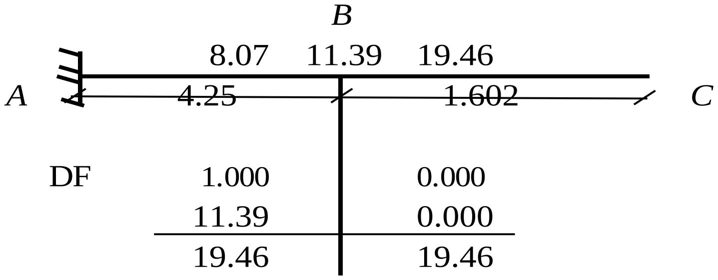
$0.2M_{s,max}$ 0.2 x 10.04 = 2.008

ΔM_s= 1.97 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 10.04 + 8.07 $)=$ 9.06

P2 & C2 on axis 4



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 1.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.000$$

C5 & P3 –on axis 1 adjustment is required.

ΔM_s= 16 - 13.3 = 2.7

$0.2M_{s,max}$ 0.2 x 16 = 3.2

ΔM_s= 2.7 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 16 + 13.3 $)=$ 14.65

P3 & P4 –on axis 2 adjustment is required.

ΔM_s= 16 - 13.73 = 2.27

$0.2M_{s,max}$ 0.2 x 16 = 3.2

ΔM_s= 2.27 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 16 + 13.73 $)=$ 14.87

P4 & P5 –on axis 3 adjustment is required.

ΔM_s= 13.73 - 11.91 = 1.82

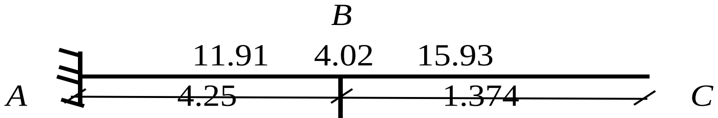
0.2$M_{s,max}$ 0.2 x 13.73 = 2.746

ΔM_s= 1.82 < 0.2$M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 13.73 + 11.91 $)=$ 12.82

P5 & C3 on axis 4



DF	1.000	0.000
	4.02	0.000
	15.93	15.93

$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 1.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.000$$

C5 & P6 –on axis 3 adjustment is required.

ΔM_s= 13.3 - 12.36 = 0.94

0.2$M_{s,max}$ 0.2 x 13.3 = 2.66

ΔM_s= 0.94 < 0.2$M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 13.3 + 12.36 $)=$ 12.83

P6 & P7 no need of adjustment

P7 & P8 –on axis 3 adjustment is required.

ΔM_s= 12.36 - 11.16 = 1.2

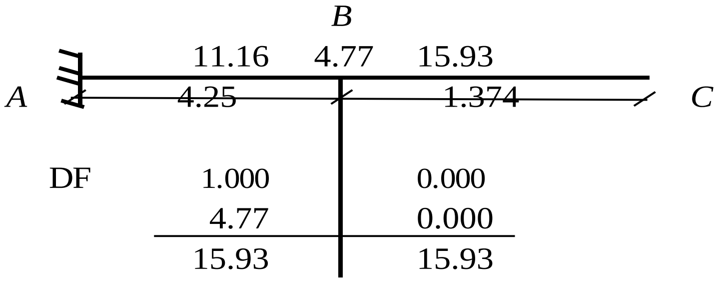
0.2$M_{s,max}$ 0.2 x 12.36 = 2.472

ΔM_s= 1.2 < 0.2$M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 12.36 + 11.16 $)=$ 11.76

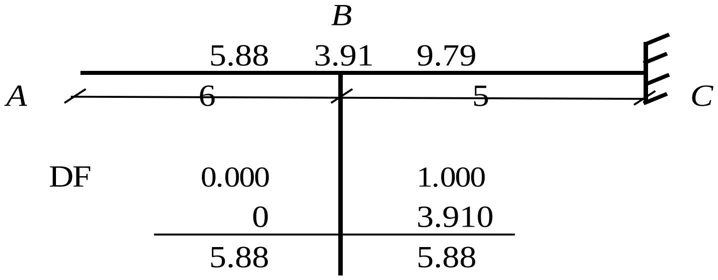
P8 & C3 on axis 4



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= \quad 1.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= \quad 0.000$$

C1 & P1 on axis A



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= \quad 0.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= \quad 1.000$$

C1 & P2 –on axis A adjustment is required.

$\Delta M_s = 5.88 - 6.3 = -0.42$

$0.2M_{s,max} \quad 0.2 \times 5.88 = 1.176$

$\Delta M_s = -0.42 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}(5.88 + 6.3) = 6.09$

P1 & P4 –on axis B adjustment is required.

$\Delta M_s = 9.79 - 10.99 = -1.2$

$0.2M_{s,max} \quad 0.2 \times 9.79 = 1.958$

$\Delta M_s = -1.2 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(9.79 + 10.99) = 10.39$

P2 & P5 –on axis B adjustment is required.

$\Delta M_s = 6.3 - 7.94 = -1.64$

$0.2M_{s,max} \quad 0.2 \times 6.3 = 1.26$

$\Delta M_s = -1.64 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(6.3 + 7.94) = 7.12$

P3 & P6 –on axis C adjustment is required.

$\Delta M_s = 12.8 - 10.99 = 1.81$

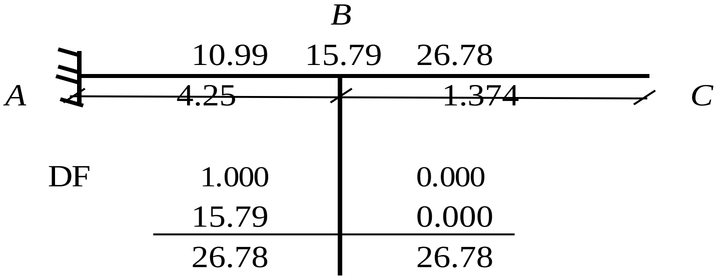
$0.2M_{s,max} \quad 0.2 \times 12.8 = 2.56$

$\Delta M_s = 1.81 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(12.8 + 10.99) = 11.90$

P6,7 & C4 on axis 4



$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 1.000$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.000$$

Second floor field adjustment

For Panel 1

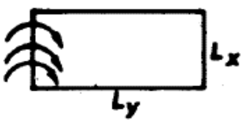
M_{xf} = 14.34 M_{Xs} = 19.29 $M_{Xs,adj}$ = 9.06

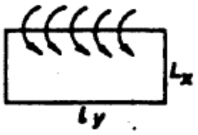
M_{yf} = 13.85 M_{ys} = 13.85 $M_{ys,adj}$ = 9.79

Ly/Lx= 1.00

ΔM_{xs} = Mxs- M_Xs,adj = 10.23 neglect

ΔM_{ys} = Mys- M_ys,adj = 4.06 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 5.059

Mxf,adj = ΔMxf+ Mxf= **19.40**

ΔM_{yf} = CyΔMxs + CyΔMys = 4.214

Myf,adj = ΔMyf+ Myf= **18.06**

Panel 2 no need adjustment

For Panel 3

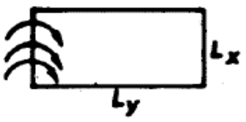
M_{xf} = 12 M_{Xs} = 16 $M_{Xs,adj}$ = 14.87

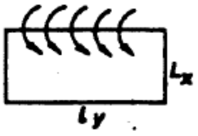
M_{yf} = 9.6 M_{ys} = 12.8 $M_{ys,adj}$ = 11.9

Ly/Lx= 1.16

ΔM_{xs} = Mxs- M_Xs,adj = 1.13 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.90 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 0.707

Mxf,adj = ΔMxf+ Mxf= **12.71**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.531

Myf,adj = ΔMyf+ Myf= **10.13**

For Panel 4

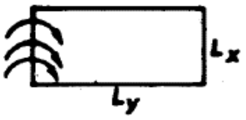
M_{xf} = 10.3 M_{Xs} = 13.73 $M_{Xs,adj}$ = 12.82

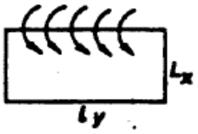
M_{yf} = 8.24 M_{ys} = 10.99 $M_{ys,adj}$ = 10.39

Ly/Lx= 1.16

ΔM_{xs} = Mxs- M_Xs,adj = 0.91 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.60 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf = CxΔMxs + CxΔMys = 0.529

Mxf,adj = ΔMxf + Mxf = **10.83**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.410

Myf,adj = ΔMyf + Myf = **8.65**

For Panel 5

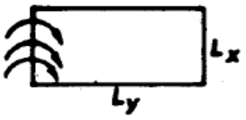
M_{xf} = 8.93 M_{Xs} = 11.91 $M_{Xs,adj}$ = 15.93

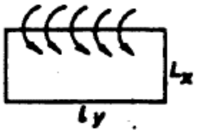
M_{yf} = 5.95 M_{ys} = 7.94 $M_{ys,adj}$ = 7.12

Ly/Lx= 1.36

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

ΔM_{ys} = Mys- M_ys,adj = 0.82 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf = CxΔMxs + CxΔMys = 0.270

Mxf,adj = ΔMxf + Mxf = **9.20**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.120

Myf,adj = ΔMyf + Myf = **6.07**

Panel 6 no need adjustment

For Panel 7

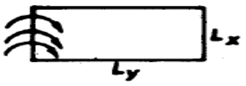
M_{xf} = 9.61 M_{Xs} = 12.36 $M_{Xs,adj}$ = 11.76

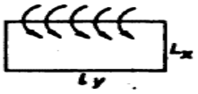
M_{yf} = 8.24 M_{ys} = 10.99 $M_{ys,adj}$ = 10.99

Ly/Lx= 1.08

ΔM_{xs} = Mxs- M_Xs,adj = 0.60 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 0.218

Mxf,adj = ΔMxf+ Mxf= **9.83**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.212

Myf,adj = ΔMyf+ Myf= **8.45**

Panel 8 no need adjustment

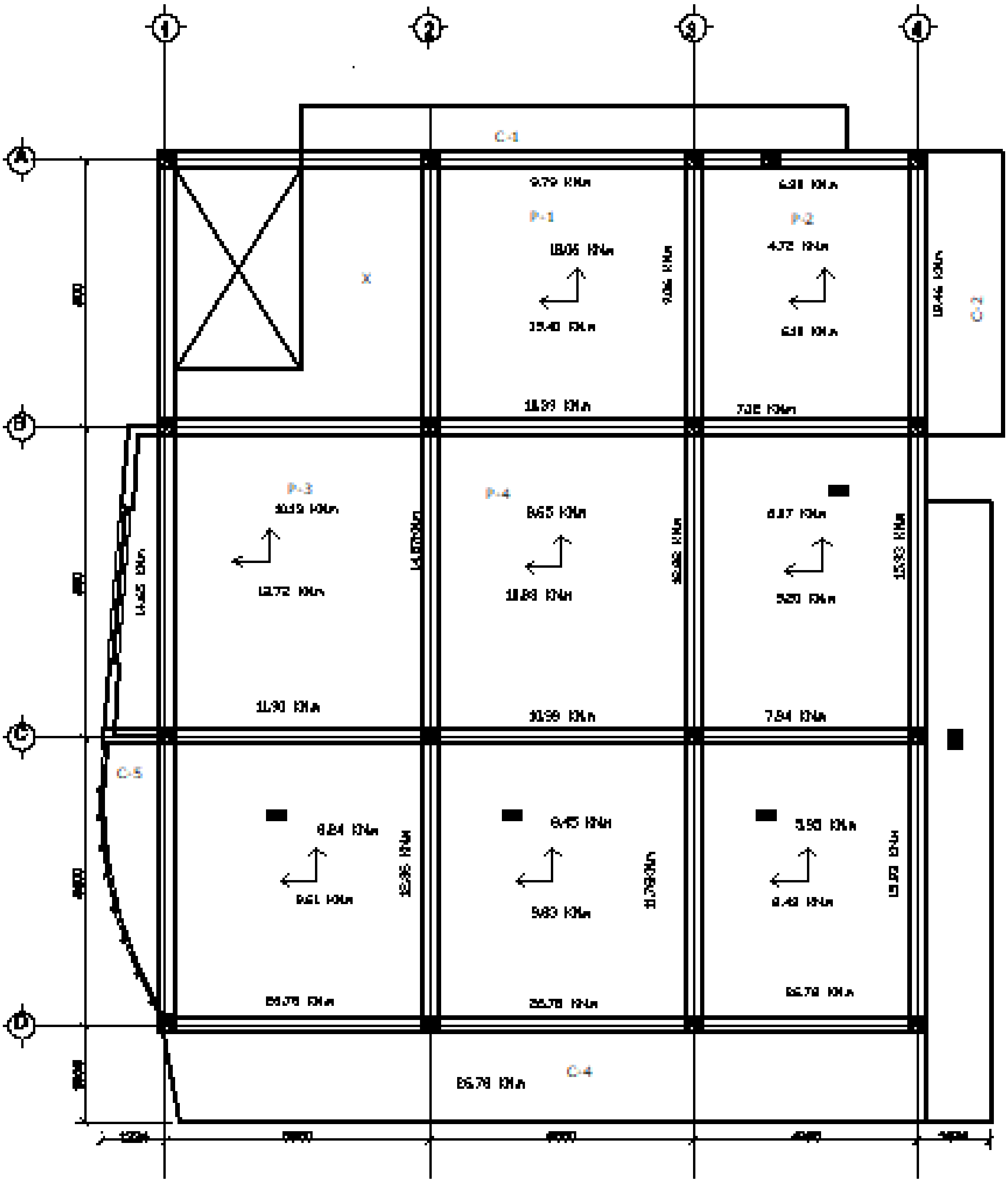


Figure 10-3second floor slab adjusted moment

Appendix B5: Shear calculation of 2nd floor slab

Table 24. Shear calculation of 2nd floor slab

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 1	1.00	5.00	12.23	β_{vxc} =	0.33	V_{xc} =	20.180
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.33	V_{yc} =	20.180
					β_{vyd} =	0	V_{yd} =	0.000
P-2	Type 1	1.18	4.25	11.61	β_{vxc} =	0.38	V_{xc} =	18.750
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.33	V_{yc} =	16.283
					β_{vyd} =	0	V_{yd} =	0.000
P-3	Type 1	1.16	5.00	13.75	β_{vxc} =	0.38	V_{xc} =	26.120
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.33	V_{yc} =	22.683
					β_{vyd} =	0	V_{yd} =	0.000
P-4	Type 1	1.16	5.00	14.37	β_{vxc} =	0.38	V_{xc} =	27.297
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.33	V_{yc} =	23.705
					β_{vyd} =	0	V_{yd} =	0.000
P-5	Type 1	1.36	4.25	14.37	β_{vxc} =	0.42	V_{xc} =	25.646
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.33	V_{yc} =	20.151
					β_{vyd} =	0	V_{yd} =	0.000
P-6	Type 1	1.08	5.00	12.15	β_{vxc} =	0.35	V_{xc} =	21.260
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.33	V_{yc} =	20.045
					β_{vyd} =	0	V_{yd} =	0.000
P-7	Type 1	1.08	5.00	12.14	β_{vxc} =	0.35	V_{xc} =	21.250
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.33	V_{yc} =	20.036
					β_{vyd} =	0	V_{yd} =	0
P-8	Type 1	1.27	4.25	12.33	β_{vxc} =	0.4	V_{xc} =	20.964
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.33	V_{yc} =	17.296
					β_{vyd} =	0	V_{yd} =	0.000

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= 175

k= 2.07 not < 2,so k=2

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

taking minimum reinforcment ∅ 10 c/c 300

$\rho_1 =$ 0.0015

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

$V_{min} =$ 77.476

$v_{Rd,c} =$ 41.867 $\leq V_{min}$ so, $v_{Rd,c} =$ 77.476

maximum shear force transfererd to beams is Vi= Vi=27.3

Vi= 27.3 KN/m

77.476>27.3,slab thickness is sufficient to support design load

Appendix B7: Second floor Reinforcement calculation

Table 25.Second floor Reinforcement calculation

panel	moment	Ast=Msd/Z*f		ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	9.06	252.00	10	311.51	300	ø	10	c/c	300
	M_{xf} =	19.4	352.11	10	222.94	200	ø	10	c/c	200
	M_{ys} =	9.79	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	18.06	327.79	10	239.48	230	ø	10	c/c	230
P-2	M_{xs} =	19.46	353.20	10	222.25	200	ø	10	c/c	200
	M_{xf} =	6.1	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	6.09	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	4.72	252.00	10	311.51	300	ø	10	c/c	300
P-3	M_{xs} =	14.65	265.90	10	295.23	290	ø	10	c/c	290
	M_{xf} =	12.72	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	11.9	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	10.13	252.00	10	311.51	300	ø	10	c/c	300
P-4	M_{xs} =	14.87	269.89	10	290.86	280	ø	10	c/c	280
	M_{xf} =	10.83	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	10.39	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	8.65	252.00	10	311.51	300	ø	10	c/c	300
P-5	M_{xs} =	12.82	252.00	10	311.51	300	ø	10	c/c	300
	M_{xf} =	9.2	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	7.12	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	6.07	252.00	10	311.51	300	ø	10	c/c	300
P-6	M_{xs} =	12.36	252.00	10	311.51	260	ø	10	c/c	260
	M_{xf} =	9.61	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	11.9	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	8.24	252.00	10	311.51	300	ø	10	c/c	300
P-7	M_{xs} =	11.76	252.00	10	311.51	300	ø	10	c/c	300
	M_{xf} =	9.83	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	10.99	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	8.45	252.00	10	311.51	300	ø	10	c/c	300
P-8	M_{xs} =	15.93	289.13	10	271.50	300	ø	10	c/c	300
	M_{xf} =	8.43	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	7.94	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	5.95	252.00	10	311.51	300	ø	10	c/c	300

C-1		9.79	330.00	10	237.88	200	ø 10 c/c 200
C-2		19.46	330.00	10	237.88	200	ø 10 c/c 200
C-3		14.65	330.00	10	237.88	200	ø 10 c/c 200
C-4		15.93	330.00	10	237.88	200	ø 10 c/c 200
C-5		26.78	371.94	10	211.06	200	ø 10 c/c 200

Appendix B8: Reinforcement Detailing

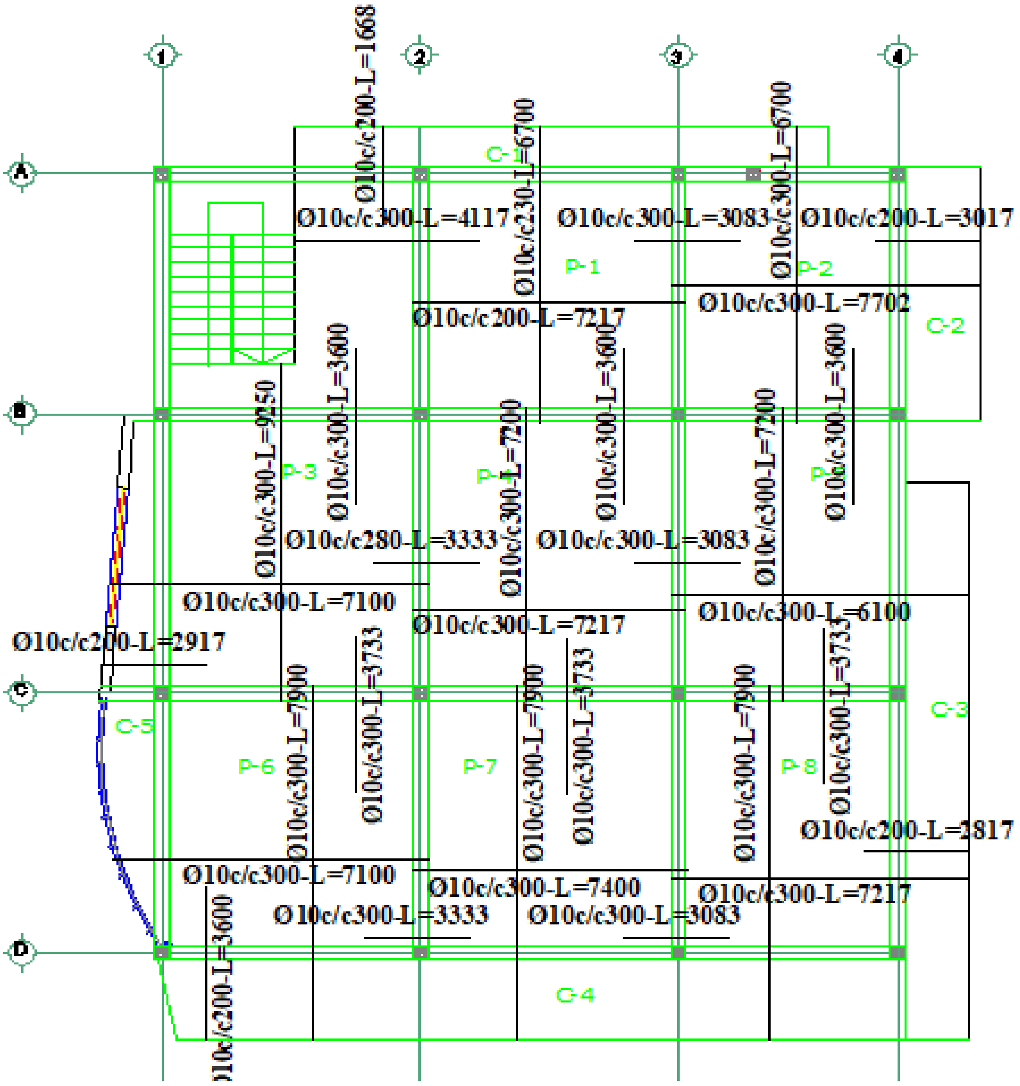


Figure 10-4 second floor reinforecement detailing

Appendix C: first floor slab design

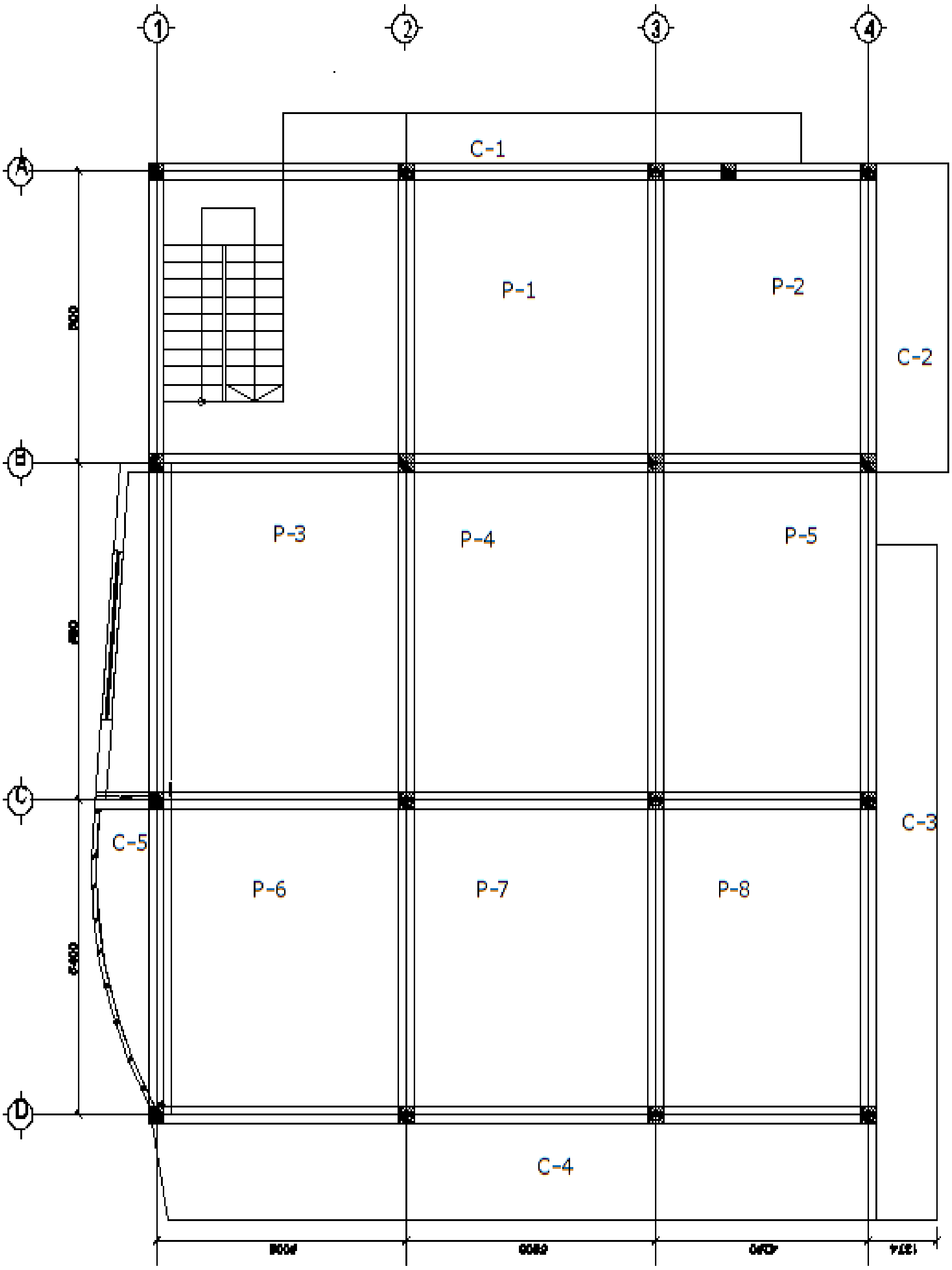


Figure 10-5first floor slab lay out

Appendix C1: First floor slab depth determination

Φbar= 10
Cnom= 20
ϒs = 1.15   fcd= $\frac{0.85*fck}{\Upsilon c}$ = 11.33   fyd= $\frac{fyk}{\Upsilon s}$ = 347.8
ϒc = 1.5
fck = 20
fyk= 400   unit wei 25
F1= 1.25
F2= 1
F3= 1
d'= 25

panel	Ly	Lx	Ly/Lx	Type	suppor	N	K	Lx/d	d	D	D providec
p1	5000	5000	1.00	two way	Interio	17.71	1.5	33.20	150.59	175.59	200
p2	5000	4250	1.18	two way	Interio	17.71	1.5	33.20	128.00	153.00	
p3	5800	5000	1.16	two way	Interio	17.71	1.5	33.20	150.59	175.59	
p4	5800	5000	1.16	two way	Interio	17.71	1.5	33.20	150.59	175.59	
p5	5800	4250	1.36	two way	Interio	17.71	1.5	33.20	128.00	153.00	
p6	5400	5000	1.08	two way	Interio	17.71	1.5	33.20	150.59	175.59	
p7	5400	5000	1.08	two way	Interio	17.71	1.5	33.20	150.59	175.59	
p8	5400	4250	1.27	two way	Interio	17.71	1.5	33.20	128.00	153.00	
c1	10350	850	12.18	one way	cantile	17.71	0.4	8.85	96.00	121.00	250
c2	5306	1602	3.31	one way	cantile	17.71	0.4	8.85	180.93	205.93	
c3	11594	1374	8.44	one way	cantile	17.71	0.4	8.85	155.18	180.18	
c4	14250	1800	7.92	one way	cantile	17.71	0.4	8.85	203.30	228.30	
c5	11200	1212	9.24	one way	cantile	17.71	0.4	8.85	136.89	161.89	

Appendix C2: Load calculation of first floor slab

Table 26Load calculation of first floor slab

<table>
<tr><td colspan="4">P1</td><td>Material</td><td>Unit weight</td><td>Dimension</td><td>Load KN/m2</td></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.200</td><td>5.00</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="3">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">PVC</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>25</td><td>25</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}}*t_{finish}=$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.240</td><td rowspan="3">3.36</td></tr>
<tr><td rowspan="2">25</td><td>(10*0.2*3)</td><td rowspan="2">0.24</td></tr>
<tr><td>6</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>4</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>10.20</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$</td><td></td><td>19.77</td><td colspan="2">KN/m^2</td></tr>
</table>

P2				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	6.75	6.75	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	14.500	14.5	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.141	1.98
	21.25	(5*0.2*3)	0.141176				
		3					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					4.63	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					12.25	KN/m^2	

P3				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	8.7	8.7	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	20.3	20.3	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.131	1.84
	29	6.34*0.2*3	0.131172				
		3.804					
LIVE LOAD Q_k					5	KN/m^2	
DEAD LOAD Gk					4.49	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					13.56	KN/m^2	

P4				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	6.25	6.25	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	22.8	22.75	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.188	2.64
	29	(9.1*0.2*3)	0.188276				
		5.46					
LIVE LOAD Q_k					5	KN/m^2	
DEAD LOAD Gk					5.29	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					14.64	KN/m^2	

P5				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	5.31	5.31	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	19.34	19.34	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.318	4.45
	24.65	13.05*0.2*	0.317647				
		7.83					
LIVE LOAD Q_k					5	KN/m^2	
DEAD LOAD Gk					7.10	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					17.08	KN/m^2	

P6				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	14.04	14.04	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	12.96	12.96	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.120	1.68
	27	(5.4*0.2*3)	0.12				
		3.24					
LIVE LOAD Q_k					5	KN/m^2	
DEAD LOAD Gk					4.33	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					13.35	KN/m^2	

P7				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.200	5.00
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	27	27	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.240	3.36
	27	10.8*0.2*3	0.24				
		6.48					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					10.20	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					19.77	KN/m^2	

<table>
<tr><td colspan="4">P8</td><td>Material</td><td>Unit weight</td><td>Dimension</td><td>Load KN/m2</td></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.200</td><td>5.00</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="3">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">PVC</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>22.95</td><td>22.95</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}} * t_{finish} =$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.141</td><td rowspan="3">1.98</td></tr>
<tr><td rowspan="2">22.95</td><td>(5.4*0.2*3)</td><td rowspan="2">0.141176</td></tr>
<tr><td>3.24</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>4</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>8.82</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35 G_k + 1.5 Q_k$</td><td></td><td>17.90224</td><td colspan="2">KN/m^2</td></tr>
</table>

<table>
<tr><td colspan="4">c1</td><td>Material</td><td>Unit weight</td><td>Dimension</td><td>Load KN/m2</td></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.250</td><td>6.25</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="3">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">Ceramic</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>8.7975</td><td>8.7975</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}} * t_{finish} =$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.814</td><td rowspan="3">11.40</td></tr>
<tr><td rowspan="2">8.7975</td><td>11.94*0.2*</td><td rowspan="2">0.814322</td></tr>
<tr><td>7.164</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>4</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>19.49</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35 G_k + 1.5 Q_k$</td><td></td><td>32.31219</td><td colspan="2">KN/m^2</td></tr>
</table>

c2				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	8.5	8.5	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.374	5.24
	8.5	(5.3*0.2*3)	0.374108				
		3.18					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					13.33	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					23.99215	KN/m^2	

c3				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	15.93	15.93	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.540	7.56
	15.93	4.342*0.2*	0.540183				
		8.6052					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					15.65	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					27.13096	KN/m^2	

c4				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	25.65	25.65	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.418	5.85
	25.65	7.85*0.2*	0.417544				
		10.71					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					13.94	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					24.81308	KN/m^2	

<table>
<tr><td colspan="4">c5</td><td>Material</td><td>Unit weight</td><td>Dimension</td><td>Load KN/m2</td></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.250</td><td>6.25</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="3">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">Ceramic</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>13.5744</td><td>13.5744</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}} * t_{finish} =$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.251</td><td rowspan="3">3.51</td></tr>
<tr><td rowspan="2">13.552</td><td>5.662*0.2*3</td><td rowspan="2">0.250679</td></tr>
<tr><td>3.3972</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>4</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>11.60</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35 G_k + 1.5 Q_k$</td><td></td><td>21.65933</td><td colspan="2">KN/m^2</td></tr>
</table>

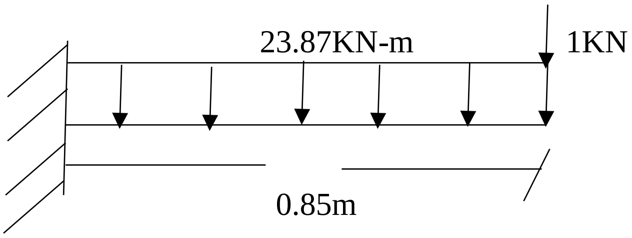
Appendix C3: First floor moment analysis

Table 27First floor moment analysis

panel	support condition	ly/lx	lx	lx^2	Pd	moment coeficient		moment	moment
						location	value	location	$Mi=\alpha_i P_d L^2 x$
P-1	Type 1	1.00	5.00	25.00	19.77	$\beta_{sx,sup=}$	0.034	M_{xs} =	16.80
						$\beta_{sx,span=}$	0.024	M_{xf} =	11.86
						$\beta_{sy,sup=}$	0.032	M_{ys} =	15.82
						$\beta_{sy,span=}$	0.024	M_{yf} =	11.86
P-2	Type 1	1.18	4.25	18.06	12.25	$\beta_{sx,sup=}$	0.041	M_{xs} =	9.07
						$\beta_{sx,span=}$	0.031	M_{xf} =	6.86
						$\beta_{sy,sup=}$	0.032	M_{ys} =	7.08
						$\beta_{sy,span=}$	0.024	M_{yf} =	5.31
P-3	Type 1	1.16	5.00	25.00	12.06	$\beta_{sx,sup=}$	0.04	M_{xs} =	12.06
						$\beta_{sx,span=}$	0.03	M_{xf} =	9.04
						$\beta_{sy,sup=}$	0.032	M_{ys} =	9.65
						$\beta_{sy,span=}$	0.024	M_{yf} =	7.23
P-4	Type 1	1.16	5.00	25.00	17.96	$\beta_{sx,sup=}$	0.04	M_{xs} =	17.96
						$\beta_{sx,span=}$	0.03	M_{xf} =	13.47
						$\beta_{sy,sup=}$	0.032	M_{ys} =	14.37
						$\beta_{sy,span=}$	0.024	M_{yf} =	10.78
P-5	Type 1	1.36	4.25	18.06	17.08	$\beta_{sx,sup=}$	0.048	M_{xs} =	14.81
						$\beta_{sx,span=}$	0.036	M_{xf} =	11.11
						$\beta_{sy,sup=}$	0.032	M_{ys} =	9.87
						$\beta_{sy,span=}$	0.024	M_{yf} =	7.40
P-6	Type 1	1.08	5.00	25.00	13.35	$\beta_{sx,sup=}$	0.036	M_{xs} =	12.01
						$\beta_{sx,span=}$	0.028	M_{xf} =	9.34
						$\beta_{sy,sup=}$	0.032	M_{ys} =	10.68
						$\beta_{sy,span=}$	0.024	M_{yf} =	8.01
P-7	Type 1	1.08	5.00	25.00	19.77	$\beta_{sx,sup=}$	0.036	M_{xs} =	17.79
						$\beta_{sx,span=}$	0.028	M_{xf} =	13.84
						$\beta_{sy,sup=}$	0.032	M_{ys} =	15.82
						$\beta_{sy,span=}$	0.024	M_{yf} =	11.86
P-8	Type 1	1.27	4.25	18.06	17.90	$\beta_{sx,sup=}$	0.045	M_{xs} =	14.55
						$\beta_{sx,span=}$	0.034	M_{xf} =	10.99
						$\beta_{sy,sup=}$	0.032	M_{ys} =	10.35
						$\beta_{sy,span=}$	0.024	M_{yf} =	7.76

Appendix C4: First floor cantilever moment calculation

C-1



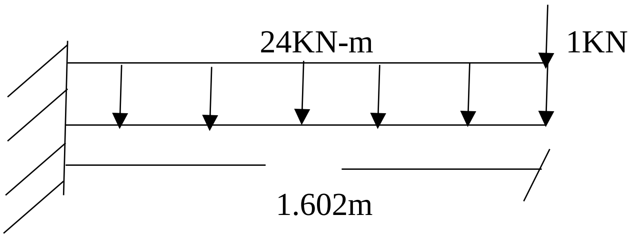
$M = \frac{wL2}{2}$+1KN*L

M=

$\frac{23.87*0.85m*0.85m}{2}$+1KN*0.85m

M=9.47KNm

C-2



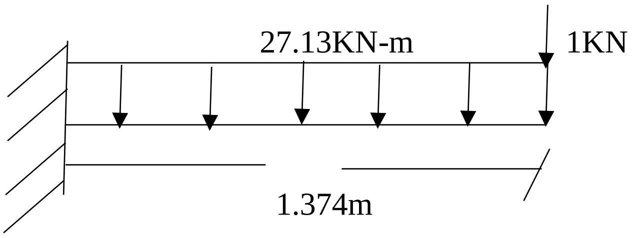
$M = \frac{wL2}{2}$+1KN*L

M=

$\frac{24*1.602m*1.602m}{2}$+1KN*1.602m

M=32.4KNm

C-3



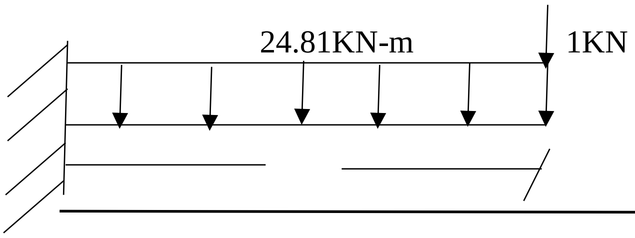
$M = \frac{wL2}{2}$+1KN*L

M=

$\frac{27.13*1.374m*1.374m}{2}$+1KN*1.374m

M=27KNm

C-4



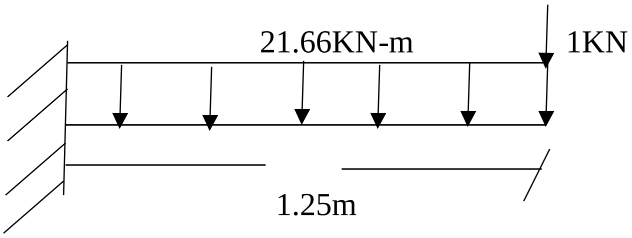
$M = \frac{wL2}{2}$+1KN*L

1.8m

$M = \frac{24.81*1.8m*1.8m}{2}+1KN*1.8m$

$M=42KNm$

C-5



$M = \frac{wL2}{2}+1KN*L$

$M = \frac{21.66*1.25m*1.25m}{2}+1KN*1.25m$

$M=18.17KNm$

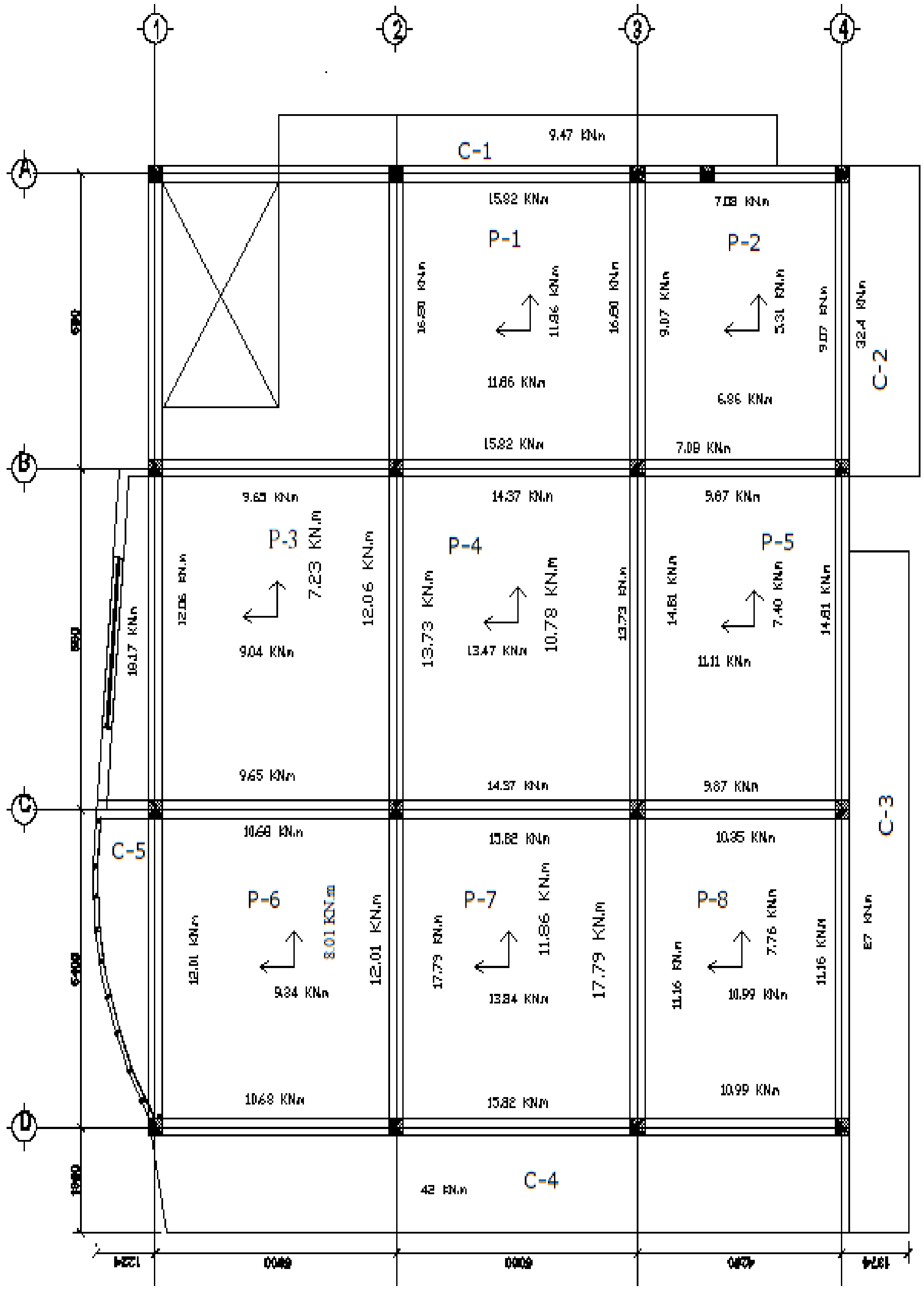
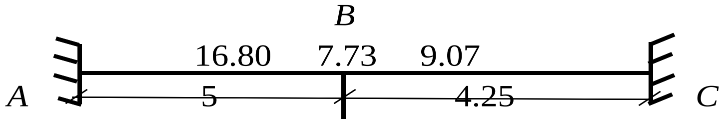


Figure 10-6First floor slab unadjusted moment

Appendix C5: First floor moment adjustment

p1&p2 on axis 3



DF	0.459	0.541
	3.55162	4.178
	13.2484	13.2

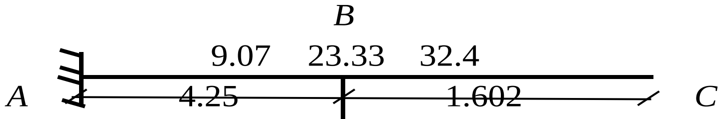
$$K_{AB} = \frac{I}{L} = \frac{I}{5} = 0.2\ I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{4.74} = 0.24\ I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.459$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.541$$

P2 & C2 on axis 4

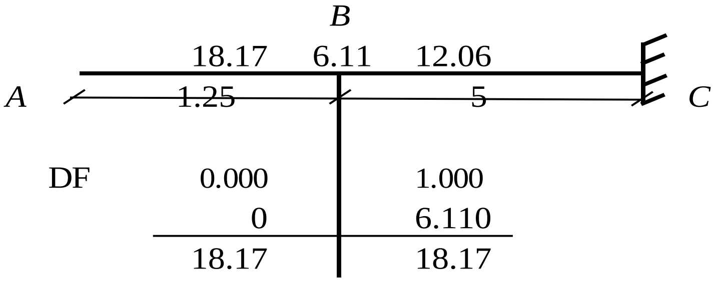


DF	1.000	0.000
	23.33	0.000
	32.40	32.40

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 1.000$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.000$$

C5 & P3 on axis 1



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 1.000$$

P3 & P4 –on axis 2 adjustment is required.

$\Delta M_s=$ 12.06 - 17.79 = -5.73

$0.2M_{s,max}$ 0.2 x 12.06 = 2.412

$\Delta M_s=$ -5.73 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 12.06 + 17.79 $)=$ 14.93

P4 & P5 –on axis 3 adjustment is required.

$\Delta M_s=$ 17.96 - 14.81 = 3.15

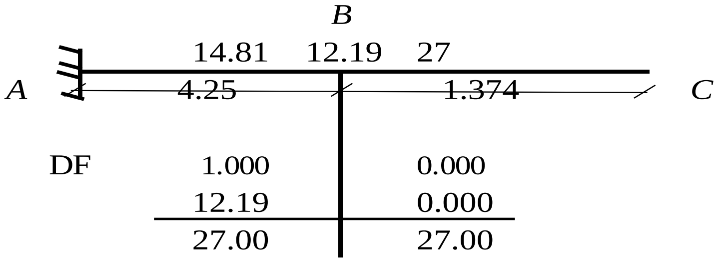
$0.2M_{s,max}$ 0.2 x 17.96 = 3.592

$\Delta M_s=$ 3.15 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 17.96 + 14.81 $)=$ 16.39

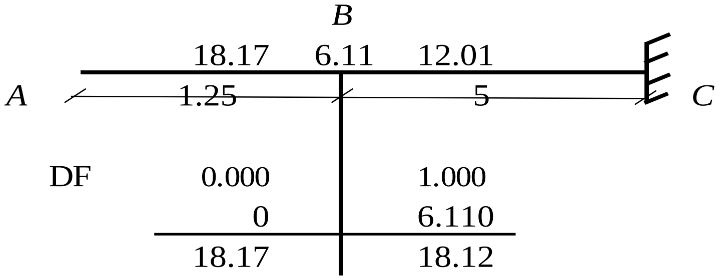
P5 & C3 on axis 4



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 1.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.000$$

C5 & P6 on axis 1



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}=0.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}=1.000$$

P6 & P7 –on axis 2 adjustment is required.

$\Delta M_s = 12.01 - 17.79 = -5.78$

$0.2M_{s,max}$ $0.2 \times 12.01 = 2.402$

$\Delta M_s = -5.78 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}(12.01 + 17.79)= 14.90$

P7 & P8 –on axis 3 adjustment is required.

$\Delta M_s = 17.79 - 14.55 = 3.24$

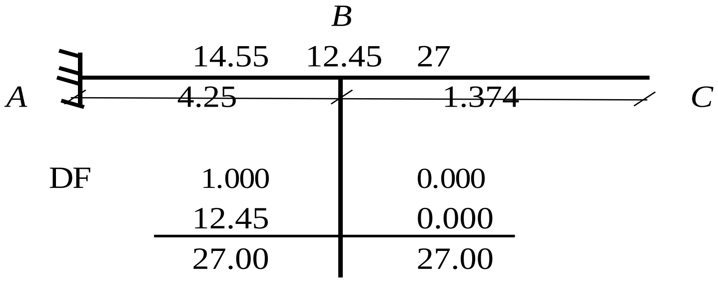
$0.2M_{s,max}$ $0.2 \times 17.79 = 3.558$

$\Delta M_s = 3.24 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}(17.79 + 14.55)= 16.17$

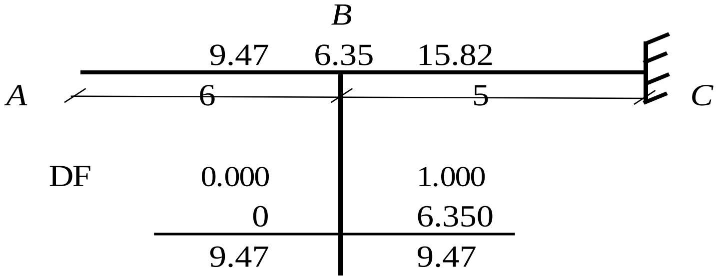
P8 & C3 on axis 4



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 1.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.000$$

C1 & P1 on axis A



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 1.000$$

C1 & P2 –on axis A adjustment is required.

$\Delta M_s =$ 9.47 - 7.08 = 2.39

$0.2M_{s,max}$ 0.2 x 9.47 = 1.894

$\Delta M_s =$ 2.39 < $0.2M_{s,max}$ FALSE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 9.47 + 7.08 $)=$ 8.28

P1 & P4 –on axis B adjustment is required.

$\Delta M_s =$ 15.82 - 14.37 = 1.45

$0.2M_{s,max}$ 0.2 x 15.82 = 3.164

$\Delta M_s =$ 1.45 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 15.82 + 14.37 $)=$ 15.10

P2 & P5 –on axis B adjustment is required.

$\Delta M_s = 7.08 - 9.87 = -2.79$

$0.2M_{s,max} \quad 0.2 \times 7.08 = 1.416$

$\Delta M_s = -2.79 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(7.08 + 9.87) = 8.48$

P3 & P6 –on axis C adjustment is required.

$\Delta M_s = 9.65 - 10.68 = -1.03$

$0.2M_{s,max} \quad 0.2 \times 9.65 = 1.93$

$\Delta M_s = -1.03 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(9.65 + 10.68) = 10.17$

P4 & P7 –on axis C adjustment is required.

$\Delta M_s = 14.37 - 15.82 = -1.45$

$0.2M_{s,max} \quad 0.2 \times 14.37 = 2.874$

$\Delta M_s = -1.45 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(14.37 + 15.82) = 15.10$

P5 & P8 –on axis C adjustment is required.

$\Delta M_s = 9.87 - 10.33 = -0.46$

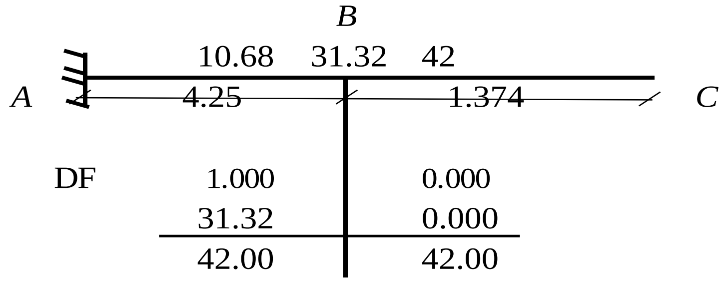
$0.2M_{s,max} \quad 0.2 \times 9.87 = 1.974$

$\Delta M_s = -0.46 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(9.87 + 10.33) = 10.10$

P6,7 & C4 on axis D



$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 1.000$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.000$$

Appendix C6 First floor field adjustment

For Panel 1

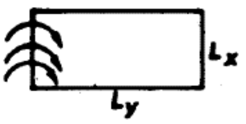
M_{xf} = 11.86 M_{Xs} = 16.8 $M_{Xs,adj}$ = 13.2

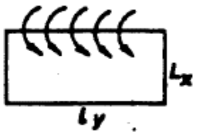
M_{yf} = 11.86 M_{ys} = 15.82 $M_{ys,adj}$ = 15.1

Ly/Lx= 1.00

ΔM_{xs} = Mxs- M_Xs,adj = 3.60 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.72 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf = CxΔMxs + CxΔMys = 1.547

Mxf ,adj = ΔMxf + Mxf = 13.41

ΔM_{yf} = CyΔMxs + CyΔMys = 1.380

Myf ,adj = ΔMyf + Myf = 13.24

Panel 2&3 no need adjustment

For Panel 4

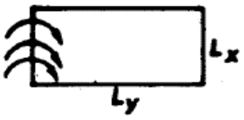
M_{xf} = 13.47 M_{Xs} = 17.96 $M_{Xs,adj}$ = 16.39

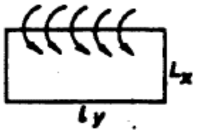
M_{yf} = 10.78 M_{ys} = 14.37 $M_{ys,adj}$ = 15.1

Ly/Lx= 1.16

ΔM_{xs} = Mxs- M_Xs,adj = 1.57 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 0.571

Mxf,adj = ΔMxf+ Mxf= 14.04

ΔM_{yf} = CyΔMxs + CyΔMys = 0.556

Myf,adj = ΔMyf+ Myf= 11.34

Panel 5&6 no need adjustment

For Panel 7

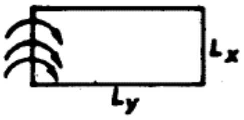
M_{xf} = 13.84 M_{Xs} = 17.79 $M_{Xs,adj}$ = 16.17

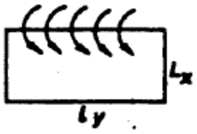
M_{yf} = 11.86 M_{ys} = 15.82 $M_{ys,adj}$ = 15.1

Ly/Lx= 1.16

ΔM_{xs} = Mxs- M_Xs,adj = 1.62 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.72 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 0.827

Mxf,adj = ΔMxf+ Mxf= 14.67

ΔM_{yf} = CyΔMxs + CyΔMys = 0.679

Myf,adj = ΔMyf+ Myf= 12.54

For Panel 8

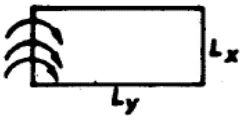
M_{xf} = 10.99 M_{Xs} = 14.55 $M_{Xs,adj}$ = 16.17

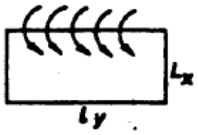
M_{yf} = 7.76 M_{ys} = 10.35 $M_{ys,adj}$ = 10.1

Ly/Lx= 1.36

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

ΔM_{ys} = Mys- M_ys,adj = 0.25 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf = CxΔMxs + CxΔMys = 0.082

Mxf ,adj = ΔMxf + Mxf = 11.07

ΔM_{yf} = CyΔMxs + CyΔMys = 0.037

Myf ,adj = ΔMyf + Myf = 7.80

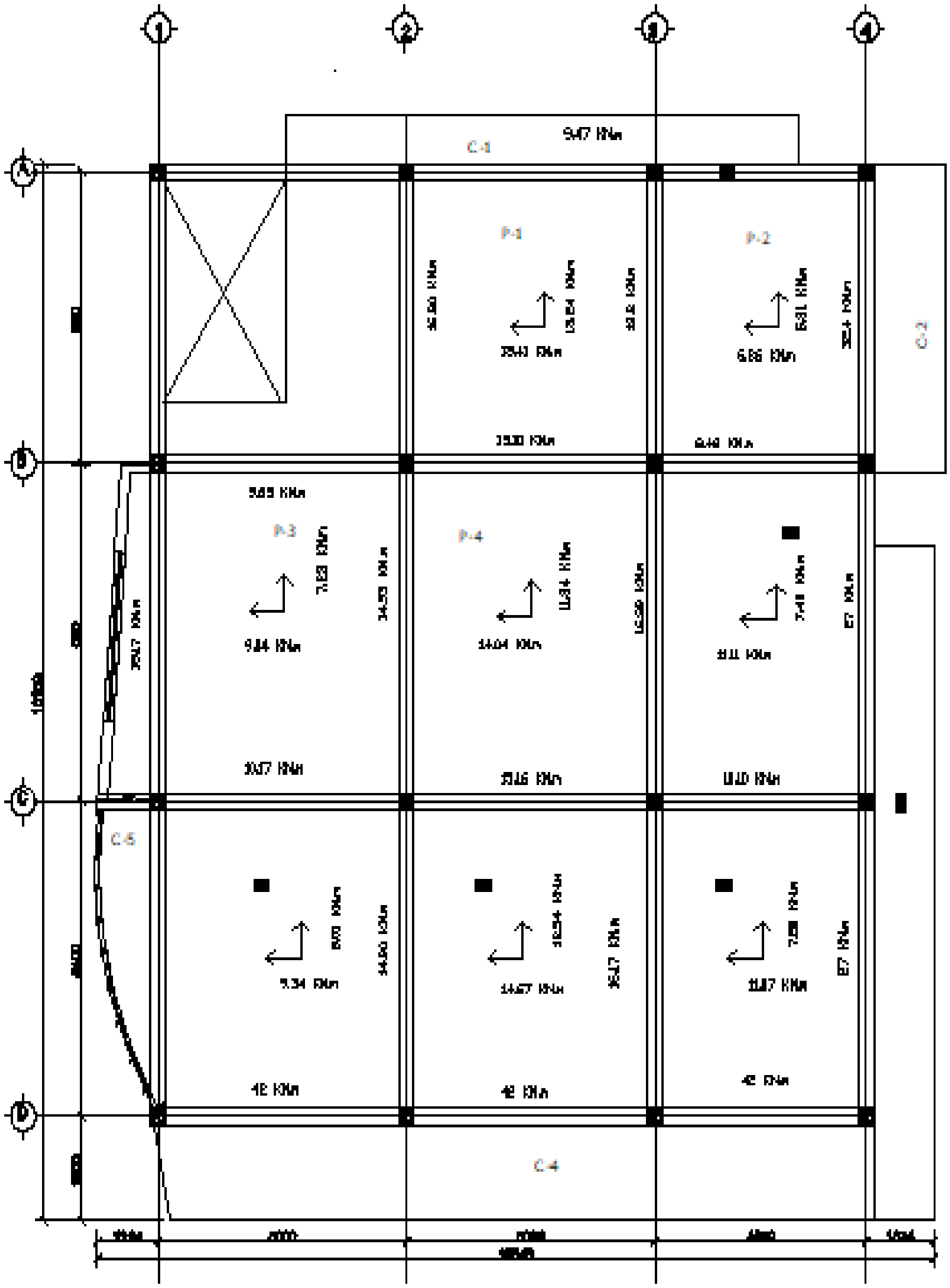


Figure 10-7First floor slab adjusted moment

Appendix C7: Load transfer to beam

Table 28Load 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 1	1.00	5.00	19.77	β_{vxc} =	0.33	V_{xc} =	32.621
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.33	V_{yc} =	32.621
					β_{vyd} =	0	V_{yd} =	0.000
P-2	Type 1	1.18	4.25	12.25	β_{vxc} =	0.38	V_{xc} =	19.777
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.33	V_{yc} =	17.175
					β_{vyd} =	0	V_{yd} =	0.000
P-3	Type 1	1.16	5.00	12.06	β_{vxc} =	0.38	V_{xc} =	22.908
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.33	V_{yc} =	19.893
					β_{vyd} =	0	V_{yd} =	0.000
P-4	Type 1	1.16	5.00	17.96	β_{vxc} =	0.38	V_{xc} =	34.123
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.33	V_{yc} =	29.634
					β_{vyd} =	0	V_{yd} =	0.000
P-5	Type 1	1.36	4.25	17.08	β_{vxc} =	0.42	V_{xc} =	30.490
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.33	V_{yc} =	23.956
					β_{vyd} =	0	V_{yd} =	0.000
P-6	Type 1	1.08	5.00	13.35	β_{vxc} =	0.35	V_{xc} =	23.355
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.33	V_{yc} =	22.020
					β_{vyd} =	0	V_{yd} =	0.000
P-7	Type 1	1.08	5.00	19.77	β_{vxc} =	0.35	V_{xc} =	34.598
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.33	V_{yc} =	32.621
					β_{vyd} =	0	V_{yd} =	0
P-8	Type 1	1.27	4.25	17.90	β_{vxc} =	0.4	V_{xc} =	30.434
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.33	V_{yc} =	25.108
					β_{vyd} =	0	V_{yd} =	0.000

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= 175

k= 2.07 not < 2,so k=2

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

taking minimum reinforcment ø 10 c/c 300

$\rho_1 =$ 0.0015

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

$V_{min} = 77.476$

$v_{Rd,c} = 41.867 \leq V_{min}$ so, $v_{Rd,c} = 77.476$

maximum shear force transfererd to beams is Vi= Vi=34.59

Vi= 34.59 KN/m

77.476>34.59,slab thickness is sufficient to support design load

Appendix C9: First floor Reinforcement calculation

Table 29Reinforcement of first floor

panel	moment	Ast=Msd/Z*f		ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =	16.8	304.92	10	257.44	250	ø	10	c/c	250
	M_{xf} =	13.41	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	15.1	274.07	10	286.43	250	ø	10	c/c	250
	M_{yf} =	13.24	252.00	10	311.51	300	ø	10	c/c	300
P-2	M_{xs} =	13.2	252.00	10	311.51	300	ø	10	c/c	300
	M_{xf} =	6.86	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	8.48	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	5.31	252.00	10	311.51	300	ø	10	c/c	300
P-3	M_{xs} =	14.93	270.98	10	289.69	250	ø	10	c/c	250
	M_{xf} =	9.04	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	9.65	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	7.23	252.00	10	311.51	300	ø	10	c/c	300
P-4	M_{xs} =	16.39	297.48	10	263.88	250	ø	10	c/c	250
	M_{xf} =	14.04	254.83	10	308.05	300	ø	10	c/c	300
	M_{ys} =	15.1	274.07	10	286.43	250	ø	10	c/c	250
	M_{yf} =	11.34	252.00	10	311.51	300	ø	10	c/c	300
P-5	M_{xs} =	16.39	297.48	10	263.88	250	ø	10	c/c	250
	M_{xf} =	11.11	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	10.1	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	7.4	252.00	10	311.51	300	ø	10	c/c	300
P-6	M_{xs} =	14.9	270.44	10	290.27	250	ø	10	c/c	250
	M_{xf} =	9.34	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	10.17	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	8.01	252.00	10	311.51	300	ø	10	c/c	300
P-7	M_{xs} =	16.17	293.49	10	267.47	250	ø	10	c/c	250
	M_{xf} =	14.67	266.26	10	294.82	250	ø	10	c/c	250
	M_{ys} =	15.1	274.07	10	286.43	250	ø	10	c/c	250
	M_{yf} =	12.54	252.00	10	311.51	300	ø	10	c/c	300
P-8	M_{xs} =	16.17	293.49	10	267.47	250	ø	10	c/c	250
	M_{xf} =	11.07	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	10.1	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	7.8	252.00	10	311.51	300	ø	10	c/c	300

C-1		9.47	330.00	10	237.88	200	ø	10	c/c	200
C-2		32.4	449.99	10	174.45	150	ø	10	c/c	150
C-3		27	375.00	10	209.34	200	ø	10	c/c	200
C-4		42	583.33	10	134.57	100	ø	10	c/c	100
C-5		18.17	330.00	10	237.88	200	ø	10	c/c	200

Appendix C10: Reinforcement Detailing

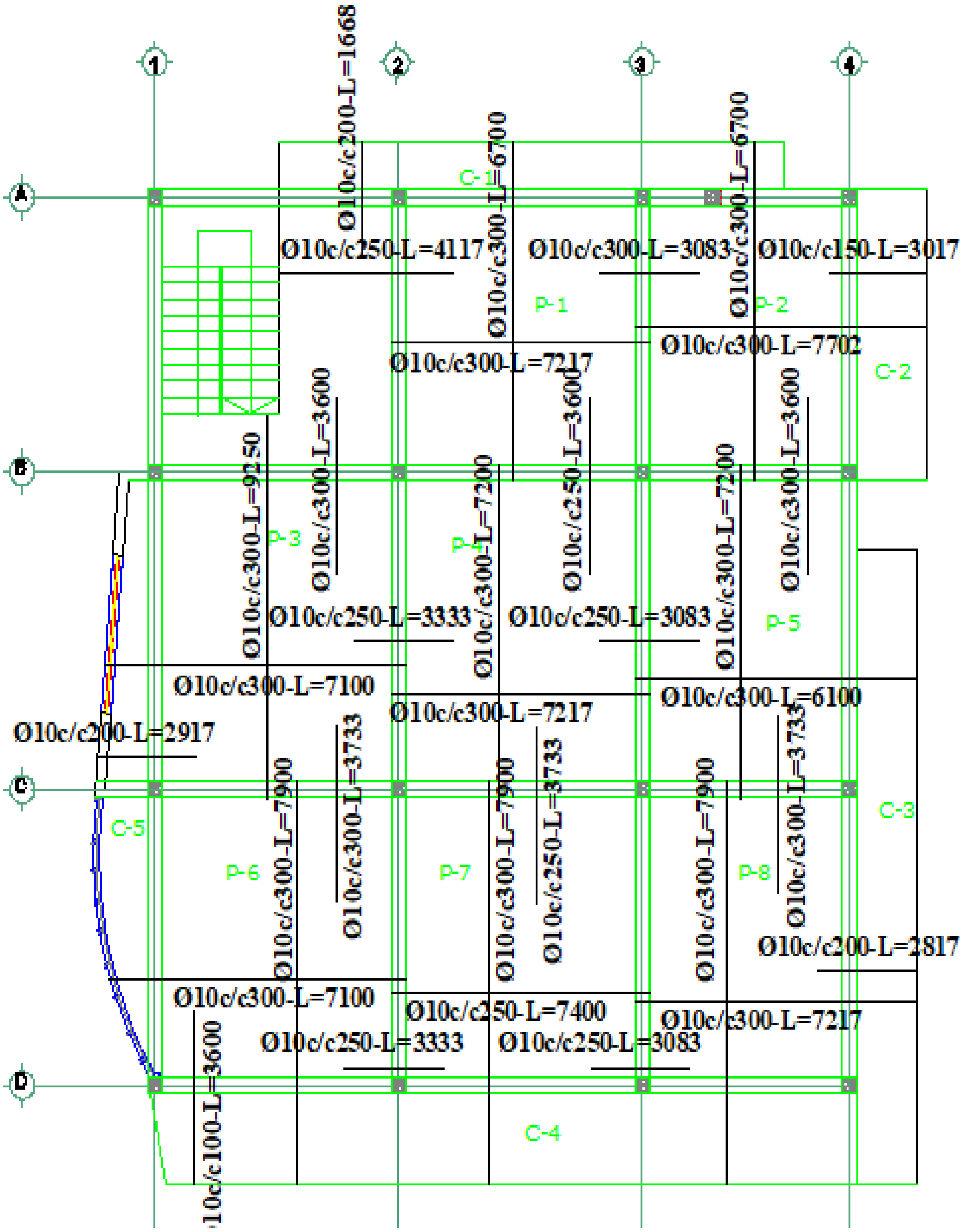


Figure 10-8Reinforcement Detailing

Appendix D: Ground floor slab design

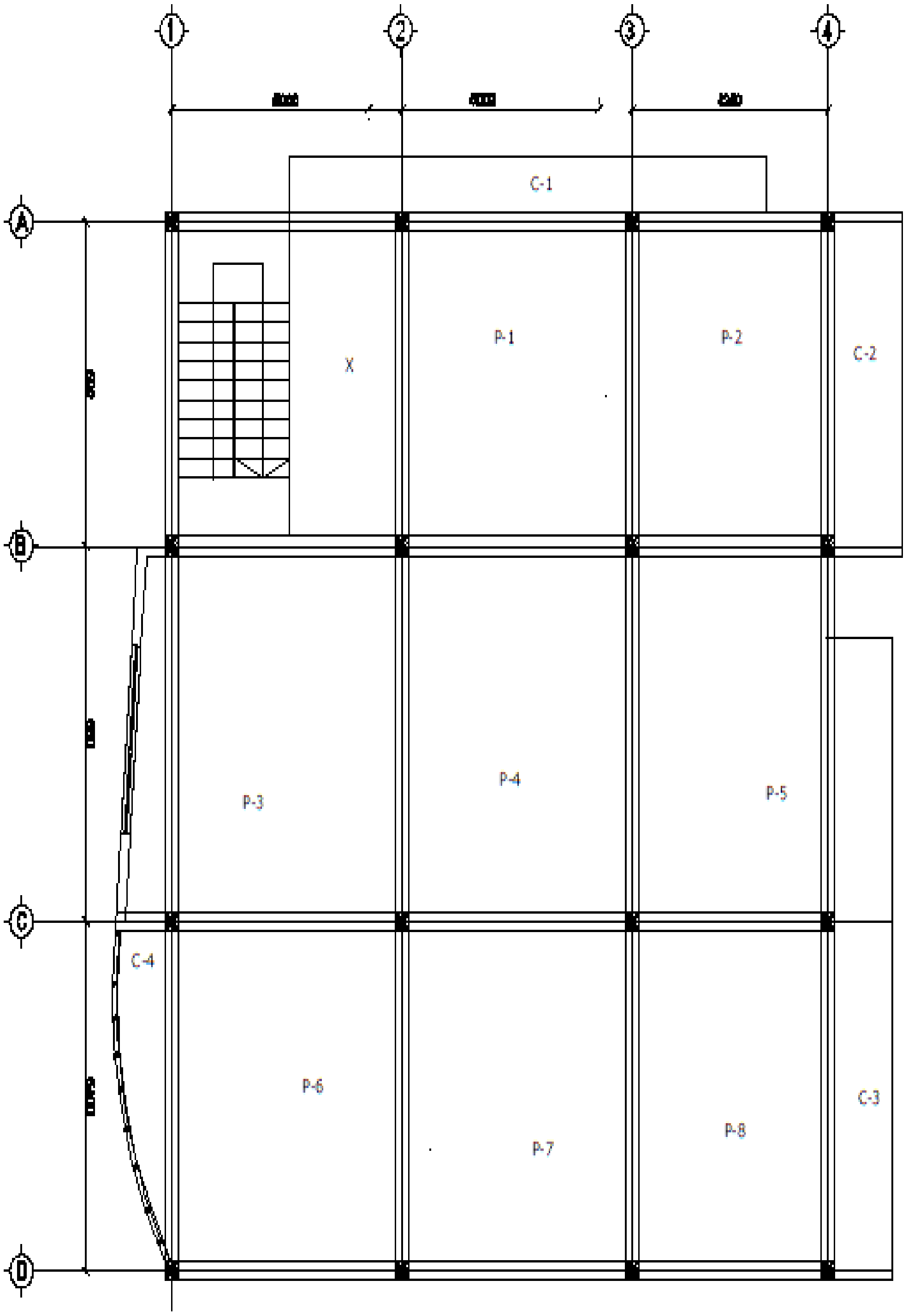


Figure 10-9Ground floor slab lay out

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 weigh 25
F1= 1.25
F2= 1
F3= 1
d'= 25

panel	Ly	Lx	Ly/Lx	Type	support	N	K	Lx/d	d	D
p1	5000	5000	1.00	two way	Interior	17.71	1.5	33.20	150.59	175.59
p2	5000	4250	1.18	two way	Interior	17.71	1.5	33.20	128.00	153.00
p3	5800	5000	1.16	two way	Interior	17.71	1.5	33.20	150.59	175.59
p4	5800	5000	1.16	two way	Interior	17.71	1.5	33.20	150.59	175.59
p5	5800	4250	1.36	two way	Interior	17.71	1.5	33.20	128.00	153.00
p6	5400	5000	1.08	two way	Interior	17.71	1.5	33.20	150.59	175.59
p7	5400	5000	1.08	two way	Interior	17.71	1.5	33.20	150.59	175.59
p8	5400	4250	1.27	two way	Interior	17.71	1.5	33.20	128.00	153.00
c1	10350	850	12.18	one way	cantilever	17.71	0.4	8.85	96.00	121.00
c2	5306	1602	3.31	one way	cantilever	17.71	0.4	8.85	180.93	205.93
c3	11594	1374	8.44	one way	cantilever	17.71	0.4	8.85	155.18	180.18
c4	11200	1212	9.24	one way	cantilever	17.71	0.4	8.85	136.89	161.89

Appendix D2: Ground floor slab load calculation

Table 30Ground floor slab load calculation

<table>
<tr><th colspan="4">P1</th><th>Material</th><th>Unit weight</th><th>Dimension</th><th>Load KN/m2</th></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.200</td><td>5.00</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="3">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">PVC</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>25</td><td>25</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}}*t_{finish}=$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.240</td><td rowspan="3">3.36</td></tr>
<tr><td rowspan="2">25</td><td>(10*0.2*3)</td><td rowspan="2">0.24</td></tr>
<tr><td>6</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>4</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>10.20</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$</td><td></td><td>19.77</td><td colspan="2">KN/m^2</td></tr>
</table>

P2				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	6.75	6.75	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	14.500	14.5	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.141	1.98
	21.25	(5*0.2*3)	0.141176				
		3					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					4.63	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					12.25	KN/m^2	

P3				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	8.7	8.7	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	20.3	20.3	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.131	1.84
	29	6.34*0.2*3	0.131172				
		3.804					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					4.49	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					12.06	KN/m^2	

P4				Material	Unit weight	Dimension	oad KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	6.25	6.25	1				
	$\frac{A_{finish}}{A_{total}} * t_{finish} =$		0.02				
	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	22.8	22.75	1				
	$\frac{A_{finish}}{A_{total}} * t_{finish} =$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.364	5.10
	29	17.6*0.2*3	0.364138				
		10.56					
LIVE LOAD Q_k					5	KN/m^2	
DEAD LOAD Gk					7.75	KN/m^2	
DESIGN LOAD $P_d = 1.35 G_k + 1.5 Q_k$					17.96	KN/m^2	

P5				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.014	0.35
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	5.31	5.31	1				
	$\frac{A_{finish}}{A_{total}} * t_{finish} =$		0.02				
	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	19.34	19.34	1				
	$\frac{A_{finish}}{A_{total}} * t_{finish} =$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.318	4.45
	24.65	13.05*0.2*	0.317647				
		7.83					
LIVE LOAD Q_k					5	KN/m^2	
DEAD LOAD Gk					7.10	KN/m^2	
DESIGN LOAD $P_d = 1.35 G_k + 1.5 Q_k$					17.08	KN/m^2	

<table>
<tr><td colspan="4">P6</td><td>Material</td><td>Unit weight</td><td>Dimension</td><td>Load KN/m2</td></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.014</td><td>0.35</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="6">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">Ceramic</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>14.04</td><td>14.04</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}}*t_{finish}=$</td><td>0.02</td></tr>
<tr><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">PVC</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>12.96</td><td>12.96</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}}*t_{finish}=$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.120</td><td rowspan="3">1.68</td></tr>
<tr><td rowspan="2">27</td><td>(5.4*0.2*3)</td><td rowspan="2">0.12</td></tr>
<tr><td>3.24</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>5</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>4.33</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$</td><td></td><td>13.35</td><td colspan="2">KN/m^2</td></tr>
</table>

<table>
<tr><td colspan="4">P7</td><td>Material</td><td>Unit weight</td><td>Dimension</td><td>Load KN/m2</td></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.200</td><td>5.00</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="3">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">PVC</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>27</td><td>27</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}}*t_{finish}=$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.240</td><td rowspan="3">3.36</td></tr>
<tr><td rowspan="2">27</td><td>10.8*0.2*3</td><td rowspan="2">0.24</td></tr>
<tr><td>6.48</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>4</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>10.20</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$</td><td></td><td>19.77</td><td colspan="2">KN/m^2</td></tr>
</table>

<table>
<tr><td colspan="4">P8</td><td>Material</td><td>Unit weight</td><td>Dimension</td><td>Load KN/m2</td></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.200</td><td>5.00</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="3">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">PVC</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>22.95</td><td>22.95</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}} * t_{finish} =$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.141</td><td rowspan="3">1.98</td></tr>
<tr><td rowspan="2">22.95</td><td>(5.4*0.2*3)</td><td rowspan="2">0.141176</td></tr>
<tr><td>3.24</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>4</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>8.82</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$</td><td></td><td>17.90224</td><td colspan="2">KN/m^2</td></tr>
</table>

<table>
<tr><td colspan="4">c1</td><td>Material</td><td>Unit weight</td><td>Dimension</td><td>Load KN/m2</td></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.250</td><td>6.25</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="3">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">Ceramic</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>8.7975</td><td>8.7975</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}} * t_{finish} =$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.814</td><td rowspan="3">11.40</td></tr>
<tr><td rowspan="2">8.7975</td><td>11.94*0.2*3</td><td rowspan="2">0.814322</td></tr>
<tr><td>7.164</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>4</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>19.49</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$</td><td></td><td>32.31219</td><td colspan="2">KN/m^2</td></tr>
</table>

<table>
<tr><td colspan="4">c2</td><td>Material</td><td>Unit weight</td><td>Dimension</td><td>Load KN/m2</td></tr>
<tr><td colspan="4">Slab</td><td>Reinforced concrete</td><td>25</td><td>0.250</td><td>6.25</td></tr>
<tr><td colspan="4">cement screed</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td colspan="4">celling plaster</td><td>concrete</td><td>23</td><td>0.030</td><td>0.69</td></tr>
<tr><td rowspan="3">Floor finish</td><td>A_{finish}</td><td>A_{total}</td><td>A_f/A_t</td><td rowspan="3">Ceramic</td><td rowspan="3">23</td><td rowspan="3">0.020</td><td rowspan="3">0.46</td></tr>
<tr><td>8.5</td><td>8.5</td><td>1</td></tr>
<tr><td colspan="2">$\frac{A_{finish}}{A_{total}} * t_{finish} =$</td><td>0.02</td></tr>
<tr><td rowspan="3">partition load</td><td>$A_{slab}(m^2)$</td><td>$V_{partition}$</td><td>A/V</td><td rowspan="3">HCB</td><td rowspan="3">14</td><td rowspan="3">0.374</td><td rowspan="3">5.24</td></tr>
<tr><td rowspan="2">8.5</td><td>(5.3*0.2*3)</td><td rowspan="2">0.374108</td></tr>
<tr><td>3.18</td></tr>
<tr><td colspan="5">LIVE LOAD Q_k</td><td>4</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="5">DEAD LOAD Gk</td><td>13.33</td><td colspan="2">KN/m^2</td></tr>
<tr><td colspan="4">DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$</td><td></td><td>23.99215</td><td colspan="2">KN/m^2</td></tr>
</table>

c3				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	PVC	23	0.020	0.46
	15.93	15.93	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.540	7.56
	15.93	4.342*0.2*	0.540183				
		8.6052					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					15.65	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					27.13096	KN/m^2	

c4				Material	Unit weight	Dimension	Load KN/m2
Slab				Reinforced concrete	25	0.250	6.25
cement screed				concrete	23	0.030	0.69
celling plaster				concrete	23	0.030	0.69
Floor finish	A_{finish}	A_{total}	A_f/A_t	Ceramic	23	0.020	0.46
	13.5744	13.5744	1				
	$\frac{A_{finish}}{A_{total}}*t_{finish}=$		0.02				
partition load	$A_{slab}(m^2)$	$V_{partition}$	A/V	HCB	14	0.251	3.51
	13.552	.662*0.2*	0.250679				
		3.3972					
LIVE LOAD Q_k					4	KN/m^2	
DEAD LOAD Gk					11.60	KN/m^2	
DESIGN LOAD $P_d = 1.35\ G_k + 1.5\ Q_k$					21.65933	KN/m^2	

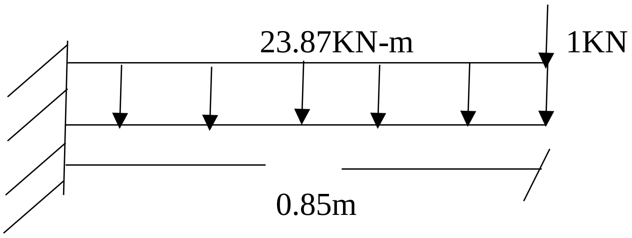
Appendix D3: Moment analysis of ground floor slab

Table 31Moment analysis of ground floor slab

panel	support condition	ly/lx	lx	lx^2	Pd	moment coeficient		moment	moment
						location	value	location	$M_i=\alpha_i P_d$
P-1	Type 1	1.00	5.00	25.00	19.77	$\beta_{sx,sup}$=	0.034	M_{xs} =	16.80
						$\beta_{sx,span}$=	0.024	M_{xf} =	11.86
						$\beta_{sy,sup}$=	0.032	M_{ys} =	15.82
						$\beta_{sy,span}$=	0.024	M_{yf} =	11.86
P-2	Type 1	1.18	4.25	18.06	12.25	$\beta_{sx,sup}$=	0.041	M_{xs} =	9.07
						$\beta_{sx,span}$=	0.031	M_{xf} =	6.86
						$\beta_{sy,sup}$=	0.032	M_{ys} =	7.08
						$\beta_{sy,span}$=	0.024	M_{yf} =	5.31
P-3	Type 1	1.16	5.00	25.00	12.06	$\beta_{sx,sup}$=	0.04	M_{xs} =	12.06
						$\beta_{sx,span}$=	0.03	M_{xf} =	9.04
						$\beta_{sy,sup}$=	0.032	M_{ys} =	9.65
						$\beta_{sy,span}$=	0.024	M_{yf} =	7.23
P-4	Type 1	1.16	5.00	25.00	17.96	$\beta_{sx,sup}$=	0.04	M_{xs} =	17.96
						$\beta_{sx,span}$=	0.03	M_{xf} =	13.47
						$\beta_{sy,sup}$=	0.032	M_{ys} =	14.37
						$\beta_{sy,span}$=	0.024	M_{yf} =	10.78
P-5	Type 1	1.36	4.25	18.06	17.08	$\beta_{sx,sup}$=	0.048	M_{xs} =	14.81
						$\beta_{sx,span}$=	0.036	M_{xf} =	11.11
						$\beta_{sy,sup}$=	0.032	M_{ys} =	9.87
						$\beta_{sy,span}$=	0.024	M_{yf} =	7.40
P-6	Type 2	1.08	5.00	25.00	13.35	$\beta_{sx,sup}$=	0.031	M_{xs} =	10.34
						$\beta_{sx,span}$=	0.035	M_{xf} =	11.68
						$\beta_{sy,sup}$=	0.037	M_{ys} =	12.34
						$\beta_{sy,span}$=	0.028	M_{yf} =	9.34
P-7	Type 2	1.08	5.00	25.00	19.77	$\beta_{sx,sup}$=	0.031	M_{xs} =	15.32
						$\beta_{sx,span}$=	0.035	M_{xf} =	17.30
						$\beta_{sy,sup}$=	0.037	M_{ys} =	18.29
						$\beta_{sy,span}$=	0.028	M_{yf} =	13.84
P-8	Type 2	1.27	4.25	18.06	17.90	$\beta_{sx,sup}$=	0.04	M_{xs} =	12.93
						$\beta_{sx,span}$=	0.043	M_{xf} =	13.90
						$\beta_{sy,sup}$=	0.037	M_{ys} =	11.96
						$\beta_{sy,span}$=	0.028	M_{yf} =	9.05

Appendix D4: Ground floor cantilever moment calculation

C-1

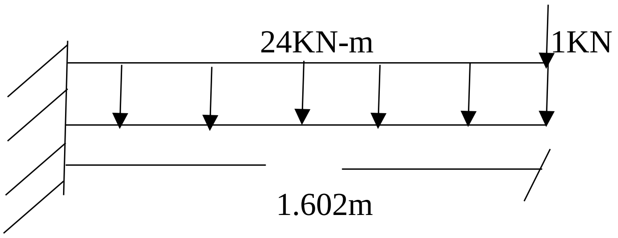


$M = \frac{wL2}{2}$+1KN*L

M=$\frac{23.87*0.85m*0.85m}{2}$+1KN*0.85m

M=9.47KNm

C-2

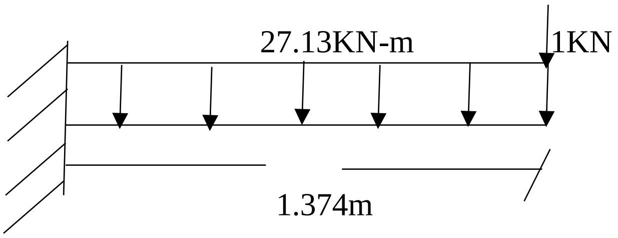


$M = \frac{wL2}{2}$+1KN*L

M=$\frac{24*1.602m*1.602m}{2}$+1KN*1.602m

M=32.4KNm

C-3

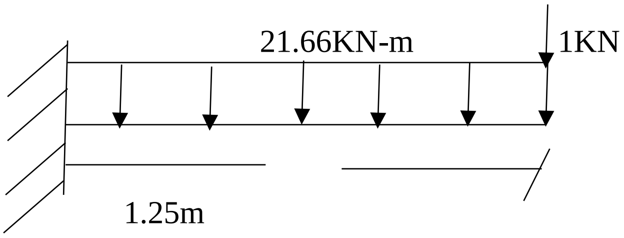


$M = \frac{wL2}{2}$+1KN*L

M=$\frac{27.13*1.374m*1.374m}{2}$+1KN*1.374m

M=27KNm

C-4



$$M = \frac{wL2}{2} + 1KN*L$$

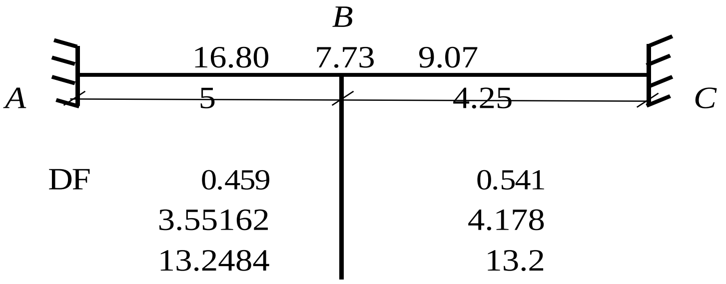
$$M = \frac{21.66*1.25m*1.25m}{2} + 1KN*1.25m$$

M=18.17KNm

Appendix D5: Moment Adjustment

Support moment adjustment

p1&p2 on axis 3



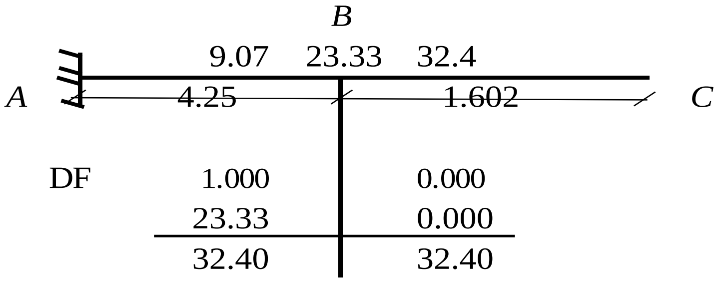
$$K_{AB} = \frac{I}{L} = \frac{I}{5} = 0.2\ I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{4.74} = 0.24\ I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.459$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.541$$

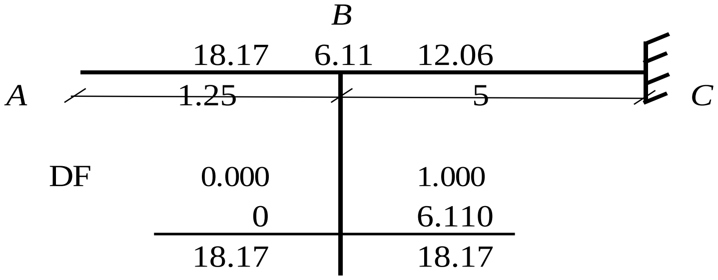
P2 & C2 on axis 4



$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 1.000$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 0.000$$

C5 & P3 on axis 1



$$DF_{AB} = \frac{K_{AB}}{K_{AB}+K_{BC}} = 0.000$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC}+K_{AB}} = 1.000$$

P3 & P4 –on axis 2 adjustment is required.

$\Delta M_s = 12.06 - 17.79 = -5.73$

$0.2M_{s,max}$ $0.2 \times 12.06 = 2.412$

$\Delta M_s = -5.73 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(12.06 + 17.79) = 14.93$

P4 & P5 –on axis 3 adjustment is required.

$\Delta M_s = 17.96 - 14.81 = 3.15$

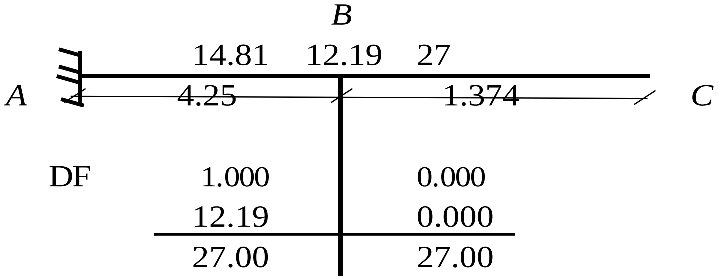
$0.2M_{s,max}$ $0.2 \times 17.96 = 3.592$

$\Delta M_s = 3.15 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(17.96 + 14.81) = 16.39$

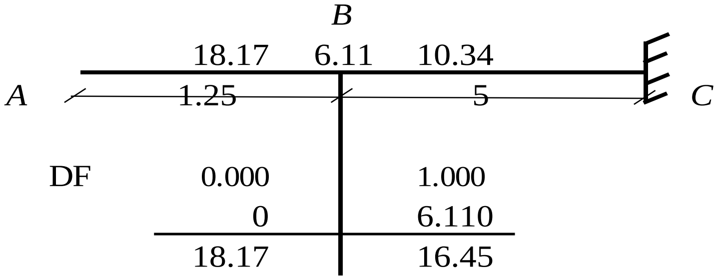
P5 & C3 on axis 4



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 1.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.000$$

C5 & P6 on axis 1



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 1.000$$

P6 & P7 –on axis 2 adjustment is required.

$\Delta M_s=$ 10.34 - 15.32 = -4.98

$0.2M_{s,max}$ 0.2 x 10.34 = 2.068

$\Delta M_s=$ -4.98 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 10.34 + 15.32 $)=$ 12.83

P7 & P8 –on axis 3 adjustment is required.

$\Delta M_s=$ 15.32 - 12.93 = 2.39

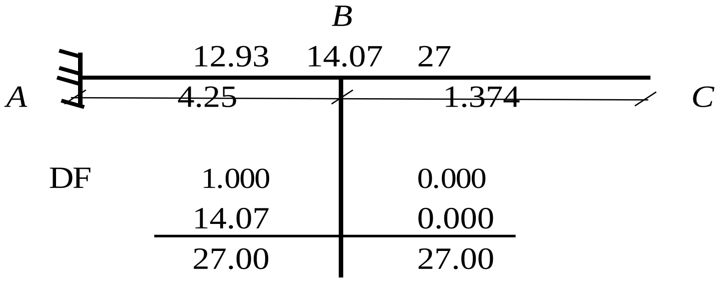
$0.2M_{s,max}$ 0.2 x 15.32 = 3.064

$\Delta M_s=$ 2.39 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 15.32 + 12.93 $)=$ 14.13

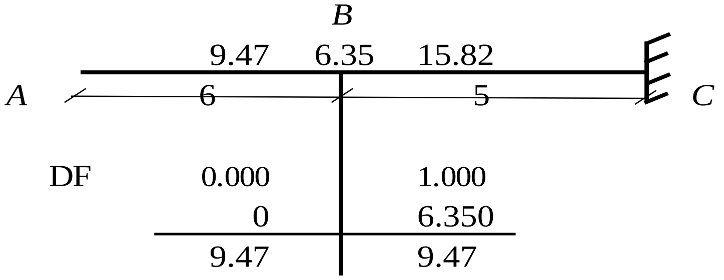
P8 & C3 on axis 4



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 1.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 0.000$$

C1 & P1 on axis A



$$DF_{AB}=\frac{K_{AB}}{K_{AB}+K_{BC}}= 0.000$$

$$DF_{BC}=\frac{K_{BC}}{K_{BC}+K_{AB}}= 1.000$$

C1 & P2 –on axis A adjustment is required.

ΔM_s= 9.47 - 7.08 = 2.39

$0.2M_{s,max}$ 0.2 x 9.47 = 1.894

ΔM_s= 2.39 < $0.2M_{s,max}$ FALSE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 9.47 + 7.08 $)=$ 8.28

P1 & P4 –on axis B adjustment is required.

ΔM_s= 15.82 - 14.37 = 1.45

$0.2M_{s,max}$ 0.2 x 15.82 = 3.164

ΔM_s= 1.45 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 15.82 + 14.37 $)=$ 15.10

P2 & P5 –on axis B adjustment is required.

$\Delta M_s = 7.08 - 9.87 = -2.79$
$0.2M_{s,max} \quad 0.2 \times 7.08 = 1.416$
$\Delta M_s = -2.79 < 0.2M_{s,max}$ TRUE
Therefore, average of the moments can be taken.
$M_{s,avg} = \frac{1}{2}(7.08 + 9.87) = 8.48$

P3 & P6 –on axis C adjustment is required.

$\Delta M_s = 9.65 - 12.34 = -2.69$
$0.2M_{s,max} \quad 0.2 \times 9.65 = 1.93$
$\Delta M_s = -2.69 < 0.2M_{s,max}$ TRUE
Therefore, average of the moments can be taken.
$M_{s,avg} = \frac{1}{2}(9.65 + 12.34) = 11.00$

P4 & P7 –on axis C adjustment is required.

$\Delta M_s = 14.37 - 18.29 = -3.92$
$0.2M_{s,max} \quad 0.2 \times 14.37 = 2.874$
$\Delta M_s = -3.92 < 0.2M_{s,max}$ TRUE
Therefore, average of the moments can be taken.
$M_{s,avg} = \frac{1}{2}(14.37 + 18.29) = 16.33$

P5 & P8 –on axis C adjustment is required.

$\Delta M_s = 9.87 - 11.96 = -2.09$
$0.2M_{s,max} \quad 0.2 \times 9.87 = 1.974$
$\Delta M_s = -2.09 < 0.2M_{s,max}$ TRUE
Therefore, average of the moments can be taken.
$M_{s,avg} = \frac{1}{2}(9.87 + 11.96) = 10.92$

Field Moment adjustment

For Panel 1

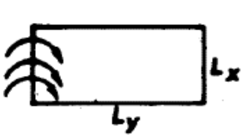
M_{xf} = 11.86 M_{Xs} = 16.8 $M_{Xs,adj}$ = 13.2

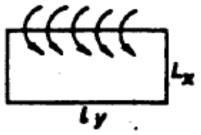
M_{yf} = 11.86 M_{ys} = 15.82 $M_{ys,adj}$ = 15.1

Ly/Lx= 1.00

ΔM_{xs} = Mxs- M_Xs,adj = 3.60 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.72 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf = CxΔMxs + CxΔMys = 1.547

Mxf,adj = ΔMxf + Mxf = **13.41**

ΔM_{yf} = CyΔMxs + CyΔMys = 1.380

Myf,adj = ΔMyf + Myf = **13.24**

Panel 2&3 no need adjustment

For Panel 4

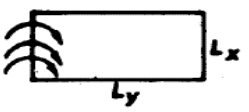
M_{xf} = 13.47 M_{Xs} = 17.96 $M_{Xs,adj}$ = 16.39

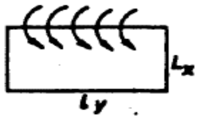
M_{yf} = 10.78 M_{ys} = 14.37 $M_{ys,adj}$ = 15.1

Ly/Lx= 1.16

ΔM_{xs} = Mxs- M_Xs,adj = 1.57 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf = CxΔMxs + CxΔMys = 0.571

Mxf,adj = ΔMxf + Mxf = **14.04**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.556

Myf,adj = ΔMyf + Myf = **11.34**

Panel 5&6 no need adjustment

For Panel 7

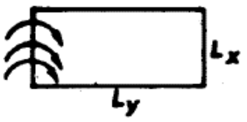
M_{xf} = 13.84 M_{Xs} = 15.32 $M_{Xs,adj}$ = 14.13

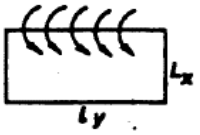
M_{yf} = 17.3 M_{ys} = 18.29 $M_{ys,adj}$ = 1633

Ly/Lx= 1.16

ΔM_{xs} = Mxs- M_Xs,adj = 1.19 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf = CxΔMxs + CxΔMys = 0.433

Mxf ,adj = ΔMxf + Mxf = **14.27**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.421

Myf ,adj = ΔMyf + Myf = **17.72**

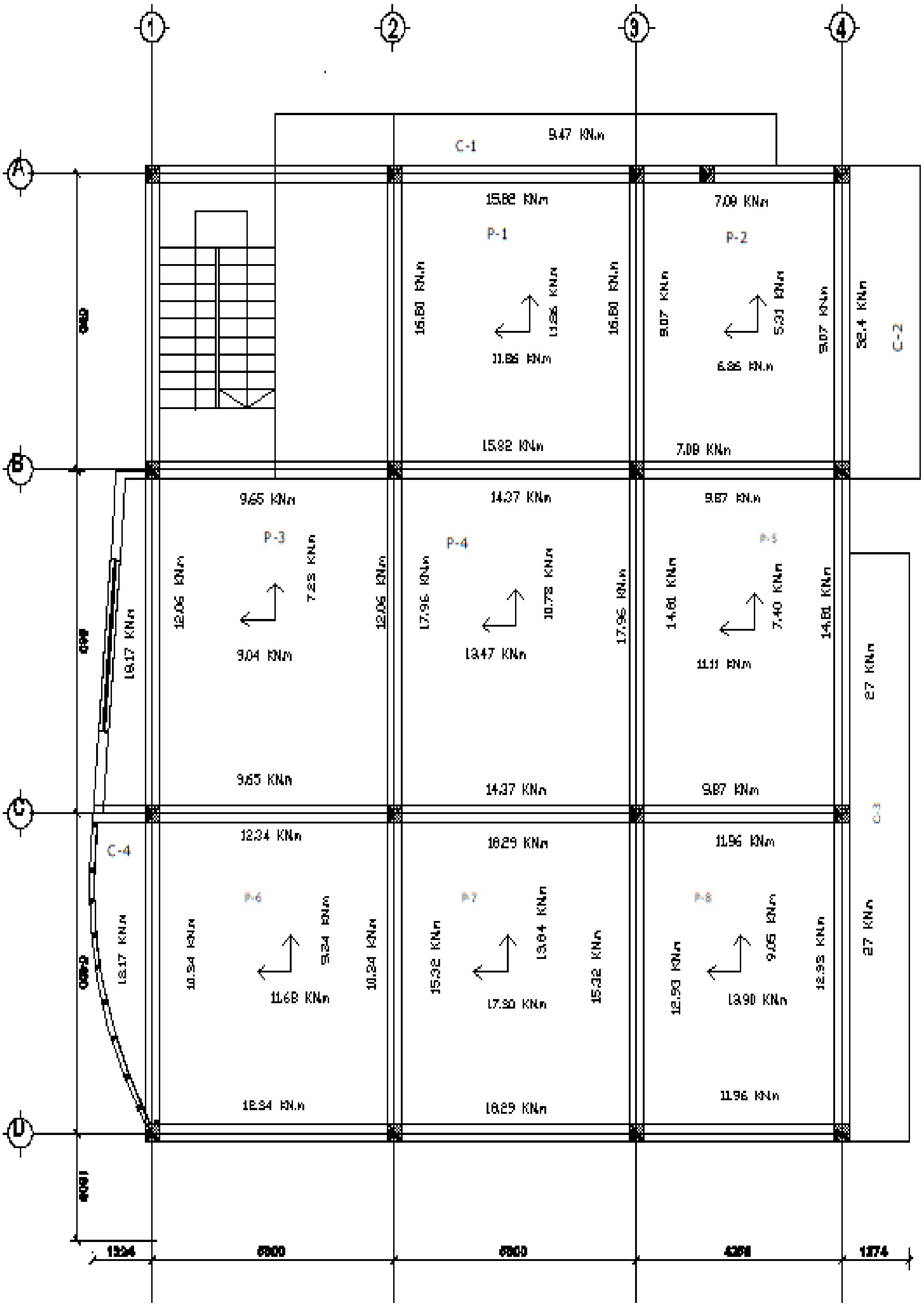


Figure 10-10Ground floor plan unadjusted moment

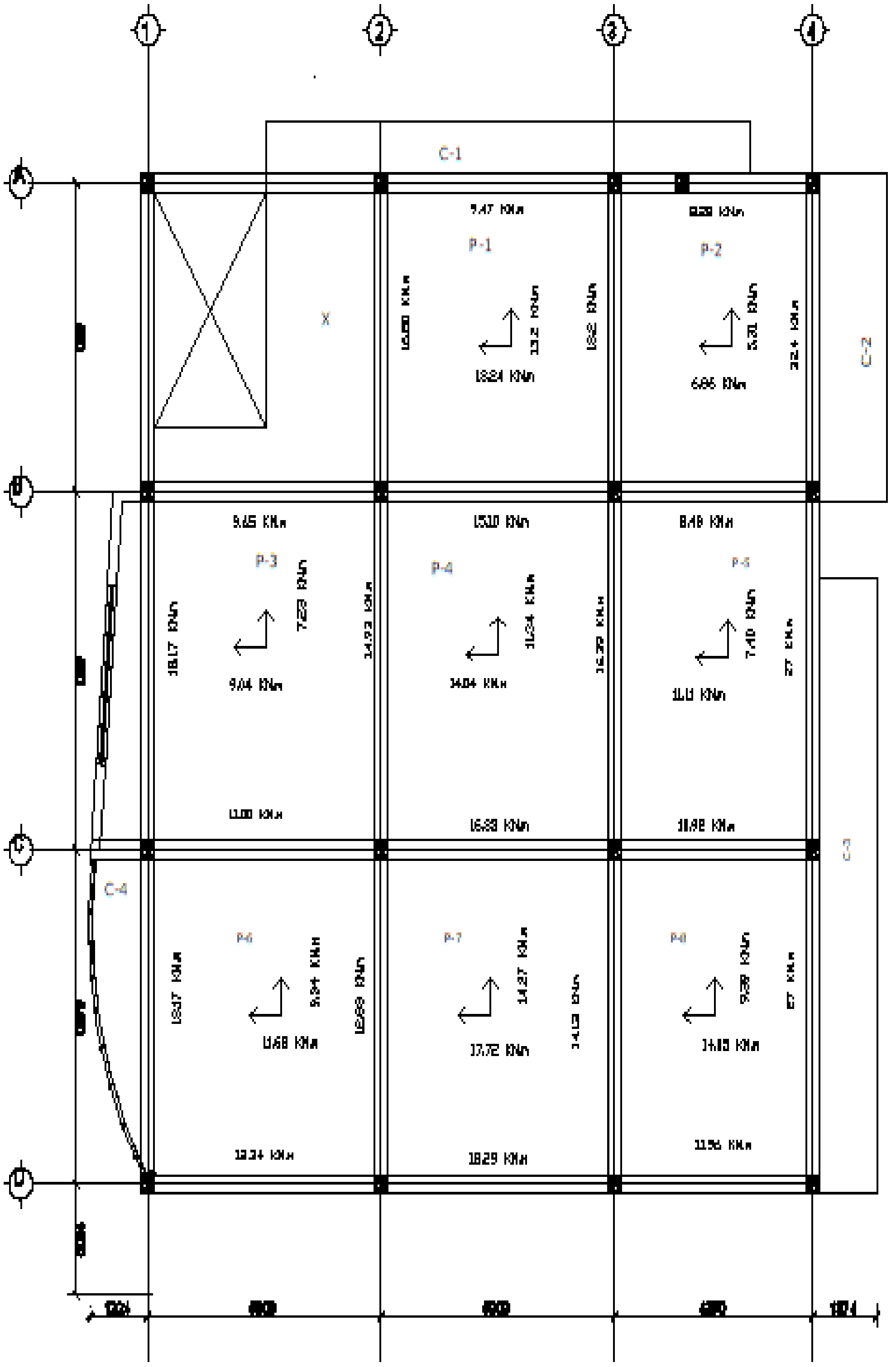


Figure 10-11Ground floor slab adjusted moment

Appendix D6: Reinforcement calculation

Table 32Reinforcement calculation

panel	moment	Ast=Msd/Z*f		ø	spacing	spacing used	Reinforcment			
P-1	M_{xs} =		252.00	10	311.51	300	ø	10	c/c	300
	M_{xf} =		252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =		252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =		252.00	10	311.51	300	ø	10	c/c	300
P-2	M_{xs} =	16.8	304.92	10	257.44	250	ø	10	c/c	250
	M_{xf} =	13.41	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	15.1	274.07	10	286.43	250	ø	10	c/c	250
	M_{yf} =	13.24	252.00	10	311.51	300	ø	10	c/c	300
P-3	M_{xs} =	13.2	252.00	10	311.51	300	ø	10	c/c	300
	M_{xf} =	6.86	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	8.48	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	5.31	252.00	10	311.51	300	ø	10	c/c	300
P-4	M_{xs} =	14.93	270.98	10	289.69	250	ø	10	c/c	250
	M_{xf} =	9.04	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	9.65	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	7.23	252.00	10	311.51	300	ø	10	c/c	300
P-5	M_{xs} =	16.39	297.48	10	263.88	250	ø	10	c/c	250
	M_{xf} =	14.04	254.83	10	308.05	300	ø	10	c/c	300
	M_{ys} =	15.1	274.07	10	286.43	250	ø	10	c/c	250
	M_{yf} =	11.34	252.00	10	311.51	300	ø	10	c/c	300
P-6	M_{xs} =	16.39	297.48	10	263.88	250	ø	10	c/c	250
	M_{xf} =	11.11	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	10.1	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	7.4	252.00	10	311.51	300	ø	10	c/c	300
P-7	M_{xs} =	18.17	329.79	10	238.03	200	ø	10	c/c	200
	M_{xf} =	11.68	252.00	10	311.51	300	ø	10	c/c	300
	M_{ys} =	11	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	9.34	252.00	10	311.51	300	ø	10	c/c	300

P-8	M_{xs} =	12.83	252.00	10	311.51	300	ø	10	c/c	300
	M_{xf} =	17.72	321.62	10	244.08	200	ø	10	c/c	200
	M_{ys} =	14.27	259.00	10	303.09	300	ø	10	c/c	300
	M_{yf} =	16.33	296.39	10	264.85	250	ø	10	c/c	250
P-9	M_{xs} =	14.13	256.46	10	306.09	300	ø	10	c/c	300
	M_{xf} =	14.05	255.01	10	307.83	300	ø	10	c/c	300
	M_{ys} =	10.92	252.00	10	311.51	300	ø	10	c/c	300
	M_{yf} =	9.39	252.00	10	311.51	300	ø	10	c/c	300
C-1		9.47	330.00	10	237.88	200	ø	10	c/c	200
C-2		32.4	449.99	10	174.45	150	ø	10	c/c	150
C-3		27	375.00	10	209.34	200	ø	10	c/c	200
C-4		18.17	330.00	10	237.88	200	ø	10	c/c	200

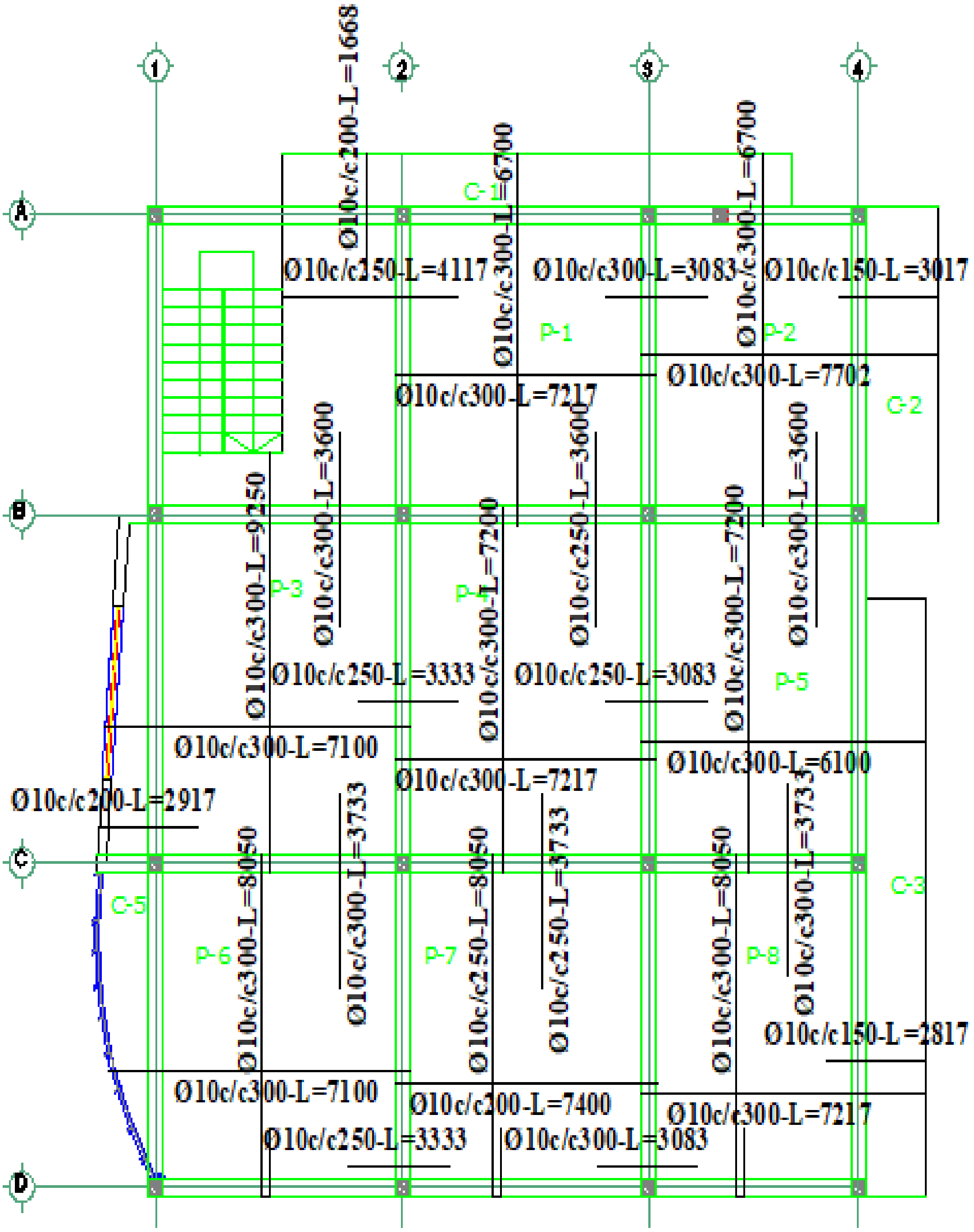


Figure 10-12ground floor slab reinforcement detailing

Appendix E: roof slab design

Appendix E1: Depth determination

We use C30/37 concrete and S400 rebar

Φbar= 10
Cnom= 20
Υs = 1.15 fcd= $\frac{0.85*fck}{\Upsilon c}$=17.00 fyd= $\frac{fyk}{\Upsilon s}$= 347.83
Υc = 1.5
fck = 30
fyk= 400 unit weigh 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	5000	5000	1.00	two way	Interior	19.22	1.5	36.03	138.77	163.77	200
p2	5000	4250	1.18	two way	Interior	19.22	1.5	36.03	117.96	142.96	
p3	5800	5000	1.16	two way	Interior	19.22	1.5	36.03	138.77	163.77	
p4	5800	5000	1.16	two way	Interior	19.22	1.5	36.03	138.77	163.77	
p5	5800	4250	1.36	two way	Interior	19.22	1.5	36.03	117.96	142.96	
p6	5400	5000	1.08	two way	Interior	19.22	1.5	36.03	138.77	163.77	
p7	5400	5000	1.08	two way	Interior	19.22	1.5	36.03	138.77	163.77	
p8	5400	4250	1.27	two way	Interior	19.22	1.5	36.03	117.96	142.96	

The worst combination is COMB 4 which is **Pd = 9.742 KN/m^2**

Appendix E2: Moment analysis of roof slab

Table 33Moment analysis of roof slab

panel	support condition	ly/lx	lx	lx^2	Pd	moment coeficient		moment location	moment $Mi=\alpha_i P_d$
						location	value		
P-1	Type 4	1	5	25.00	9.74	$\beta_{sx,sup}=$	0.039	$M_{xs}=$	9.50
						$\beta_{sx,span}=$	0.029	$M_{xf}=$	7.06
						$\beta_{sy,sup}=$	0.037	$M_{ys}=$	9.01
						$\beta_{sy,span}=$	0.028	$M_{yf}=$	6.82
P-2	Type 2	1.00	5.000	25.00	9.74	$\beta_{sx,sup}=$	0.039	$M_{xs}=$	9.50
						$\beta_{sx,span}=$	0.029	$M_{xf}=$	7.06
						$\beta_{sy,sup}=$	0.037	$M_{ys}=$	9.01
						$\beta_{sy,span}=$	0.028	$M_{yf}=$	6.82
P-3	Type 4	1.18	4.250	18.06	9.74	$\beta_{sx,sup}=$	0.061	$M_{xs}=$	10.73
						$\beta_{sx,span}=$	0.046	$M_{xf}=$	8.09
						$\beta_{sy,sup}=$	0.045	$M_{ys}=$	7.92
						$\beta_{sy,span}=$	0.034	$M_{yf}=$	5.98
P-4	Type 3	1.16	5	25	9.74	$\beta_{sx,sup}=$	0.053	$M_{xs}=$	12.91
						$\beta_{sx,span}=$	0.040	$M_{xf}=$	9.74
						$\beta_{sy,sup}=$	0.037	$M_{ys}=$	9.01
						$\beta_{sy,span}=$	0.028	$M_{yf}=$	6.82
P-5	Type 1	1.16	5	25.00	9.74	$\beta_{sx,sup}=$	0.041	$M_{xs}=$	9.99
						$\beta_{sx,span}=$	0.031	$M_{xf}=$	7.55
						$\beta_{sy,sup}=$	0.032	$M_{ys}=$	7.79
						$\beta_{sy,span}=$	0.024	$M_{yf}=$	5.85
P-6	Type 3	1.36	4.25	18.06	9.74	$\beta_{sx,sup}=$	0.066	$M_{xs}=$	11.61
						$\beta_{sx,span}=$	0.049	$M_{xf}=$	8.62
						$\beta_{sy,sup}=$	0.037	$M_{ys}=$	6.51
						$\beta_{sy,span}=$	0.028	$M_{yf}=$	4.93
P-7	Type 4	1.08	5	25	9.74	$\beta_{sx,sup}=$	0.054	$M_{xs}=$	13.15
						$\beta_{sx,span}=$	0.041	$M_{xf}=$	9.99
						$\beta_{sy,sup}=$	0.045	$M_{ys}=$	10.96
						$\beta_{sy,span}=$	0.031	$M_{yf}=$	7.55
P-8	Type 2	1.08	5	25.00	9.74	$\beta_{sx,sup}=$	0.044	$M_{xs}=$	10.72
						$\beta_{sx,span}=$	0.033	$M_{xf}=$	8.04
						$\beta_{sy,sup}=$	0.037	$M_{ys}=$	9.01
						$\beta_{sy,span}=$	0.028	$M_{yf}=$	6.82
P-9	Type 4	1.27	4.25	18.06	9.74	$\beta_{sx,sup}=$	0.067	$M_{xs}=$	11.79
						$\beta_{sx,span}=$	0.049	$M_{xf}=$	8.62
						$\beta_{sy,sup}=$	0.045	$M_{ys}=$	7.92
						$\beta_{sy,span}=$	0.031	$M_{yf}=$	5.45

Appendix E3 moment adjustment roof slab

Support adjustment

P2&P3 –on axis C adjustment is required.

$\Delta M_s = 9.5 - 10.73 = -1.23$

$0.2M_{s,max}$ $0.2 \times 9.5 = 1.9$

$\Delta M_s = -1.23 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(9.5 + 10.73) = 10.12$

P4&P5 –on axis C adjustment is required.

$\Delta M_s = 9.99 - 12.91 = -2.92$

$0.2M_{s,max}$ $0.2 \times 9.99 = 1.998$

$\Delta M_s = -2.92 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(9.99 + 12.91) = 11.45$

P5&P6 –on axis C adjustment is required.

$\Delta M_s = 9.99 - 11.61 = -1.62$

$0.2M_{s,max}$ $0.2 \times 9.99 = 1.998$

$\Delta M_s = -1.62 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(9.99 + 11.61) = 10.80$

P7&P8 –on axis C adjustment is required.

$\Delta M_s = 13.15 - 10.72 = 2.43$

$0.2M_{s,max}$ $0.2 \times 13.15 = 2.63$

$\Delta M_s = 2.43 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(13.15 + 10.72) = 11.94$

P8&P9 –on axis C adjustment is required.

$\Delta M_s = 10.72 - 11.79 = -1.07$

$0.2M_{s,max} \quad 0.2 \times 10.72 = 2.144$

$\Delta M_s = -1.07 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(10.72 + 11.79) = 11.26$

P2&P5 –on axis C adjustment is required.

$\Delta M_s = 9.01 - 7.79 = 1.22$

$0.2M_{s,max} \quad 0.2 \times 9.01 = 1.802$

$\Delta M_s = 1.22 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(9.01 + 7.79) = 8.40$

P3&P6 –on axis C adjustment is required.

$\Delta M_s = 7.92 - 6.51 = 1.41$

$0.2M_{s,max} \quad 0.2 \times 7.92 = 1.584$

$\Delta M_s = 1.41 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(7.92 + 6.51) = 7.22$

P4&P7 –on axis C adjustment is required.

$\Delta M_s = 9.01 - 10.96 = -1.95$

$0.2M_{s,max} \quad 0.2 \times 9.01 = 1.802$

$\Delta M_s = -1.95 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(9.01 + 10.96) = 9.99$

P5&P8 –on axis C adjustment is required.

$\Delta M_s = 7.79 - 9.01 = -1.22$

$0.2M_{s,max} \quad 0.2 \times 7.79 = 1.558$

$\Delta M_s = -1.22 < 0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg} = \frac{1}{2}(7.79 + 9.01) = 8.40$

P6&P9 –on axis C adjustment is required.

ΔM_s= 6.51 - 7.92 = -1.41

$0.2M_{s,max}$ 0.2 x 6.51 = 1.302

ΔM_s= -1.41 < $0.2M_{s,max}$ TRUE

Therefore, average of the moments can be taken.

$M_{s,avg}=\frac{1}{2}($ 6.51 + 7.92 $)=$ 7.22

For Panel 2

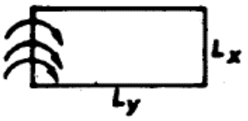
M_{xf} = 7.06 M_{Xs} = 9.5 $M_{Xs,adj}$ = 10.2

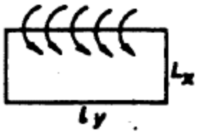
M_{yf} = 6.82 M_{ys} = 9.01 $M_{ys,adj}$ = 8.4

Ly/Lx= 1.27

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

ΔM_{ys} = Mys- M_ys,adj = 0.61 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf = CxΔMxs + CxΔMys = 0.201

Mxf ,adj = ΔMxf + Mxf = **7.261**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.089

Myf ,adj = ΔMyf + Myf = **6.909**

For Panel 4

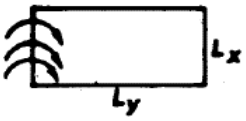
M_{xf} = 9.74 M_{Xs} = 12.91 $M_{Xs,adj}$ = 11.45

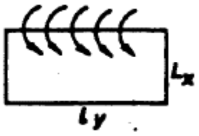
M_{yf} = 6.82 M_{ys} = 9.01 $M_{ys,adj}$ = 9.99

Ly/Lx= 1.27

ΔM_{xs} = Mxs- M_Xs,adj = 1.46 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.00 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 0.531

Mxf,adj = ΔMxf+ Mxf= **10.271**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.517

Myf,adj = ΔMyf+ Myf= **7.337**

For Panel 7

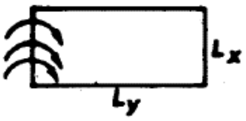
M_{xf} = 9.99 M_{Xs} = 13.15 $M_{Xs,adj}$ = 11.94

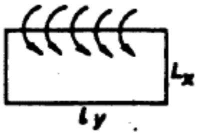
M_{yf} = 7.55 M_{ys} = 10.96 $M_{ys,adj}$ = 9.99

Ly/Lx= 1.27

ΔM_{xs} = Mxs- M_Xs,adj = 1.21 neglect

ΔM_{ys} = Mys- M_ys,adj = 0.97 neglect





Cx = 0.329 Cx = 0.364

Cy = 0.146 Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 0.760

Mxf,adj = ΔMxf+ Mxf= **10.750**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.570

Myf,adj = ΔMyf+ Myf= **8.120**

For Panel 8

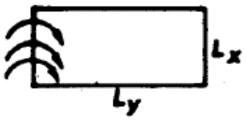
M_{xf} = 8.04 M_{Xs} = 10.72 $M_{Xs,adj}$ = 11.26

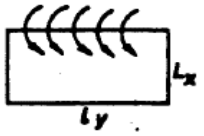
M_{yf} = 6.82 M_{ys} = 9.01 $M_{ys,adj}$ = 8.4

Ly/Lx= 1.27

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

ΔM_{ys} = Mys- M_ys,adj = 0.61 neglect





Cx = 0.329   Cx = 0.364

Cy = 0.146   Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 0.201

Mxf,adj = ΔMxf+ Mxf= **8.241**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.089

Myf,adj = ΔMyf+ Myf= **6.909**

For Panel 9

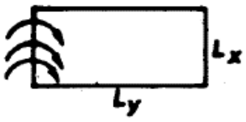
M_{xf} = 8.04 M_{Xs} = 10.72 $M_{Xs,adj}$ = 11.26

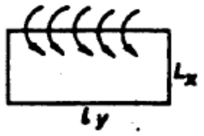
M_{yf} = 6.82 M_{ys} = 9.01 $M_{ys,adj}$ = 8.4

Ly/Lx= 1.27

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

ΔM_{ys} = Mys- M_ys,adj = 0.61 neglect





Cx = 0.329   Cx = 0.364

Cy = 0.146   Cy = 0.354

ΔMxf= CxΔMxs + CxΔMys = 0.201

Mxf,adj = ΔMxf+ Mxf= **8.241**

ΔM_{yf} = CyΔMxs + CyΔMys = 0.089

Myf,adj = ΔMyf+ Myf= **6.909**

Appendix E4: Load transfer to top tie beam

Table 34Load 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	5	9.74	β_{vxc} =	0.36	V_{xc} =	17.532
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.36	V_{yc} =	17.532
					β_{vyd} =	0.24	V_{yd} =	11.688
P-2	Type 2	1.00	5.00	9.74	β_{vxc} =	0.36	V_{xc} =	17.532
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.36	V_{yc} =	17.532
					β_{vyd} =	0.24	V_{yd} =	11.688
P-3	Type 4	1.18	4.25	9.74	β_{vxc} =	0.46	V_{xc} =	19.042
					β_{vxd} =	0.31	V_{xd} =	13
					β_{vyc} =	0.4	V_{yc} =	16.558
					β_{vyd} =	0.24	V_{yd} =	9.935
P-4	Type 3	1.16	5.00	9.74	β_{vxc} =	0.42	V_{xc} =	20.454
					β_{vxd} =	0.28	V_{xd} =	14
					β_{vyc} =	0.36	V_{yc} =	17.532
					β_{vyd} =	0	V_{yd} =	0.000
P-5	Type 1	1.16	5.00	9.74	β_{vxc} =	0.38	V_{xc} =	18.506
					β_{vxd} =	0	V_{xd} =	0.000
					β_{vyc} =	0.33	V_{yc} =	16.071
					β_{vyd} =	0	V_{yd} =	0.000
P-6	Type 3	1.36	4.25	9.74	β_{vxc} =	0.48	V_{xc} =	19.870
					β_{vxd} =	0.32	V_{xd} =	13.246
					β_{vyc} =	0.36	V_{yc} =	14.902
					β_{vyd} =	0	V_{yd} =	0.000
P-7	Type 4	1.08	5.00	9.74	β_{vxc} =	0.43	V_{xc} =	20.941
					β_{vxd} =	0.28	V_{xd} =	14
					β_{vyc} =	0.4	V_{yc} =	19.480
					β_{vyd} =	0.24	V_{yd} =	11.688
P-8	Type 2	1.08	5.00	9.74	β_{vxc} =	0.38	V_{xc} =	18.506
					β_{vxd} =	0	V_{xd} =	0
					β_{vyc} =	0.36	V_{yc} =	17.532
					β_{vyd} =	0.24	V_{yd} =	12

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= 175

k= 2.07 not < 2,so k=2

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

taking minimum reinforcment ∅ 10 c/c 300

$\rho_1 =$ 0.0015

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

$V_{min} =$ 77.476

$v_{Rd,c} =$ 41.867 $\leq V_{min}$ so, $v_{Rd,c} =$ 77.476

maximum shear force transfererd to beams is Vi= Vi=20.45

Vi= 20.45 KN/m

77.476>2045,slab thickness is sufficient to support design load

Table 35roof slab reinforcement

panel	moment	Ast=Msd/Z*f		ø	spacing	spacing used	Reinforcment			
P-1	$M_{xs} =$	9.5	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	7.06	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	9.01	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-2	$M_{xs} =$	10.2	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	7.26	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	8.4	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	6.91	250.00	10	314.00	300	ø	10	c/c	300
P-3	$M_{xs} =$	10.2	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	8.09	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	7.22	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	5.98	250.00	10	314.00	300	ø	10	c/c	300
P-4	$M_{xs} =$	11.45	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	10.27	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	9.99	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	7.34	250.00	10	314.00	300	ø	10	c/c	300
P-5	$M_{xs} =$	10.8	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	7.55	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	8.4	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	5.85	250.00	10	314.00	300	ø	10	c/c	300
P-6	$M_{xs} =$	10.8	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	8.62	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	7.22	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	4.93	250.00	10	314.00	300	ø	10	c/c	300
P-7	$M_{xs} =$	11.94	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	10.75	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	9.99	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	8.12	250.00	10	314.00	300	ø	10	c/c	300
P-8	$M_{xs} =$	11.26	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	8.24	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	8.4	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	6.91	250.00	10	314.00	300	ø	10	c/c	300
P-9	$M_{xs} =$	11.26	250.00	10	314.00	300	ø	10	c/c	300
	$M_{xf} =$	8.24	250.00	10	314.00	300	ø	10	c/c	300
	$M_{ys} =$	7.22	250.00	10	314.00	300	ø	10	c/c	300
	$M_{yf} =$	6.91	250.00	10	314.00	300	ø	10	c/c	300

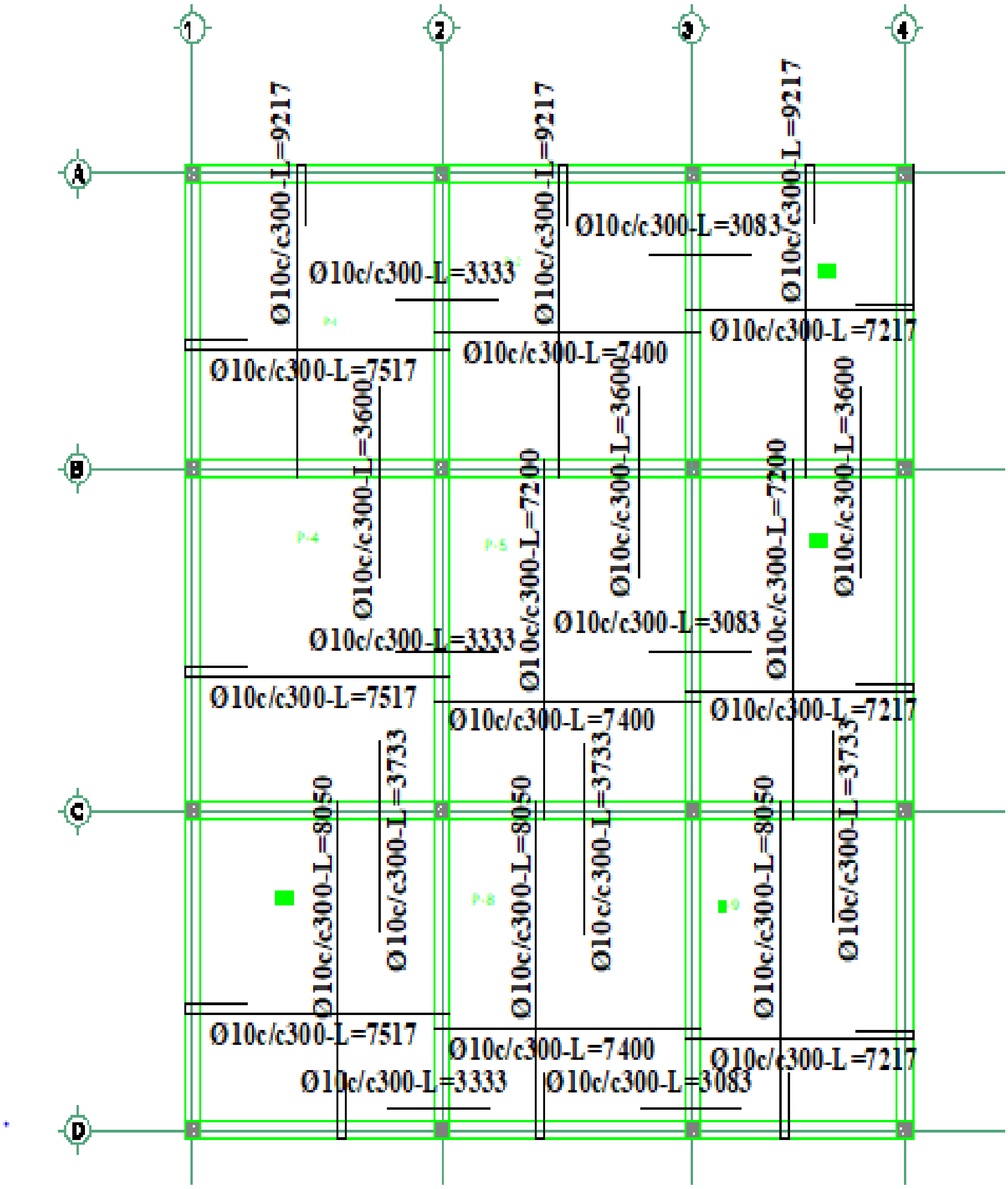


Figure 10-13roof slab reinforcement detailing

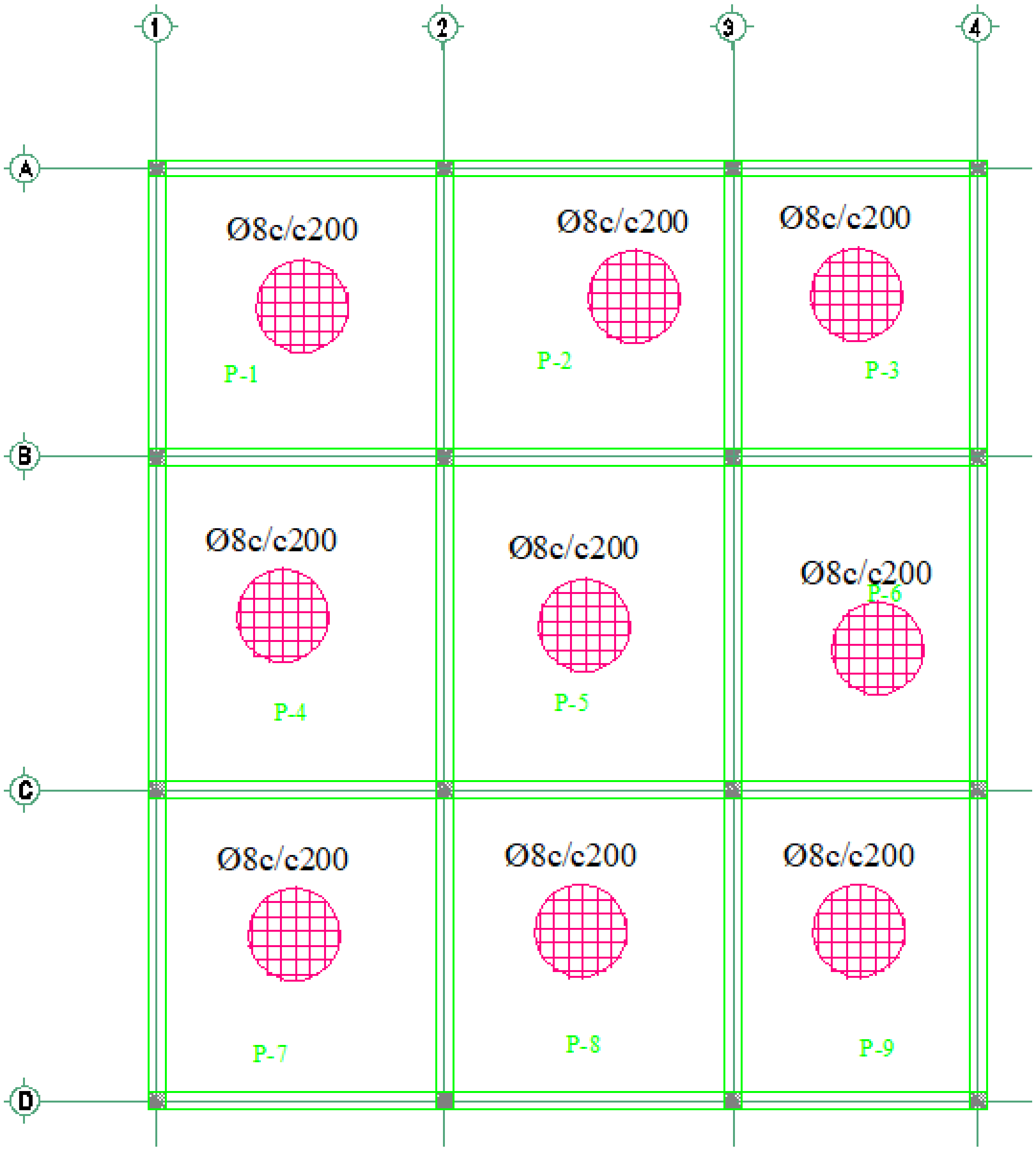


Figure 10-14 basement floor reinforcement

Appendix F Beam design

Appendix F beam longitudinal reinforcement

Table 36beam longitudinal reinforcement

story- 6

Type	Loc	Moment (KNm)	b(m	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	28.36	300	450	0.0233	440.5312	single	193.05	5400	185.0834	193.05	0.61	2Ø20
span	1-2	41.49	300	450	0.0341	436.0036	single	193.05	5400	273.5843	273.5843	0.87	2Ø20
support	2	35.13	300	450	0.0289	438.2087	single	193.05	5400	230.4809	230.4809	0.73	2Ø20
span	2-3	67.71	300	450	0.0557	426.6581	single	193.05	5400	456.2582	456.2582	1.45	2Ø20
support	3	50.75	300	450	0.0418	432.7511	single	193.05	5400	337.1597	337.1597	1.07	2Ø20
span	3-4	30.32	300	450	0.0250	439.8614	single	193.05	5400	198.176	198.176	0.63	2Ø20
support	4	29.01	300	450	0.0239	440.3093	single	193.05	5400	189.4208	193.05	0.61	2Ø20

story- 5

Type	Loc	Moment (KNm)	b(m	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	29.5	300	450	0.0243	440.1419	single	193.05	5400	192.6935	193.05	0.61	2Ø20
span	1-2	62.28	300	450	0.0513	428.6287	single	193.05	5400	417.7392	417.7392	1.33	2Ø20
support	2	60.24	300	450	0.0496	429.3641	single	193.05	5400	403.3639	403.3639	1.28	2Ø20
span	2-3	91.86	300	450	0.0756	417.6497	single	193.05	5400	632.3421	632.3421	2.01	3Ø20
support	3	74.27	300	450	0.0611	424.2514	single	193.05	5400	503.3012	503.3012	1.60	2Ø20
span	3-4	46.22	300	450	0.0380	434.3485	single	193.05	5400	305.9352	305.9352	0.97	2Ø20
support	4	52.85	300	450	0.0435	432.0064	single	193.05	5400	351.7164	351.7164	1.12	2Ø20

story- 4

Type	Loc	Moment (KNm)	b(m	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	24.1	300	450	0.0198	441.9799	single	193.05	5400	156.7662	193.05	0.61	2Ø20
span	1-2	61.11	300	450	0.0503	429.0508	single	193.05	5400	409.4882	409.4882	1.30	2Ø20
support	2	60.46	300	450	0.0498	429.285	single	193.05	5400	404.9117	404.9117	1.29	2Ø20
span	2-3	99.91	300	450	0.0822	414.5519	single	193.05	5400	692.8958	692.8958	2.21	3Ø20
support	3	76.67	300	450	0.0631	423.3636	single	193.05	5400	520.6547	520.6547	1.66	2Ø20
span	3-4	45.17	300	450	0.0372	434.7171	single	193.05	5400	298.7317	298.7317	0.95	2Ø20
support	4	40.72	300	450	0.0335	436.2718	single	193.05	5400	268.3419	268.3419	0.85	2Ø20

story- 3

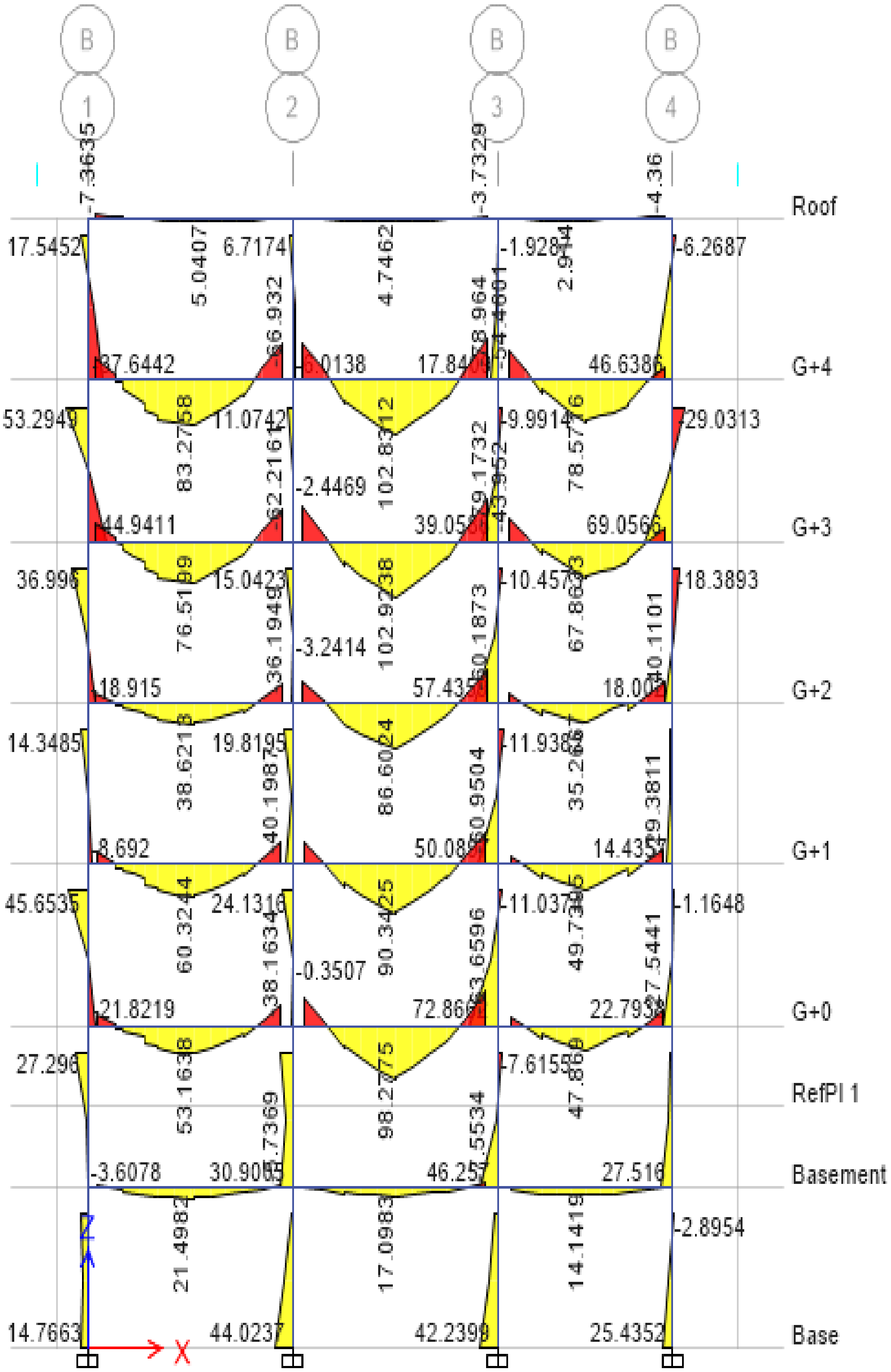
Type	Loc	Moment (KNm)	b(m	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	23.2	300	450	0.0191	442.2848	single	193.05	5400	150.8078	193.05	0.61	2Ø20
span	1-2	60.02	300	450	0.0494	429.4433	single	193.05	5400	401.8167	401.8167	1.28	2Ø20
support	2	61.66	300	450	0.0507	428.8525	single	193.05	5400	413.3647	413.3647	1.32	2Ø20
span	2-3	98.56	300	450	0.0811	415.0749	single	193.05	5400	682.672	682.672	2.17	3Ø20
support	3	75.27	300	450	0.0620	423.882	single	193.05	5400	510.5224	510.5224	1.63	2Ø20
span	3-4	45.43	300	450	0.0374	434.6259	single	193.05	5400	300.5142	300.5142	0.96	2Ø20
support	4	36.88	300	450	0.0304	437.6042	single	193.05	5400	242.2966	242.2966	0.77	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	8.16	300	450	0.0067	447.3169	single	193.05	5400	52.44604	193.05	0.61	2Ø20
span	1-2	18.99	300	450	0.0156	443.705	single	193.05	5400	123.0463	193.05	0.61	2Ø20
support	2	3.8	300	450	0.0031	448.7545	single	193.05	5400	24.34516	193.05	0.61	2Ø20
span	2-3	18.17	300	450	0.0150	443.9806	single	193.05	5400	117.66	193.05	0.61	2Ø20
support	3	11.52	300	450	0.0095	446.2026	single	193.05	5400	74.22637	193.05	0.61	2Ø20
span	3-4	14.98	300	450	0.0123	445.0493	single	193.05	5400	96.77018	193.05	0.61	2Ø20
support	4	4.57	300	450	0.0038	448.5013	single	193.05	5400	29.29479	193.05	0.61	2Ø20

story- 8

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	7.99	300	250	0.0213	245.2073	single	107.25	3000	93.68092	107.25	0.34	2Ø20
span	1-2	6.17	300	250	0.0165	246.3157	single	107.25	3000	72.01632	107.25	0.34	2Ø20
support	2	1.19	300	250	0.0032	249.2979	single	107.25	3000	13.72354	107.25	0.34	2Ø20
span	2-3	3.86	300	250	0.0103	247.708	single	107.25	3000	44.80073	107.25	0.34	2Ø20
support	3	2.944	300	250	0.0079	248.2558	single	107.25	3000	34.09387	107.25	0.34	2Ø20
span	3-4	5.75	300	250	0.0153	246.57	single	107.25	3000	67.04485	107.25	0.34	2Ø20
support	4	6.89	300	250	0.0184	245.8784	single	107.25	3000	80.56319	107.25	0.34	2Ø20


B
1
B
2
B
3
B
4
-7.3635
-3.7329
-4.36
Roof
17.5452
5.0407
6.7174
66.932
4.7462
-1.9284
2.914
58.964
34.4801
-6.2687
37.6442
6.0138
17.8409
46.6386
G+4
53.2949
83.2758
11.0742
62.2161
-2.4469
102.8312
79.1732
43.952
-9.9914
78.5776
29.0313
44.9411
39.0569
69.0566
G+3
36.996
76.5199
15.0423
36.1949
-3.2414
102.9238
60.1873
-10.4573
67.8633
40.1101
18.3893
18.915
57.4356
18.009
G+2
14.3485
38.6218
19.8195
40.1987
86.6024
60.9504
-11.9382
35.2667
29.3811
8.692
50.0854
14.4357
G+1
45.6535
60.3244
24.1316
38.1634
-0.3507
90.3425
83.6596
-11.0378
49.7385
27.5441
-1.1648
21.8219
72.8684
22.7934
G+0
27.296
53.1638
7.7369
98.2775
5534
-7.6155
47.869
RefPl 1
-3.6078
30.9065
46.257
27.516
Basement
21.4982
17.0983
14.1419
-2.8954
Z
14.7663
44.0237
42.2399
25.4352
X
Base

Axis B

story- 7

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	40.01	300	450	0.0329	436.5188	single	193.05	5400	263.5139	263.5139	0.84	2Ø20
span	1-2	83.78	300	450	0.0690	420.7099	single	193.05	5400	572.5264	572.5264	1.82	2Ø20
support	2	66.92	300	450	0.0551	426.946	single	193.05	5400	450.6308	450.6308	1.44	2Ø20
span	2-3	102.83	300	450	0.0846	413.4156	single	193.05	5400	715.1067	715.1067	2.28	3Ø20
support	3	78.96	300	450	0.0650	422.5128	single	193.05	5400	537.2855	537.2855	1.71	2Ø20
span	3-4	78.57	300	450	0.0647	422.6579	single	193.05	5400	534.4481	534.4481	1.70	2Ø20
support	4	25.67	300	450	0.0211	441.4472	single	193.05	5400	167.1803	193.05	0.61	2Ø20

story- 6

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	34.47	300	450	0.0284	438.4363	single	193.05	5400	226.0334	226.0334	0.72	2Ø20
span	1-2	76.52	300	450	0.0630	423.4192	single	193.05	5400	519.5678	519.5678	1.65	2Ø20
support	2	63.97	300	450	0.0527	428.0174	single	193.05	5400	429.6875	429.6875	1.37	2Ø20
span	2-3	102.98	300	450	0.0848	413.357	single	193.05	5400	716.2513	716.2513	2.28	3Ø20
support	3	79.17	300	450	0.0652	422.4346	single	193.05	5400	538.8142	538.8142	1.72	2Ø20
span	3-4	67.86	300	450	0.0559	426.6034	single	193.05	5400	457.3276	457.3276	1.46	2Ø20
support	4	23.53	300	450	0.0194	442.173	single	193.05	5400	152.9916	193.05	0.61	2Ø20

story- 5

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	19.82	300	450	0.0163	443.4258	single	193.05	5400	128.5052	193.05	0.61	2Ø20
span	1-2	38.62	300	450	0.0318	437.0015	single	193.05	5400	254.0781	254.0781	0.81	2Ø20
support	2	36.19	300	450	0.0298	437.8428	single	193.05	5400	237.6338	237.6338	0.76	2Ø20
span	2-3	86.6	300	450	0.0713	419.6473	single	193.05	5400	593.2958	593.2958	1.89	2Ø20
support	3	60.19	300	450	0.0495	429.3821	single	193.05	5400	403.0122	403.0122	1.28	2Ø20
span	3-4	35.27	300	450	0.0290	438.1604	single	193.05	5400	231.4249	231.4249	0.74	2Ø20
support	4	40.1	300	450	0.0330	436.4875	single	193.05	5400	264.1256	264.1256	0.84	2Ø20

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	22.7	300	450	0.0187	442.4539	single	193.05	5400	147.5012	193.05	0.61	2Ø20
span	1-2	60.32	300	450	0.0496	429.3353	single	193.05	5400	403.9267	403.9267	1.29	2Ø20
support	2	40.19	300	450	0.0331	436.4562	single	193.05	5400	264.7373	264.7373	0.84	2Ø20
span	2-3	90.34	300	450	0.0744	418.2291	single	193.05	5400	621.0173	621.0173	1.98	2Ø20
support	3	60.95	300	450	0.0502	429.1085	single	193.05	5400	408.3612	408.3612	1.30	2Ø20
span	3-4	49.74	300	450	0.0409	433.1083	single	193.05	5400	330.1772	330.1772	1.05	2Ø20
support	4	29.38	300	450	0.0242	440.1829	single	193.05	5400	191.8918	193.05	0.61	2Ø20

story- 3

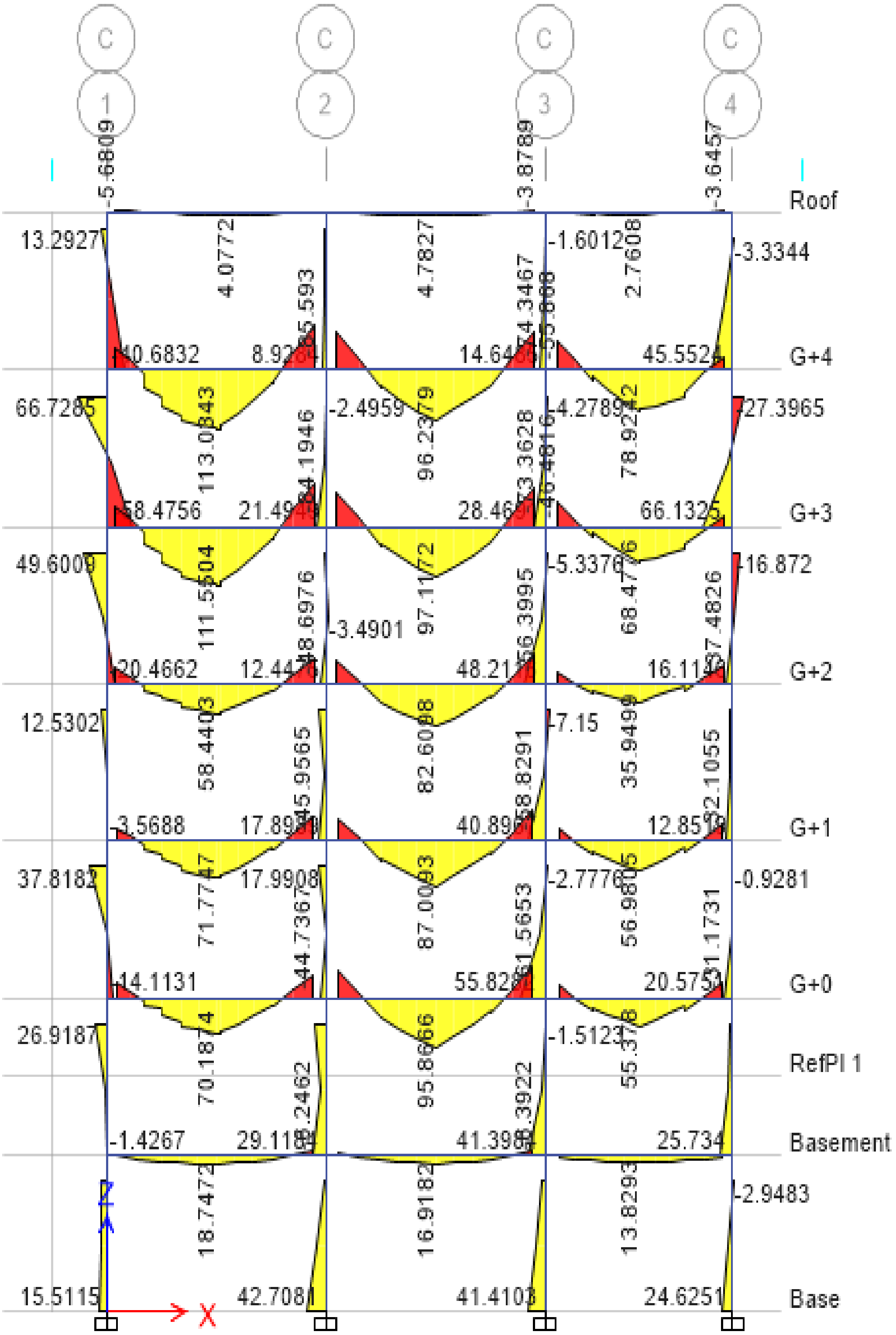
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	22.52	300	450	0.0185	442.5148	single	193.05	5400	146.3115	193.05	0.61	2Ø20
span	1-2	53.16	300	450	0.0438	431.8962	single	193.05	5400	353.8697	353.8697	1.13	2Ø20
support	2	49.67	300	450	0.0409	433.1331	single	193.05	5400	329.6937	329.6937	1.05	2Ø20
span	2-3	98.27	300	450	0.0809	415.1871	single	193.05	5400	680.4794	680.4794	2.17	3Ø20
support	3	63.66	300	450	0.0524	428.1297	single	193.05	5400	427.4931	427.4931	1.36	2Ø20
span	3-4	47.87	300	450	0.0394	433.7681	single	193.05	5400	317.2807	317.2807	1.01	2Ø20
support	4	27.54	300	450	0.0227	440.8109	single	193.05	5400	179.6179	193.05	0.61	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	4.59	300	450	0.0038	448.4947	single	193.05	5400	29.42342	193.05	0.61	2Ø20
span	1-2	21.49	300	450	0.0177	442.8628	single	193.05	5400	139.5099	193.05	0.61	2Ø20
support	2	6.74	300	450	0.0055	447.7861	single	193.05	5400	43.27401	193.05	0.61	2Ø20
span	2-3	17.1	300	450	0.0141	444.3396	single	193.05	5400	110.6417	193.05	0.61	2Ø20
support	3	7.55	300	450	0.0062	447.5186	single	193.05	5400	48.50357	193.05	0.61	2Ø20
span	3-4	14.14	300	450	0.0116	445.3298	single	193.05	5400	91.28627	193.05	0.61	2Ø20
support	4	1.4	300	450	0.0012	449.5419	single	193.05	5400	8.953558	193.05	0.61	2Ø20

story- 8

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	7.36	300	250	0.0196	245.5922	single	107.25	3000	86.15911	107.25	0.34	2Ø20
span	1-2	5.04	300	250	0.0134	246.9988	single	107.25	3000	58.66426	107.25	0.34	2Ø20
support	2	2.29	300	250	0.0061	248.6454	single	107.25	3000	26.47847	107.25	0.34	2Ø20
span	2-3	4.75	300	250	0.0127	247.1735	single	107.25	3000	55.24966	107.25	0.34	2Ø20
support	3	3.73	300	250	0.0099	247.7859	single	107.25	3000	43.27829	107.25	0.34	2Ø20
span	3-4	2.91	300	250	0.0078	248.2761	single	107.25	3000	33.69737	107.25	0.34	2Ø20
support	4	4.36	300	250	0.0116	247.408	single	107.25	3000	50.6653	107.25	0.34	2Ø20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Base
X

Axis C

story 7

Type	Loc	Moment	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,ma	As,cal	As,pro	No of	Remar
suppor	1	43.13	300	450	0.0355	435.4	single	193.1	5400	284.8	284.8	0.91	2Ø20
span	1-2	113	300	450	0.0930	409.4	single	193.1	5400	793.8	793.8	2.53	3Ø20
suppor	2	85.59	300	450	0.0704	420	single	193.1	5400	585.8	585.8	1.87	2Ø20
span	2-3	96.24	300	450	0.0792	416	single	193.1	5400	665.2	665.2	2.12	3Ø20
suppor	3	74.35	300	450	0.0612	424.2	single	193.1	5400	503.9	503.9	1.60	2Ø20
span	3-4	78.92	300	450	0.0650	422.5	single	193.1	5400	537	537	1.71	2Ø20
suppor	4	24.34	300	450	0.0200	441.9	single	193.1	5400	158.4	193.1	0.61	2Ø20

story-6

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of ba	Remark
support	1	40.24	300	450	0.0331	436.439	single	193.05	5400	265.077	265.077	0.84	2Ø20
span	1-2	111.55	300	450	0.0918	409.981	single	193.05	5400	782.247	782.247	2.49	3Ø20
support	2	84.19	300	450	0.0693	420.556	single	193.05	5400	575.539	575.539	1.83	2Ø20
span	2-3	97.12	300	450	0.0799	415.631	single	193.05	5400	671.797	671.797	2.14	3Ø20
support	3	73.36	300	450	0.0604	424.587	single	193.05	5400	496.742	496.742	1.58	2Ø20
span	3-4	68.48	300	450	0.0564	426.377	single	193.05	5400	461.751	461.751	1.47	2Ø20
support	4	21.22	300	450	0.0175	442.954	single	193.05	5400	137.729	193.05	0.61	2Ø20

story-5

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of b	Remark
support	1	29.84	300	450	0.0246	440.03	single	193.05	5400	194.97	194.97	0.62	2Ø20
span	1-2	58.44	300	450	0.0481	430.01	single	193.05	5400	390.72	390.72	1.24	2Ø20
support	2	48.69	300	450	0.0401	433.48	single	193.05	5400	322.93	322.93	1.03	2Ø20
span	2-3	82.61	300	450	0.0680	421.15	single	193.05	5400	563.94	563.94	1.80	2Ø20
support	3	56.39	300	450	0.0464	430.74	single	193.05	5400	376.37	376.37	1.20	2Ø20
span	3-4	35.95	300	450	0.0296	437.93	single	193.05	5400	236.01	236.01	0.75	2Ø20
support	4	37.48	300	450	0.0308	437.4	single	193.05	5400	246.36	246.36	0.78	2Ø20

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	25.52	300	450	0.0210	441.4981	single	193.05	5400	166.1842	193.05	0.61	2Ø20
span	1-2	71.77	300	450	0.0591	425.172	single	193.05	5400	485.3065	485.3065	1.55	2Ø20
support	2	45.95	300	450	0.0378	434.4434	single	193.05	5400	304.0816	304.0816	0.97	2Ø20
span	2-3	87.01	300	450	0.0716	419.4924	single	193.05	5400	596.3249	596.3249	1.90	2Ø20
support	3	58.83	300	450	0.0484	429.8709	single	193.05	5400	393.4582	393.4582	1.25	2Ø20
span	3-4	56.98	300	450	0.0469	430.5339	single	193.05	5400	380.4985	380.4985	1.21	2Ø20
support	4	32.1	300	450	0.0264	439.2513	single	193.05	5400	210.1018	210.1018	0.67	2Ø20

story-3

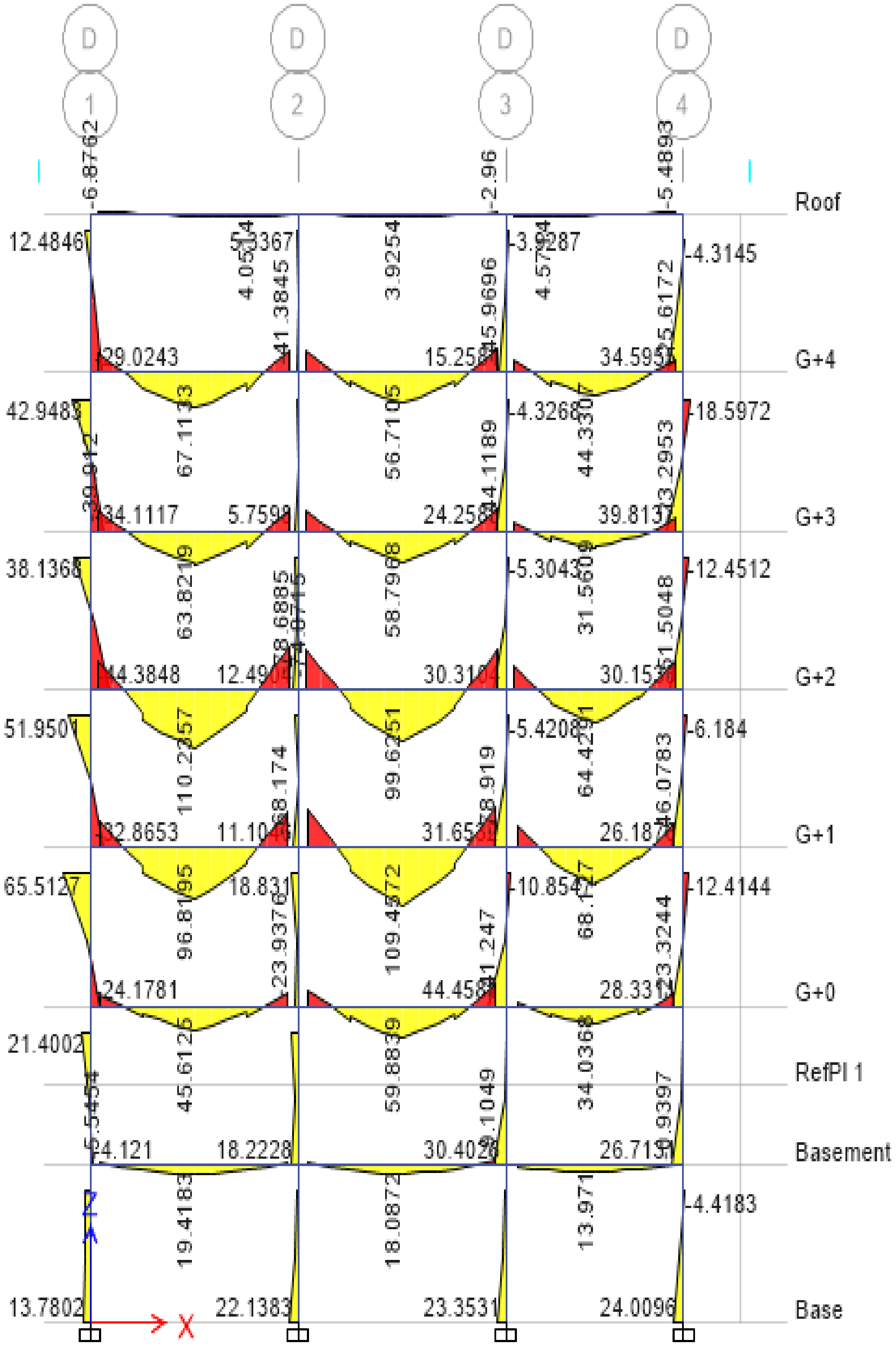
Type	Loc	Moment	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of b	Remark
support	1	27.21	300	450	0.0224	440.92	single	193.05	5400	177.42	193.05	0.61	2Ø20
span	1-2	70.19	300	450	0.0578	425.75	single	193.05	5400	473.98	473.98	1.51	2Ø20
support	2	44.74	300	450	0.0368	434.87	single	193.05	5400	295.79	295.79	0.94	2Ø20
span	2-3	95.87	300	450	0.0789	416.11	single	193.05	5400	662.38	662.38	2.11	3Ø20
support	3	61.57	300	450	0.0507	428.88	single	193.05	5400	412.73	412.73	1.31	2Ø20
span	3-4	55.38	300	450	0.0456	431.11	single	193.05	5400	369.32	369.32	1.18	2Ø20
support	4	31.17	300	450	0.0257	439.57	single	193.05	5400	203.87	203.87	0.65	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 b	Remark
support	1	0.83	300	450	0.0007	449.73	single	193.05	5400	5.306	193.05	0.61	2Ø20
span	1-2	18.75	300	450	0.0154	443.79	single	193.05	5400	121.47	193.05	0.61	2Ø20
support	2	6.25	300	450	0.0051	447.95	single	193.05	5400	40.113	193.05	0.61	2Ø20
span	2-3	16.91	300	450	0.0139	444.4	single	193.05	5400	109.4	193.05	0.61	2Ø20
support	3	6.39	300	450	0.0053	447.9	single	193.05	5400	41.016	193.05	0.61	2Ø20
span	3-4	13.82	300	450	0.0114	445.44	single	193.05	5400	89.199	193.05	0.61	2Ø20
support	4	3.58	300	450	0.0029	448.83	single	193.05	5400	22.932	193.05	0.61	2Ø20

story-8

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of b	Remark
support	1	5.68	300	250	0.0151	246.61	single	107.25	3000	66.217	107.25	0.34	2Ø20
span	1-2	4.08	300	250	0.0109	247.58	single	107.25	3000	47.379	107.25	0.34	2Ø20
support	2	2.31	300	250	0.0062	248.63	single	107.25	3000	26.711	107.25	0.34	2Ø20
span	2-3	4.78	300	250	0.0127	247.16	single	107.25	3000	55.603	107.25	0.34	2Ø20
support	3	3.88	300	250	0.0103	247.7	single	107.25	3000	45.035	107.25	0.34	2Ø20
span	3-4	2.76	300	250	0.0074	248.37	single	107.25	3000	31.949	107.25	0.34	2Ø20
support	4	3.65	300	250	0.0097	247.83	single	107.25	3000	42.342	107.25	0.34	2Ø20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Base
Z
X

Axis D

story-7

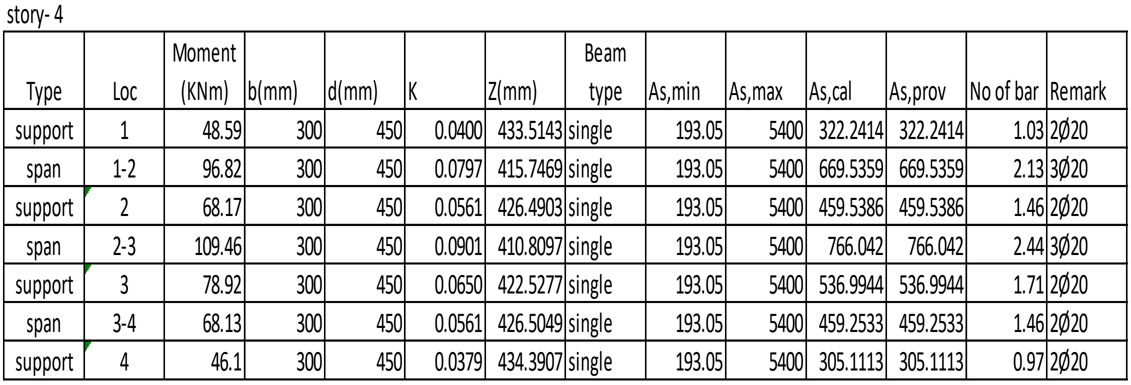
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	38.56	300	450	0.0317	437.022	single	193.05	5400	253.671	253.671	0.81	2Ø20
span	1-2	67.11	300	450	0.0552	426.877	single	193.05	5400	451.983	451.983	1.44	2Ø20
support	2	41.38	300	450	0.0341	436.042	single	193.05	5400	272.835	272.835	0.87	2Ø20
span	2-3	56.71	300	450	0.0467	430.631	single	193.05	5400	378.611	378.611	1.21	2Ø20
support	3	45.97	300	450	0.0378	434.436	single	193.05	5400	304.219	304.219	0.97	2Ø20
span	3-4	44.33	300	450	0.0365	435.011	single	193.05	5400	292.978	292.978	0.93	2Ø20
support	4	25.62	300	450	0.0211	441.464	single	193.05	5400	166.848	193.05	0.61	2Ø20

story- 6

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of ba	Remark
support	1	39.91	300	450	0.0328	436.554	single	193.05	5400	262.834	262.834	0.84	2Ø20
span	1-2	63.82	300	450	0.0525	428.072	single	193.05	5400	428.626	428.626	1.37	2Ø20
support	2	39.4	300	450	0.0324	436.731	single	193.05	5400	259.37	259.37	0.83	2Ø20
span	2-3	58.8	300	450	0.0484	429.882	single	193.05	5400	393.248	393.248	1.25	2Ø20
support	3	44.12	300	450	0.0363	435.085	single	193.05	5400	291.541	291.541	0.93	2Ø20
span	3-4	31.56	300	450	0.0260	439.437	single	193.05	5400	206.48	206.48	0.66	2Ø20
support	4	23.19	300	450	0.0191	442.288	single	193.05	5400	150.742	193.05	0.61	2Ø20

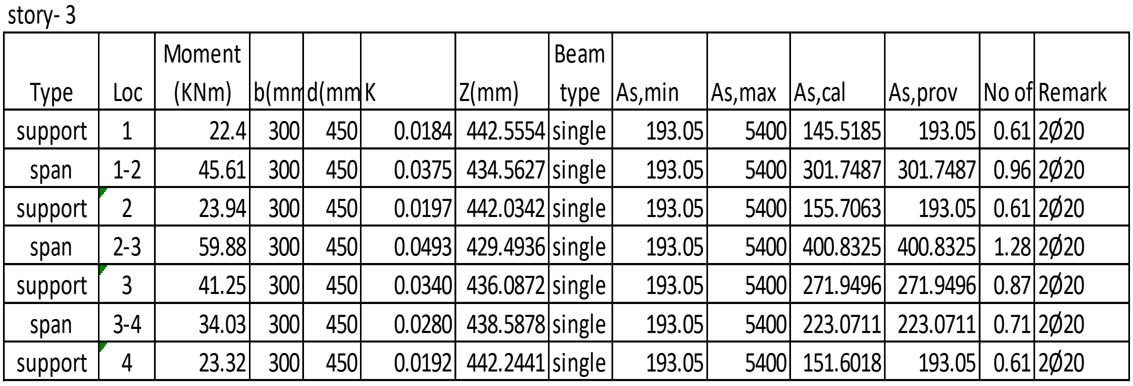
story- 5

Type	Loc	Moment (KNm)	b(mn	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	54.55	300	450	0.0449	431.4016	single	193.05	5400	363.5389	363.5389	1.16	2Ø20
span	1-2	110.24	300	450	0.0907	410.5008	single	193.05	5400	772.0814	772.0814	2.46	3Ø20
support	2	78.69	300	450	0.0648	422.6133	single	193.05	5400	535.321	535.321	1.70	2Ø20
span	2-3	99.62	300	450	0.0820	414.6643	single	193.05	5400	690.6972	690.6972	2.20	3Ø20
support	3	74.68	300	450	0.0615	424.1	single	193.05	5400	506.2603	506.2603	1.61	2Ø20
span	3-4	64.43	300	450	0.0530	427.8507	single	193.05	5400	432.946	432.946	1.38	2Ø20
support	4	51.5	300	450	0.0424	432.4854	single	193.05	5400	342.3526	342.3526	1.09	2Ø20



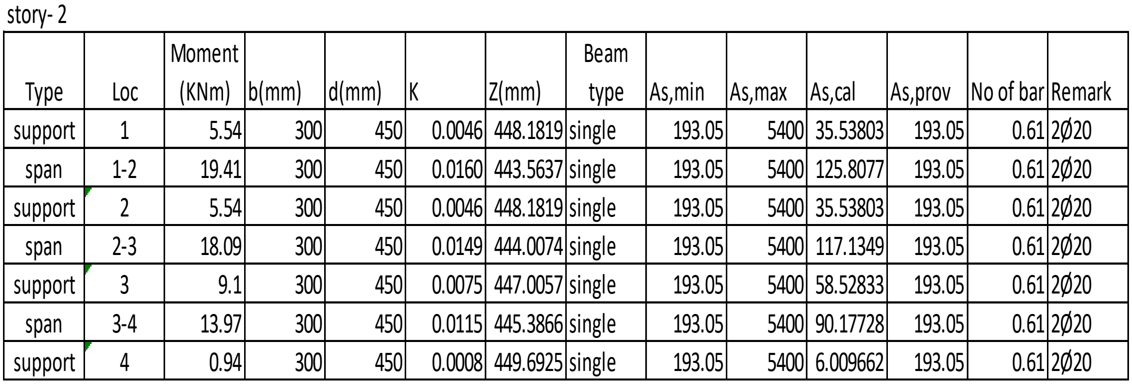
story- 4

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	48.59	300	450	0.0400	433.5143	single	193.05	5400	322.2414	322.2414	1.03	2Ø20
span	1-2	96.82	300	450	0.0797	415.7469	single	193.05	5400	669.5359	669.5359	2.13	3Ø20
support	2	68.17	300	450	0.0561	426.4903	single	193.05	5400	459.5386	459.5386	1.46	2Ø20
span	2-3	109.46	300	450	0.0901	410.8097	single	193.05	5400	766.042	766.042	2.44	3Ø20
support	3	78.92	300	450	0.0650	422.5277	single	193.05	5400	536.9944	536.9944	1.71	2Ø20
span	3-4	68.13	300	450	0.0561	426.5049	single	193.05	5400	459.2533	459.2533	1.46	2Ø20
support	4	46.1	300	450	0.0379	434.3907	single	193.05	5400	305.1113	305.1113	0.97	2Ø20



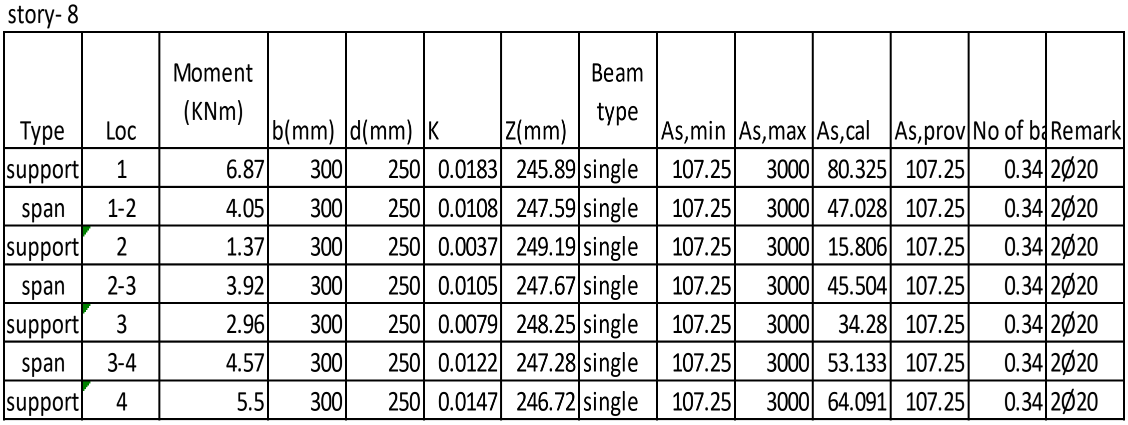
story- 3

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	22.4	300	450	0.0184	442.5554	single	193.05	5400	145.5185	193.05	0.61	2Ø20
span	1-2	45.61	300	450	0.0375	434.5627	single	193.05	5400	301.7487	301.7487	0.96	2Ø20
support	2	23.94	300	450	0.0197	442.0342	single	193.05	5400	155.7063	193.05	0.61	2Ø20
span	2-3	59.88	300	450	0.0493	429.4936	single	193.05	5400	400.8325	400.8325	1.28	2Ø20
support	3	41.25	300	450	0.0340	436.0872	single	193.05	5400	271.9496	271.9496	0.87	2Ø20
span	3-4	34.03	300	450	0.0280	438.5878	single	193.05	5400	223.0711	223.0711	0.71	2Ø20
support	4	23.32	300	450	0.0192	442.2441	single	193.05	5400	151.6018	193.05	0.61	2Ø20



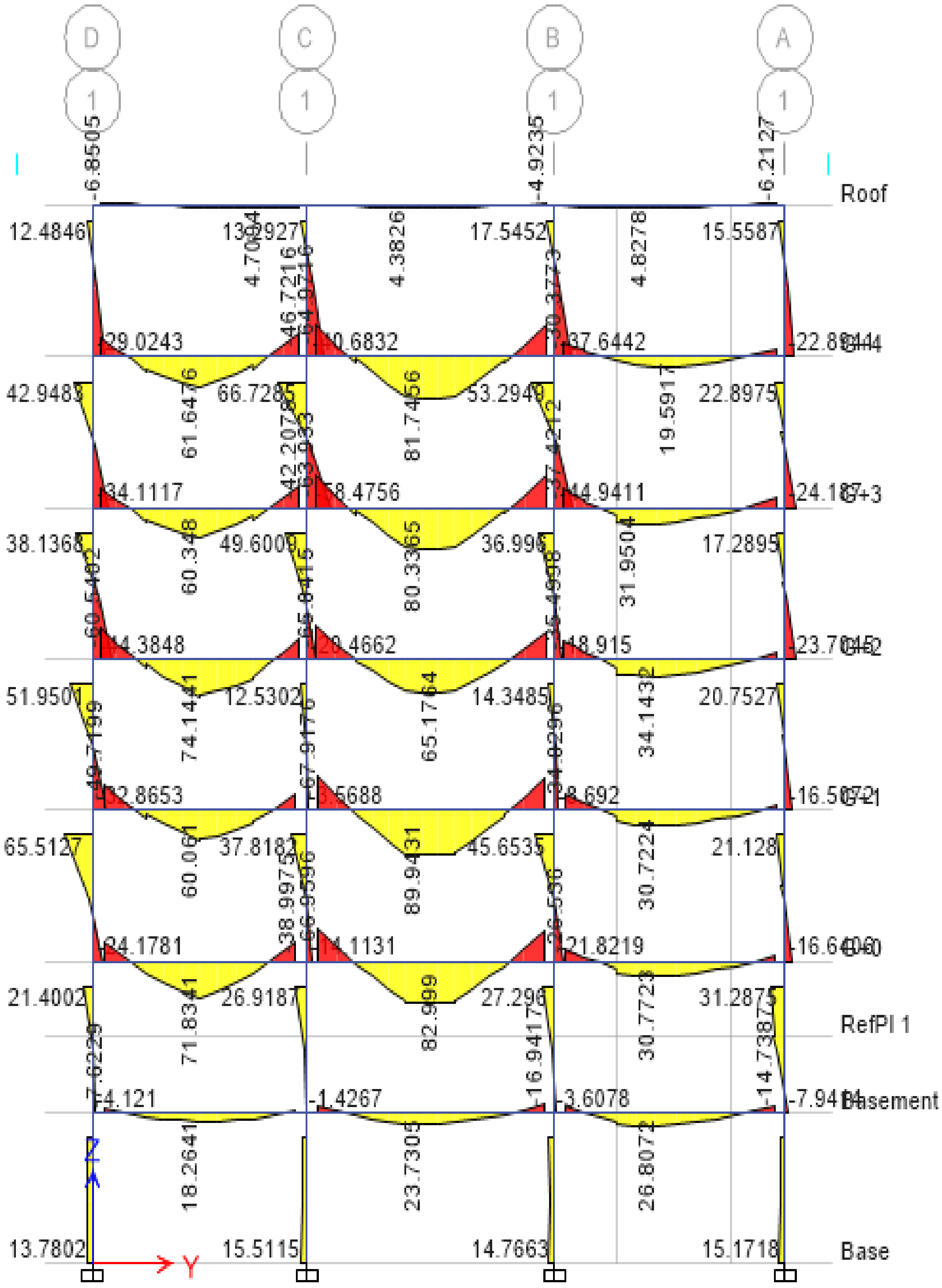
story- 2

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	5.54	300	450	0.0046	448.1819	single	193.05	5400	35.53803	193.05	0.61	2Ø20
span	1-2	19.41	300	450	0.0160	443.5637	single	193.05	5400	125.8077	193.05	0.61	2Ø20
support	2	5.54	300	450	0.0046	448.1819	single	193.05	5400	35.53803	193.05	0.61	2Ø20
span	2-3	18.09	300	450	0.0149	444.0074	single	193.05	5400	117.1349	193.05	0.61	2Ø20
support	3	9.1	300	450	0.0075	447.0057	single	193.05	5400	58.52833	193.05	0.61	2Ø20
span	3-4	13.97	300	450	0.0115	445.3866	single	193.05	5400	90.17728	193.05	0.61	2Ø20
support	4	0.94	300	450	0.0008	449.6925	single	193.05	5400	6.009662	193.05	0.61	2Ø20



story- 8

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of ba	Remark
support	1	6.87	300	250	0.0183	245.89	single	107.25	3000	80.325	107.25	0.34	2Ø20
span	1-2	4.05	300	250	0.0108	247.59	single	107.25	3000	47.028	107.25	0.34	2Ø20
support	2	1.37	300	250	0.0037	249.19	single	107.25	3000	15.806	107.25	0.34	2Ø20
span	2-3	3.92	300	250	0.0105	247.67	single	107.25	3000	45.504	107.25	0.34	2Ø20
support	3	2.96	300	250	0.0079	248.25	single	107.25	3000	34.28	107.25	0.34	2Ø20
span	3-4	4.57	300	250	0.0122	247.28	single	107.25	3000	53.133	107.25	0.34	2Ø20
support	4	5.5	300	250	0.0147	246.72	single	107.25	3000	64.091	107.25	0.34	2Ø20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Basement
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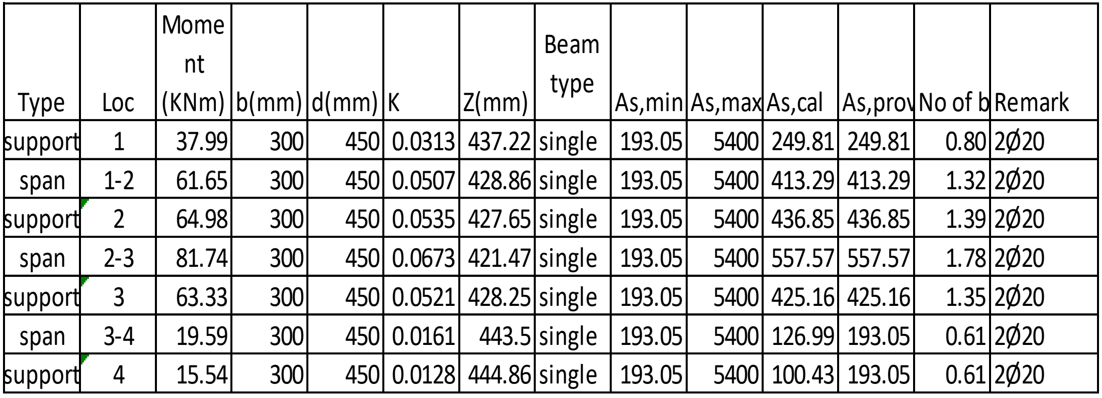
14.7663
15.1718
Base
Z
Y

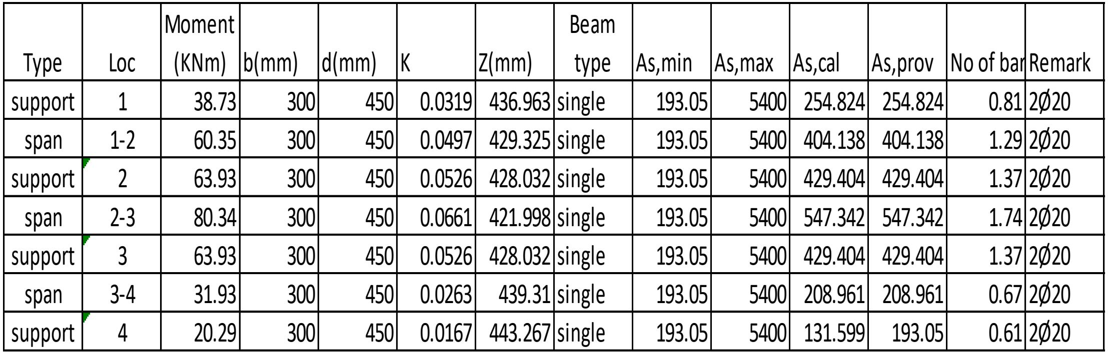
Axis 1

story- 7



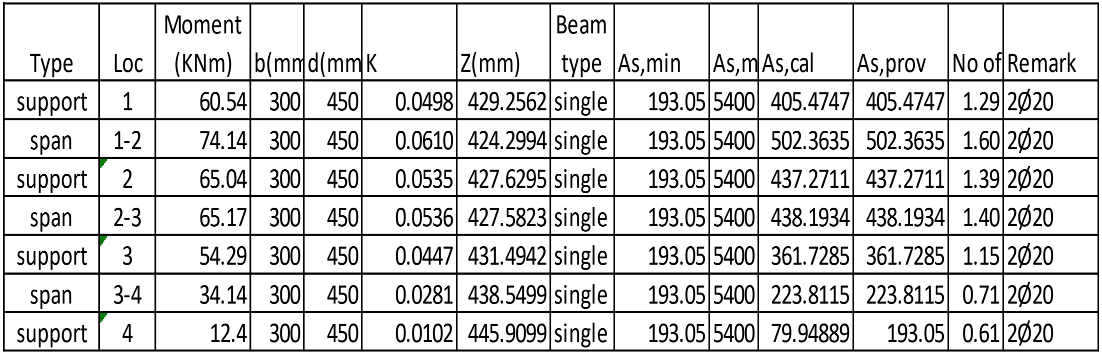
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of b	Remark
support	1	37.99	300	450	0.0313	437.22	single	193.05	5400	249.81	249.81	0.80	2Ø20
span	1-2	61.65	300	450	0.0507	428.86	single	193.05	5400	413.29	413.29	1.32	2Ø20
support	2	64.98	300	450	0.0535	427.65	single	193.05	5400	436.85	436.85	1.39	2Ø20
span	2-3	81.74	300	450	0.0673	421.47	single	193.05	5400	557.57	557.57	1.78	2Ø20
support	3	63.33	300	450	0.0521	428.25	single	193.05	5400	425.16	425.16	1.35	2Ø20
span	3-4	19.59	300	450	0.0161	443.5	single	193.05	5400	126.99	193.05	0.61	2Ø20
support	4	15.54	300	450	0.0128	444.86	single	193.05	5400	100.43	193.05	0.61	2Ø20

story- 6



Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	1	38.73	300	450	0.0319	436.963	single	193.05	5400	254.824	254.824	0.81	2Ø20
span	1-2	60.35	300	450	0.0497	429.325	single	193.05	5400	404.138	404.138	1.29	2Ø20
support	2	63.93	300	450	0.0526	428.032	single	193.05	5400	429.404	429.404	1.37	2Ø20
span	2-3	80.34	300	450	0.0661	421.998	single	193.05	5400	547.342	547.342	1.74	2Ø20
support	3	63.93	300	450	0.0526	428.032	single	193.05	5400	429.404	429.404	1.37	2Ø20
span	3-4	31.93	300	450	0.0263	439.31	single	193.05	5400	208.961	208.961	0.67	2Ø20
support	4	20.29	300	450	0.0167	443.267	single	193.05	5400	131.599	193.05	0.61	2Ø20

story- 5



Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	60.54	300	450	0.0498	429.2562	single	193.05	5400	405.4747	405.4747	1.29	2Ø20
span	1-2	74.14	300	450	0.0610	424.2994	single	193.05	5400	502.3635	502.3635	1.60	2Ø20
support	2	65.04	300	450	0.0535	427.6295	single	193.05	5400	437.2711	437.2711	1.39	2Ø20
span	2-3	65.17	300	450	0.0536	427.5823	single	193.05	5400	438.1934	438.1934	1.40	2Ø20
support	3	54.29	300	450	0.0447	431.4942	single	193.05	5400	361.7285	361.7285	1.15	2Ø20
span	3-4	34.14	300	450	0.0281	438.5499	single	193.05	5400	223.8115	223.8115	0.71	2Ø20
support	4	12.4	300	450	0.0102	445.9099	single	193.05	5400	79.94889	193.05	0.61	2Ø20

story- 4

Type	Loc	Moment (KNm)	b(m	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	49.72	300	450	0.0409	433.1154	single	193.05	5400	330.0391	330.0391	1.05	2Ø20
span	1-2	60.1	300	450	0.0495	429.4145	single	193.05	5400	402.3793	402.3793	1.28	2Ø20
support	2	69.92	300	450	0.0575	425.8505	single	193.05	5400	472.0436	472.0436	1.50	2Ø20
span	2-3	89.94	300	450	0.0740	418.3813	single	193.05	5400	618.0427	618.0427	1.97	2Ø20
support	3	61.37	300	450	0.0505	428.9571	single	193.05	5400	411.3203	411.3203	1.31	2Ø20
span	3-4	30.77	300	450	0.0253	439.7073	single	193.05	5400	201.1878	201.1878	0.64	2Ø20
support	4	9.23	300	450	0.0076	446.9627	single	193.05	5400	59.37017	193.05	0.61	2Ø20

story- 3

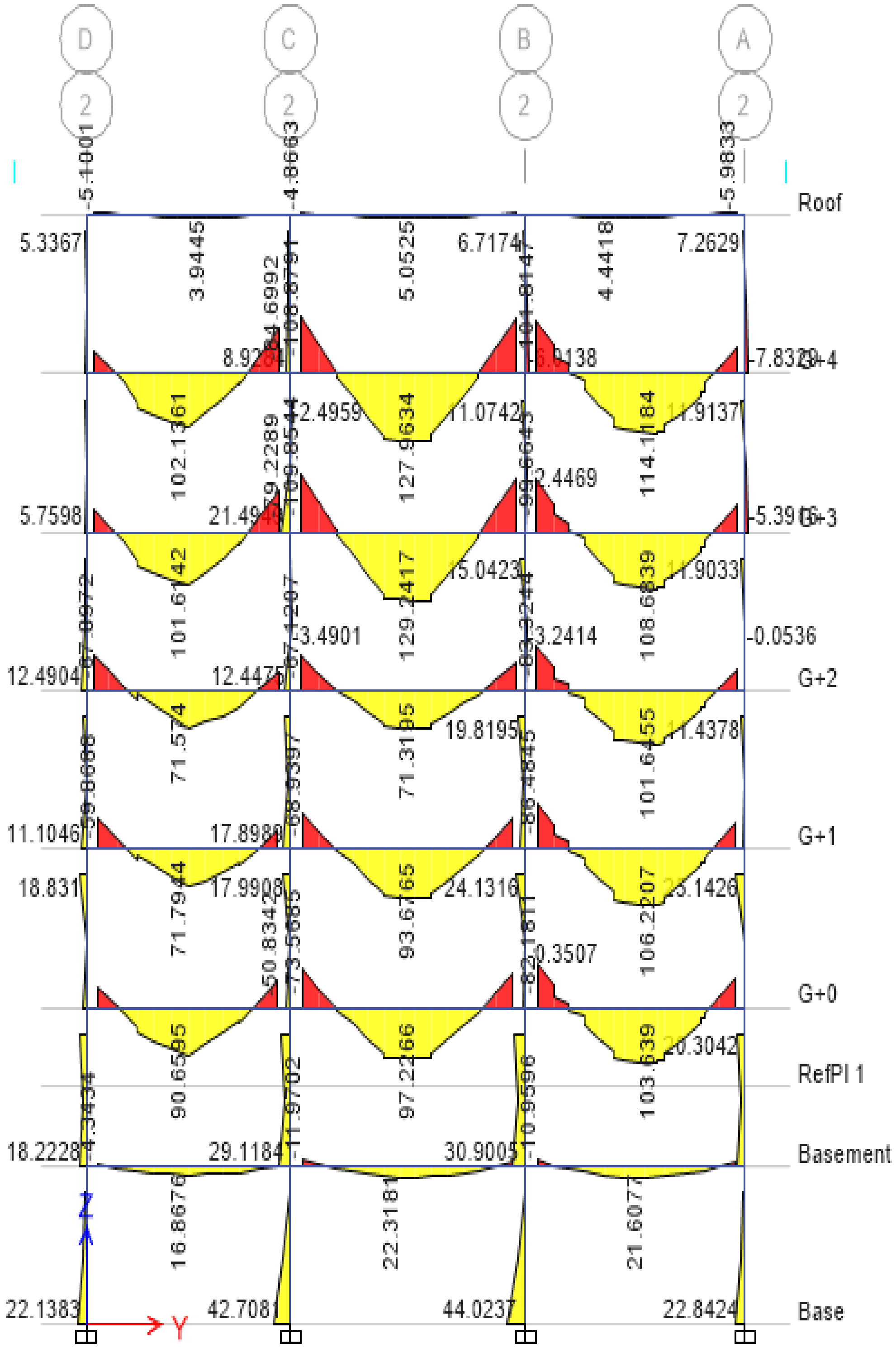
Type	Loc	Moment (KNm)	b(m	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	36.4	300	450	0.0300	437.7702	single	193.05	5400	239.0524	239.0524	0.76	2Ø20
span	1-2	71.83	300	450	0.0591	425.15	single	193.05	5400	485.7374	485.7374	1.55	2Ø20
support	2	66.95	300	450	0.0551	426.9351	single	193.05	5400	450.8443	450.8443	1.44	2Ø20
span	2-3	82.9	300	450	0.0682	421.0403	single	193.05	5400	566.0682	566.0682	1.80	2Ø20
support	3	26.5	300	450	0.0218	441.165	single	193.05	5400	172.6962	193.05	0.61	2Ø20
span	3-4	30.8	300	450	0.0253	439.6971	single	193.05	5400	201.3886	201.3886	0.64	2Ø20
support	4	9.23	300	450	0.0076	446.9627	single	193.05	5400	59.37017	193.05	0.61	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(m	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	7.62	300	450	0.0063	447.4954	single	193.05	5400	48.95581	193.05	0.61	2Ø20
span	1-2	18.26	300	450	0.0150	443.9504	single	193.05	5400	118.2508	193.05	0.61	2Ø20
support	2	13.6	300	450	0.0112	445.51	single	193.05	5400	87.76458	193.05	0.61	2Ø20
span	2-3	23.7	300	450	0.0195	442.1155	single	193.05	5400	154.117	193.05	0.61	2Ø20
support	3	16.94	300	450	0.0139	444.3933	single	193.05	5400	109.5932	193.05	0.61	2Ø20
span	3-4	26.81	300	450	0.0221	441.0595	single	193.05	5400	174.7582	193.05	0.61	2Ø20
support	4	14.74	300	450	0.0121	445.1295	single	193.05	5400	95.20263	193.05	0.61	2Ø20

story- 8

Type	Loc	Moment (KNm)	b(m	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	6.85	300	250	0.0183	245.9028	single	107.25	3000	80.08755	107.25	0.34	2Ø20
span	1-2	4.7	300	250	0.0125	247.2036	single	107.25	3000	54.66143	107.25	0.34	2Ø20
support	2	2.1	300	250	0.0056	248.7583	single	107.25	3000	24.27054	107.25	0.34	2Ø20
span	2-3	4.38	300	250	0.0117	247.396	single	107.25	3000	50.90018	107.25	0.34	2Ø20
support	3	4.9	300	250	0.0131	247.0831	single	107.25	3000	57.01522	107.25	0.34	2Ø20
span	3-4	4.83	300	250	0.0129	247.1253	single	107.25	3000	56.19113	107.25	0.34	2Ø20
support	4	6.21	300	250	0.0166	246.2914	single	107.25	3000	72.49034	107.25	0.34	2Ø20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C
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A
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Roof
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-108.8791
5.0525
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8.9204
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Basement
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21.6077
22.1383
42.7081
44.0237
22.8424
Base
Z
Y

Axis 2

story- 7

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of ba	Remark
support	1	42.18	300	450	0.0347	435.763	single	193.05	5400	278.288	278.288	0.89	2Ø20
span	1-2	102.1	300	450	0.0840	413.7	single	193.05	5400	709.541	709.541	2.26	3Ø20
support	2	108.88	300	450	0.0896	411.039	single	193.05	5400	761.558	761.558	2.43	3Ø20
span	2-3	127.96	300	450	0.1053	403.337	single	193.05	5400	912.102	912.102	2.90	3Ø20
support	3	106.3	300	450	0.0875	412.056	single	193.05	5400	741.677	741.677	2.36	3Ø20
span	3-4	114.12	300	450	0.0939	408.956	single	193.05	5400	802.274	802.274	2.56	3Ø20
support	4	52.73	300	450	0.0434	432.049	single	193.05	5400	350.883	350.883	1.12	2Ø20

story- 6

Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of ba	Remark
support	1	41.04	300	450	0.0338	436.16	single	193.05	5400	270.52	270.52	0.86	2Ø20
span	1-2	101.6	300	450	0.0836	413.895	single	193.05	5400	705.734	705.734	2.25	3Ø20
support	2	109.85	300	450	0.0904	410.655	single	193.05	5400	769.06	769.06	2.45	3Ø20
span	2-3	129.2	300	450	0.1063	402.825	single	193.05	5400	922.112	922.112	2.94	3Ø20
support	3	99.6	300	450	0.0820	414.672	single	193.05	5400	690.546	690.546	2.20	3Ø20
span	3-4	108.7	300	450	0.0895	411.11	single	193.05	5400	760.167	760.167	2.42	3Ø20
support	4	52.1	300	450	0.0429	432.273	single	193.05	5400	346.512	346.512	1.10	2Ø20

story- 5

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	60.54	300	450	0.0498	429.2562	single	193.05	5400	405.4747	405.4747	1.29	2Ø20
span	1-2	74.14	300	450	0.0610	424.2994	single	193.05	5400	502.3635	502.3635	1.60	2Ø20
support	2	65.04	300	450	0.0535	427.6295	single	193.05	5400	437.2711	437.2711	1.39	2Ø20
span	2-3	65.17	300	450	0.0536	427.5823	single	193.05	5400	438.1934	438.1934	1.40	2Ø20
support	3	54.29	300	450	0.0447	431.4942	single	193.05	5400	361.7285	361.7285	1.15	2Ø20
span	3-4	34.14	300	450	0.0281	438.5499	single	193.05	5400	223.8115	223.8115	0.71	2Ø20
support	4	12.4	300	450	0.0102	445.9099	single	193.05	5400	79.94889	193.05	0.61	2Ø20

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	59.9	300	450	0.0493	429.4865	single	193.05	5400	400.9731	400.9731	1.28	2Ø20
span	1-2	71.8	300	450	0.0591	425.161	single	193.05	5400	485.522	485.522	1.55	2Ø20
support	2	68.94	300	450	0.0567	426.209	single	193.05	5400	465.0359	465.0359	1.48	2Ø20
span	2-3	93.67	300	450	0.0771	416.9576	single	193.05	5400	645.8721	645.8721	2.06	3Ø20
support	3	86.5	300	450	0.0712	419.6851	single	193.05	5400	592.5574	592.5574	1.89	2Ø20
span	3-4	106.2	300	450	0.0874	412.0956	single	193.05	5400	740.9082	740.9082	2.36	3Ø20
support	4	50.4	300	450	0.0415	432.875	single	193.05	5400	334.7387	334.7387	1.07	2Ø20

story- 3

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	37.06	300	450	0.0305	437.542	single	193.05	5400	243.5138	243.5138	0.78	2Ø20
span	1-2	90.7	300	450	0.0747	418.0921	single	193.05	5400	623.6964	623.6964	1.99	2Ø20
support	2	73.58	300	450	0.0606	424.5059	single	193.05	5400	498.3264	498.3264	1.59	2Ø20
span	2-3	97.2	300	450	0.0800	415.6004	single	193.05	5400	672.4008	672.4008	2.14	3Ø20
support	3	81.2	300	450	0.0668	421.677	single	193.05	5400	553.6228	553.6228	1.76	2Ø20
span	3-4	103.64	300	450	0.0853	413.0991	single	193.05	5400	721.2918	721.2918	2.30	3Ø20
support	4	55.26	300	450	0.0455	431.1484	single	193.05	5400	368.4868	368.4868	1.17	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	4.34	300	450	0.0036	448.577	single	193.05	5400	27.81574	193.05	0.61	2Ø20
span	1-2	16.87	300	450	0.0139	444.4167	single	193.05	5400	109.1346	193.05	0.61	2Ø20
support	2	11.97	300	450	0.0099	446.053	single	193.05	5400	77.15171	193.05	0.61	2Ø20
span	2-3	22.32	300	450	0.0184	442.5824	single	193.05	5400	144.99	193.05	0.61	2Ø20
support	3	10.96	300	450	0.0090	446.3887	single	193.05	5400	70.5887	193.05	0.61	2Ø20
span	3-4	21.61	300	450	0.0178	442.8222	single	193.05	5400	140.3018	193.05	0.61	2Ø20
support	4	6.09	300	450	0.0050	448.0006	single	193.05	5400	39.08198	193.05	0.61	2Ø20

story- 8

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	5.1	300	250	0.0136	246.9626	single	107.25	3000	59.37134	107.25	0.34	2Ø20
span	1-2	3.94	300	250	0.0105	247.6601	single	107.25	3000	45.7381	107.25	0.34	2Ø20
support	2	4.87	300	250	0.0130	247.1012	single	107.25	3000	56.66201	107.25	0.34	2Ø20
span	2-3	5.05	300	250	0.0135	246.9927	single	107.25	3000	58.78209	107.25	0.34	2Ø20
support	3	3.88	300	250	0.0103	247.696	single	107.25	3000	45.03504	107.25	0.34	2Ø20
span	3-4	4.44	300	250	0.0118	247.3599	single	107.25	3000	51.60496	107.25	0.34	2Ø20
support	4	5.98	300	250	0.0159	246.4308	single	107.25	3000	69.76603	107.25	0.34	2Ø20

Axis 3

story- 7

Type	Loc	Moment (KNm)	b(mn	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	42.46	300	450	0.0349	435.6652	single	193.05	5400	280.1979	280.1979	0.89	2Ø20
span	1-2	98.9	300	450	0.0814	414.9433	single	193.05	5400	685.2442	685.2442	2.18	3Ø20
support	2	106.6	300	450	0.0877	411.9383	single	193.05	5400	743.9828	743.9828	2.37	3Ø20
span	2-3	123.9	300	450	0.1020	405.0038	single	193.05	5400	879.5287	879.5287	2.80	3Ø20
support	3	104.8	300	450	0.0863	412.6451	single	193.05	5400	730.1675	730.1675	2.33	3Ø20
span	3-4	119.73	300	450	0.0985	406.6995	single	193.05	5400	846.3835	846.3835	2.70	3Ø20
support	4	52.9	300	450	0.0435	431.9886	single	193.05	5400	352.0637	352.0637	1.12	2Ø20

story- 6

Type	Loc	Moment (KNm)	b(mn	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	38.8	300	450	0.0319	436.9391	single	193.05	5400	255.2988	255.2988	0.81	2Ø20
span	1-2	84.42	300	450	0.0695	420.4692	single	193.05	5400	577.2301	577.2301	1.84	2Ø20
support	2	107.21	300	450	0.0882	411.6981	single	193.05	5400	748.6766	748.6766	2.38	3Ø20
span	2-3	127.32	300	450	0.1048	403.6011	single	193.05	5400	906.9475	906.9475	2.89	3Ø20
support	3	96.32	300	450	0.0793	415.9396	single	193.05	5400	665.7697	665.7697	2.12	3Ø20
span	3-4	99.23	300	450	0.0817	414.8155	single	193.05	5400	687.7425	687.7425	2.19	3Ø20
support	4	47.52	300	450	0.0391	433.8914	single	193.05	5400	314.8714	314.8714	1.00	2Ø20

story- 5

Type	Loc	Moment (KNm)	b(mn	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	66.03	300	450	0.0543	427.2698	single	193.05	5400	444.3006	444.3006	1.41	2Ø20
span	1-2	68.69	300	450	0.0565	426.3004	single	193.05	5400	463.2502	463.2502	1.48	2Ø20
support	2	73.73	300	450	0.0607	424.4506	single	193.05	5400	499.4073	499.4073	1.59	2Ø20
span	2-3	79.7	300	450	0.0656	422.2371	single	193.05	5400	542.675	542.675	1.73	2Ø20
support	3	76.76	300	450	0.0632	423.3302	single	193.05	5400	521.307	521.307	1.66	2Ø20
span	3-4	95.14	300	450	0.0783	416.3936	single	193.05	5400	656.8966	656.8966	2.09	3Ø20
support	4	32.66	300	450	0.0269	439.059	single	193.05	5400	213.8608	213.8608	0.68	2Ø20

story- 4

Type	Loc	Moment (KNm)	b(mn	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	66.65	300	450	0.0549	427.0443	single	193.05	5400	448.7093	448.7093	1.43	2Ø20
span	1-2	83.23	300	450	0.0685	420.9164	single	193.05	5400	568.4887	568.4887	1.81	2Ø20
support	2	75.8	300	450	0.0624	423.6859	single	193.05	5400	514.3551	514.3551	1.64	2Ø20
span	2-3	98.97	300	450	0.0815	414.9162	single	193.05	5400	685.774	685.774	2.18	3Ø20
support	3	74.67	300	450	0.0615	424.1037	single	193.05	5400	506.1881	506.1881	1.61	2Ø20
span	3-4	95.5	300	450	0.0786	416.2552	single	193.05	5400	659.6014	659.6014	2.10	3Ø20
support	4	39.36	300	450	0.0324	436.7447	single	193.05	5400	259.0988	259.0988	0.83	2Ø20

story- 3

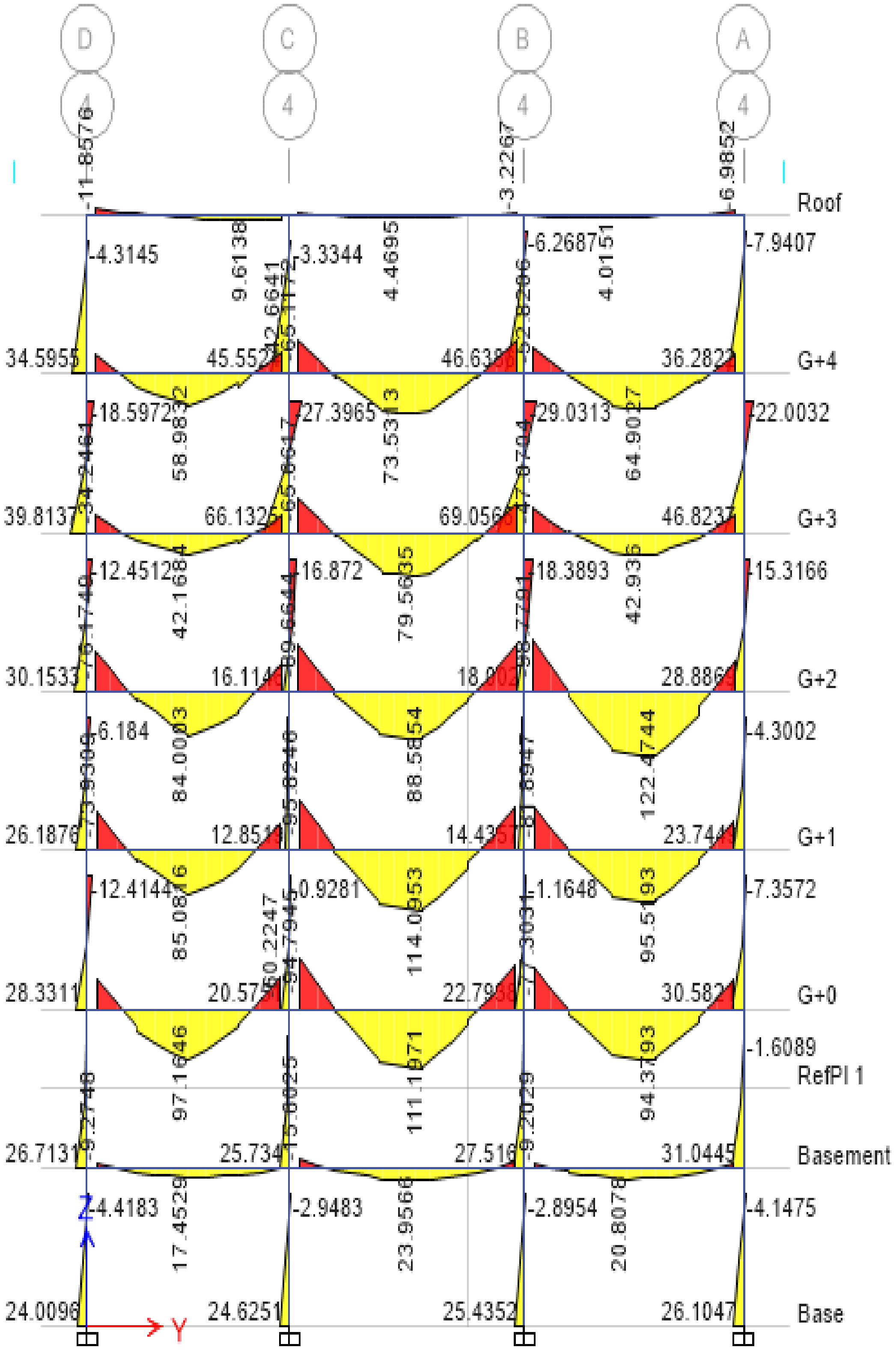
Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	44.38	300	450	0.0365	434.9939	single	193.05	5400	293.3202	293.3202	0.93	2Ø20
span	1-2	96.1	300	450	0.0791	416.0243	single	193.05	5400	664.1138	664.1138	2.12	3Ø20
support	2	81.01	300	450	0.0667	421.748	single	193.05	5400	552.2344	552.2344	1.76	2Ø20
span	2-3	106.7	300	450	0.0878	411.8989	single	193.05	5400	744.7519	744.7519	2.37	3Ø20
support	3	70.74	300	450	0.0582	425.5501	single	193.05	5400	477.9168	477.9168	1.52	2Ø20
span	3-4	92.83	300	450	0.0764	417.2791	single	193.05	5400	639.5869	639.5869	2.04	3Ø20
support	4	43.67	300	450	0.0359	435.2424	single	193.05	5400	288.4628	288.4628	0.92	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	4.9	300	450	0.0040	448.3927	single	193.05	5400	31.41777	193.05	0.61	2Ø20
span	1-2	16.57	300	450	0.0136	444.5173	single	193.05	5400	107.1696	193.05	0.61	2Ø20
support	2	11.8	300	450	0.0097	446.1095	single	193.05	5400	76.04635	193.05	0.61	2Ø20
span	2-3	22.14	300	450	0.0182	442.6432	single	193.05	5400	143.8009	193.05	0.61	2Ø20
support	3	8.31	300	450	0.0068	447.2673	single	193.05	5400	53.41605	193.05	0.61	2Ø20
span	3-4	19.64	300	450	0.0162	443.4863	single	193.05	5400	127.3207	193.05	0.61	2Ø20
support	4	2.82	300	450	0.0023	449.0764	single	193.05	5400	18.05372	193.05	0.61	2Ø20

story- 8

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	5.13	300	250	0.0137	246.9445	single	107.25	3000	59.72496	107.25	0.34	2Ø20
span	1-2	3.84	300	250	0.0102	247.72	single	107.25	3000	44.56644	107.25	0.34	2Ø20
support	2	4.93	300	250	0.0131	247.0651	single	107.25	3000	57.36849	107.25	0.34	2Ø20
span	2-3	5.23	300	250	0.0139	246.8842	single	107.25	3000	60.90406	107.25	0.34	2Ø20
support	3	3.73	300	250	0.0099	247.7859	single	107.25	3000	43.27829	107.25	0.34	2Ø20
span	3-4	4.32	300	250	0.0115	247.432	single	107.25	3000	50.1956	107.25	0.34	2Ø20
support	4	6.14	300	250	0.0164	246.3339	single	107.25	3000	71.66087	107.25	0.34	2Ø20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Base
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Axis 4

story- 7

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	37.13	300	450	0.0306	437.5178	single	193.05	5400	243.9872	243.9872	0.78	2Ø20
span	1-2	58.98	300	450	0.0485	429.8171	single	193.05	5400	394.5109	394.5109	1.26	2Ø20
support	2	65.12	300	450	0.0536	427.6004	single	193.05	5400	437.8387	437.8387	1.39	2Ø20
span	2-3	73.53	300	450	0.0605	424.5243	single	193.05	5400	497.9661	497.9661	1.59	2Ø20
support	3	64.8	300	450	0.0533	427.7166	single	193.05	5400	435.5688	435.5688	1.39	2Ø20
span	3-4	64.9	300	450	0.0534	427.6803	single	193.05	5400	436.278	436.278	1.39	2Ø20
support	4	39.1	300	450	0.0322	436.8349	single	193.05	5400	257.334	257.334	0.82	2Ø20

story- 6

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	34.25	300	450	0.0282	438.5121	single	193.05	5400	224.552	224.552	0.72	2Ø20
span	1-2	42.2	300	450	0.0347	435.756	single	193.05	5400	278.4242	278.4242	0.89	2Ø20
support	2	65.7	300	450	0.0541	427.3898	single	193.05	5400	441.956	441.956	1.41	2Ø20
span	2-3	79.56	300	450	0.0655	422.2893	single	193.05	5400	541.6548	541.6548	1.73	2Ø20
support	3	55.52	300	450	0.0457	431.0557	single	193.05	5400	370.3002	370.3002	1.18	2Ø20
span	3-4	42.9	300	450	0.0353	435.5116	single	193.05	5400	283.2014	283.2014	0.90	2Ø20
support	4	34.4	300	450	0.0283	438.4604	single	193.05	5400	225.562	225.562	0.72	2Ø20

story- 5

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	76.14	300	450	0.0627	423.56	single	193.05	5400	516.8158	516.8158	1.65	2Ø20
span	1-2	84	300	450	0.0691	420.6272	single	193.05	5400	574.1426	574.1426	1.83	2Ø20
support	2	89.7	300	450	0.0738	418.4725	single	193.05	5400	616.2591	616.2591	1.96	2Ø20
span	2-3	88.6	300	450	0.0729	418.8902	single	193.05	5400	608.0949	608.0949	1.94	2Ø20
support	3	98.8	300	450	0.0813	414.982	single	193.05	5400	684.4875	684.4875	2.18	3Ø20
span	3-4	122.5	300	450	0.1008	405.5749	single	193.05	5400	868.3661	868.3661	2.77	3Ø20
support	4	58.12	300	450	0.0478	430.1256	single	193.05	5400	388.4795	388.4795	1.24	2Ø20

story- 4

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	73.9	300	450	0.0608	424.3879	single	193.05	5400	500.6328	500.6328	1.59	2Ø20
span	1-2	85	300	450	0.0700	420.2509	single	193.05	5400	581.4979	581.4979	1.85	2Ø20
support	2	95.8	300	450	0.0788	416.1398	single	193.05	5400	661.8569	661.8569	2.11	3Ø20
span	2-3	114.1	300	450	0.0939	408.9641	single	193.05	5400	802.1181	802.1181	2.55	3Ø20
support	3	81.9	300	450	0.0674	421.4151	single	193.05	5400	558.7425	558.7425	1.78	2Ø20
span	3-4	85.52	300	450	0.0704	420.0549	single	193.05	5400	585.3282	585.3282	1.86	2Ø20
support	4	53.1	300	450	0.0437	431.9176	single	193.05	5400	353.4529	353.4529	1.13	2Ø20

story- 3

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	55.4	300	450	0.0456	431.0985	single	193.05	5400	369.4631	369.4631	1.18	2Ø20
span	1-2	97.2	300	450	0.0800	415.6004	single	193.05	5400	672.4008	672.4008	2.14	3Ø20
support	2	94.8	300	450	0.0780	416.5241	single	193.05	5400	654.3438	654.3438	2.08	3Ø20
span	2-3	111.19	300	450	0.0915	410.1238	single	193.05	5400	779.4507	779.4507	2.48	3Ø20
support	3	83.13	300	450	0.0684	420.954	single	193.05	5400	567.7551	567.7551	1.81	2Ø20
span	3-4	94.4	300	450	0.0777	416.6777	single	193.05	5400	651.3428	651.3428	2.07	3Ø20
support	4	55.2	300	450	0.0454	431.1698	single	193.05	5400	368.0684	368.0684	1.17	2Ø20

story- 2

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	9.27	300	450	0.0076	446.9494	single	193.05	5400	59.62923	193.05	0.61	2Ø20
span	1-2	17.46	300	450	0.0144	444.2189	single	193.05	5400	113.0017	193.05	0.61	2Ø20
support	2	15.6	300	450	0.0128	444.842	single	193.05	5400	100.8223	193.05	0.61	2Ø20
span	2-3	23.95	300	450	0.0197	442.0308	single	193.05	5400	155.7725	193.05	0.61	2Ø20
support	3	9.2	300	450	0.0076	446.9726	single	193.05	5400	59.17589	193.05	0.61	2Ø20
span	3-4	20.8	300	450	0.0171	443.0955	single	193.05	5400	134.9596	193.05	0.61	2Ø20
support	4	7.2	300	450	0.0059	447.6342	single	193.05	5400	46.24311	193.05	0.61	2Ø20

story- 8

Type	Loc	Moment (KNm)	b(mm	d(mm	K	Z(mm)	Beam type	As,min	As,m	As,cal	As,prov	No of	Remark
support	1	11.86	300	250	0.0316	242.8159	single	107.25	3000	140.4253	140.4253	0.45	2Ø20
span	1-2	9.61	300	250	0.0256	244.2121	single	107.25	3000	113.1342	113.1342	0.36	2Ø20
support	2	3.02	300	250	0.0081	248.2104	single	107.25	3000	34.9804	107.25	0.34	2Ø20
span	2-3	4.5	300	250	0.0120	247.3239	single	107.25	3000	52.30996	107.25	0.34	2Ø20
support	3	3.2	300	250	0.0085	248.1029	single	107.25	3000	37.08138	107.25	0.34	2Ø20
span	3-4	4.01	300	250	0.0107	247.6181	single	107.25	3000	46.55859	107.25	0.34	2Ø20
support	4	6.98	300	250	0.0186	245.8237	single	107.25	3000	81.63372	107.25	0.34	2Ø20

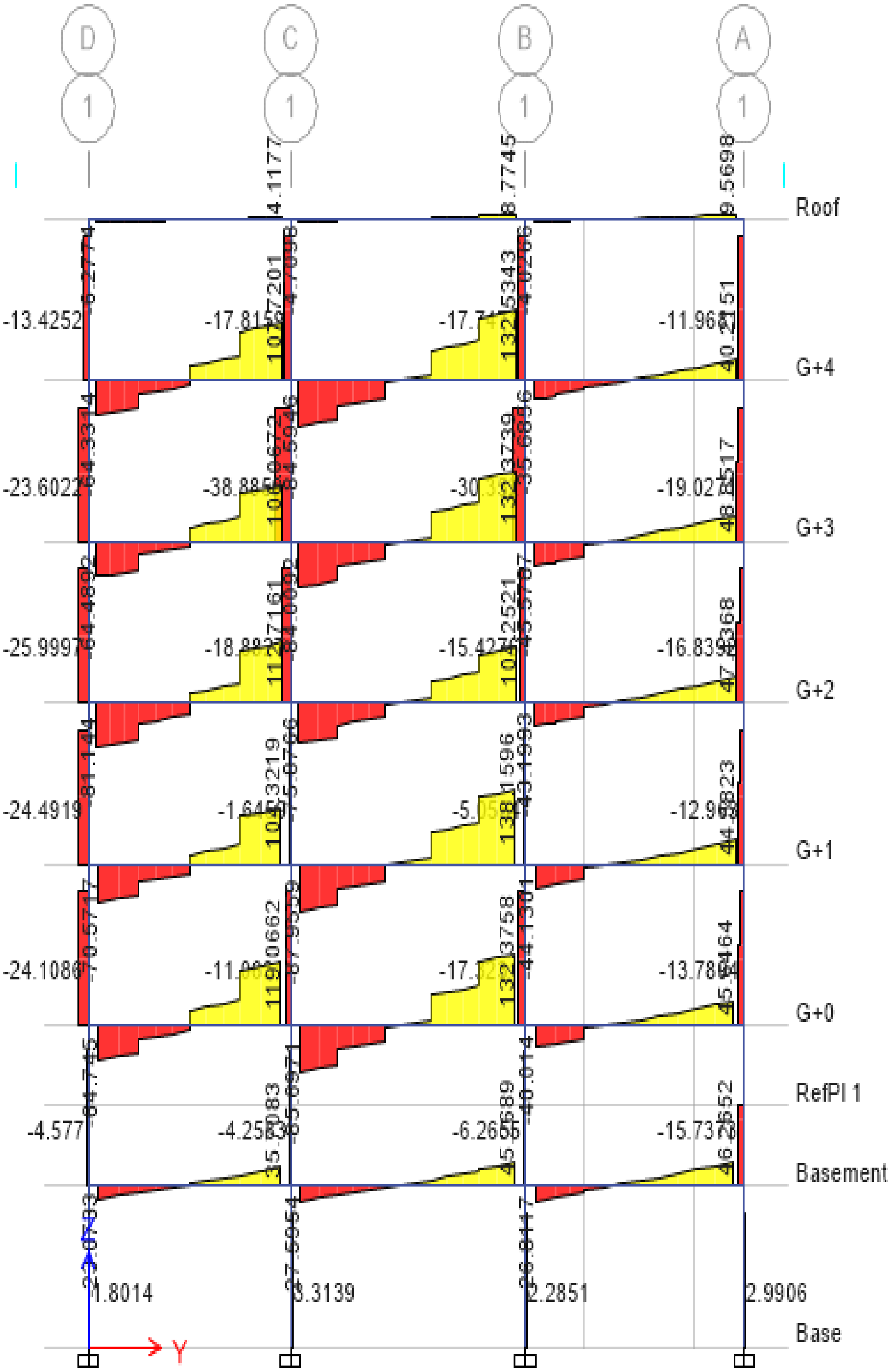
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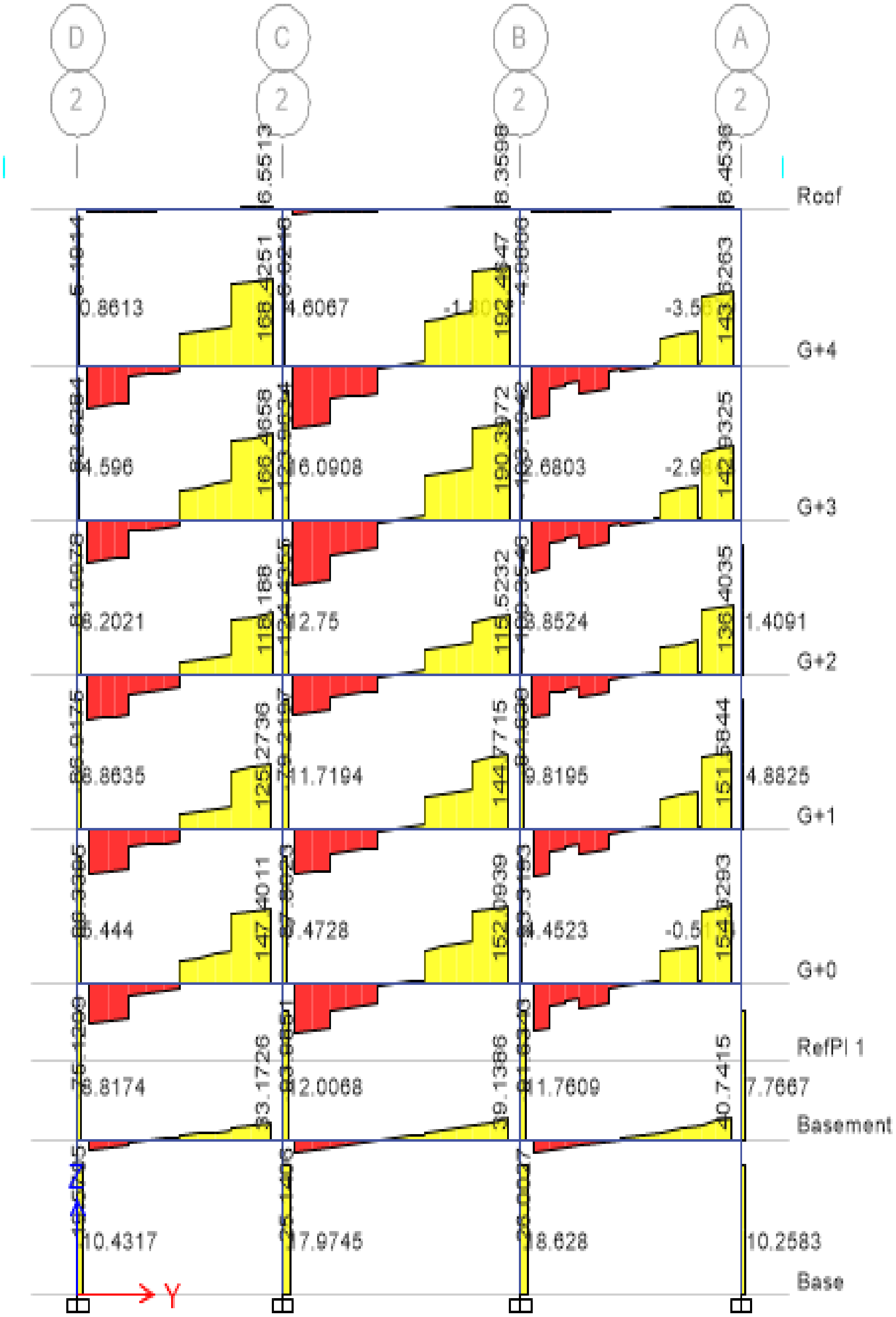
Table 37beam stirrup calculation

axis 1

Story -8

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	6.3	5.47	337.5	5054.0	292.6	337.50	330
B-C	8.8	7.87	337.5	3512.8	292.6	337.50	330
A-B	9.6	8.59	337.5	3218.4	292.6	337.50	300
Story -7							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	107.7	88.59	337.5	312.1	292.6	312.06	300
B-C	132.53	110.07	337.5	251.2	292.6	251.16	250
A-B	40.22	31.11	337.5	888.6	292.6	337.50	330
Story -6							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	106.1	87.15	337.5	317.2	292.6	317.22	310
B-C	132.4	110.01	337.5	251.3	292.6	251.30	250
A-B	48.85	37.52	337.5	736.8	292.6	337.50	330
Story -5							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	112.72	91.18	337.5	303.2	292.6	303.20	300
B-C	104.2	84.02	337.5	329.0	292.6	329.04	320
A-B	47.44	35.66	337.5	775.3	292.6	337.50	330
Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	104.3	83.25	337.5	332.1	292.6	332.08	330
B-C	138.16	112.82	337.5	245.0	292.6	245.04	240
A-B	44.58	33.05	337.5	836.5	292.6	337.50	330
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	119.1	96.97	337.5	285.1	292.6	285.09	280
B-C	132.37	107.93	337.5	256.1	292.6	256.14	250
A-B	45.65	34.52	337.5	800.9	292.6	337.50	330
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	35.7	28.62	337.5	966.0	292.6	337.50	330
B-C	45.25	37.1	337.5	745.2	292.6	337.50	330
A-B	46.26	40.52	337.5	682.3	292.6	337.50	330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C
B
A
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2.6803
-2.9
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142.9325
G+3
8.2021
12.75
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136.4035
1.4091
G+2
8.8635
11.7194
9.8195
125.2736
144.7715
151.5844
4.8825
G+1
5.444
7.4728
4.4523
-0.5
147.4011
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154.3293
G+0
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33.1726
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Basement
10.4317
17.9745
18.628
10.2583
Base
Z
Y

axis 2

Story -8

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	6.6	5.73	337.5	4824.7	292.6	337.50	330
B-C	8.36	7.29	337.5	3792.3	292.6	337.50	330
A-B	8.45	7.37	337.5	3751.1	292.6	337.50	330

Story -7

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	188.4	158.29	337.5	174.7	292.6	174.65	170
B-C	192.46	157.01	337.5	176.1	292.6	176.08	170
A-B	143.6	111.52	337.5	247.9	292.6	247.90	240

Story -6

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	116.46	92.58	337.5	298.6	292.6	298.61	290
B-C	190.4	155.12	337.5	178.2	292.6	178.22	170
A-B	142.9	111.27	337.5	248.5	292.6	248.46	240

Story -5

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	118.52	93.51	337.5	295.6	292.6	295.64	290
B-C	115.52	93.7	337.5	295.0	292.6	295.04	290
A-B	136.4	107.66	337.5	256.8	292.6	256.79	250

Story -4

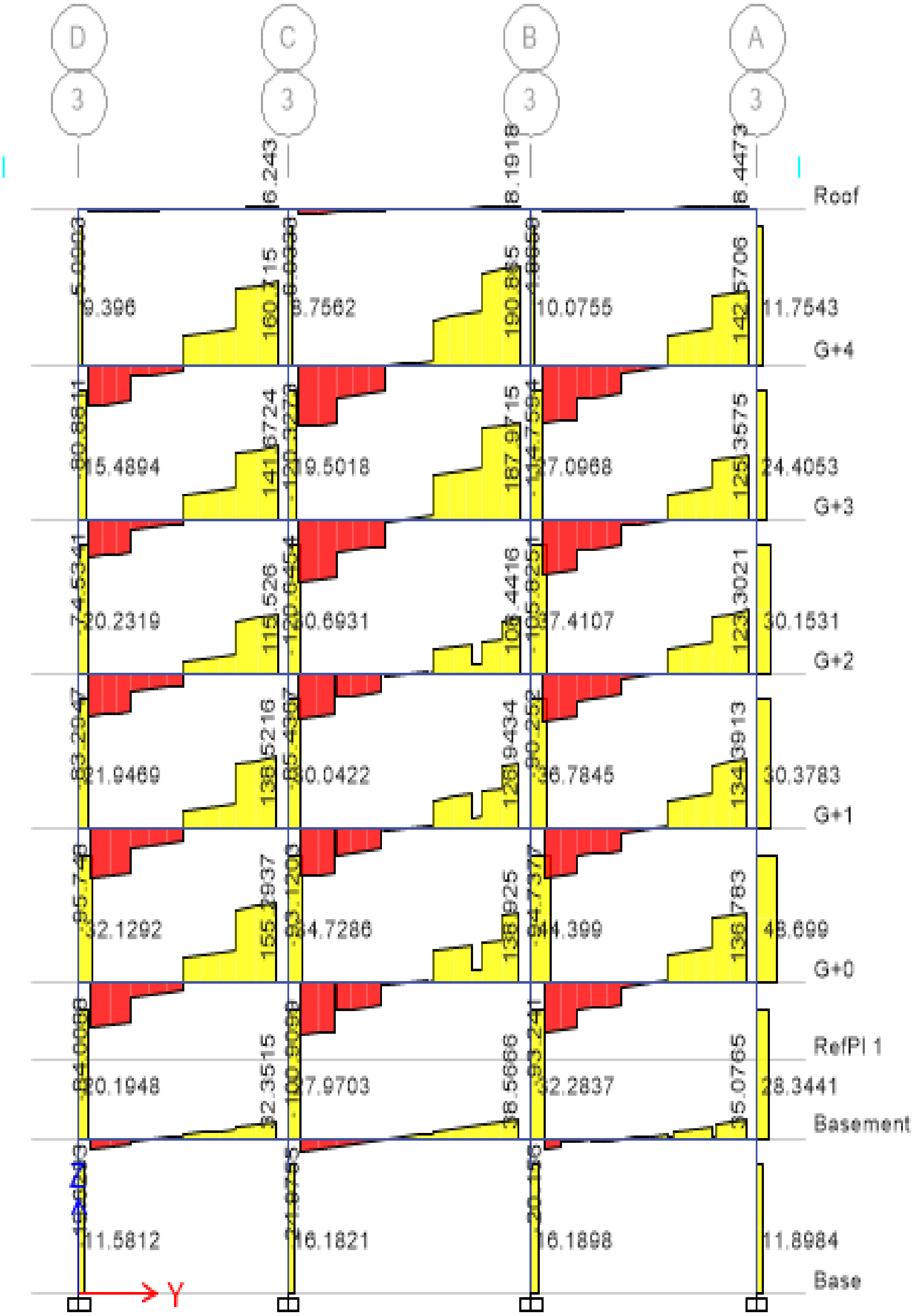
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	125.3	97.87	337.5	282.5	292.6	282.47	280
B-C	144.7	116.64	337.5	237.0	292.6	237.02	230
A-B	151.6	117.31	337.5	235.7	292.6	235.66	230

Story -3

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	147.4	118.55	337.5	233.2	292.6	233.20	230
B-C	152.1	122.4	337.5	225.9	292.6	225.86	220
A-B	154.32	119.89	337.5	230.6	292.6	230.59	230

Story -2

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	33.2	26.36	337.5	1048.8	292.6	337.50	330
B-C	39.14	31.38	337.5	881.0	292.6	337.50	330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C
B
A
3
Roof
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G+3
G+2
G+1
G+0
RefPl 1
Basement
Base
Z
Y
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15.4894
9.5018
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20.2319
0.6931
7.4107
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21.9469
0.0422
6.7845
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2.1292
4.7286
4.399
48.699
20.1948
7.9703
2.2837
28.3441
32.3515
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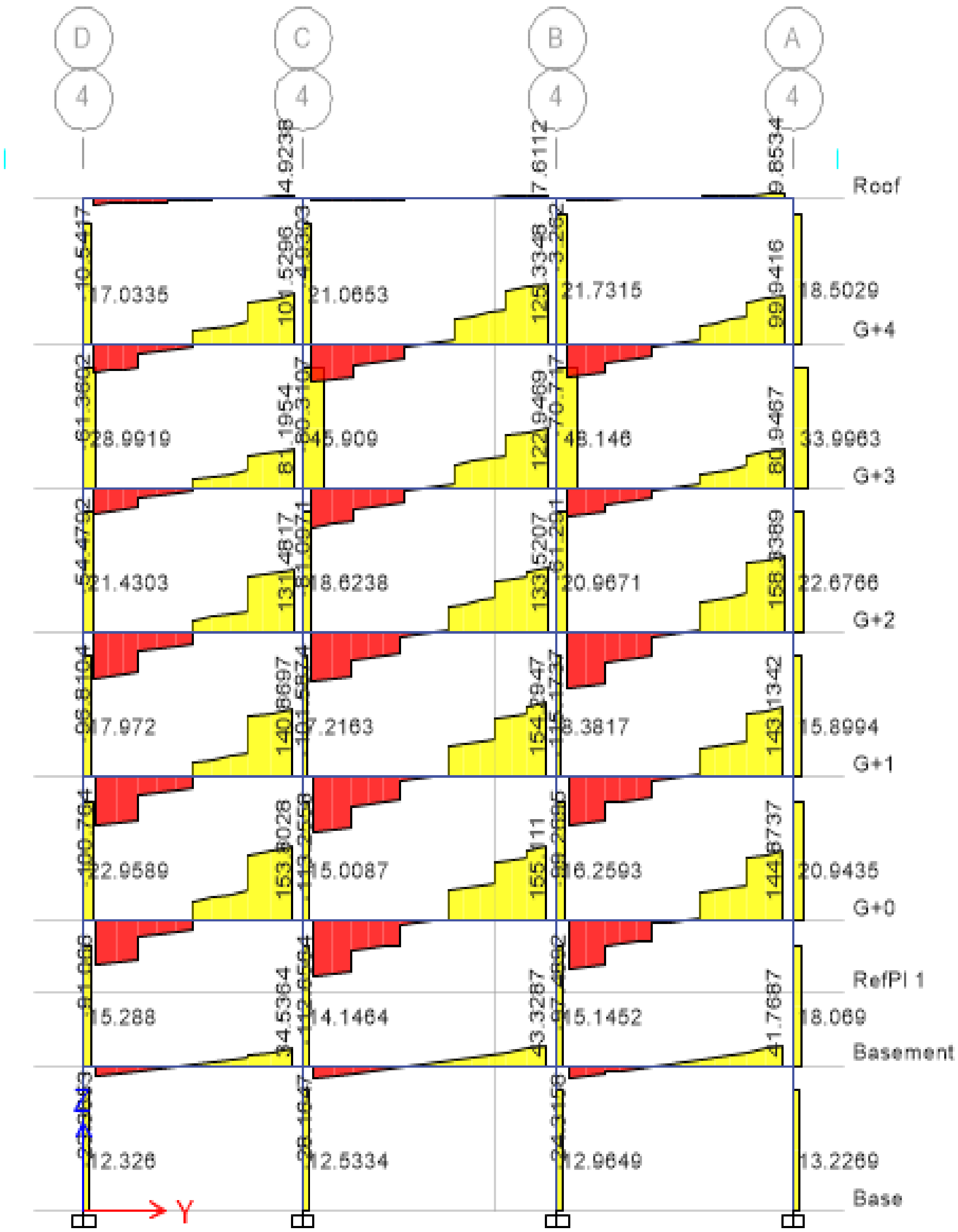
11.5812
16.1821
16.1898
11.8984

axis 3							
Story -8							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	6.24	5.4	337.5	5119.6	292.6	337.50	330
B-C	8.2	7.22	337.5	3829.0	292.6	337.50	330
A-B	8.45	7.53	337.5	3671.4	292.6	337.50	330
Story -7							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	160.7	131.62	337.5	210.0	292.6	210.04	210
B-C	187.97	153.39	337.5	180.2	292.6	180.23	180
A-B	142.57	109.12	337.5	253.4	292.6	253.35	250

Story -5							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	115.53	91.6	337.5	301.8	292.6	301.81	300
B-C	108.44	86.71	337.5	318.8	292.6	318.83	300
A-B	123.3	95.54	337.5	289.4	292.6	289.36	280
Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	138.52	108.15	337.5	255.6	292.6	255.62	250
B-C	126.94	100.38	337.5	275.4	292.6	275.41	270
A-B	134.4	102.32	337.5	270.2	292.6	270.19	270
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	155.3	124.28	337.5	222.4	292.6	222.45	220
B-C	138.93	109.99	337.5	251.3	292.6	251.35	250
A-B	136.78	104.58	337.5	264.3	292.6	264.35	260
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	32.35	25.61	337.5	1079.5	292.6	337.50	330
B-C	38.57	30.9	337.5	894.7	292.6	337.50	330
A-B	35.1	27.36	337.5	1010.4	292.6	337.50	330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48.146
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G+3
21.4303
18.6238
20.9671
22.6766
G+2
17.972
7.2163
8.3817
15.8994
G+1
22.9589
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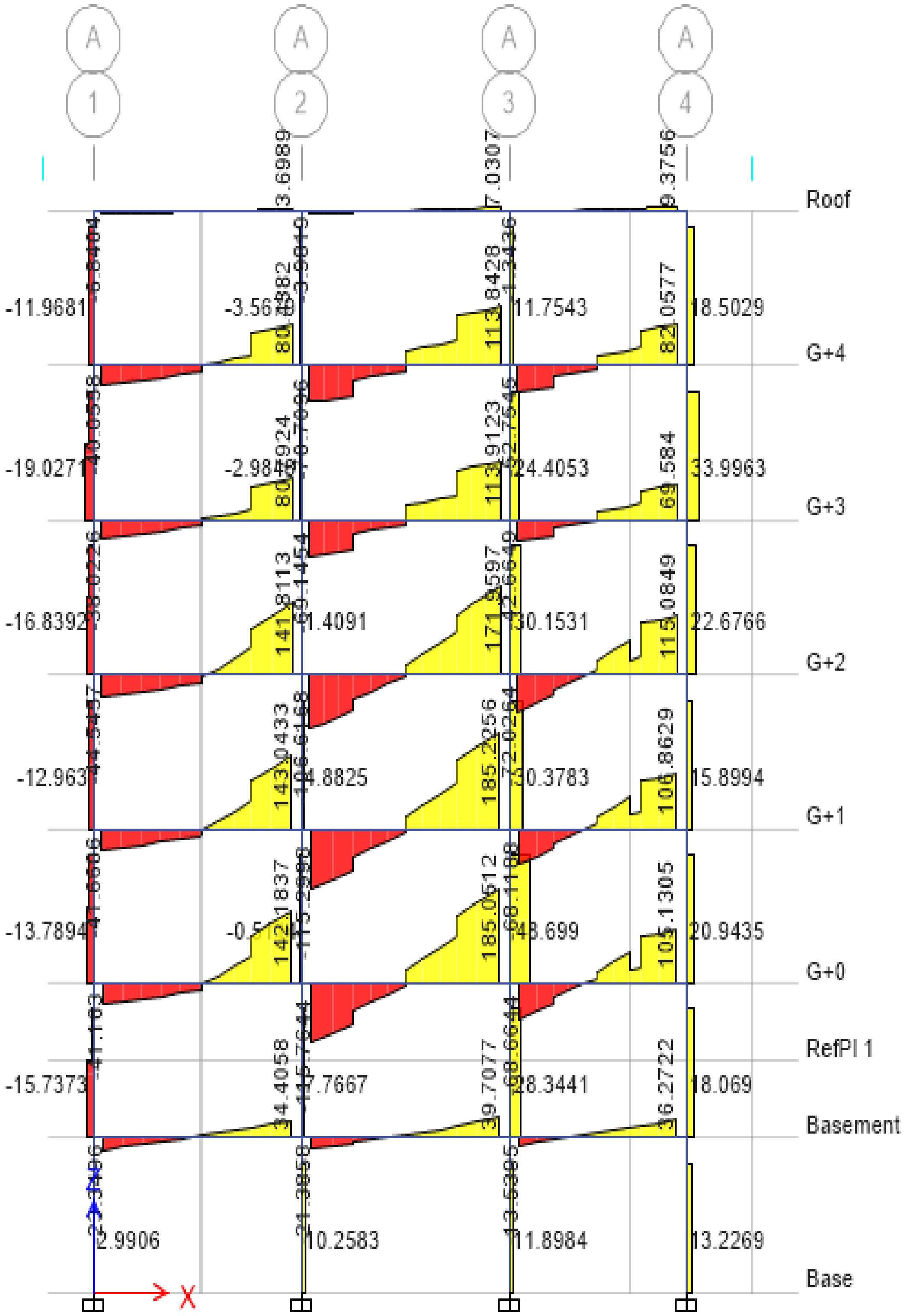
16.2593
20.9435
G+0
RefPl 1
15.288
34.5364
14.1464
43.3287
15.1452
41.7687
18.069
Basement
12.326
12.5334
12.9649
13.2269
Base
Z
Y

axis 4							
Story -8							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	10.54	9.39	337.5	2944.2	292.6	337.50	330
B-C	7.6	6.74	337.5	4101.7	292.6	337.50	330
A-B	9.85	8.8	337.5	3141.6	292.6	337.50	330
Story -7							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	101.53	81.92	337.5	337.5	292.6	337.47	330
B-C	125.33	102.28	337.5	270.3	292.6	270.29	270
A-B	99.94	77.75	337.5	355.6	292.6	337.50	330
Story -6							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
C-D	81.2	64.87	337.5	426.2	292.6	337.50	330
B-C	122.98	100.07	337.5	276.3	292.6	276.26	270
A-B	80.9	62.41	337.5	443.0	292.6	337.50	330

Story -5

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remrk
C-D	131.48	104	337.5	265.8	292.6	265.82	260
B-C	133.52	107.17	337.5	258.0	292.6	257.96	250
A-B	158.3	122.75	337.5	225.2	292.6	225.22	220
Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remrk
C-D	140.87	109.56	337.5	252.3	292.6	252.33	250
B-C	154.3	122.01	337.5	226.6	292.6	226.59	220
A-B	143.18	109.24	337.5	253.1	292.6	253.07	250
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remrk
C-D	151.8	120.31	337.5	229.8	292.6	229.79	220
B-C	155.1	122.79	337.5	225.1	292.6	225.15	220
A-B	144.87	115.62	337.5	239.1	292.6	239.11	230
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remrk
C-D	34.54	27.03	337.5	1022.8	292.6	337.50	330
B-C	43.32	34.69	337.5	796.9	292.6	337.50	330
A-B	41.77	32.52	337.5	850.1	292.6	337.50	330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A
3
A
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24.4053
33.9963
G+3
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1.4091
30.1531
22.6766
G+2
-12.9637
4.8825
30.3783
15.8994
G+1
-13.7894
48.699
20.9435
G+0
RefPl 1
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28.3441
18.069
Basement
2.9906
10.2583
11.8984
13.2269
Base
X

axis A

Story -8

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	6.84	5.58	337.5	4954.4	292.6	337.5	330
2-3	5.71	5.71	337.5	4841.6	292.6	337.5	330
3-4	7.87	10.58	337.5	2613.0	292.6	337.5	330

Story -7

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	80.44	65.98	337.5	419.0	292.6	337.5	330
2-3	113.84	91.69	337.5	301.5	292.6	301.5	300
3-4	82.06	63.03	337.5	438.6	292.6	337.5	330

Story -6

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	80.79	66.53	337.5	415.5	292.6	337.5	330
2-3	113.91	91.94	337.5	300.7	292.6	300.7	300
3-4	69.6	53.75	337.5	514.3	292.6	337.5	290

Story -5

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	141.81	119.45	337.5	231.4	292.6	231.4	230
2-3	171.96	138.53	337.5	199.6	292.6	199.6	190
3-4	115.1	88.68	337.5	311.7	292.6	311.7	310

Story -4

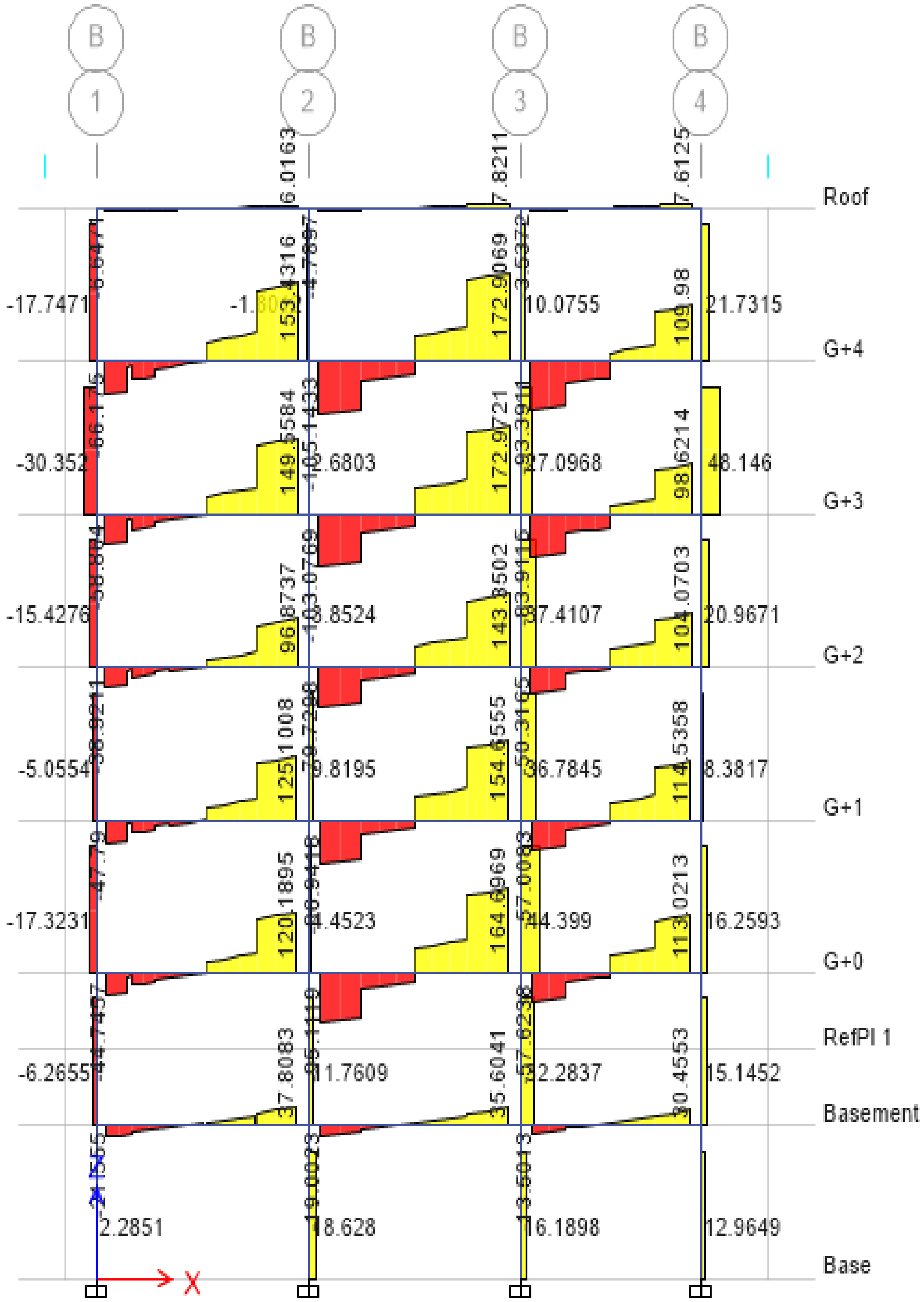
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	143.04	120.88	337.5	228.7	292.6	228.7	220
2-3	185.2	146.14	337.5	189.2	292.6	189.2	180
3-4	106.9	80.13	337.5	345.0	292.6	337.5	330

Story -3

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	142.2	118.36	337.5	233.6	292.6	233.6	230
2-3	185.05	145.94	337.5	189.4	292.6	189.4	180
3-4	105.13	78.55	337.5	351.9	292.6	337.5	330

Story -2

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	34.4	26.86	337.5	1029.2	292.6	337.5	330
2-3	39.7	31.76	337.5	870.5	292.6	337.5	330
3-4	36.3	28.67	337.5	964.3	292.6	337.5	330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1
B
2
B
3
B
4
Roof
G+4
G+3
G+2
G+1
G+0
RefPl 1
Basement
Base
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20.9671
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-79.7298
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50.3165
36.7845
114.5358
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164.6969
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44.399
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-95.1119
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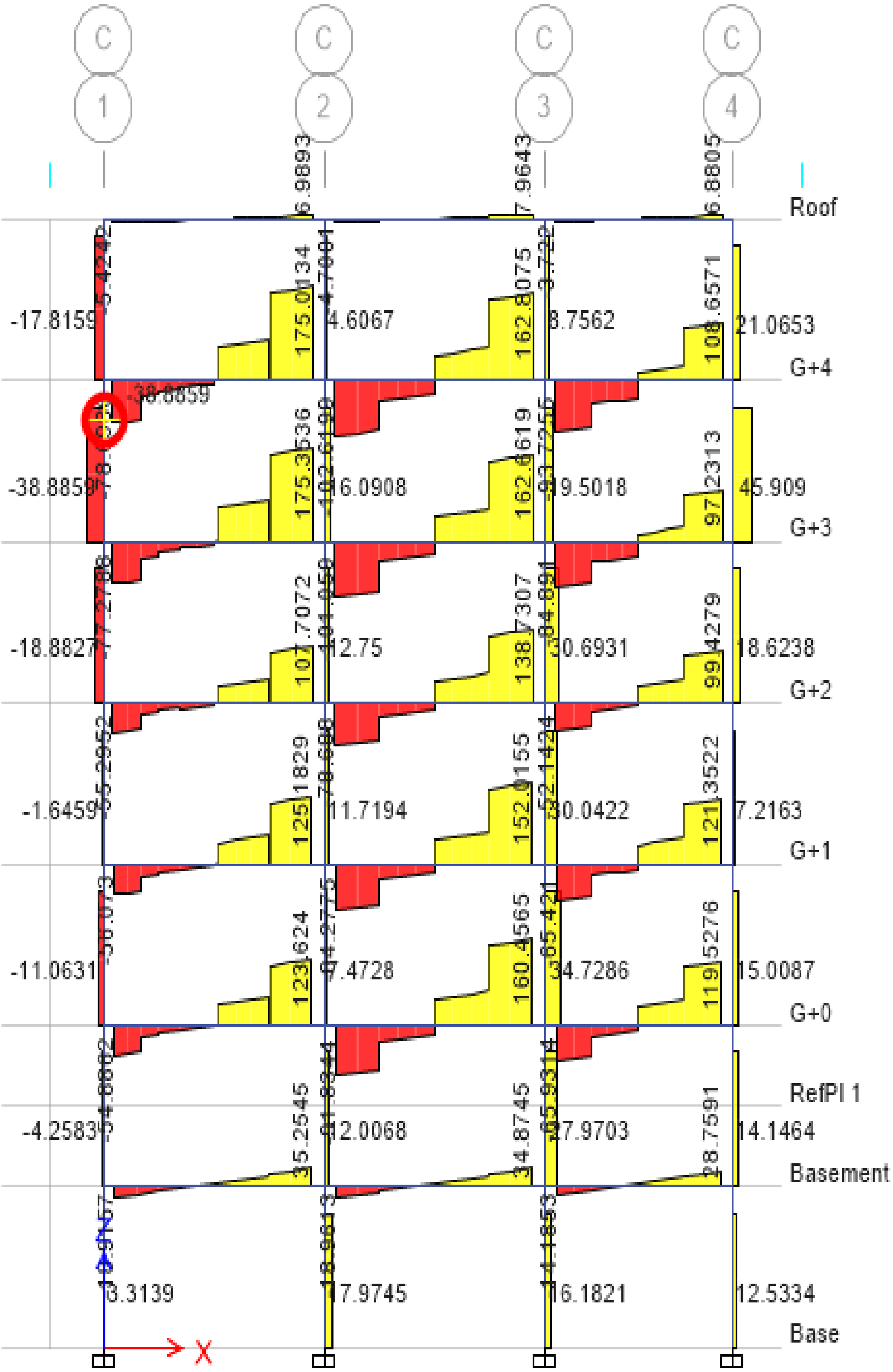
35.6041
57.6238
32.2837
30.4553
15.1452
-21.1565
2.2851
19.0023
18.628
13.5013
16.1898
12.9649

axis B							
Story -8							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	6.65	5	337.5	5529.1	292.6	337.5	330
2-3	7.8	6.16	337.5	4487.9	292.6	337.5	330
3-4	7.6	5.9	337.5	4685.7	292.6	337.5	330
Story -7							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	153.4	124.85	337.5	221.4	292.6	221.4	220
2-3	172.9	136.75	337.5	202.2	292.6	202.2	200
3-4	109.98	78.87	337.5	350.5	292.6	337.5	330
Story -6							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	149.56	122.46	337.5	225.8	292.6	225.8	220
2-3	172.97	137.08	337.5	201.7	292.6	201.7	200
3-4	98.6	70.69	337.5	391.1	292.6	337.5	330
Story -5							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	96.87	79.22	337.5	349.0	292.6	337.5	330
2-3	143.35	114.35	337.5	241.8	292.6	241.8	240
3-4	104.1	80.48	337.5	343.5	292.6	337.5	330

Story -4

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	125.1	100.9	337.5	274.0	292.6	274.0	270
2-3	154.65	120.83	337.5	228.8	292.6	228.8	220
3-4	114.54	86.28	337.5	320.4	292.6	320.4	320
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	120.2	97.11	337.5	284.7	292.6	284.7	280
2-3	164.7	128.33	337.5	215.4	292.6	215.4	210
3-4	113.02	84.91	337.5	325.6	292.6	325.6	320
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	37.8	29.49	337.5	937.5	292.6	337.5	330
2-3	35.6	27.96	337.5	988.8	292.6	337.5	330
3-4	30.45	23.21	337.5	1191.1	292.6	337.5	330


C 1
C 2
C 3
C 4
Roof
G+4
G+3
G+2
G+1
G+0
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Base
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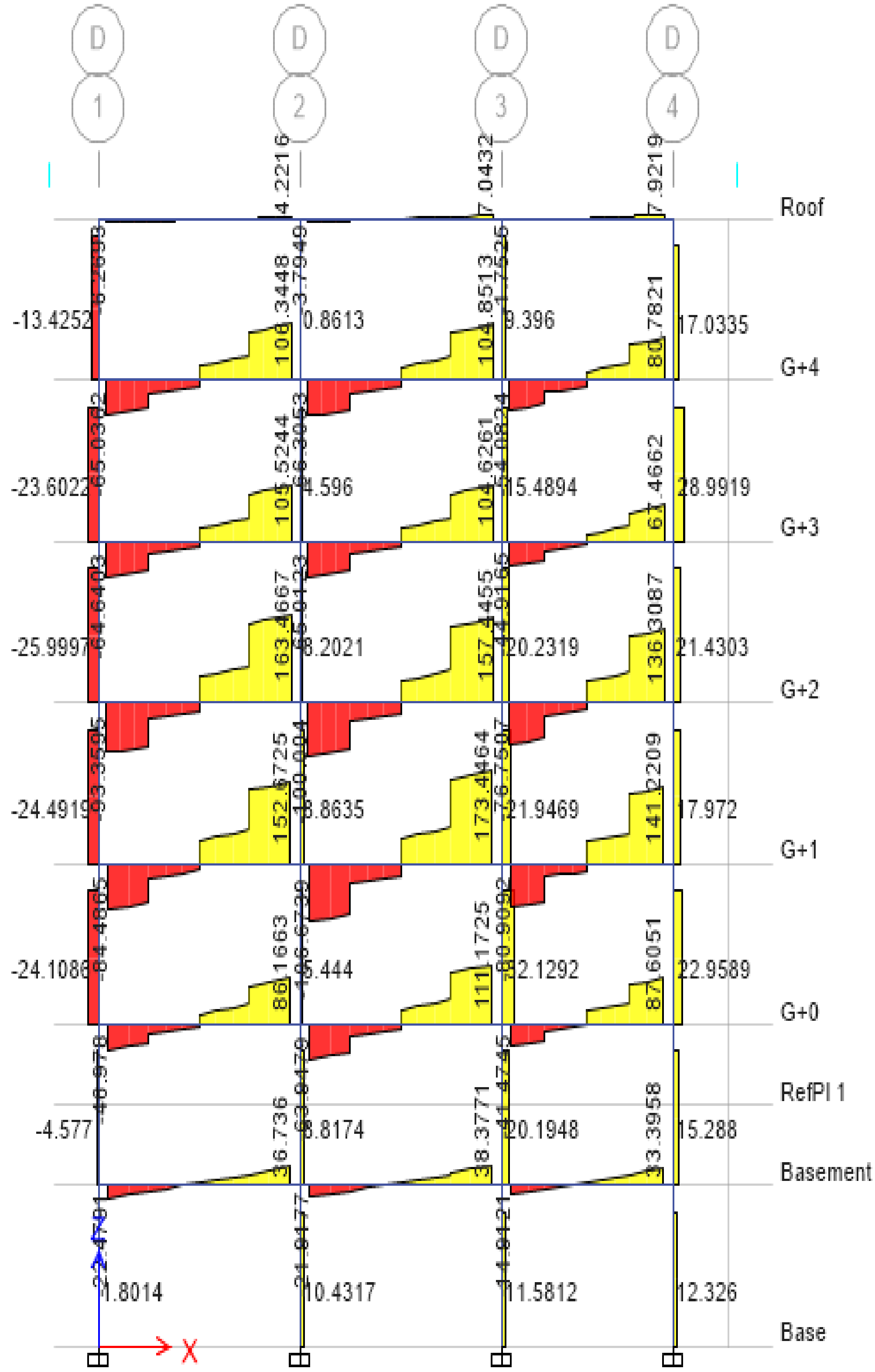
28.7591
14.1464
8.3139
17.9745
16.1821
12.5334

axis C							
Story -8							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	6.98	5.25	337.5	5265.8	292.6	337.5	330
2-3	7.9	6.14	337.5	4502.5	292.6	337.5	330
3-4	6.9	5.15	337.5	5368.1	292.6	337.5	330
Story -7							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	175.02	142.04	337.5	194.6	292.6	194.6	190
2-3	162.8	128.3	337.5	215.5	292.6	215.5	210
3-4	108.66	77.71	337.5	355.8	292.6	337.5	330
Story -6							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	175.36	142.52	337.5	194.0	292.6	194.0	190
2-3	162.7	128.41	337.5	215.3	292.6	215.3	210
3-4	97.2	69.35	337.5	398.6	292.6	337.5	330
Story -5							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	107.7	86.51	337.5	319.6	292.6	319.6	310
2-3	137.7	109.57	337.5	252.3	292.6	252.3	250
3-4	99.43	76.25	337.5	362.6	292.6	337.5	330
Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	125.2	101.63	337.5	272.0	292.6	272.0	270
2-3	152.1	119	337.5	232.3	292.6	232.3	230
3-4	121.3	90.55	337.5	305.3	292.6	305.3	300

Story -3

span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	123.6	98.61	337.5	280.4	292.6	280.4	280
2-3	160.45	125.14	337.5	220.9	292.6	220.9	220
3-4	119.5	88.96	337.5	310.8	292.6	310.8	310
Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	35.25	27.67	337.5	999.1	292.6	337.5	330
2-3	34.87	27.33	337.5	1011.5	292.6	337.5	330
3-4	28.76	21.69	337.5	1274.6	292.6	337.5	330


D
1
D
2
D
3
D
4
Roof
G+4
G+3
G+2
G+1
G+0
RefPl 1
Basement
Base
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8.8635
21.9469
17.972
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20.1948
15.288
1.8014
10.4317
11.5812
12.326
Z
X

axis D							
Story -8							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	6.27	4.8	337.5	5759.5	292.6	337.5	330
2-3	7.04	5.52	337.5	5008.3	292.6	337.5	330
3-4	792	6.78	337.5	4077.5	292.6	337.5	330
Story -7							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	106.34	85.77	337.5	322.3	292.6	322.3	320
2-3	104.85	84.31	337.5	327.9	292.6	327.9	320
3-4	80.8	61.76	337.5	447.6	292.6	337.5	330
Story -6							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	105.52	85.1	337.5	324.9	292.6	324.9	320
2-3	104.62	84.26	337.5	328.1	292.6	328.1	320
3-4	67.45	51.59	337.5	535.9	292.6	337.5	330
Story -5							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	163.47	132.65	337.5	208.4	292.6	208.4	200
2-3	157.44	126.55	337.5	218.5	292.6	218.5	210
3-4	136.3	106.22	337.5	260.3	292.6	260.3	260
Story -4							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	152.7	119.49	337.5	231.4	292.6	231.4	230
2-3	173.4	134.19	337.5	206.0	292.6	206.0	200
3-4	141.2	104.62	337.5	264.2	292.6	264.2	260
Story -3							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	86.16	67.52	337.5	409.4	292.6	337.5	330
2-3	111.2	86.68	337.5	318.9	292.6	318.9	310
3-4	87.6	66.34	337.5	416.7	292.6	337.5	330

Story -2							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
1-2	36.74	28.45	337.5	971.7	292.6	337.5	330
2-3	38.4	29.96	337.5	922.8	292.6	337.5	330
3-4	33.4	25.44	337.5	1086.7	292.6	337.5	330

Appendix G: Column design parameter

Table 38. effective length of column

location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
Roof	C1	300	300	300	300	2700	0	5000	5000	0	1.85	1.85	1.85	1.85	2436.08	2436.08
Roof	C2	300	300	300	300	2700	5000	5000	5000	0	0.93	0.93	1.85	1.85	2258.48	2436.08
Roof	C3	300	300	300	300	2700	5000	4250	5000	0	0.85	0.85	1.85	1.85	2233.00	2436.08
Roof	C4	300	300	300	300	2700	4250	0	5000	0	1.57	1.57	1.85	1.85	2399.86	2436.08
Roof	C5	300	300	300	300	2700	4250	0	5800	5000	1.57	1.57	0.99	0.99	2399.86	2279.44
Roof	C6	400	400	300	300	2700	5000	4250	5800	5000	2.69	2.69	3.14	3.14	2506.47	2530.93
Roof	C7	400	400	300	300	2700	5000	5000	5800	5000	2.93	2.93	3.14	3.14	2520.07	2530.93
Roof	C8	300	300	300	300	2700	0	5000	5800	5000	1.85	1.85	0.99	0.99	2436.08	2279.44
Roof	C9	300	300	300	300	2700	0	5000	5400	5800	1.85	1.85	1.04	1.04	2436.08	2291.11
Roof	C10	400	400	300	300	2700	5000	5000	5400	5800	2.93	2.93	3.27	3.27	2520.07	2536.84
Roof	C11	400	400	300	300	2700	5000	4250	5400	5800	2.69	2.69	3.27	3.27	2506.47	2536.84
Roof	C12	300	300	300	300	2700	4250	0	5400	5800	1.57	1.57	1.04	1.04	2399.86	2291.11
Roof	C13	300	300	300	300	2700	4250	0	0	5400	1.57	1.57	2.00	2.00	2399.86	2452.04
Roof	C14	300	300	300	300	2700	5500	5500	0	5400	1.02	1.02	2.00	2.00	2286.32	2452.04
Roof	C15	300	300	300	300	2700	5500	5500	0	5400	1.02	1.02	2.00	2.00	2286.32	2452.04
Roof	C16	300	300	300	300	2700	0	5500	0	5400	2.04	2.04	2.00	2.00	2455.73	2452.04

Table 39check slenderness

locatio	**column**	**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	21.87	8.10	13.70	23.28	15.88	28.13	28.13	142.17	88.02	short	short
Roof	C2	7.742	65.79	7.304	10.09	22.43	8.19	7.75	66.24	10.54	26.08	28.13	163.68	100.12	short	short
Roof	C3	7.861	71.8	4.815	18.88	21.74	8.30	5.25	72.24	19.32	25.78	28.13	167.21	150.66	short	short
Roof	C4	7.157	50.2	7.846	35.14	21.45	7.59	8.28	50.63	35.57	27.71	28.13	164.61	155.81	short	short
Roof	C5	1.878	11.88	6.236	46.17	23.32	2.34	6.70	12.34	46.63	27.71	26.32	153.79	158.50	short	short
Roof	C6	0.911	10.27	1.591	16.87	38.79	1.69	2.37	11.05	17.65	21.71	21.92	162.90	164.86	short	short
Roof	C7	1.42	19.76	6.518	9.318	40.68	2.23	7.33	20.57	10.13	21.82	21.92	163.59	100.37	short	short
Roof	C8	3.18	36.13	17	32.76	27.55	3.73	17.55	36.68	33.31	28.13	26.32	149.75	109.91	short	short
Roof	C9	7.158	6.86	13.39	40.25	23.01	7.32	13.85	7.62	40.71	28.13	26.46	75.78	139.39	short	short
Roof	C10	8.183	9.846	4.331	9.428	40.09	8.99	5.13	10.65	10.23	21.82	21.97	88.66	124.08	short	short
Roof	C11	9.037	9.105	1.593	14.68	39.47	9.83	2.38	9.89	15.47	21.71	21.97	73.77	161.35	short	short
Roof	C12	9.957	6.256	3.364	45.42	21.35	6.68	3.79	10.38	45.85	27.71	26.46	112.42	172.12	short	short
Roof	C13	15.33	25.82	4.328	34.86	23.17	15.80	4.79	26.28	35.33	27.71	28.31	112.28	159.82	short	short
Roof	C14	12.21	33.36	3.903	15.3	21.55	12.64	4.33	33.79	15.73	26.40	28.31	140.46	150.90	short	short
Roof	C15	12.27	34.35	5.431	2.049	21.81	12.70	2.48	34.79	5.87	26.40	28.31	140.58	134.43	short	short
Roof	C16	12.83	26.65	12.61	28.7	21.22	13.26	13.03	27.07	29.12	28.36	28.31	129.22	133.74	short	short

Table 40.column reinforcement

colmn	location	MED	MEDy	Vsd	μsdy	μsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal	shear reinforcment	
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

location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
G+4	C1	400	400	300	500	2500	0	5000	5000	0	1.37	1.37	1.37	1.37	2190.14	2190.14
G+4	C2	400	400	300	500	2500	5000	5000	5000	0	0.68	0.68	1.37	1.37	2003.38	2190.14
G+4	C3	400	400	300	500	2500	5000	4250	5000	0	0.63	0.63	1.37	1.37	1977.87	2190.14
G+4	C4	400	400	300	500	2500	4250	0	5000	0	1.16	1.16	1.37	1.37	2150.74	2190.14
G+4	C5	400	400	300	500	2500	4250	0	5800	5000	1.16	1.16	0.73	0.73	2150.74	2024.61
G+4	C6	400	400	300	500	2500	5000	4250	5800	5000	0.63	0.63	0.73	0.73	1977.87	2024.61
G+4	C7	400	400	300	500	2500	5000	5000	5800	5000	0.68	0.68	0.73	0.73	2003.38	2024.61
G+4	C8	400	400	300	500	2500	0	5000	5800	5000	1.37	1.37	0.73	0.73	2190.14	2024.61
G+4	C9	400	400	300	500	2500	0	5000	5400	5800	1.37	1.37	0.76	0.76	2190.14	2036.51
G+4	C10	400	400	300	500	2500	5000	5000	5400	5800	0.68	0.68	0.76	0.76	2003.38	2036.51
G+4	C11	400	400	300	500	2500	5000	4250	5400	5800	0.63	0.63	0.76	0.76	1977.87	2036.51
G+4	C12	400	400	300	500	2500	4250	0	5400	5800	1.16	1.16	0.76	0.76	2150.74	2036.51
G+4	C13	400	400	300	500	2500	4250	0	0	5400	1.16	1.16	1.47	1.47	2150.74	2207.73
G+4	C14	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+4	C15	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+4	C16	400	400	300	500	2500	0	5500	0	5400	1.50	1.50	1.47	1.47	2211.81	2207.73

locatior	colum	Mxtop	Mxbotm	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slender	slender
G+4	C1	12.4371	51.7719	22.9987	14.7932	124.928	14.94	17.29	54.27	25.50	18.97	18.97	83.58	59.94	short	short
G+4	C2	42.4275	94.2784	12.0093	8.9785	249.803	47.42	13.97	99.27	17.01	17.35	18.97	50.71	36.43	short	short
G+4	C3	43.0431	89.6839	13.3691	38.0242	254.729	48.14	18.46	94.78	43.12	17.13	18.97	48.97	52.25	short	short
G+4	C4	24.9092	44.6646	24.1858	48.7522	149.394	27.90	27.17	47.65	51.74	18.63	18.97	59.79	63.02	short	short
G+4	C5	0.8662	28.4211	30.4003	69.955	255.98	5.99	35.52	33.54	75.07	18.63	17.53	62.36	50.28	short	short
G+4	C6	1.14	35.5514	12.6232	39.686	521.601	11.57	23.06	45.98	50.12	17.13	17.53	41.58	35.60	short	short
G+4	C7	10.8251	45.91	13.3298	6.7187	518.492	21.19	17.09	56.28	23.70	17.35	17.53	38.11	28.19	short	short
G+4	C8	22.8536	46.7561	47.4323	41.0013	225.755	27.37	45.52	51.27	51.95	18.97	17.53	50.89	35.95	short	short
G+4	C9	12.312	7.6799	67.7935	59.5416	277.791	13.24	65.10	17.87	73.35	18.97	17.64	37.74	31.96	short	short
G+4	C10	22.48	13.1482	2.8222	20.0514	530.706	23.76	13.44	33.09	30.67	17.35	17.64	27.95	35.91	short	short
G+4	C11	26.1766	16.6699	4.7781	28.0555	502.093	26.71	14.82	36.22	38.10	17.13	17.64	28.16	38.36	short	short
G+4	C12	16.3581	10.5711	27.8051	66.3249	246.927	15.51	32.74	21.30	71.26	18.63	17.64	40.55	51.76	short	short
G+4	C13	37.5432	29.9065	18.0832	39.3415	144.44	32.80	20.97	40.43	42.23	18.63	19.12	48.49	65.65	short	short
G+4	C14	56.8143	48.9886	3.5167	22.7758	236.11	53.71	8.24	61.54	27.50	17.59	19.12	35.30	59.76	short	short
G+4	C15	58.7994	54.2754	5.4858	4.9741	249.655	59.27	9.97	63.79	10.48	17.59	19.12	31.99	31.07	short	short
G+4	C16	39.2377	35.9474	44.0393	34.4288	155.998	39.07	37.55	42.36	47.16	19.15	19.12	40.83	47.45	short	short

colmn	location	MEDx	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal reinforcment	shear reinforcment	
G+4	C1	54.27	25.50	0.1	0.04	0.07	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+4	C2	99.27	17.01	0.1	0.02	0.14	0.2	1226.4	320	6400	1226.36	20	3.91	4ø20	ø 10 c/c	400
G+4	C3	94.78	43.12	0.1	0.06	0.13	0.2	1226.4	320	6400	1226.36	20	3.91	4ø20	ø 10 c/c	400
G+4	C4	47.65	51.74	0.1	0.07	0.07	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+4	C5	33.54	75.07	0.1	0.10	0.05	0.15	919.77	320	6400	919.77	20	2.93	4ø20	ø 10 c/c	400
G+4	C6	45.98	50.12	0.3	0.07	0.06	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+4	C7	56.28	23.70	0.3	0.03	0.08	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+4	C8	51.27	51.95	0.1	0.07	0.07	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+4	C9	17.87	73.35	0.2	0.10	0.02	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+4	C10	33.09	30.67	0.3	0.04	0.05	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+4	C11	36.22	38.10	0.3	0.05	0.05	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+4	C12	21.30	71.26	0.1	0.10	0.03	0.05	306.59	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+4	C13	40.43	42.23	0.1	0.06	0.06	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+4	C14	61.54	27.50	0.1	0.04	0.08	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+4	C15	63.79	10.48	0.1	0.01	0.09	0.15	919.77	320	6400	919.77	20	2.93	4ø20	ø 10 c/c	400
G+4	C16	42.36	47.16	0.1	0.07	0.06	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400

location	colun	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
G+3	C1	400	400	300	500	2500	0	5000	5000	0	1.37	1.37	1.37	1.37	2190.14	2190.14
G+3	C2	400	400	300	500	2500	5000	5000	5000	0	0.68	0.68	1.37	1.37	2003.38	2190.14
G+3	C3	400	400	300	500	2500	5000	4250	5000	0	0.63	0.63	1.37	1.37	1977.87	2190.14
G+3	C4	400	400	300	500	2500	4250	0	5000	0	1.16	1.16	1.37	1.37	2150.74	2190.14
G+3	C5	400	400	300	500	2500	4250	0	5800	5000	1.16	1.16	0.73	0.73	2150.74	2024.61
G+3	C6	400	400	300	500	2500	5000	4250	5800	5000	0.63	0.63	0.73	0.73	1977.87	2024.61
G+3	C7	400	400	300	500	2500	5000	5000	5800	5000	0.68	0.68	0.73	0.73	2003.38	2024.61
G+3	C8	400	400	300	500	2500	0	5000	5800	5000	1.37	1.37	0.73	0.73	2190.14	2024.61
G+3	C9	400	400	300	500	2500	0	5000	5400	5800	1.37	1.37	0.76	0.76	2190.14	2036.51
G+3	C10	400	400	300	500	2500	5000	5000	5400	5800	0.68	0.68	0.76	0.76	2003.38	2036.51
G+3	C11	400	400	300	500	2500	5000	4250	5400	5800	0.63	0.63	0.76	0.76	1977.87	2036.51
G+3	C12	400	400	300	500	2500	4250	0	5400	5800	1.16	1.16	0.76	0.76	2150.74	2036.51
G+3	C13	400	400	300	500	2500	4250	0	0	5400	1.16	1.16	1.47	1.47	2150.74	2207.73
G+3	C14	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+3	C15	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+3	C16	400	400	300	500	2500	0	5500	0	5400	1.50	1.50	1.47	1.47	2211.81	2207.73

locatior	colum	Mxtop	Mxbotm	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slenderr	slenderr
G+3	C1	7.71	40.7528	28.6765	22.8979	227.849	12.27	27.45	45.31	33.23	18.97	18.97	62.08	37.96	short	short
G+3	C2	32.1668	82.3735	10.7201	4.4475	479.198	41.75	14.03	91.96	20.30	17.35	18.97	37.32	30.22	short	short
G+3	C3	28.9456	76.9937	13.343	42.3538	472.241	38.39	22.79	86.44	51.80	17.13	18.97	37.89	38.02	short	short
G+3	C4	17.4696	86.7369	15.8257	27.1147	266.619	22.80	21.16	92.07	32.45	18.63	18.97	58.32	42.08	short	short
G+3	C5	2.5164	3.2715	18.6593	15.6071	473.706	11.99	25.08	12.75	28.13	18.63	17.53	22.87	24.36	short	short
G+3	C6	4.3185	9.6752	10.9637	53.1738	985.17	24.02	30.67	29.38	72.88	17.13	17.53	18.43	26.72	short	short
G+3	C7	0.9086	3.9904	16.6558	9.7413	990.114	20.71	29.54	23.79	36.46	17.35	17.53	17.29	18.54	slender	short
G+3	C8	10.1146	26.3924	33.5739	17.3876	451.005	19.13	26.41	35.41	42.59	18.97	17.53	35.80	33.35	short	short
G+3	C9	11.0461	1.711	50.9774	22.3414	529.214	12.30	32.93	21.63	61.56	18.97	17.64	32.25	33.21	short	short
G+3	C10	23.1934	3.1423	2.9068	8.4585	1020.77	23.56	23.32	43.61	28.87	17.35	17.64	23.80	18.31	short	short
G+3	C11	35.064	11.5875	4.4783	44.3156	950.415	30.60	23.49	54.07	63.32	17.13	17.64	24.12	28.27	short	short
G+3	C12	23.4959	8.0258	16.371	13.9449	459.846	17.22	23.14	32.69	25.57	18.63	17.64	35.87	24.31	short	short
G+3	C13	26.7855	15.6302	11.3516	28.3156	253.581	20.70	16.42	31.86	33.39	18.63	19.12	43.24	49.74	short	short
G+3	C14	37.623	5.8562	3.8517	27.7665	436.515	14.59	12.58	46.35	36.50	17.59	19.12	43.48	42.53	short	short
G+3	C15	44.703	7.6136	7.3911	10.8242	476.023	17.13	16.91	54.22	20.34	17.59	19.12	41.59	26.11	short	short
G+3	C16	34.8068	22.9126	39.6457	45.2477	291.433	28.74	45.47	40.64	51.08	19.15	19.12	38.13	31.10	short	short

colmn	location	MEDx	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal reinforcment	shear reinforcment	
G+3	C1	45.31	33.23	0.1	0.05	0.06	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+3	C2	91.96	20.30	0.3	0.03	0.13	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+3	C3	86.44	51.80	0.3	0.07	0.12	0.15	919.77	320	6400	919.77	20	2.93	4ø20	ø 10 c/c	400
G+3	C4	92.07	32.45	0.1	0.04	0.13	0.2	1226.4	320	6400	1226.36	20	3.91	4ø20	ø 10 c/c	400
G+3	C5	12.75	28.13	0.3	0.04	0.02	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C6	29.38	72.88	0.5	0.10	0.04	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C7	23.79	36.46	0.5	0.05	0.03	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C8	35.41	42.59	0.2	0.06	0.05	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C9	21.63	61.56	0.3	0.08	0.03	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C10	43.61	28.87	0.6	0.04	0.06	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C11	54.07	63.32	0.5	0.09	0.07	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+3	C12	32.69	25.57	0.3	0.04	0.05	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C13	31.86	33.39	0.1	0.05	0.04	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C14	46.35	36.50	0.2	0.05	0.06	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C15	54.22	20.34	0.3	0.03	0.07	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+3	C16	40.64	51.08	0.2	0.07	0.06	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400

location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
G+2	C1	400	400	300	500	2500	0	5000	5000	0	1.37	1.37	1.37	1.37	2190.14	2190.14
G+2	C2	400	400	300	500	2500	5000	5000	5000	0	0.68	0.68	1.37	1.37	2003.38	2190.14
G+2	C3	400	400	300	500	2500	5000	4250	5000	0	0.63	0.63	1.37	1.37	1977.87	2190.14
G+2	C4	400	400	300	500	2500	4250	0	5000	0	1.16	1.16	1.37	1.37	2150.74	2190.14
G+2	C5	400	400	300	500	2500	4250	0	5800	5000	1.16	1.16	0.73	0.73	2150.74	2024.61
G+2	C6	400	400	300	500	2500	5000	4250	5800	5000	0.63	0.63	0.73	0.73	1977.87	2024.61
G+2	C7	400	400	300	500	2500	5000	5000	5800	5000	0.68	0.68	0.73	0.73	2003.38	2024.61
G+2	C8	400	400	300	500	2500	0	5000	5800	5000	1.37	1.37	0.73	0.73	2190.14	2024.61
G+2	C9	400	400	300	500	2500	0	5000	5400	5800	1.37	1.37	0.76	0.76	2190.14	2036.51
G+2	C10	400	400	300	500	2500	5000	5000	5400	5800	0.68	0.68	0.76	0.76	2003.38	2036.51
G+2	C11	400	400	300	500	2500	5000	4250	5400	5800	0.63	0.63	0.76	0.76	1977.87	2036.51
G+2	C12	400	400	300	500	2500	4250	0	5400	5800	1.16	1.16	0.76	0.76	2150.74	2036.51
G+2	C13	400	400	300	500	2500	4250	0	0	5400	1.16	1.16	1.47	1.47	2150.74	2207.73
G+2	C14	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+2	C15	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+2	C16	400	400	300	500	2500	0	5500	0	5400	1.50	1.50	1.47	1.47	2211.81	2207.73

locatio	colum	Mxtop	Mxbot	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slende	slender
G+2	C1	8.4	40.3	33.9	35.5	331	15.01	40.51	46.94	42.09	18.97	18.97	49.76	26.59	short	short
G+2	C2	27.2	63	9.86	0.68	803	43.28	16.75	79.04	25.93	17.35	18.97	26.66	24.39	short	short
G+2	C3	22.6	56	14.6	41.4	769	38.03	30.01	71.41	56.76	17.13	18.97	27.60	27.69	short	short
G+2	C4	35.2	65.1	5.62	21.7	488	44.96	15.38	74.87	31.47	18.63	18.97	32.63	35.96	short	short
G+2	C5	18.6	4.59	5.32	10.1	804	20.67	21.40	34.70	26.21	18.63	17.53	25.54	20.44	short	short
G+2	C6	18.3	2.93	13.3	44.1	1342	29.77	40.13	45.17	70.94	17.13	17.53	18.63	20.30	short	short
G+2	C7	17.8	4.05	20.9	4.51	1345	30.96	31.41	44.73	47.80	17.35	17.53	18.02	18.64	short	short
G+2	C8	6.14	25	10.4	9.08	652	19.18	22.11	37.99	23.47	18.97	17.53	30.70	19.46	short	short
G+2	C9	7.25	6.42	12.7	5.95	795	22.32	21.85	23.14	28.57	18.97	17.64	17.11	21.76	slende	short
G+2	C10	14.6	4.29	15.5	12	1363	31.55	39.25	41.81	42.73	17.35	17.64	16.79	13.88	slende	slender
G+2	C11	24.7	4.19	6.21	34.1	1288	29.95	31.96	50.41	59.89	17.13	17.64	20.21	21.31	short	short
G+2	C12	21.3	8.02	7.95	9.6	780	23.62	23.55	36.91	25.20	18.63	17.64	24.89	17.97	short	short
G+2	C13	16.8	5.45	4.77	22.9	545	16.34	15.67	27.67	33.75	18.63	19.12	31.17	34.72	short	short
G+2	C14	1.76	7.14	3.43	28.6	777	17.30	18.98	22.68	44.18	17.59	19.12	22.04	29.88	short	short
G+2	C15	2.51	12.4	8.19	8.48	856	19.63	25.31	29.56	25.60	17.59	19.12	23.22	15.94	short	slender
G+2	C16	16.6	6.67	54	34.3	510	16.87	44.54	26.77	64.22	19.15	19.12	31.07	29.22	short	short

colmn	location	MEDx	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal reinforcment	shear reinforcment	
G+2	C1	46.94	42.09	0.2	0.06	0.06	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C2	79.04	25.93	0.4	0.04	0.11	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C3	71.41	56.76	0.4	0.08	0.10	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+2	C4	74.87	31.47	0.3	0.04	0.10	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C5	34.70	26.21	0.4	0.04	0.05	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C6	45.17	70.94	0.7	0.10	0.06	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+2	C7	44.73	47.80	0.7	0.07	0.06	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+2	C8	37.99	23.47	0.4	0.03	0.05	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C9	23.14	28.57	0.4	0.04	0.03	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C10	41.81	42.73	0.8	0.06	0.06	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C11	50.41	59.89	0.7	0.08	0.07	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+2	C12	36.91	25.20	0.4	0.03	0.05	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C13	27.67	33.75	0.3	0.05	0.04	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C14	22.68	44.18	0.4	0.06	0.03	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C15	29.56	25.60	0.5	0.04	0.04	0	0	320	6400	320	20	1.02	4ø20	ø 10 c/c	400
G+2	C16	26.77	64.22	0.3	0.09	0.04	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400

location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
G+1	C1	400	400	300	500	2500	0	5000	5000	0	1.37	1.37	1.37	1.37	2190.14	2190.14
G+1	C2	400	400	300	500	2500	5000	5000	5000	0	0.68	0.68	1.37	1.37	2003.38	2190.14
G+1	C3	400	400	300	500	2500	5000	4250	5000	0	0.63	0.63	1.37	1.37	1977.87	2190.14
G+1	C4	400	400	300	500	2500	4250	0	5000	0	1.16	1.16	1.37	1.37	2150.74	2190.14
G+1	C5	400	400	300	500	2500	4250	0	5800	5000	1.16	1.16	0.73	0.73	2150.74	2024.61
G+1	C6	500	500	300	500	2500	5000	4250	5800	5000	1.53	1.53	1.79	1.79	2216.13	2248.90
G+1	C7	500	500	300	500	2500	5000	5000	5800	5000	1.67	1.67	1.79	1.79	2234.25	2248.90
G+1	C8	400	400	300	500	2500	0	5000	5800	5000	1.37	1.37	0.73	0.73	2190.14	2024.61
G+1	C9	400	400	300	500	2500	0	5000	5400	5800	1.37	1.37	0.76	0.76	2190.14	2036.51
G+1	C10	500	500	300	500	2500	5000	5000	5400	5800	1.67	1.67	1.86	1.86	2234.25	2256.94
G+1	C11	500	500	300	500	2500	5000	4250	5400	5800	1.53	1.53	1.86	1.86	2216.13	2256.94
G+1	C12	400	400	300	500	2500	4250	0	5400	5800	1.16	1.16	0.76	0.76	2150.74	2036.51
G+1	C13	400	400	300	500	2500	4250	0	0	5400	1.16	1.16	1.47	1.47	2150.74	2207.73
G+1	C14	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+1	C15	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+1	C16	400	400	300	500	2500	0	5500	0	5400	1.50	1.50	1.47	1.47	2211.81	2207.73

locatio	column	Mxtop	Mxbot	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slender	slender
G+1	C1	7.906	43.81	35.94	46.89	429.7	16.50	44.54	52.40	55.49	18.97	18.97	43.81	28.38	short	short
G+1	C2	54.15	119.3	23.06	8.759	1157	77.28	31.89	142.39	46.20	17.35	18.97	22.31	19.47	short	short
G+1	C3	45.47	105	24.63	69.7	1088	67.22	46.38	126.78	91.45	17.13	18.97	23.26	23.71	short	short
G+1	C4	44.34	104.5	9.031	30.12	691	58.16	22.85	118.33	43.94	18.63	18.97	30.14	29.43	short	short
G+1	C5	5.032	18.01	2.433	21.3	1126	27.54	24.94	40.52	43.81	18.63	17.53	19.94	22.10	short	short
G+1	C6	8.638	13.82	13.1	69.34	1732	43.27	47.73	48.45	103.97	15.35	15.58	15.89	24.44	short	short
G+1	C7	12.29	0.313	25.69	7.798	1748	35.28	42.77	47.26	60.66	15.48	15.58	18.69	19.50	short	short
G+1	C8	16.92	52.19	42.07	22.03	885.4	34.63	39.74	69.89	59.78	18.97	17.53	26.54	22.81	short	short
G+1	C9	27.7	11.99	39.83	17.62	1072	33.43	39.06	49.15	61.28	18.97	17.64	20.42	21.28	short	short
G+1	C10	23.2	6.236	17.97	3.56	1734	40.92	38.25	57.89	52.66	15.48	15.64	19.54	19.16	short	short
G+1	C11	18.24	3.99	3.238	49.7	1678	37.55	36.80	51.80	83.27	15.35	15.64	19.51	25.17	short	short
G+1	C12	32.03	9.161	1.162	17.79	1132	31.81	23.81	54.68	40.44	18.63	17.64	21.79	21.65	short	short
G+1	C13	25.23	53.3	11.84	26.13	846.2	42.16	28.77	70.22	43.05	18.63	19.12	24.79	23.26	short	short
G+1	C14	15.89	49.33	9.253	41.51	1152	38.93	32.30	72.37	64.56	17.59	19.12	22.45	23.17	short	short
G+1	C15	4.642	41.97	20.81	4.852	1238	29.41	29.62	66.73	45.57	17.59	19.12	23.47	19.57	short	short
G+1	C16	18.35	37.24	67.39	27.34	714.7	32.64	41.63	51.53	81.68	19.15	19.12	26.16	29.19	short	short

colmn	location	MEDx	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal	shear reinforcment	
G+1	C1	52.40	55.49	0.2	0.08	0.07	0.15	919.8	320	6400	919.8	20	2.93	4ø20	ø 10 c/c	400
G+1	C2	142.39	46.20	0.6	0.06	0.20	0.4	2453	320	6400	2453	20	7.81	4ø20	ø 10 c/c	400
G+1	C3	126.78	91.45	0.6	0.13	0.17	0.4	2453	320	6400	2453	20	7.81	4ø20	ø 10 c/c	400
G+1	C4	118.33	43.94	0.4	0.06	0.16	0.4	2453	320	6400	2453	20	7.81	4ø20	ø 10 c/c	400
G+1	C5	40.52	43.81	0.6	0.06	0.06	0.15	919.8	320	6400	919.8	20	2.93	4ø20	ø 10 c/c	400
G+1	C6	48.45	103.97	0.6	0.07	0.03	0	0	500	10000	500	20	1.59	4ø20	ø 10 c/c	400
G+1	C7	47.26	60.66	0.6	0.04	0.03	0	0	500	10000	500	20	1.59	4ø20	ø 10 c/c	400
G+1	C8	69.89	59.78	0.5	0.08	0.10	0.1	613.2	320	6400	613.2	20	1.95	4ø20	ø 10 c/c	400
G+1	C9	49.15	61.28	0.6	0.08	0.07	0.15	919.8	320	6400	919.8	20	2.93	4ø20	ø 10 c/c	400
G+1	C10	57.89	52.66	0.6	0.04	0.04	0	0	500	10000	500	20	1.59	4ø20	ø 10 c/c	400
G+1	C11	51.80	83.27	0.6	0.06	0.04	0	0	500	10000	500	20	1.59	4ø20	ø 10 c/c	400
G+1	C12	54.68	40.44	0.6	0.06	0.08	0.15	919.8	320	6400	919.8	20	2.93	4ø20	ø 10 c/c	400
G+1	C13	70.22	43.05	0.5	0.06	0.10	0.1	613.2	320	6400	613.2	20	1.95	4ø20	ø 10 c/c	400
G+1	C14	72.37	64.56	0.6	0.09	0.10	0.15	919.8	320	6400	919.8	20	2.93	4ø20	ø 10 c/c	400
G+1	C15	66.73	45.57	0.7	0.06	0.09	0.2	1226	320	6400	1226	20	3.91	4ø20	ø 10 c/c	400
G+1	C16	51.53	81.68	0.4	0.11	0.07	0.1	613.2	320	6400	613.2	20	1.95	4ø20	ø 10 c/c	400

location	column	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
G+0	C1	400	400	300	500	2500	0	5000	5000	0	1.37	1.37	1.37	1.37	2190.14	2190.14
G+0	C2	400	400	300	500	2500	5000	5000	5000	0	0.68	0.68	1.37	1.37	2003.38	2190.14
G+0	C3	400	400	300	500	2500	5000	4250	5000	0	0.63	0.63	1.37	1.37	1977.87	2190.14
G+0	C4	400	400	300	500	2500	4250	0	5000	0	1.16	1.16	1.37	1.37	2150.74	2190.14
G+0	C5	400	400	300	500	2500	4250	0	5800	5000	1.16	1.16	0.73	0.73	2150.74	2024.61
G+0	C6	500	500	300	500	2500	5000	4250	5800	5000	1.53	1.53	1.79	1.79	2216.13	2248.90
G+0	C7	500	500	300	500	2500	5000	5000	5800	5000	1.67	1.67	1.79	1.79	2234.25	2248.90
G+0	C8	400	400	300	500	2500	0	5000	5800	5000	1.37	1.37	0.73	0.73	2190.14	2024.61
G+0	C9	400	400	300	500	2500	0	5000	5400	5800	1.37	1.37	0.76	0.76	2190.14	2036.51
G+0	C10	500	500	300	500	2500	5000	5000	5400	5800	1.67	1.67	1.86	1.86	2234.25	2256.94
G+0	C11	500	500	300	500	2500	5000	4250	5400	5800	1.53	1.53	1.86	1.86	2216.13	2256.94
G+0	C12	400	400	300	500	2500	4250	0	5400	5800	1.16	1.16	0.76	0.76	2150.74	2036.51
G+0	C13	400	400	300	500	2500	4250	0	0	5400	1.16	1.16	1.47	1.47	2150.74	2207.73
G+0	C14	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+0	C15	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
G+0	C16	400	400	300	500	2500	0	5500	0	5400	1.50	1.50	1.47	1.47	2211.81	2207.73

locatio	colum	Mxtop	Mxbotm	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slender	slender
G+0	C1	16.8657	52.642	47.0282	31.3873	536.012	27.59	42.11	63.36	57.75	18.97	18.97	35.82	27.50	short	short
G+0	C2	36.2032	47.3813	19.5063	10.1527	1509.68	66.40	40.35	77.57	49.70	17.35	18.97	14.24	14.99	slender	slender
G+0	C3	30.5301	42.0938	11.0145	29.0376	1408.13	58.69	39.18	70.26	57.20	17.13	18.97	15.11	17.74	slender	slender
G+0	C4	32.3336	43.537	2.0515	24.7965	893.212	50.20	19.92	61.40	42.66	18.63	18.97	19.36	27.05	short	short
G+0	C5	6.1099	22.8518	3.1268	20.3531	1444.57	35.00	32.02	51.74	49.24	18.63	17.53	17.66	18.11	slender	short
G+0	C6	2.3846	24.2077	7.993	31.7216	2137.89	45.14	50.75	66.97	74.48	15.35	15.58	18.18	18.06	short	short
G+0	C7	6.8716	15.3106	30.3822	16.1688	2164.6	50.16	59.46	58.60	73.67	15.48	15.58	14.87	15.73	slender	short
G+0	C8	6.6503	19.5786	25.3051	11.8361	1113.69	28.92	34.11	41.85	47.58	18.97	17.53	19.82	19.32	short	short
G+0	C9	10.289	4.4454	27.0389	5.4891	1356.03	31.57	32.61	37.41	54.16	18.97	17.64	15.25	19.55	slender	short
G+0	C10	11.0662	9.8846	23.8894	20.9408	2135.85	52.60	63.66	53.78	66.61	15.48	15.64	12.80	13.20	slender	slender
G+0	C11	14.3597	14.5622	1.9122	33.3093	2097.5	56.31	43.86	56.51	75.26	15.35	15.64	12.59	19.99	slender	short
G+0	C12	17.5444	11.0671	4.0873	21.6113	1492.33	40.91	33.93	47.39	51.46	18.63	17.64	14.20	17.66	slender	short
G+0	C13	57.3312	15.8423	1.355	25.3061	1030.09	36.44	21.96	77.93	45.91	18.63	19.12	25.18	24.96	short	short
G+0	C14	59.4077	16.2495	2.1155	28.6303	1369.95	43.65	29.51	86.81	56.03	17.59	19.12	21.21	20.78	short	short
G+0	C15	58.8089	18.3116	16.7528	16.9714	1449.81	47.31	45.75	87.81	45.97	17.59	19.12	20.00	12.14	short	slender
G+0	C16	41.9632	17.5592	20.1266	4.3408	846.962	34.50	21.28	58.90	37.07	19.15	19.12	25.11	25.37	short	short

colmn	location	MEDx	MEDy	Vsd	µsdy	µsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal reinforcment	shear reinforcment	
G+0	C1	63.36	57.75	0.3	0.08	0.09	0.15	919.77	320	6400	919.77	20	2.93	4ø20	ø 10 c/c	400
G+0	C2	77.57	49.70	0.8	0.07	0.11	0.25	1533	320	6400	1532.95	20	4.88	4ø20	ø 10 c/c	400
G+0	C3	70.26	57.20	0.8	0.08	0.10	0.25	1533	320	6400	1532.95	20	4.88	4ø20	ø 10 c/c	400
G+0	C4	61.40	42.66	0.5	0.06	0.08	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400
G+0	C5	51.74	49.24	0.8	0.07	0.07	0.25	1533	320	6400	1532.95	20	4.88	4ø20	ø 10 c/c	400
G+0	C6	66.97	74.48	0.8	0.05	0.05	0.1	958.09	500	10000	958.094	20	3.05	4ø20	ø 10 c/c	400
G+0	C7	58.60	73.67	0.8	0.05	0.04	0.1	958.09	500	10000	958.094	20	3.05	4ø20	ø 10 c/c	400
G+0	C8	41.85	47.58	0.6	0.07	0.06	0.15	919.77	320	6400	919.77	20	2.93	4ø20	ø 10 c/c	400
G+0	C9	37.41	54.16	0.7	0.07	0.05	0.2	1226.4	320	6400	1226.36	20	3.91	4ø20	ø 10 c/c	400
G+0	C10	53.78	66.61	0.8	0.05	0.04	0	0	500	10000	500	20	1.59	4ø20	ø 10 c/c	400
G+0	C11	56.51	75.26	0.7	0.05	0.04	0	0	500	10000	500	20	1.59	4ø20	ø 10 c/c	400
G+0	C12	47.39	51.46	0.8	0.07	0.07	0.25	1533	320	6400	1532.95	20	4.88	4ø20	ø 10 c/c	400
G+0	C13	77.93	45.91	0.6	0.06	0.11	0.15	919.77	320	6400	919.77	20	2.93	4ø20	ø 10 c/c	400
G+0	C14	86.81	56.03	0.8	0.08	0.12	0.25	1533	320	6400	1532.95	20	4.88	4ø20	ø 10 c/c	400
G+0	C15	87.81	45.97	0.8	0.06	0.12	0.25	1533	320	6400	1532.95	20	4.88	4ø20	ø 10 c/c	400
G+0	C16	58.90	37.07	0.5	0.05	0.08	0.1	613.18	320	6400	613.18	20	1.95	4ø20	ø 10 c/c	400

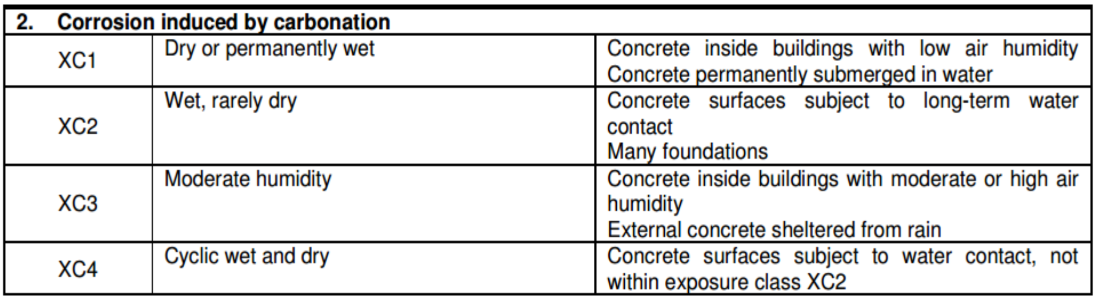
location	colum	bc	hc	bb	hb	Lc	lbxl	lbxr	lbyl	lbyr	k1x	k2x	k1y	k2y	Lox	Loy
Basement	C1	400	400	300	500	2500	0	5000	5000	0	1.37	1.37	1.37	1.37	2190.14	2190.14
Basement	C2	400	400	300	500	2500	5000	5000	5000	0	0.68	0.68	1.37	1.37	2003.38	2190.14
Basement	C3	400	400	300	500	2500	5000	4250	5000	0	0.63	0.63	1.37	1.37	1977.87	2190.14
Basement	C4	400	400	300	500	2500	4250	0	5000	0	1.16	1.16	1.37	1.37	2150.74	2190.14
Basement	C5	400	400	300	500	2500	4250	0	5800	5000	1.16	1.16	0.73	0.73	2150.74	2024.61
Basement	C6	500	500	300	500	2500	5000	4250	5800	5000	1.53	1.53	1.79	1.79	2216.13	2248.90
Basement	C7	500	500	300	500	2500	5000	5000	5800	5000	1.67	1.67	1.79	1.79	2234.25	2248.90
Basement	C8	400	400	300	500	2500	0	5000	5800	5000	1.37	1.37	0.73	0.73	2190.14	2024.61
Basement	C9	400	400	300	500	2500	0	5000	5400	5800	1.37	1.37	0.76	0.76	2190.14	2036.51
Basement	C10	500	500	300	500	2500	5000	5000	5400	5800	1.67	1.67	1.86	1.86	2234.25	2256.94
Basement	C11	500	500	300	500	2500	5000	4250	5400	5800	1.53	1.53	1.86	1.86	2216.13	2256.94
Basement	C12	400	400	300	500	2500	4250	0	5400	5800	1.16	1.16	0.76	0.76	2150.74	2036.51
Basement	C13	400	400	300	500	2500	4250	0	0	5400	1.16	1.16	1.47	1.47	2150.74	2207.73
Basement	C14	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
Basement	C15	400	400	300	500	2500	5500	5500	0	5400	0.75	0.75	1.47	1.47	2031.61	2207.73
Basement	C16	400	400	300	500	2500	0	5500	0	5400	1.50	1.50	1.47	1.47	2211.81	2207.73

location	column	Mxtop	Mxbotn	Mytop	Mybot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slenderr	slenderr
Baseme	C1	2.2315	19.25	29.14	5.543	611.6	14.46	17.77	31.48	41.37	18.97	18.97	32.89	33.68	short	short
Baseme	C2	1.0237	23.4	7.713	27.35	1613	33.28	39.97	55.66	59.61	17.35	18.97	17.99	16.81	short	slender
Baseme	C3	0.0009	22.59	7.322	26.74	1499	29.99	37.31	52.57	56.73	17.13	18.97	19.13	17.65	short	slender
Baseme	C4	2.055	22.04	1.981	28.63	960.9	21.27	21.20	41.26	47.84	18.63	18.97	25.05	26.59	short	short
Baseme	C5	4.1598	22.54	1.612	26.5	1541	34.99	32.44	53.37	57.32	18.63	17.53	17.44	18.94	slender	short
Baseme	C6	7.2799	40.77	10.91	44.13	2273	52.75	56.38	86.24	89.60	15.35	15.58	18.71	18.41	short	short
Baseme	C7	11.573	43.02	4.596	47.99	2316	57.90	50.92	89.34	94.32	15.48	15.58	17.91	19.76	short	short
Baseme	C8	21.642	23.74	12.07	16.21	1229	46.23	36.65	48.33	40.80	18.97	17.53	13.90	14.99	slender	slender
Baseme	C9	11.299	22.62	10.38	15.68	1455	40.41	39.48	51.72	44.79	18.97	17.64	15.79	14.07	slender	slender
Baseme	C10	13.64	41.25	3.539	42.62	2274	59.13	49.03	86.74	88.11	15.48	15.64	17.50	19.65	short	short
Baseme	C11	10.222	39.34	8.154	40.49	2230	54.83	52.76	83.95	85.10	15.35	15.64	18.17	18.74	short	short
Baseme	C12	9.0477	19.77	2.226	24.24	1585	40.74	33.92	51.46	55.93	18.63	17.64	14.96	18.01	slender	short
Baseme	C13	5.1465	17.97	3.728	22.75	1091	26.97	25.55	39.79	44.57	18.63	19.12	20.29	22.37	short	short
Baseme	C14	5.3552	19.59	4.154	21.52	1459	34.54	33.34	48.77	50.70	17.59	19.12	17.02	17.90	slender	slender
Baseme	C15	7.3558	20.37	4.964	20.5	1545	38.25	35.86	51.26	51.39	17.59	19.12	15.91	16.72	slender	slender
Baseme	C16	11.987	19.04	12.59	12.46	913.2	30.25	30.72	37.31	30.86	19.15	19.12	19.29	15.28	short	slender

colmn	location	MEDx	MEDy	Vsd	μsdy	μsdx	ω	As	As,min	As,max	As,prov	ø	no of bar	longtudinal	shear reinforcment	
Baseme	C1	31.48	41.37	0.3	0.06	0.04	0	0	320	6400	320	20	1.02	4ø20	10 c/c	400
Baseme	C2	55.66	59.61	0.9	0.08	0.08	0.3	1840	320	6400	1840	20	5.86	4ø20	10 c/c	400
Baseme	C3	52.57	56.73	0.8	0.08	0.07	0.25	1533	320	6400	1533	20	4.88	4ø20	10 c/c	400
Baseme	C4	41.26	47.84	0.5	0.07	0.06	0.1	613.2	320	6400	613.2	20	1.95	4ø20	10 c/c	400
Baseme	C5	53.37	57.32	0.9	0.08	0.07	0.3	1840	320	6400	1840	20	5.86	4ø20	10 c/c	400
Baseme	C6	86.24	89.60	0.8	0.06	0.06	0.25	2395	500	10000	2395	20	7.63	4ø20	10 c/c	400
Baseme	C7	89.34	94.32	0.8	0.07	0.06	0.25	2395	500	10000	2395	20	7.63	4ø20	10 c/c	400
Baseme	C8	48.33	40.80	0.7	0.06	0.07	0.2	1226	320	6400	1226	20	3.91	4ø20	10 c/c	400
Baseme	C9	51.72	44.79	0.8	0.06	0.07	0.25	1533	320	6400	1533	20	4.88	4ø20	10 c/c	400
Baseme	C10	86.74	88.11	0.8	0.06	0.06	0.25	2395	500	10000	2395	20	7.63	4ø20	10 c/c	400
Baseme	C11	83.95	85.10	0.8	0.06	0.06	0.25	2395	500	10000	2395	20	7.63	4ø20	10 c/c	400
Baseme	C12	51.46	55.93	0.9	0.08	0.07	0.3	1840	320	6400	1840	20	5.86	4ø20	10 c/c	400
Baseme	C13	39.79	44.57	0.6	0.06	0.05	0.15	919.8	320	6400	919.8	20	2.93	4ø20	10 c/c	400
Baseme	C14	48.77	50.70	0.8	0.07	0.07	0.25	1533	320	6400	1533	20	4.88	4ø20	10 c/c	400
Baseme	C15	51.26	51.39	0.9	0.07	0.07	0.3	1840	320	6400	1840	20	5.86	4ø20	10 c/c	400
Baseme	C16	37.31	30.86	0.5	0.04	0.05	0	0	320	6400	320	20	1.02	4ø20	10 c/c	400

Appendix H: Exposure classes related to environmental conditions in accordance with ESEN 206-1

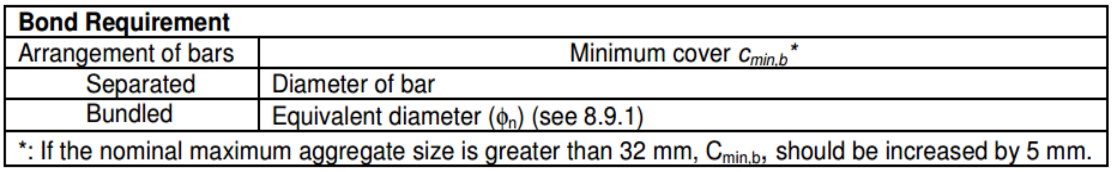
Table 41Exposure classes related to environmental conditions in accordance with ESEN 206-1



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

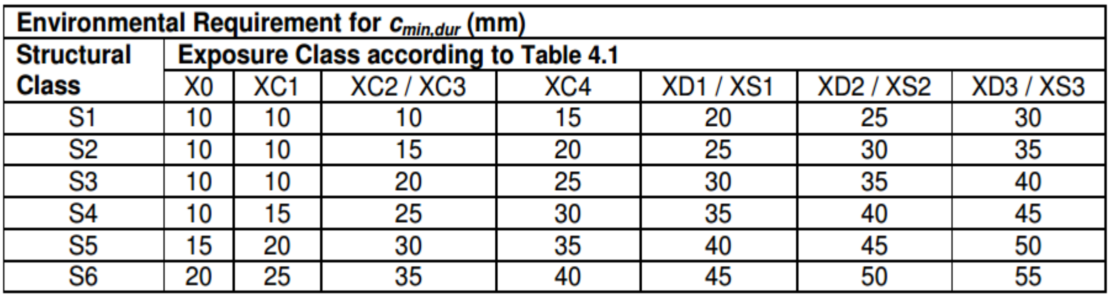
Table 42Minimum cover, cmin,b,requirements with regard to bond



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.

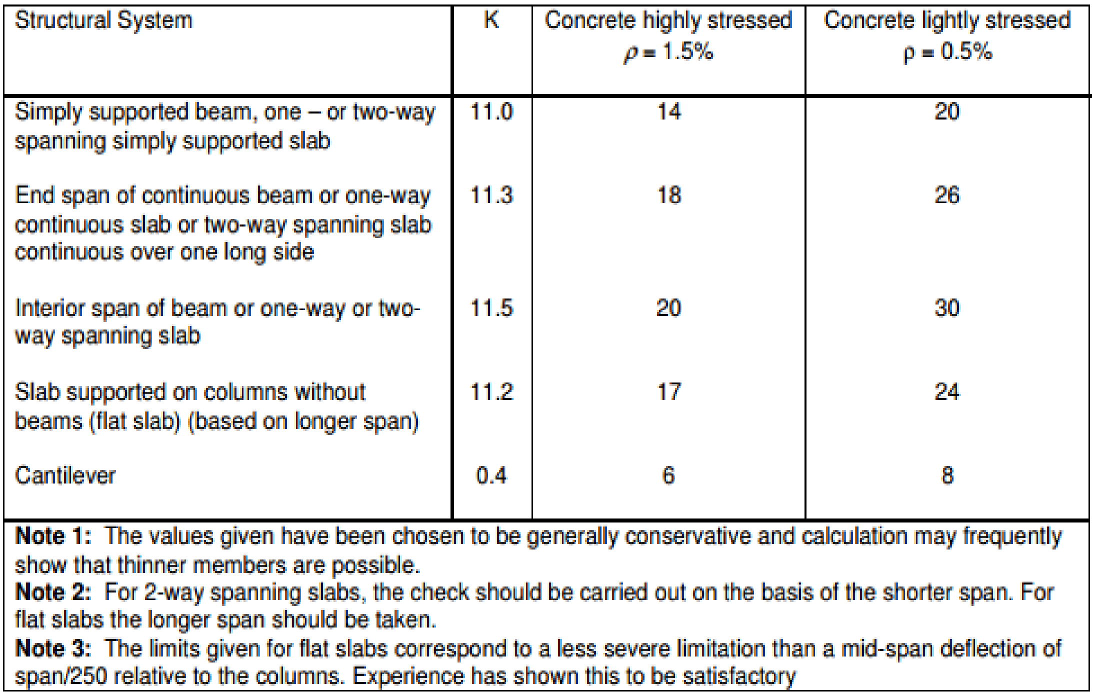
Table 43Values of minimum cover, cmin,dur, requirements



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 44K factor for design of beam (EBCS 2, 2015)

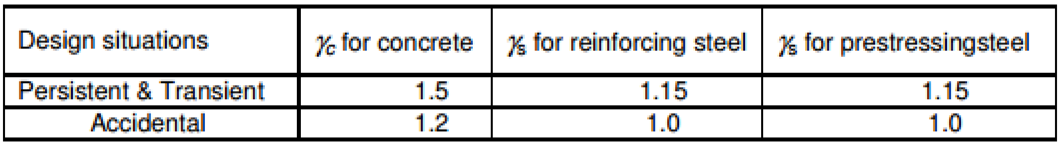


Structural System	K	Concrete highly stressed $\rho = 1.5\%$	Concrete lightly stressed $\rho = 0.5\%$
Simply supported beam, one – or two-way spanning simply supported slab	11.0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	11.3	18	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*, 2015b)(2.4.2.4)

Table 45partial 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