

Wolkite University

College of Engineering and Technology

Civil Engineering Department

Structural design of of G+5 Hotel Building in Addis Ababa

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Date; December, 25,2020

DECLARATION

We declare that the project work entitled as “Structural design of a G+5 hotel building” is our own original work carried out at “Wolkite University” under the guidance of Instructor Desalegn during the whole project time.

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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 **Bruktawit Kassu, Senait Worku, Fitsum Takele, YordanosWele, Simret Mesfin**entitled **Structural Sesign of G+5 Hotel Building in Addis Ababa**, and recommended for acceptance as a fulfillment of the requirement of **B.Sc. in Civil Engineering**

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Preface

This document is a report as a part of fulfillment for a Bachelor of Science in Civil Engineering. The project is conducted by Bruktawit Kassu, Senait Worku, Fitsum Takele, YordanosWele, Simret Mesfin members.

Acknowledgement

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EXECUTIVE SUMMARY

This project is about the structural design and analysis of G+5 hotel building considering all the effects that the building receives in its service life. The project is located in Addis Ababa city at 2355m above sea level.

Structural design involves proportioning of structural members of the building and providing reinforcement which results economical.

In short, the main structural designs included the paper is as follows.

The roof is a flat RC roof. It design deals with analysis of the external forces that the roof is exposed then checked for their 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 dog logged 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 load analysis. We have calculated the base shear and distributed to the frames according to their stiffness by finding the frame shears. The frame was analyzed using ETABS 2015.

After generating the analysis result, we designed beams and columns by assuming a preliminary cross sections 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 the elevator.

An isolated footing type is selected for our foundation. The design was made by taking the loads coming from the super structure through columns..

Finally, all the structural members are designed as per the specification of Ethiopian Building code of Standards, EBCS and we used limit state design method.

Throughout the whole process, different software will be used such as:

- MS – office (word, excel,)
- Analysis & design software (ETABS & SAP)
- Management software (MS – project)

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SYMBOL

A_c: Gross Area of concrete
A_s: Area of tensile reinforcement bar
A_s: Area of compressive reinforcement bar
b, h: Dimension of rectangular section (width and depth respectively)
CALT: Altitude factor
C_d: Dynamic coefficient
CDIR: Direction factor
C_e: Exposure coefficient
C_p: Pressure coefficient
C_r: Roughness coefficient
C_t: Topographic coefficient
CTEM: Temperature coefficient
d: Effective depth
E: Modulus of elasticity
E_c: Modulus of elasticity of concrete
ε_c: Maximum compressive strain of concrete
EBCS: Ethiopian Building code of standard
E_s: Modulus of elasticity of steel
f_{cd}: Design Compressive strength of concrete in compression
f_{ck}: Characteristic cylindrical compressive strength of concrete
f_{ctd}: Tensile strength of concrete
f_{yd}: Design yield strength of Reinforcement Bar in Tension and Compression
f_{yk}: Characteristic strength of Reinforcement Bar in Tension
HCB: Hollow Concrete Block
I: Second Moment of Inertia
K: Equivalent roughness
KT: Terrain factor
λ: Slenderness ratio
ϕ: Diameter of Reinforcement Bar
Pd: Design value
qref: Reference mean velocity pressure
r: Radius of gyration
RC: Reinforced Concrete
t: Plate thickness
Vm: Mean wind velocity
V_{ref}: Relative wind velocity factor
γ : Unit Weight
Z: Height above the ground
Z_{min}: Minimum height
Z_o: Roughness length
γ_c: Partial safety factor of concrete

γ_s: Partial safety factor of steel
ρ: Density
W_e: external pressure
C_{pe}: external pressure coefficient
C_{pi}: internal pressure coefficient
Qk: live load
G_k: dead load
F_b: base shear
L_y: long dimension
L_x: short direction
M_{ys}: support moment in the longer direction of panel
M_{xs}: support moment in the shorter direction of panel
M_{xf}: field moment in the shorter direction of the panel
M_{yf}: field moment in the longer direction of panel

CHAPTER ONE

1. INTRODUCTION

Project Back Ground and Technical Information

The aim of this project is, to examine the integrated techniques used in the design of abuilding. Now a day's it has being practical to choose types of structural members for different criteria especially economy after assuring safety. This project deals about the structural analysis and design of **G+5 hotel building** considering all the external effects according to EBCS, 1995. The structural design of this typical building involves design flat roof slab, floor slabs, stairs, beams, columns, foundation and analysis of frames for lateral loads. The building is composed of solid slab for the floors. This structural design is executed based on the Ethiopian Building Code of Standard (EBCS) prepared in 1995 E.C. This code follows the Limit State design approach. Limit state is a state beyond which the structure no longer satisfies the design performance requirements. Its main concern is the safety of structure and people.

Serviceability Limit states are those associated to conditions beyond which a structure does not accomplish specified service requirements. It is mainly concerned about the function of construction works, comfort of people, and appearance. The design of any structure is categorized in to functional design and structural design.The intended building for this project is located in Adidis ababa. The town is placed under a categoryIII and Seismic zone 3 according to EBCS-. Hence, analysis for the wind load becomes vital and the building shall be wind pressure load resistant, which provides an adequate energy dissipation capacity to the structure without substantial reduction of its overall resistance against external and internal wind pressure. The overall design of a building includes design of roof, design of slab, design of beam, design of stair, design of column, frame analysis, and design of foundation. The analysis will be compute using applicable software ETAB 2016

General Information about Project

The building under consideration has the following information

- ✓ Height of building is 20m and Height of foundation column is 1.50m.
- ✓ Thickness of slab is 210 mm from calculated value.
- ✓ Roof type is flat roof.
- ✓ Building is located in Addis Ababa (seismic region zone 3) and it has 2355m altitude.

- ✓ The total area of the building is 401.5m^2.

General objective

- ✓ The main objective of the project is to carry out a structural design and analysis of a G+5 hotel building.

Specific Objective

This Project has wide objectives, which are based on knowledge. This project has an objective:

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

Basic structural components

All structures are composed of a number of inter connected elements such as; slabs, beams, columns, walls and foundations. Collectively, they enable the internal and external loads acting on the structure to be safely transmitted down to the ground. The actual way that this is achieved is difficult to model and many simplifying, but conservative assumptions have to be made.

Lay out of the report

Our given project is G+5 hotel building The structural design is including; design of roof, design of slab, design of structural frame such as design of beam and column, design of foundation. Limit state design method has been adopted for the whole components. Ethiopian building code of standards EBCS-1, and EBCS-2 are referred for the design of the building. We have also analyzed the building using ETABS.

Design Data and Material Property specification

The Building is hotel building and it has the following material property.

Concrete: Class I work

- ✓ Unit weight of concrete, ϒc = 25KN/m^3
- ✓ Safety factor for concrete γs=1.5…… EBCS-2, 1995

For C-25fck =0.8*25Mpa=20Mpa

fctk = 0.21*$(fck)^{2/3}$=0.21$(20)^{2/3}$ = 1.547Mpa

fcd =$0.85 * \frac{20}{1.5}$= 11.33Mpa

fctd= $\frac{1.547}{1.5}$Mpa= 1.03Mpa

For C-3Ofck =0.8*30Mpa=24Mpa

fcd =$0.85 * \frac{24}{1.5}$= 13.6Mpa

Reinforcement:

Steel S-500

- ✓ Partial safety factor for concrete γs= 1.15`
- ✓ Characteristic strength, f_{yk}=500 Mpa
- ✓ Design strength, fyd $= \frac{fyk}{\Upsilon s} = \frac{500}{1.15} =$ 434.78Mpa

Partial safety factors for load

- ✓ 1.35 for dead load
- ✓ 1.50 for live load

2. WIND LOAD ANALYSIS AND DESIGN OF ROOF

Wind load Analysis of Roof

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

A roof consists of the following:

- Roof cover – prevents any kind of weather disturbance & serves as a protecting sheet for the building. In our case, it is concrete covering roof. Therefore, the roof design comprises determination of different loadings acting on the roof, checking whether the roof cover can carry those loads or not, choosing the appropriate effective depth and determining its dimension and designing of the roof cover.

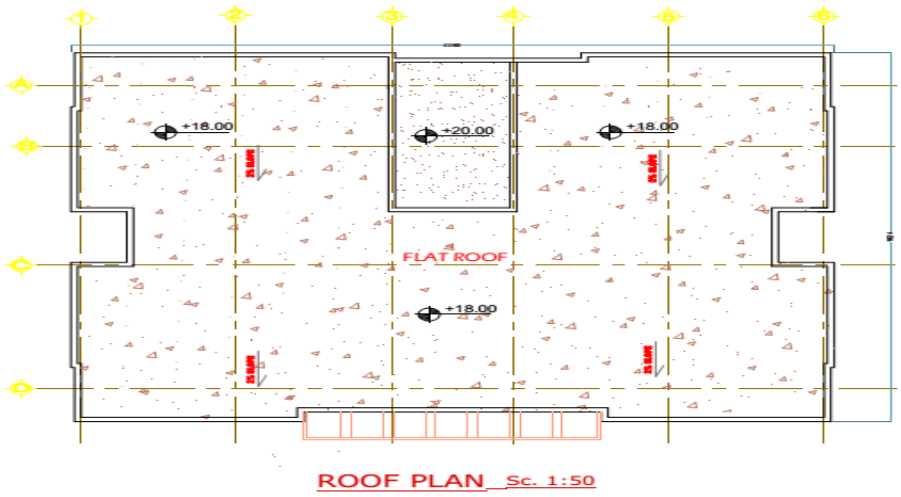


Figure 2-1 Roof floor plan

Analysis Information

The building under consideration has the following properties: -

- The height of building above the ground level is 20m including slab thickness.
- The building roof is Flat roof from the given.
- Building is located in Addis Ababa, which is seismic region 3.
- Addis Ababa is Category III based on terrain category as set in ES EN.
- Addis Ababa altitude is 2355m above sea level.
- Width and length of the buildings is 22m and 18.25m respectively taken from floor plan.

- Height of parapet of the roof=0.8m
- Thickness of RC solid slab=210mm
- Thickness of Water proof material(bitumen)=20mm
- Thickness of lightweight concrete=50mm
- Thickness of cement screed=30mm
- Thickness of Plastering Cielling=30mm

Types of loads acting on roof surface

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

- Pd = 1.35LL+1.5LL+0.9WL

Where: Pd - design load

DL - total dead load on slab

LL - total live load on slab

WL - total wind load on slab

i. Live load

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

ii. **Dead load(DL)**

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

iii. Wind load (W.L)

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

Wind pressure

A. External wind pressure (We)

It is the wind pressure acting on the external surfaces of a structure.

B. Internal wind pressure(Wi)

It is the wind pressure acting on the internal surfaces of a structure.

- Wind load = $q_p(Z)C_P$
  - Wnet = We-Wi = $q_p(Z)C_Pe - q_p(Z)C_Pi$
  - Wnet = $q_p(Z)(C_{Pe} - C_{Pi})$

Where

Wnet: - Net wind pressure load

We: - External wind Pressure

Wi: - Internal wind Pressure

$q_p(Z)$: - is the Peck velocity Pressure

External wind Pressure on the roof floor

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

- Wex=$q_p(Z)C_{Pe}$……EBCS-1, 2015

Cpi:-is the internal pressure coefficient

Cpe- is the external pressure coefficient

Determination of $q_p(Z)$ =Peck velocity Pressure

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

Where:

$I_v(z)$ - Turbulence intensity

ρair- Air density

$V_m (z)$ -Mean wind velocity

A. Turbulence intensity($I_v(z)$)

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

Where: -V_m (z) -Mean wind velocity

σv – Standard deviation of Turbulence

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

Table 2-1Terrain Categories and Related Parameters

Category	Terrain classification	Zo(m)	Zmin
0	Sea or coastal area exposed to the open sea	0.003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0.01	2
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle height	0.05	4
III	Area with regular cover of vegetation or buildings or with Isolated obstacles with separations of maximum 20 obstacle heights(such as villages, suburban terrain, permanent forest)	0.3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1	10

We use this formula $I_v(z)= \frac{\sigma v}{V_m(Z)}$, b/c Zmin< Z <Zmax 10<20<200 for roof 1 and 10<18<200

B. Mean wind velocity V_m (z)

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

Where: $C_r(Z)$-Roughness factor

$C_o(Z)$- Terrian Topography

V_b-basic wind velocity

$V_b = C_{dir}C_{season}V_{b,o}$,Where, C_{dir} =Directional factor =1.0

C_{season} =Seasonal factor =1.0

$V_{b,o}$=Fundamental value of basic wind velocity =22 m/s

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

=1*1*22m/s=**22m/s**

C. Roughness factor ($C_r(Z)$)

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

- ✓ $C_r(Z) = K_r \ln(\frac{Z}{Zo})$, for $Zmin \leq Z \leq Zmax$

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

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

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

=**0.2343**

Case 1 For roof 1(h=18m)

$C_r(Z)$=0.2343ln(18/1)=**0.677**

Case 2 For roof 2(h=20m)

$C_r(Z)$=.0234ln(20/1)=**0.701**

D. Terrain orography ($C_o(Z)$): Based on EBCS-1, 2015 Art 3.8.4 (Page 58) defined by

Ct (z) = 1 + 2sΦ for 0.05 < Φ 0.3

Ct (z) = 1 + 0.6s for Φ > 0.3

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

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

Ct (z) = 1 for Φ < 0.05

After finding terrain orography we can calculate Vm

- ➢ $V_m(z) = C_r(Z)C_o(Z)V_b$

=0.677*1*22m/s=**14.894m/s**

E. Standard deviation of Turbulence (σv)

- ➢ $\sigma v = K_r V_b K_1$

Where, K_r-Terrian factor=0.2343from above

K_1-Turbulence Factor=1 recommended value

V_b -basic wind velocity=22m/s

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

$I_v(z) = \frac{\sigma v}{V_m(Z)} = \frac{5.06}{14.894} = \mathbf{0.34}$

F. Air density

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

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

Site altitude above sea level (m)	Air density (kg/m^3)
0	1.20
500	1.12
1000	1.06
1500	1.00
2000	0.94

Therefore

The altitude of Addis Ababa is 2355m which is greater than 2000m. so we take the maximum value of air density 0.94 from the above table.

Now we ca calculate the **peack velocity**

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

$$=(1+7*0.34)(0.5*0.94*(14.894)^2)$$

$$=352.4N/m^2$$

$$q_p(Z)=\mathbf{0.352KN/m^2}$$

Geometric data of the building

- The height of building above the ground level is 20m including slab thickness.
- The building roof is Flat roof from the given.
- Width and length of the buildings is 20m and 18.85m respectively taken from floor plan.
- Height of parapet of the roof=1m
- Our building is G+5 which have 5 Storys

.

20m<18.85m (h<b), and from EBCS-1 Section A.2.2it is stated that “for building whose height, h is less than b, shall be considered to be one part

Roof 1 at h= 18m

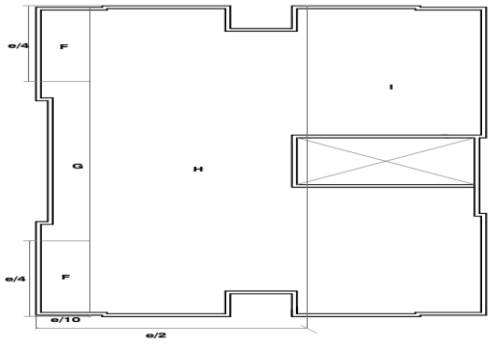


Figure 2-2 roof 1 zone

e= b or 2h whichever is the smaller

Then, $e \leq \min \begin{cases} b = 22m \\ 2h = 2 * 18.25 = 36.5m \end{cases} \Longrightarrow e = \mathbf{22m}$

Areas of each roof zone

- ✓ e/4 = 22/4 = **5.5m**
- ✓ e/10 = 22/10 = **2.2m**
- ✓ e/2 = 22/2 = **11m**
- ✓ b=**22m**
- ✓ d=**18.25m**

Area F = **24.2m^2**

Area G=**25.52m^2**

Area H= **182.17m^2**

Area I= **132.82m^2**

Determine external wind pressure coefficient for roof 1 at h=18m

- ✓ Height of parapet wall hp=1m
- ✓ Height of the building h = 18m

For $\frac{hp}{h} = \frac{1}{18} = 0.055$

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

For roof 1

External wind pressure coefficient

Table 2-3shows the external pressure coefficient (Cpe) values.

Zone	F		G		H		I	
Area	24.2m^2		25.52m^2		182.174m^2		132.82m^2	
d/h	C_{pe},10	C_{pe},1	C_{pe},10	C_{pe},1	C_{pe},10	C_{pe},1	C_{pe},10	C_{pe},1
	-1.38	-1.98	-0.9	-1.58	-0.7	-1.2	+0.2	-0.2
Cpe	-1.38	-0.9			-0.7		0.2	

Internal wind pressure coefficients

- ✓ For closed building with internal partitions and opening windows the extreme values are
  - ✓ $c_{pi} = 0.8$ for pressure
  - ✓ $c_{pi} = -0.5$ for suction

Table 2-4net wind load calculation

Region	F		G		H		I	
Area	24.2m^2		25.52m^2		182.174m^2		132.82m^2	
q_p(Zi)	0.352		0.352		0.352		0.352	
Cpi(+ve) for pressure	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Cpi(-ve) for suction	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Cpe	-1.38	0	-0.9	0	-0.7	0	0.2	0
$W_{net}=q_p(Zi)*(Cpe-Cpi_{(+ve)})$	-0.31	0.17	-0.14	0.17	-0.07	0.17	0.24	-0.11
$W_{net}=q_p(Zi)*(Cpe-Cpi_{(-ve)})$	-0.77	-0.28	-0.6	-0.28	-0.56	-0.28	-0.21	-0.35

Therefore,-

- $W_{net(+ve)}$= +0.17
- $W_{net(-ve)}$= -0.77

Roof 2 at h= 20m

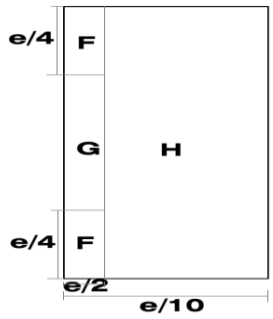


Figure 2-3roof 2 zone

- e= b or 2h whichever is the smaller

Then, $e \leq \min \begin{cases} b = 7.6m \\ 2h = 2 * 3.8 = 7.6m \end{cases} \Longrightarrow e = \mathbf{7.6m}$

Areas of each roof zone

- e/4 = 7.6/4 = **1.9m**
- e/10 = 7.6/10 = **0.76m**
- e/2 = 7.6/2 = **3.6m**
- b=**7.6m**
- d=**3.8m**

Area F = **2.88m^2**

Area G=**2.88m^2**

Area H= **23.1m^2**

Determine external wind pressure coefficient

For roof 2 at h=20

- Height of parapet wall hp=1m
- Height of the building h = 20m

For $\frac{hp}{h} = \frac{1}{20} = 0.05$,

Then using EBCS-1, 2015 Table 7.2 (Page 33) to determine external pressure coefficient as in a table

External wind pressure coefficient

Table 2-5shows the external pressure coefficient (Cpe) values.

Zone	F		G		H	
Area	2.88m^2		2.88m^2		23.1m^2	
	C_{pe},10	C_{pe},1	C_{pe},10	C_{pe},1	C_{pe},10	C_{pe},1
	-1.4	-2	-0.9	-1.6	-0.7	-1.2
Cpe	0	-2	0	-1.6	-0.7	0

Peck velocity For roof 2(h=20m)

- ✓ $C_r(Z)$=.0234ln(20/1)=**0.701**
- ✓ $V_m(z) = C_r(Z)C_o(Z)\ V_b$=0.701*1*22m/s=15.4m/s
- ✓ $I_v(z)=\frac{\sigma v}{V_m(Z)}=\frac{5.06}{15.44}=0.3277$
- ✓ $q_p(Z)=[1+7I_v(z)]*0.5*\rho_{air}*V^2_m(z)$

= (1+7*0.327)(0.5*0.94*(15.44)^2

=368.51N/m^2=**0.368KN/m^2**

Internal wind pressure coefficients

- ✓ For closed building with internal partitions and opening windows the extreme values are
  - ✓ $c_{pi} = 0.8$ for pressure and $c_{pi} = -0.5$ for suction

Table 2-6wind load for roof two

Region	F		G		H	
Area	2.88m^2		2.88m^2		23.1m^2	
$q_p(Zi)$	0.368		0.368		0.368	
Cpi(+ve) for pressure	0.8	0.8	0.8	0.8	0.8	0.8
Cpi(-ve) for suction	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Cpe	0	-2	0	-1.6	-0.7	0
$W_{net}=q_p(Zi)*(Cpe-Cpi_{(+ve)})$	0.184	0.552	0.184	-0.4	-0.07	0.184
$W_{net}=q_p(Zi)*(Cpe-Cpi_{(-ve)})$	-0.3	-1.03	-0.3	-0.88	-0.55	-0.3

Therefore;-

$W_{net(+ve)}$= **+0.552 KN/m^2**

$W_{net(-ve)}$= **-1.03KN/m^2**

We chose the maximum value from the four net wind load. That is **-1.03KN/m^2**. The negative sign indicate the direction of the wind.

Total design load on the roof floor surface

STEP 1: Load determination

A. Dead Load

For roof 1

- ✓ 210mm RC solid slab=0.21*25 = **5.25 kN/m^2**
- ✓ 30mm thick cement screed =0.03*23 = **0.69 kN/m^2**
- ✓ Porcelain tiles=0.02*21=**0.42 kN/m^2**

For roof 2

- ✓ 20 mm plastering on one sides = 0.03*23 = **0.69kN/m^2**
- ✓ 2mm bitumen layertile finishing material= 0.02*14 = **0.28 kN/m^2**
- ✓ Lightweight concrete=0.05*10=**0.5kN/m^2**
- ✓ Rc slab(self weight)=0.15*25=**3.75 kN/m^2**

Total Dead Load, DL = 5.25+0.69+0.42+0.5+0.28+0.69+3.75**=10.08kN/m^2**

B. Live Load

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

- ✓ It is category H roof, hence
- ✓ The distributed live load, qk=**0.5KN/m^2**
- ✓ The concentrated live load, Qk=**1KN**

C. Wind Load

- ✓ Wind Load (Suction) = **-1.03KN/m^2**
- ✓ Wind Load (Pressure) = **0.552KN/m^2**

Determination Effective depth of flat slab roof floor

Concrete

Class I workmanship and ordinary loading condition is used.

- ✓ Partial safety factor for concrete γc= 1.5,

Reinforcement

Steel S-500

- ✓ Partial safety factor for concrete γs= 1.15

Concrete cover

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

Cmin = max of (12, 10+ 0 – 0-0, 10mm) Cmin = 12mm

Cnom= 12+10 = 22mm take Ø=12mm for slab and concrete cover of 22mm.

Partial safety factors for load

-1.35 for dead load

-1.50 for live load Concrete

According to EBCS-2 2015(page 133) slab determination can be calculated by using the following formula and consideration.

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

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

$$N = \left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \text{ if } \rho > \rho_o$$

$$N = \left[11 + 1.5\sqrt{f_{ck}}\right] \text{ if } \rho = \rho_o$$

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

  K=1.3 for end span,
  K=1.5 for interior span
  K=0.4 for cantilever
  } according to EBCS-2,2015 Table 7.4 (Page 134)

- ρ_o:-is the reference reinforcement ratio , $\rho_o = \frac{\sqrt{f_{ck}}}{1000} = \frac{\sqrt{20}}{1000} x100 = \mathbf{0.447}\%$
- 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)
- $F1 = \frac{500}{f_{yk}} = \frac{500}{500} = 1$
- ,F2= 1,*F*3 = 1

Panel 1

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

$$N = \left[11 + 1.5\sqrt{f_{ck}}\right] \text{ if } \rho = \rho_o \text{ , } N = \left[11 + 1.5\sqrt{20}\right] = \mathbf{17.71}$$

$$\frac{l}{d} = 17.71 * 1.3 * 1 * 1 * 1 = \mathbf{23.023} \quad \text{, where} \quad l = lx$$

$$d \geq \frac{4300mm}{23.032} = \mathbf{186.77mm}$$

Using Φ12mm bar and clear cover of 22mm

- Total depth $D = 186.77 + \frac{12}{2} + 22 = \mathbf{214.76mm use 210mm}$

Table 2-7depth determination of roof slabs

Panel	Ly	Lx	Ly/Lx	Slab Type	Support Type	N*K*F1*F2*F3	D	D_{calc}
P-1	6.1	4.3	1.42	Two way	End span	23.023	149.36	177.414
P-2	6.1	4.6	1.33	Two way	End span	23.023	159.84	187.84
P-3	6.1	4.6	1.33	Two way	End span	23.023	159.84	187.84
P-4	6.1	4.3	1.42	Two way	End span	23.023	149.41	177.414
P-5	2.95	2.8	1.05	Two way	End span	23.023	97.29	125.29
P-6	4.6	2.8	1.64	Two way	Interior span	26.56	84.32	112.32
P-7	3.6	2.8	1.28	Two way	Interior span	26.56	84.32	112.32
P-8	4.6	2.8	1.64	Two way	Interior span	26.56	84.32	112.32
P-9	2.95	2.8	1.05	Two way	End span	23.023	97.29	125.29
P-10	6.1	4.3	1.42	Two way	End span	23.023	149.36	177.414
P-11	6.1	4.6	1.33	Two way	End span	23.023	159.84	187.84
P-12	6.1	3.6	1.69	Two way	End span	23.023	125.09	153.09
P-13	6.1	4.6	1.33	Two way	End span	23.023	159.84	187.84
P-14	6.1	4.3	1.42	Two way	End span	23.023	149.41	177.414
C-1	2	1.62	1.23	Two way	Cantilever	7.084	182.9	210.95
C-2	4.01	1.1	3.65	One way	Cantilever	7.084	124.2	152.2
C-3	2.5	1	2.5	One way	Cantilever	7.084	112.93	140.93
C-4	1.91	1.62	1.179	Two way	Cantilever	7.084	182.9	210.95
C-5	2.5	1	2.5	One way	Cantilever	7.084	112.93	140.93
C-6	2	1.1	1.82	Two way	Cantilever	7.084	124.22	152.22
C-7	2	1.62	1.23	Two way	Cantilever	7.084	182.9	210.95
C-8	2	1.63	1.23	Two way	Cantilever	7.084	184.1	212.1
C-9	4.01	1.1	3.65	One way	Cantilever	7.084	124.2	152.2
C-10	2.6	1.2	2.2	One way	Cantilever	7.084	135.52	163.52
C-11	2.6	1.2	2.2	One way	Cantilever	7.084	135.52	163.52
C-12	4.01	1.1	3.65	One way	Cantilever	7.084	124.2	152.2
C-13	2	1.63	1.23	Two way	Cantilever	7.084	184.1	212.1
							D=	**210mm**
							$D_{provided}$=	**210mm**

STEP 2: Load combination

1. **Combination -1 , Dead load + Live load**
   - ✓ Pd= 1.35DL + 1.50LL = 1.35(10.08) + 1.5(0.5) = **14.36 KN/m^2**
2. **Combination -2 , Dead load + Wind load**
   - ✓ Pd = 1.35DL + 1.50WL= 1.35(10.08) + 1.5(-1.03) **= 12.06 KN/m^2**
3. **Combination -3 , Dead load - Wind load**
   - ✓ Pd = 1.35DL - 1.50WL = 1.35(10.08) - 1.5(-1.03) = **15.15KN/m^2**
4. **Combination -4 , Dead load + live load + Wind load**
   - ✓ Pd = 1.35DL +1.50LL + 0.9WL= 1.35(10.08) + 1.5(0.5) + 0.9(-1.03) = **13.433 KN/m^2**
5. **Combination -5 , Dead load + live load - Wind load**
   - ✓ Pd = 1.35DL + 1.50LL + 0.9WL= 1.35(10.08) + 1.5(0.5) - 0.9(-1.03) = **15.29KN/m^2**

- The maximum design load combinations is**15.29KN/m**

Moment analysis of slabs for support and span

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.

Individual Moment Determination Moments for each panel with edge either simply supported or fully fixed are calculated from,

$$Mi= \alpha i \, Pd \, L_x^2$$

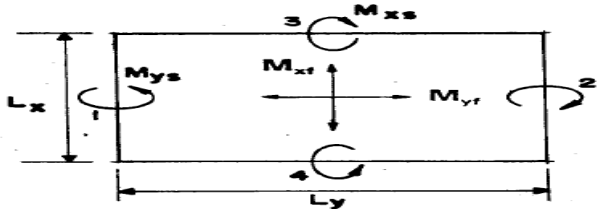


Figure 2-4Support and span moment notation

Support moments

- ✓ $M_{ys} = \alpha_{ys} \times P_d \times l_x^{\,2}$
- ✓ $M_{xs} = \alpha_{xs} \times P_d \times l_x^{\,2}$

Span moments

- ✓ $M_{yf} = \alpha_{yf} \times P_d \times l_x^{\,2}$
- ✓ $M_{xf} = \alpha_{xf} \times P_d \times l_x^{\,2}$

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

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

Pd = Design uniform load (KN/m2)

Lx, Ly = shorter and longer spans of the panel, respectively

The subscripts for moments are: s = support f = field (span)

x & y = -*are directions of shorter & longer span, respectively

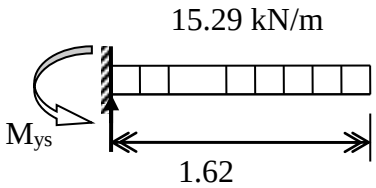
Computation of moments for all panels

Table 2-8 computation of moment for all typical floors from 1st to 5th floor panel

Panel	Type	Ly	Lx	Ly/Lx	Pd	α_{xs}	α_{xf}	α_{ys}	α_{yf}	Mxs	Mxf	Mys	Myf
P-1	4	6.1	4.3	1.42	15.29	0.075	0.056	0.045	0.034	21.20	15.83	12.72	9.61
P-2	4	6.1	4.6	1.33	15.29	0.071	0.052	0.045	0.034	22.97	16.82	14.56	11.00
P-3	4	6.1	4.6	1.33	15.29	0.071	0.052	0.045	0.034	22.97	16.82	14.56	11.00
P-4	4	6.1	4.3	1.42	15.29	0.075	0.056	0.045	0.034	21.20	15.83	12.72	9.61
P-6	1	4.6	2.8	1.64	15.29	0.056	0.042	0.032	0.024	6.71	5.035	3.836	2.877
P-7	1	3.6	2.8	1.28	15.29	0.045	0.034	0.032	0.024	5.394	4.076	3.836	2.877
P-8	1	4.6	2.8	1.64	15.29	0.056	0.042	0.032	0.024	6.71	5.035	3.836	2.877
P-10	4	6.1	4.3	1.42	15.29	0.075	0.056	0.045	0.034	21.20	15.83	12.72	9.61
P-11	2	6.1	4.6	1.33	15.29	0.053	0.039	0.037	0.028	17.15	12.62	11.97	9.06
P-12	2	6.1	3.6	1.69	15.29	0.062	0.046	0.037	0.028	12.29	9.12	7.332	5.548
P-13	2	6.1	4.6	1.33	15.29	0.053	0.039	0.037	0.028	17.15	12.62	11.97	9.06
P-14	4	6.1	4.3	1.42	15.29	0.075	0.056	0.045	0.034	21.20	15.83	12.72	9.61

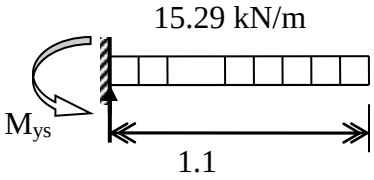
Cantilever moment Analysis calculation for C1, C7, C8 and C13

Cantilever moment: M_{ys} =15.29 * $1.62^2/2$ =**20.06kNm**



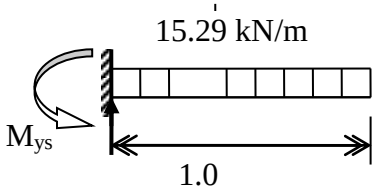
Cantilever moment Analysis calculation for C2, C9 and C12

Cantilever moment: M_{ys} = 15.29 * $1.1^2/2$ =**9.25kNm**



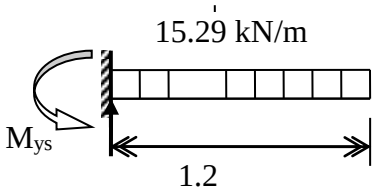
Cantilever moment Analysis calculation for C3 and C5

Cantilever moment: M_{ys} = 15. * 1^2/2 =**7.645kNm**



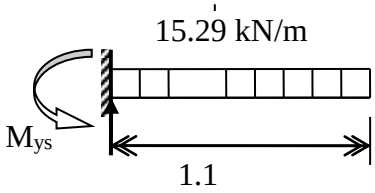
Cantilever moment Analysis calculation for C10 and C11

Cantilever moment: M_{ys} = 15.29 * 1.2^2/2 =**11.01kNm**



Cantilever moment Analysis calculation for C4 and C6

Cantilever moment: M_{ys} =15.29 * 1.1^2/2 =**9.25kNm**



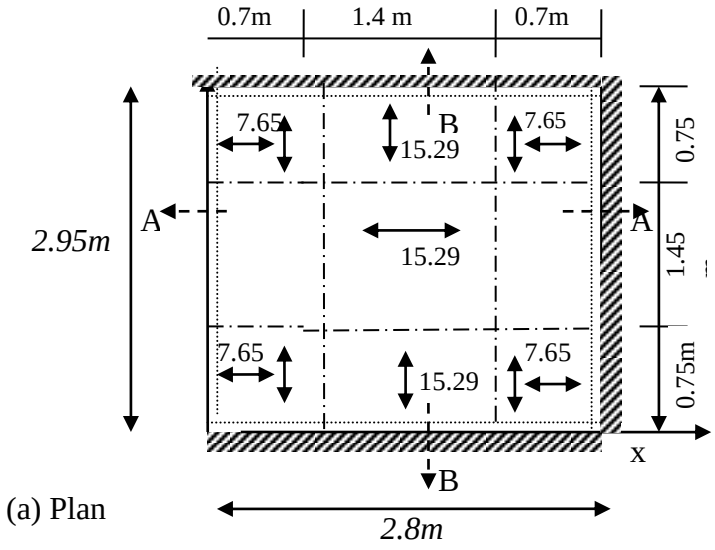
Strips in the slabs for Panel 5 and 9

The discontinuity lines are selected as shown in the figure below.

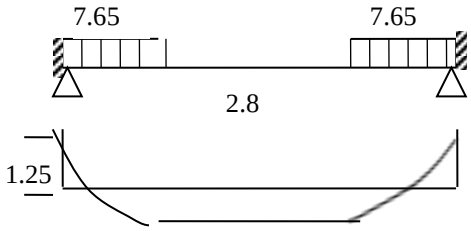
Edge strip width = b/4 = 2.8/4 = 0.7 m

In the corners the load is divided equally in the two directions; elsewhere 100 percent of load is assigned to the direction indicated by the arrows.

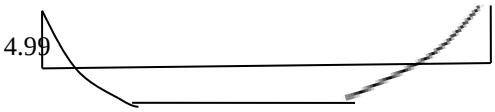
A ratio of support moment to the span moment of 2 is used **W=15.29KN/M**

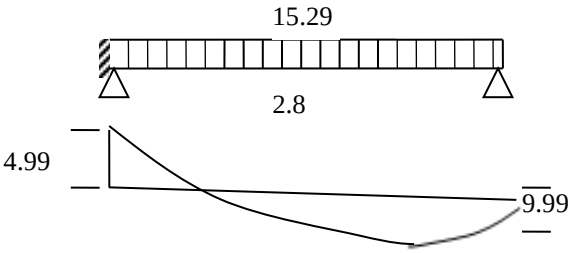


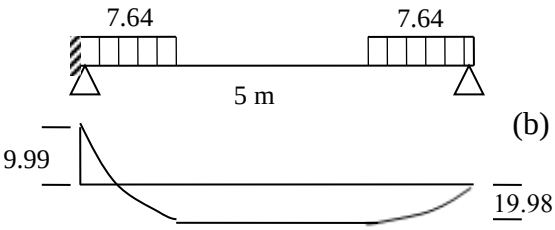
(a) Plan



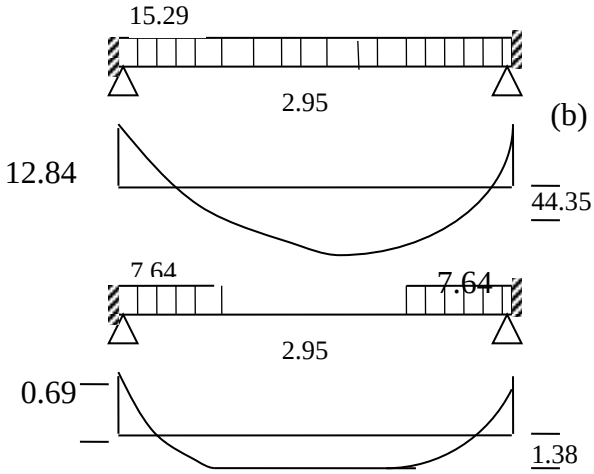
(b) w_x and m_x along Edge strip-1







(b) w_x and m_x along Edge strip-2



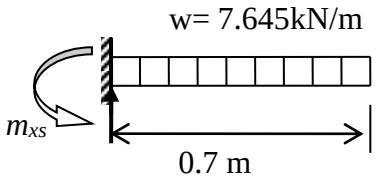
(b) w_x and m_x along Edge strip-B-B

(b) w_y and m_y along Edge strip

\

❖ **Calculation of moments**

X direction edge strip -1:



Cantilever moment: $m_x = 7.645 * 0.7^2/2 = 1.87$kNm

Negative Moment: $m_{xs} = 1.87 * 2/3 = 1.25$kNm

Positive moment: $m_{xf} = 1.87 * 1/3 = 0.62$kNm

X direction middle strip along A-A-1:

Simply supported span moment: $m_x = 15.29 * 2.8^2/8 = 14.984$ kNm

Negative Moment: $m_{xs} = 14.984 * 2/3 = 9.99$ kNm

Positive moment: $m_{xf} = 14.984 * 1/3 = 4.99$ kNm

X direction middle strip along A-A-2:

Simply supported span moment: $m_x = 15.29 * 2.8^2/8 = 14.984$ kNm

Negative Moment: $m_{xs} = 14.984 * 2/3 = 9.99$ kNm

Positive moment: $m_{xf} = 14.984 * 1/3 = 4.99$ kNm

X direction Edge strip-2:

Simply supported span moment: $m_x = 7.645 * 2.8^2/2 = 29.97$ kNm

Negative Moment: $m_{xs} = 29.97 * 2/3 = 19.98$kNm

Positive moment: $m_{xf} = 29.97* 1/3 = 9.99$kNm

Y direction middle strip along B-B:

Simply supported span moment: $m_y = 15.29 * 2.95^2/2 = 66.5$kNm

Negative Moment: $m_{ys} = 66.5* 2/3 = 44.35$kNm

Positive moment: $m_{yf} = 66.5 * 1/3 = 22.17$kNm

Y direction edge strip:

Cantilever moment: $m_y = 7.64 * 0.75^2/2 = 2.08$kNm

Negative Moment: $m_{ys} = 2.08 * 2/3 = 1.38$kNm

Positive moment: $m_{yf} = 2.08 * 1/3 = 0.69$kNm

Table 2-9 Governing moments obtained from strip method analysis

Direction	Moment	Value	Direction	Moment	Value
X	Support	19.98	Y	Support	10.48
	Span	14.36		Span	7.81

Moment Adjustment

Support moment adjustment

- **Method-1** it may be used when difference between initial support moment are less than 10 % of the larger moment. When such a case has occurred dimensioning is normally carried out either based on the average initial moment at the support.
- **Method-2** it may be used when difference between initial support moment are greater than 10 % of the larger moment. When such a case has occurred dimensioning is carried out using the stiffness of the slab and the effective length. Here we use moment distribution method to adjust support moment

Adjustment for P-1 &P-2, P-3&P-4

Mxs (P-1) = 21.2KNM

Mxs (P-2) = 22.97 KNM

$$100 * \frac{Mxs - Mxs}{Mxs} = 100 * \frac{22.97 - 21.2}{22.97} = 7.7\ \%$$

7.7 %< 10% so, we use Average method.

$$\text{Ms (adjusted)} = \frac{22.97 + 21.20}{2} = \mathbf{22.1KNm}$$

Adjustment for P-6 &P-7,P-7&P-8

Mys (P-6) = 3.836KNM

Mys (P-7) = 3.836 KNM

$$100 * \frac{Mys - Mys}{Mys} = 100 * \frac{3.836 - 3.836}{3.836} = 0\ \%$$

0%<10% so, we use Average method.

$$\text{Ms (adjusted)} = \frac{3.836 + 3.836}{2} = \mathbf{3.836KNm}$$

Adjustment for P-5 & P-10, P-9 & P-14,P-1&P-5,P-9&P-4,

Mys (P-1=P-9=P-10=P-14) = 12.72 KNM

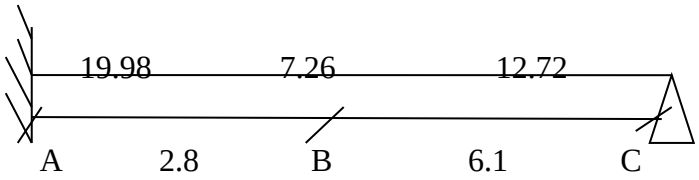
Mys (P-5=P-9) = 19.98KNM

$$100 * \frac{\text{Mys} - \text{Mys}}{\text{Mys}} = 100 * \frac{19.98 - 12.72}{19.98}$$

$$= 36.34\ \%$$

36.34%>10% so, we use moment Distribution method.

$\Delta M = \text{Mys} - \text{Mys} = 19.98 - 12.72 = 7.26$



Member	Relative stiffness(k)	Sum of k	DF
AB	KAB=$\frac{I}{L}=\frac{1}{2.8}$=**0.357**	**0.479**	DF=KAB/(KAB+KBC)=**0.745**
BC	KBC=$\frac{3I}{4L}=\frac{3}{4*6.1}$=**0.123**		DF=KBC/(KBC+KAB)=**0.256**

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

$$M_d = 19.98 - 0.745 * 7.26 = 14.57 KNm$$

$$M_d = M_L + \frac{K_L}{K_R + K_L} * \Delta M$$, taking the left……

$$M_d = 12.72 + 0.256 * 7.26 = 14.57 KNm$$

Where:

K = Relative stiffness

DF = distribution factor

Therefore, the adjusted moment For P-5=P-9=P-10=P-14ΔMs=**14.57KNm**

Table 2-10 Support moment adjustment of all typical floors (1st to 5th) of all panels

Panel 6and 2 ,Panel 8and 3		
Joint	C	
Member	CB	CD
Length	2.8	6.1
Stiffness factor	0.123	0.357
Sum of k	0.479	
DF	0.256	0.745
FEM	14.56	6.72
Balance	2.01	5.84
Adjusted	12.56	12.56

Panel-5 and 6, Panel 9 & 8		
Joint	B	
Member	BA	BC
Length	2.95	4.6
Stiffness factor	0.254	0.167
Sum of k	0.421	
DF	0.603	0.397
FEM	10.48	3.836
Balance	4.004	2.64
Adjusted	6.47	6.47

Panel 6 and 11, Panel 8 and 13,		
Joint	C	
Member	CB	CD
length	2.8	6.1
Stiffness factor	0.357	0.123
Sum of k	0.479	
DF	0.745	0.256
FEM	11.97	6.71
balance	3.92	1.34
Adjusted	8.05	8.05

Panel 11and 12 ,Panel 12and 13		
Joint	C	
Member	CB	CD
Length	4.6	3.6
Stiffness factor	0.21	0.27
Sum of k	0.48	
DF	0.437	0.56
FEM	17.15	12.29
Balance	2.12	2.72
Adjusted	15.01	15.01

Panel-7 and 12		
Joint	B	
Member	BA	BC
Length	2.8	6.1
Stiffness factor	0.268	0.123
Sum of k	0.391	
DF	0.685	0.315
FEM	7.332	5.39
Balance	1.33	0.61
Adjusted	6.002	6.002

Panel 10 and 11, Panel 14 and 13,		
Joint	C	
Member	CB	CD
Length	4.3	4.6
Stiffness factor	0.174	0.167
Sum of k	0.341	
DF	0.51	0.489
FEM	21.2	17.15
balance	2.066	1.981
Adjusted	19.13	19.13

Span /field/ moment adjustment for ground floor

Span moment is adjusted in the direction where maximum support moments exist. To adjust span moment, we add the maximum

Adjusted support moment and span moment in that direction then deduct the adjusted support moment

Span moment are adjusted only in the direction in which support moment is decreased during the support moment adjustment because The decreased moment must distribute to the field.

Support Moment on Panel 1

Mxf=15.83

Myf=9.61

Support Moment on Panel 2

Mxf=21.2+22.9722.1=21.97

Myf=14.56+6.7112.56=8.71

Support Moment on Panel 3

Mxf=21.2+22.97-22.1=21.97

Myf=14.56+6.71-12.56=8.71

Support Moment on Panel 4

Mxf= 15.83

Myf=9.61

Support Moment on Panel 5

Mxf=19.98+12.72-14.57=18.13.

Myf=10.48+3.83-6.47=7.84

Support Moment on Panel 6

Mxf=5.01

Myf=2.862

Support Moment on Panel 7

Mxf=4.076

Myf=2.877

Support Moment on Panel 8

Mxf= 5.035

Myf=2.877

Support Moment on Panel 9

Mxf=19.98+12.72-14.57=18.13

Myf=10.48+3.83-6.47=7.84

Support Moment on Panel 10

Mxf=21.2+17.1519.13=19.22

Myf=9.61

Support Moment on Panel 11

Mxf=17.15+12.29-15.01=14.43

Myf=11.97+6.71-8.05=10.63

Support Moment on Panel 12

Mxf=9.12

Myf=7.332+5.3946.002=6.724

Support Moment on Panel 13

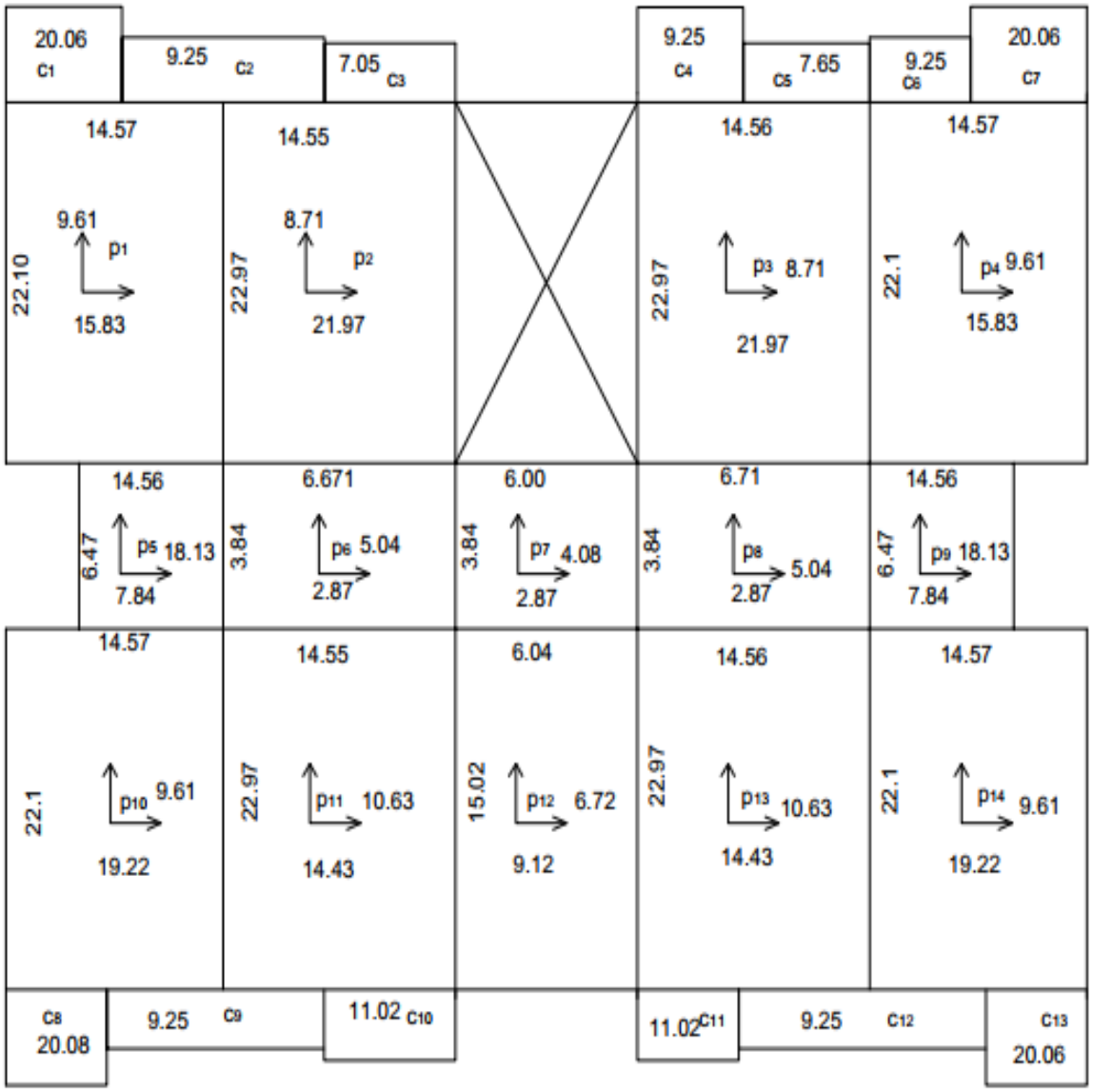
Mxf=17.15+12.29-15.01=14.43

Myf=11.97+6.71-8.05=10.63

Support Moment on Panel 14

Mxf=21.2+17.1519.13=19.22

Myf=9.61



ROOF

Figure 2-5adjusted span and support moment for roof

Flexural Reinforcement

Material data;

- Concrete c-25
- Steel grade s-400

Total depth of slab, D = 210

Effective **depth, d:**

In x-direction; 210-22-12/2=182

In y-direction; 210-22-12-12/2=170

Width, b=1000mm

Check depth for flexure Maximum moment,

Mmax from all floor is 22.971KNm and take k = 0.15,

$d = \sqrt{\frac{Mmax}{0.295 * fcd * b}}$ for singly reinforced section

$d = \sqrt{\frac{22.97 \times 10^6}{0.295 * 11.33 * 1000}}$ = **82.9mm < 210mm (depth provide for serviceability)**

Therefore the depth provides for serviceability requirement is adequate for flexure and depth of 210mm satisfy flexural requirement for all floors. .

K=Msd/d2*b*fck

Z= d/2(1+ ($\sqrt{1.353} * k$)

As=Msd/fyd*z

$as=\frac{\pi d2}{4}$

S=b*as/As

Maximum spacing is maximum of { 400 & 3h

Smax=400

Panel	moment notation	adjusted Moment	D (mm)	As ,min	As ,max	µsd	Z	As	NO. of bar	Spacing	Spacing Provided	AS Provide
P-1	Mxs	22.10	182	261.5	8000	0.027	177.611	357.555	12	316.147	350	322.971
	Mxf	15.83	182	261.5	8000	0.019	178.878	254.298	12	432.288	400	282.600
	Mys	14.57	170	244.3	8000	0.020	166.921	250.824	12	450.675	350	322.971
	Myf	9.61	170	244.3	8000	0.013	167.982	164.392	12	687.624	400	282.600
P-2	Mxs	22.97	182	261.5	8000	0.028	177.434	372.002	12	303.869	350	322.971
	Mxf	21.97	182	261.5	8000	0.027	177.637	355.399	12	318.065	400	282.600
	Mys	14.56	170	244.3	8000	0.020	166.923	250.649	12	450.990	350	322.971
	Myf	8.71	170	244.3	8000	0.012	168.173	148.827	12	759.539	400	282.600
P-3	Mxs	22.97	182	261.5	8000	0.028	177.434	372.002	12	303.869	350	322.971
	Mxf	21.97	182	261.5	8000	0.027	177.637	355.399	12	318.065	400	282.600
	Mys	14.56	142	204.0	8000	0.029	138.286	302.554	12	373.619	300	376.800
	Myf	8.71	170	244.3	8000	0.012	168.173	148.827	12	759.539	400	282.600
P-4	Mxs	22.10	182	261.5	8000	0.027	177.611	357.555	12	316.147	300	376.800
	Mxf	15.83	182	261.5	8000	0.019	178.878	254.298	12	444.517	400	282.600
	Mys	14.57	170	244.3	8000	0.020	166.921	250.824	12	450.675	200	565.200
	Myf	9.61	170	244.3	8000	0.013	167.982	164.392	12	687.624	400	282.600
P-5	Mxs	14.56	182	261.5	8000	0.018	179.133	233.564	12	432.288	400	282.600
	Mxf	18.13	182	261.5	8000	0.022	178.416	292.002	12	387.121	300	376.800
	Mys	6.47	170	244.3	8000	0.009	168.647	110.242	12	1025.381	150	753.600
	Myf	7.84	170	244.3	8000	0.011	168.357	133.815	12	462.803	400	282.600
P-6	Mxs	6.71	182	261.5	8000	0.008	180.690	106.711	12	1059.310	300	376.800
	Mxf	5.04	182	261.5	8000	0.006	181.019	79.928	12	1414.281	350	322.971
	Mys	3.84	170	244.3	8000	0.005	169.200	65.148	12	1735.138	300	376.800
	Myf	2.87	170	244.3	8000	0.004	169.402	48.684	12	2321.930	300	376.800
P-7	Mxs	6.00	182	261.5	8000	0.007	180.829	95.410	12	1184.782	350	322.971
	Mxf	4.08	182	261.5	8000	0.005	181.206	64.701	12	1747.122	400	282.600
	Mys	3.86	170	244.3	8000	0.005	169.195	65.557	12	1724.298	350	322.971
	Myf	2.87	170	244.3	8000	0.004	169.402	48.684	12	2321.930	400	282.600
P-8	Mxs	6.71	182	261.5	8000	0.008	180.690	106.711	12	1059.310	350	322.971
	Mxf	5.04	182	261.5	8000	0.006	181.019	79.928	12	1414.281	400	282.600
	Mys	3.84	170	244.3	8000	0.005	169.200	65.148	12	1735.138	350	322.971
	Myf	2.87	170	244.3	8000	0.004	169.402	48.684	12	2321.930	400	282.600
P-9	Mxs	14.56	182	261.5	8000	0.018	179.133	233.564	12	483.978	350	322.971
	Mxf	18.13	182	261.5	8000	0.022	178.416	292.002	12	387.121	300	376.800
	Mys	6.47	170	244.3	8000	0.009	168.647	110.242	12	1025.381	350	322.971
	Myf	7.84	170	244.3	8000	0.011	168.357	133.815	12	462.803	400	282.600

P-10	Mxs	22.10	182	261.5	8000	0.027	177.611	357.555	12	316.147	350	322.971
	Mxf	19.22	182	261.5	8000	0.023	178.195	309.940	12	432.288	400	282.600
	Mys	14.57	170	244.3	8000	0.020	166.921	250.824	12	450.675	300	376.800
	Myf	9.61	170	244.3	8000	0.013	167.982	164.392	12	687.624	400	282.600
P-11	Mxs	22.97	182	261.5	8000	0.028	177.434	372.002	12	303.869	350	322.971
	Mxf	14.43	182	261.5	8000	0.017	179.159	231.445	12	488.409	400	282.600
	Mys	14.56	170	244.3	8000	0.020	166.923	250.649	12	450.990	300	376.800
	Myf	10.63	170	244.3	8000	0.015	167.765	182.076	12	620.840	400	282.600
P-12	Mxs	15.02	182	261.5	8000	0.018	179.041	241.067	12	468.914	350	322.971
	Mxf	9.12	182	261.5	8000	0.011	180.215	145.420	12	777.333	400	282.600
	Mys	6.04	170	244.3	8000	0.008	168.737	102.860	12	1098.970	300	376.800
	Myf	6.72	170	244.3	8000	0.009	168.594	114.538	12	986.925	400	282.600
P-13	Mxs	22.97	182	261.5	8000	0.028	177.434	372.002	12	303.869	300	376.800
	Mxf	14.43	182	261.5	8000	0.017	179.159	231.445	12	488.409	400	282.600
	Mys	14.56	170	244.3	8000	0.020	166.923	250.649	12	450.990	300	376.800
	Myf	10.63	170	244.3	8000	0.015	167.765	182.076	12	620.840	400	282.600
P-14	Mxs	22.10	182	261.5	8000	0.027	177.611	357.555	12	316.147	350	322.971
	Mxf	19.22	182	261.5	8000	0.023	178.195	309.940	12	432.288	400	282.600
	Mys	14.57	170	244.3	8000	0.020	166.921	250.824	12	450.675	300	376.800
	Myf	9.61	170	244.3	8000	0.013	167.982	164.392	12	687.624	400	282.600
C-1,7,8,13	Mxs	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	300	376.800
	Mxf	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	400	282.600
	Mys	20.06	142	204.0	8000	0.040	136.829	421.284	12	268.323	300	376.800
	Myf	0.00	142	204.0	8000	0.000	142.000	0.000	12	0.000	400	282.600
C-2,9,12	Mxs	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	350	322.971
	Mxf	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	400	282.600
	Mys	9.25	142	204.0	8000	0.018	139.664	190.317	12	593.955	300	376.800
	Myf	0.00	142	204.0	8000	0.000	142.000	0.000	12	0.000	400	282.600
C-3,&5	Mxs	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	350	322.971
	Mxf	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	400	282.600
	Mys	7.65	142	204.0	8000	0.015	140.075	156.833	12	720.766	300	376.800
	Myf	0.00	142	204.0	8000	0.000	142.000	0.000	12	0.000	400	282.600
C-10&11	Mxs	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	350	322.971
	Mxf	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	400	282.600
	Mys	11.01	142	204.0	8000	0.022	139.210	227.267	12	497.388	300	376.800
	Myf	0.00	142	204.0	8000	0.000	142.000	0.000	12	0.00	400	282.600

Table 2-11 reinforcement calculation

Load transfer from roof floor slab to top tie beam

Sample calculation for P-1

- Shear coefficient for panel **1**

  Lx=4.3m Ly=6.1 Ly/Lx=1.42

  DL= 10.08KN/m^2 LL=0.5KN/m^2

  Bvx = 0.524

  Bvxd = 0.342

  Bvyc =0.4

  Bvyd = 0.26

- Dead load and live load transferred for panel 1

Dead load	Live load
Vxc=0.524*10.08*4.3= 27.712 KN	Vxc=0.524*0.5*4.3= 1.127KN
Vxd = 0.324*10.08*4.3= 14.04KN	Vxd = 0.324*0.5*4.3= 0.69KN
Vyc = 0.4*10.08*4.3= 17.34 KN	Vyc = 0.4*0.5*4.3= 0.86 KN
Vyd =0.26*10.08*4.3= 11.28 KN	Vyd =0.26*0.5*4.3= 0.56 KN

The rest of the panels are computed as above in tabular form as follow;

Shear coefficient and load transferred value

Table 2-12 Shear coefficient and load transferred value

Panel	Shear coefficient		DL	LL	Pd	Ly	Lx	Ly/lx	Dead load transferred	Live load Transferred	pd Transferred
	Bvxc	0.524	10.08	0.5	15.29	6.1	4.3	1.42	Vxc=22.7	1.13	35.59
	Bvxd	0.324	10.08	0.5	15.29	6.1	4.3	1.42	Vxd=14.04	0.69	21.30
	Bvyc	0.4	10.08	0.5	15.29	6.1	4.3	1.42	Vyc=17.29	0.86	26.3
	Bvyd	0.26	10.08	0.5	15.29	6.1	4.3	1.42	Vyd=11.26	0.559	17.1
P-2	Bvxc	0.506	10.08	0.5	15.29	6.1	4.6	1.33	Vxc=23.39	1.16	35.59
	Bvxd	0.33	10.08	0.5	15.29	6.1	4.6	1.33	Vxd=15.30	0.76	23.2
	Bvyc	0.4	10.08	0.5	15.29	6.1	4.6	1.33	Vyc=18.55	0.92	28.13
	Bvyd	0.26	10.08	0.5	15.29	6.1	4.6	1.33	Vyd=12.1	0.598	18.3
P-3	Bvxc	0.506	10.08	0.5	15.29	6.1	4.6	1.33	Vxc=23.39	1.16	35.59
	Bvxd	0.33	10.08	0.5	15.29	6.1	4.6	1.33	Vxd=15.30	0.76	23.2
	Bvyc	0.4	10.08	0.5	15.29	6.1	4.6	1.33	Vyc=18.55	0.92	28.13
	Bvyd	0.26	10.08	0.5	15.29	6.1	4.6	1.33	Vyd=12.1	0.598	18.3
P-4	Bvxc	0.524	10.08	0.5	15.29	6.1	4.3	1.42	Vxc=22.7	1.13	34.45
	Bvxd	0.324	10.08	0.5	15.29	6.1	4.3	1.42	Vxd=14.04	0.69	21.30
	Bvyc	0.4	10.08	0.5	15.29	6.1	4.3	1.42	Vyc=17.29	0.86	26.3
	Bvyd	0.26	10.08	0.5	15.29	6.1	4.3	1.42	Vyd=11.26	0.559	17.1

P-5	Bvxc	0.376	10.08	0.5	15.29	2.95	2.8	1.053	Vxc=10.61	0.53	16.1
	Bvxd	0	10.08	0.5	15.29	2.95	2.8	1.053	Vxd=0	0	0
	Bvyc	0.36	10.08	0.5	15.29	2.95	2.8	1.053	Vyc=10.2	0.504	15.412
	Bvyd	0.24	10.08	0.5	15.29	2.95	2.8	1.053	Vyd=6.774	0.336	10.275
P-6	Bvxc	0.467	10.08	0.5	15.29	4.6	2.8	1.643	Vxc=13.181	0.654	19.99
	Bvxd	0	10.08	0.5	15.29	4.6	2.8	1.643	Vxd=0	0	0
	Bvyc	0.33	10.08	0.5	15.29	4.6	2.8	1.643	Vyc=9.31	0.46	23.21
	Bvyd	0	10.08	0.5	15.29	4.6	2.8	1.643	Vyd=0	0	0
P-7	Bvxc	0.464	10.08	0.5	15.29	3.6	2.8	1.286	Vxc=13.096	0.65	19.86
	Bvxd	0.306	10.08	0.5	15.29	3.6	2.8	1.286	Vxd=8.636	0.4280	13.1
	Bvyc	0.36	10.08	0.5	15.29	3.6	2.8	1.286	VYc=10.16	0.504	15.41
	Bvyd	0	10.08	0.5	15.29	3.6	2.8	1.286	Vyd=0	0	0
P-8	Bvxc	0.467	10.08	0.5	15.29	4.6	2.8	1.643	Vxc=13.181	0.654	19.99
	Bvxd	0	10.08	0.5	15.29	4.6	2.8	1.643	Vxd=0	0	0
	Bvyc	0.33	10.08	0.5	15.29	4.6	2.8	1.643	Vyc=9.31	0.46	23.21
	Bvyd	0	10.08	0.5	15.29	4.6	2.8	1.643	Vyd=0	0	0
9	Bvxc	0.376	10.08	0.5	15.29	2.95	2.8	1.053	Vxc=10.61	0.53	16.1
	Bvxd	0	10.08	0.5	15.29	2.95	2.8	1.053	Vxd=0	0	0
	Bvyc	0.36	10.08	0.5	15.29	2.95	2.8	1.053	Vyc=10.2	0.504	15.412
	Bvyd	0.24	10.08	0.5	15.29	2.95	2.8	1.053	Vyd=6.774	0.336	10.275
10	Bvxc	0.524	10.08	0.5	15.29	6.1	4.3	1.42	Vxc=22.7	1.13	34.45
	Bvxd	0.324	10.08	0.5	15.29	6.1	4.3	1.42	Vxd=14.04	0.69	21.30
	Bvyc	0.4	10.08	0.5	15.29	6.1	4.3	1.42	Vyc=17.29	0.86	26.3
	Bvyd	0.26	10.08	0.5	15.29	6.1	4.3	1.42	Vyd=11.26	0.559	17.1
11	Bvxc	0.437	10.08	0.5	15.29	6.1	4.6	1.33	Vxc=20.26	1.01	30.74
	Bvxd	0	10.08	0.5	15.29	6.1	4.6	1.33	Vxd=0	0	0
	Bvyc	0.36	10.08	0.5	15.29	6.1	4.6	1.33	Vyc=16.7	0.828	25.32
	Bvyd	0.24	10.08	0.5	15.29	6.1	4.6	1.33	Vyd=11.13	0.552	16.88
12	Bvxc	0.493	10.08	0.5	15.29	6.1	3.6	1.69	Vxc=17.89	0.89	27.14
	Bvxd	0	10.08	0.5	15.29	6.1	3.6	1.69	Vxd=0	0	0
	Bvyc	0.36	10.08	0.5	15.29	6.1	3.6	1.69	Vyc=13.1	0.65	19.82
	Bvyd	0.24	10.08	0.5	15.29	6.1	3.6	1.69	Vyd=8.75	0.43	13.21
13	Bvxc	0.437	10.08	0.5	15.29	6.1	4.6	1.33	Vxc=20.26	1.01	30.74
	Bvxd	0	10.08	0.5	15.29	6.1	4.6	1.33	Vxd=0	0	0
	Bvyc	0.36	10.08	0.5	15.29	6.1	4.6	1.33	Vyc=16.7	0.828	25.32
	Bvyd	0.24	10.08	0.5	15.29	6.1	4.6	1.33	Vyd=11.13	0.552	16.88
14	Bvxc	0.524	10.08	0.5	15.29	6.1	4.3	1.42	Vxc=22.7	1.13	34.45
	Bvxd	0.324	10.08	0.5	15.29	6.1	4.3	1.42	Vxd=14.04	0.69	21.30
	Bvyc	0.4	10.08	0.5	15.29	6.1	4.3	1.42	Vyc=17.29	0.86	26.3
	Bvyd	0.26	10.08	0.5	15.29	6.1	4.3	1.42	Vyd=11.26	0.559	17.1

Load transfer for cantilevers

Cantilever 1, 7,8,13

DL=10.08KN/m^2*1.62m=16.33KN/m

LL=0.5KN/m^2*1.62m=0.81KN/m

PD=15.29KN/m^2*1.62=24.77KN/m

Cantilever 2, 9&12

DL=10.08KN/m^2*1.1m = 11.09KN/m

LL=0.5 KN/m^2*1.1m=0.55KN/m

PD=15.29KN/m^2*1.1= 16.82KN/m

Cantilever 3&5

DL=10.08KN/m^2*1m=10.08KN/m

LL=0.5 KN/m^2*1.m=0.5 KN/m

PD=15.29 KN/m^2*1= 15.29KN/m

Cantilever 10&11

DL=10.08KN/m^2*1.20 = 12.09KN/m

LL=0.5 KN/m^2*1.2m=0.6 KN/m

PD=15.29 KN/m^2*1.2= 18.35KN/m

Cantilever 4 and 6

DL=10.08KN/m^2*1.1m = 11.09KN/m

LL=0.5 KN/m^2*1.1m=0.55KN/m

PD=15.29 KN/m^2*1.1= 16.82KN/m

Table 2-13Shear load transfer from roof slab to beam

		Total pd including plaster (KN/M)	cantilever load on beam	Beam self weight(250*400) 1.35(self weight of beam)	shear load	total load
P-1						
Axis(B, 1&2)	Vxc	15.29	0	13	34.45	62.74
Axis(A, 1&2)	Vxd	15.29	29.205	13	21.3	78.79
Axis(2, A&B)	Vyc	15.29	0	13	26.3	54.59
Axis(1, A&B)	Vyd	15.29	0	13	17.1	45.39
P-2						
Axis(B, 2&3)	Vxc	15.29	0	13	35.59	63.88
Axis(A, 2&3)	Vxd	15.29	29.205	13	23.2	80.07
Axis(3, A&B)	Vyc	15.29	0	13	28.13	56.42
Axis(2, A&B)	Vyd	15.29	0	13	18.13	46.42
P-3						
Axis(B, 4&5)	Vxc	15.29	0	13	35.59	63.88
Axis(A, 4&5)	Vxd	15.29	32.11	13	23.2	83.6
Axis(5, A&B)	Vyc	15.29	0	13	28.13	56.42
Axis(4, A&B)	Vyd	15.29	0	13	18.13	46.42
P-4						
Axis(B, 5&6)	Vxc	15.29	0	13	34.45	62.74
Axis(A, 5&6)	Vxd	15.29	33.64	13	21.3	83.23
Axis(5, A&B)	Vyc	15.29	0	13	26.3	54.59
Axis(6, A&B)	Vyd	15.29	0	13	17.1	45.39
P-5						
Axis(2, B&C)	Vxc	15.29	0	13	16.01	44.3
Axis(1, B&C)	Vxd	15.29	0	13	0	28.29
Axis(C, 1&2)	Vyc	15.29	0	13	15.41	43.7
Axis(B, 1&2)	Vyd	15.29	0	13	10.275	38.57
P-6						
Axis(3, B&C)	Vxc	15.29	0	13	19.99	48.28
Axis(2, B&C)	Vxd	15.29	0	13	0	28.29
Axis(C, 2&3)	Vyc	15.29	0	13	23.21	51.5
Axis(B, 1&2)	Vyd	15.29	0	13	0	28.29
P-7						
Axis(4, B&C)	Vxc	15.29	0	13	19.86	48.15
Axis(3, B&C)	Vxd	15.29	0	13	13.1	41.39
Axis(C, 3&4)	Vyc	15.29	0	13	15.41	43.7

Axis(B, 3&4)	Vyd	15.29	0	13	0	28.29
P-8						
Axis(5, B&C)	Vxc	15.29	0	13	19.99	48.28
Axis(4, B&C)	Vxd	15.29	0	13	0	28.29
Axis(C, 4&5)	Vyc	15.29	0	13	23.21	51.5
Axis(B, 4&5)	Vyd	15.29	0	13	0	28.29
P-9						
Axis(5, B&C)	Vxc	15.29	0	13	16.01	44.3
Axis(6, B&C)	Vxd	15.29	0	13	0	28.29
Axis(C, 5&6)	Vyc	15.29	0	13	15.41	43.7
Axis(B, 6&5)	Vyd	15.29	0	13	10.275	38.57
P-10						
Axis(C, 1&2)	Vxc	15.29	0	13	34.45	62.74
Axis(D, 1&2)	Vxd	15.29	29.04	13	21.3	78.79
Axis(2, C&D)	Vyc	15.29	0	13	26.3	54.59
Axis(1, C&D)	Vyd	15.29		13	17.1	45.39
P-11						
Axis(C, 2&3)	Vxc	15.29		13	30.74	59.03
Axis(D, 2&3)	Vxd	15.29	30.74	13	0	28.29
Axis(2, C&D)	Vyc	15.29	0	13	25.32	53.61
Axis(3, C&D)	Vyd	15.29	0	13	16.88	45.17
P-12						
Axis(C, 3&4)	Vxc	15.29	0	13	27.14	55.43
Axis(D, 3&4)	Vxd	15.29	0	13	0	28.29
Axis(4, C&D)	Vyc	15.29	0	13	19.82	48.11
Axis(3, C&D)	Vyd	15.29	0	13	13.21	41.5
P-13						
Axis(C, 4&5)	Vxc	15.29	0	13	30.74	59.03
Axis(D, 4&5)	Vxd	15.29	30.74	13	0	59.03
Axis(5, C&D)	Vyc	15.29	0	13	25.32	53.61
Axis(4, C&D)	Vyd	15.29	0	13	16.88	45.17
P-14						
Axis(C, 5&6)	Vxc	15.29	0	13	38.07	66.94
Axis(D, 5&6)	Vxd	15.29	29.21	13	23.78	81.28
Axis(5, C&D)	Vyc	15.29	0	13	29.36	71.36
Axis(6, C&D)	Vyd	15.29	0	13	19.08	47.95

3. SLAB DESIGN AND ANALYSIS

Introduction to Slab Design

There are two types of slabs based on the load transferring mechanisms.

These are;-

One way slabs transmit their load in one direction

Two-way slabs resist the applied load two directions.

These types of slabs are composed of rectangular panels supported at all four edges by walls or beams stiff enough to be treated as unyielding. In our case most of the slabs are two-way and need to be analyzed based on the principle of two way actions.

Solid Slab Design Procedures

Requirement for Effective Depth

The minimum depth of a slab for deflection obtained from EBCS-2 shall be provided unless computation of deflection indicates that smaller thickness may be used without exceeding the limit

Table 3-1 exposure class

Class Designation	Description of the Environment	Informative examples where exposure classes may occur
1. 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

Indicative strength classes for durability

(1) The choice of adequately durable concrete for corrosion protection of reinforcement and protection of concrete attack requires consideration of the composition of concrete. This may result in a higher compressive strength of the concrete than is required for structural design. The relationship between concrete strength classes and exposure classes (see Table 4.1) may be described by indicative strength classes.

(2) When the chosen strength is higher than that required for structural design the value of fctm should be associated with the higher strength in the calculation of minimum reinforcement according to 7.3.2 and 9.2.1.1 and crack width control according to 7.3.3 and 7.3.4.

Note: For use of strength classes, refer to National Annex. The recommended values are given in Table E.1N.

Table 3-2 Indicative minimum strength class

	Exposure Classes according to Table 4.1									
Corrosion										
	Carbonation-induced corrosion				Chloride-induced corrosion			Chloride-induced corrosion from sea-water		
	XC1	XC2	XC3	XC 4	XD1	XD2	XD3	XS1	XS2	XS3
Indicative Strength Class	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	
Damage to Concrete										
	No risk	Freeze/Thaw Attack			Chemical Attack					
	X0	XF1	XF2	XF3	XA1	XA2	XA3			
Indicative Strength Class	C12/15	C30/37	C25/30	C30/37	C30/37		C35/45			

Requirements for durability

(1)P In order to achieve the required design working life of the structure, adequate measures shall be taken to protect each structural element against the relevant environmental actions.

(2)P the requirements for durability shall be included when considering the following:

- Structural conception, - Execution,
- Material selection, -Inspection,
- Construction details, - Verifications,
- Quality Control,
- Special measures (e.g. use of stainless steel, coatings, cathodic protection).

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 concret surface

$$C_{nom}=C_{min}+ \triangle C_{dev}$$

Where, Cmin, Minimum cover

C_{dev} allowance in design for devation

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 Cmin satisfying the requirements for both bond and environmental conditions shall be used.

$$c_{min} = \max \{c_{min,b};\ c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add};\ 10\ mm\} \quad (4.2)$$

where $c_{min,b}$ minimum cover due to bond requirement,
$c_{min,\ dur}$ minimum cover due to environmental conditions, see
$\Delta c_{dur,\gamma}$ additive safety element,
$\Delta c_{dur,st}$ reduction of minimum cover for use of stainless steel, see
$\Delta c_{dur,add}$ reduction of minimum cover for use of additional

A. Cmin, b minimum cover due to bond requirement

Table 3-3Minimum cover due to bond requirement

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.	

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

B. Cmin, dur minimum cover due to environmental conditions

Environmental Requirement for cmin,dur (mm) Structural Class Exposure Class according to Table 4.1

Table 3-4 minimum bond due to environmental condition

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

Therefore, As the structural class is S1 & Exposure class is X0, the minimum cover due to environmental requirement, Cmin, dur is 10mm.

C. ΔCdur, γ additive safety element

The concrete cover should be increased by the additive safety element $\Delta cdur_{\gamma}$.

Note: The value of $\Delta c_{dur,\gamma}$ for use in a Country may be found in its National Annex. The recommended value is 10mm.

Therefore, the concrete cover for safety element, $\Delta C_{dur,\gamma}$ is 10mm.

D. $\Delta C_{dur,st}$ reduction of minimum cover for use of stainless steel

Where stainless steel is used or other special measures have been taken, the minimum cover may be reduced by $\Delta c_{dur,\gamma}$. For such situations the effects on all material properties should be considered, including bond.

Note: The recommended value $\Delta c_{dur,st}$, without further specification, is 0mm.

Therefore, the reduction of minimum cover for use of stainless steel, $\Delta C_{dur,st}$ is 0mm.

E. $\Delta C_{dur,add}$ reduction of minimum cover for use of additional protection

For concrete with additional protection (e.g. coating) the minimum cover may be reduced by $\Delta c_{dur,add}$.

Note: The recommended value, for $\Delta c_{dur,add}$ without further specification, is 0 mm. Therefore, the reduction of minimum cover for use of additional protection, $\Delta C_{dur,add}$ is 0mm.

Finally, $c_{min} = \max \{c_{min,b}; c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10\text{ mm}$ $c_{min} = =10mm$

- $c_{nom} = c_{min} + \Delta c_{dev} 10+12=22mm$ concrete cover for slab =22mm

Allowance in design for deviation

(1)P To calculate the nominal cover, c_{nom} an addition to the minimum cover shall be made in design to allow for the deviation (Δc_{dev}).The required minimum cover shall be increased by the absolute value of the accepted negative deviation.

Table 3-5Exposure Class according to Table 4.1

Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2/ XS1	XD3/ XS2/ XS3
Design working life of 100 years	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2
Strength Class[1)2)]	≥C30/37 reduce class by 1	≥C30/37 reduce class by 1	≥C35/45 reduce class by 1	≥C40/50 reduce class by 1	≥C40/50 reduce class by 1	≥C40/50 reduce class by 1	≥C45/55 reduce class by 1
Member with slab geometry	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Design Data and Material Property specification we have considered

- **Material**
  - Concrete**:** Class I work.
  - Grade of concrete we used C-25.
  - Unit weight of concrete, ϒc = 25KN/m^3
  - Partial Safety factor for ordinary loading γc=1.5…...EBCS-2, 2015Table 2.1N
- Design strength value

In compression

fcd =$0.85 * \frac{fck}{\gamma c}$ …….EBCS-2, 2015 Page 20.

Where: fck - Characteristic of compressive strength of concrete.

γc-Partial Safety factor for ordinary loading=1.5.

- Characteristic of compressive strength of concrete(fck)

For C-25 fck =0.8*25Mpa=20Mpa or from EBCS-2, 2015 Table 3.1 (Page 25).

fcd =$0.85 * \frac{20}{1.5}$= 11.33 Mpa

In tension

fctd= $\frac{fctk}{\Upsilon c}$…….EBCS-2, 2015 Page 20

Where: fctd - Designof tensile strength of concrete.

fctk -Characteristic tensile strength of concrete.

γc-Partial Safety factor for ordinary loading=1.5.

- Characteristic of tensile strength of concrete(fctk),

fctk =0.7*fctm……..EBCS-2, 2015 Page 25

Where: fctm -Characteristiccylinder compressive strength.

- Characteristic cylinder compressive strength(fctm),

fctm = 0.3*$(fck)^{2/3}$……..EBCS-2, 2015Table 3.1 (Page 25)

Where: fck - Characteristic compressive strength of concrete=20Mpa.

fctm = 0.3*$(20)^{2/3}$=0.3$(20)^{2/3}$ = **2.2104Mpa**

fctk = 0.7*$(fctm)^{2/3}$=0.7$(2.2104)^{2/3}$ = **1.547Mpa**

Then, fctd= $\frac{1.547}{1.5}$Mpa= **1.03 Mpa**

Design strength of steel(fyd)

$$fyd = \frac{fyk}{\Upsilon s}$$

Where: fyk - Grade of steel, S-500 EBCS-2, 2015 (page 35)

γs-Reinforcing steel factor =1.15 Mpa for ordinary loading and class I work….EBCS-2, 1995 Table 3.1 (Page 22)

Then, $fyd = \frac{fyk}{\gamma s} = \frac{500}{1.15} = \mathbf{434.78Mpa}$

- Secant modulus of elasticity (Ecm)= 30Gpa….EBCS-2, 2015 Table 3.1 (Page25)
- Modulus of elasticity (Es)= 200Gpa….EBCS-2, 2015(Page37)

Slab depth determination

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

Where;

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

$$N = \left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \quad \text{if } \rho > \rho_o$$

$$N = \left[11 + 1.5\sqrt{f_{ck}}\right] \quad \text{if } \rho = \rho_o$$

Table 3-6 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 two-way spanning simply supported slab	1.1	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1.13	18	26
Interior span of beam or one-way or two-way spanning slab	1.15	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1.12	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

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

  K=1.3 for end span,
  K=1.5 for interior span } according to EBCS-2,2015 Table 7.4
  K=0.4 for cantilever (Page 134)

- ρ_o:-is the reference reinforcement ratio , $\rho_o = \frac{\sqrt{f_{ck}}}{1000} = \frac{\sqrt{20}}{1000} x100 = \mathbf{0.447\%}$
- 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)
- $F1 = \frac{500}{f_{yk}} = \frac{500}{400} = 1.25$,
- $F2 = \frac{\boldsymbol{As,prov}}{\boldsymbol{Asreq}} < 1.5$ $for first trial$: $use F2=1$
- $F3$ = 1 [$rectangular$section, solid slab and T section with b/bw≤ 3]

Panel 1

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

$N = \left[11 + 1.5\sqrt{f_{ck}}\right]$ if $\rho = \rho_o$, $N = \left[11 + 1.5\sqrt{20}\right]$ =**17.71**

$$\frac{l}{d} = 17.71 * 1.3 * 1.25 * 1 * 1 = \mathbf{28.779} \qquad , where \quad l = lx$$

$$d \geq \frac{4300mm}{28.779} = \mathbf{149.414mm}$$

Depth determination: Serviceability Requirement For end pane(p-)l
Using Φ12mm bar and clear cover of 22mm

- Total depth
  - $D = 149.414 + \frac{12}{2} + 2 = 177.414\boldsymbol{mm\ use\ 210mm}$

Depth determination: Serviceability Requirement For interior panel (p-6)
Using Φ12mm bar and clear cover of 22mm

- Total depth
  - $D = 84.32 + \frac{12}{2} + 22 = \mathbf{112.32mm\ use\ 210mm}$

Therefore, the depth provide for serviceability requirement for slab depth for deflection is **210mm**

- Depth in shorter direction
  - $dx = 210 - 12/2 - 22 = \mathbf{182}\boldsymbol{mm}$
- Depth in the longer direction
  - $dy = 210 - \frac{12}{2} - 12 - 20 = \mathbf{170}\boldsymbol{mm}$

Table 3-7 Calculation of depth for all panels

Panel	Ly	Lx	Ly/Lx	Slab Type	Support Type	N*K*F1*F2*F3	D	D_{calc}
P-1	6.1	4.3	1.42	Two way	End span	28.779	149.36	177.414
P-2	6.1	4.6	1.33	Two way	End span	28.779	159.84	187.84
P-3	6.1	4.6	1.33	Two way	End span	28.779	159.84	187.84
P-4	6.1	4.3	1.42	Two way	End span	28.779	149.41	177.414
P-5	2.95	2.8	1.05	Two way	End span	28.779	97.29	125.29
P-6	4.6	2.8	1.64	Two way	Interior span	33.206	84.32	112.32
P-7	3.6	2.8	1.28	Two way	Interior span	33.206	84.32	112.32
P-8	4.6	2.8	1.64	Two way	Interior span	33.206	84.32	112.32
P-9	2.95	2.8	1.05	Two way	End span	28.779	97.29	125.29
P-10	6.1	4.3	1.42	Two way	End span	28.779	149.36	177.414
P-11	6.1	4.6	1.33	Two way	End span	28.779	159.84	187.84
P-12	6.1	3.6	1.69	Two way	End span	28.779	125.09	153.09
P-13	6.1	4.6	1.33	Two way	End span	28.779	159.84	187.84
P-14	6.1	4.3	1.42	Two way	End span	28.779	149.41	177.414
C-1	2	1.62	1.23	Two way	Cantilever	8.855	182.9	210.95
C-2	4.01	1.1	3.65	One way	Cantilever	8.855	124.2	152.2
C-3	2.5	1	2.5	One way	Cantilever	8.855	112.93	140.93
C-4	1.91	1.62	1.179	Two way	Cantilever	8.855	182.9	210.95
C-5	2.5	1	2.5	One way	Cantilever	8.855	112.93	140.93
C-6	2	1.1	1.82	Two way	Cantilever	8.855	124.22	152.22
C-7	2	1.62	1.23	Two way	Cantilever	8.855	182.9	210.95
C-8	2	1.62	1.23	Two way	Cantilever	8.855	184.1	212.1
C-9	4.01	1.1	3.65	One way	Cantilever	8.855	124.2	152.2
C-10	2.6	1.2	2.2	One way	Cantilever	8.855	135.52	163.52
C-11	2.6	1.2	2.2	One way	Cantilever	8.855	135.52	163.52
C-12	4.01	1.1	3.65	One way	Cantilever	8.855	124.2	152.2
C-13	2	1.62	1.23	Two way	Cantilever	8.855	184.1	212.1
							D=	**210mm**
							$D_{provided}$=	**210mm**

Loading

- **Dead load**
  - 210mm RC solid slab=0.210*25=**5.25kN/m^2**
  - 20mm porcelain tile= 0.02*21= **0.42 kN/m**
  - 30mm thick cement screed =0.03*23=**0.69 kN/m^2**
  - 20 mm plastering and painting = 0.02*23 = **0.46kN/m^2**

Total Dead load=5.25+0.42+0.69+0.46= **6.82kN/m^2**

- **Live load$LL = Qk$**From EBCS-1, section 2.6.3, Table 2.10 Imposed Loads for, category B-C-2 Area with fixed seats
- **Load combinations**

Load combinations formula appropriate for this case is given as follow:

Pd = 1.35 DL + 1.5 LL

Where, Pd = design load

DL = total dead load on slab

LL = total live load on the roof

Partition load

Material data

Thickness of HCB=150mm, Thickness of plastering =20mm,

Unit weight of HCB=14 KN/m3, Unit weight of plastering =23 KN/m3

Height of wall=3.00m

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

Table 3-8 Total Partition Load on each panel

Panel	Partition wall	Thickness (m)	Length (m)	Height (m)	Unit weight (kN/m3)	Dead load	Total dead load	Area	Total DL DL/A
P-1	Wall	0.15	2.00	3.00	14	12.6	18.12	26.23	0.69
	Plastering	0.04	2.00	3.00	23	5.52			
P-2	Wall	0.15	2.00	3.00	14	12.6	18.12	28.06	0.65
	Plastering	0.04	2.00	3.00	23	5.52			
P-3	Wall	0.15	1.8	3.00	14	11.34	16.31	28.06	0.58
	Plastering	0.04	1.8	3.00	23	4.97			
P-4	Wall	0.15	4.1	3.00	14	25.83	37.15	26.23	1.42
	Plastering	0.04	4.1	3.00	23	11.32			
P-5	Wall	0.15	0	3.00	14	0	0	8.26	0
	Plastering	0.04	0	3.00	23	0			
P-6	Wall	0.15	0	3.00	14	0	0	12.88	0
	Plastering	0.04	0	3.00	23	0			
P-7	Wall	0.15	0	3.00	14	0	0	10.08	0

	Plastering	0.04	0	3.00	23	0			
P-8	Wall	0.15	0	3.00	14	0	0	12.88	0
	Plastering	0.04	0	3.00	23	0			
P-9	Wall	0.15	0	3.00	14	0	0	8.26	0
	Plastering	0.04	0	3.00	23	0			
P-10	Wall	0.15	4	3.00	14	25.2	36.24	26.23	1.382
	Plastering	0.04	4	3.00	23	11.04			
P-11	Wall	0.15	4	3.00	14	25.2	36.24	28.06	1.29
	Plastering	0.04	4	3.00	23	11.04			
P-12	Wall	0.15	0	3.00	14	0	0	21.96	0
	Plastering	0.04	0	3.00	23	0			
P-13	Wall	0.15	4	3.00	14	25.2	36.24	28.06	1.29
	Plastering	0.04	4	3.00	23	11.04			
P-14	Wall	0.15	4	3.00	14	25.2	36.24	26.23	1.38
	Plastering	0.04	4	3.00	23	11.04			
C-1	Wall	0.15	3.37	1.15	14	8.14	11.7	2.7	4.25
	Plastering	0.04	3.37	1.15	23	3.56			
C-2	Wall	0.15	4.41	3.00	14	27.78	39.95	4.41	9.05
	Plastering	0.04	4.41	3.00	23	12.17			
C-3	Wall	0.15	2.5	1.15	14	6.04	8.68	2.75	3.2
	Plastering	0.04	2.5	1.15	23	2.65			
C-4	Wall	0.15	2.01	3.00	14	12.66	18.21	1.91	9.53
	Plastering	0.04	2.01	3.00	23	5.55			
C-5	Wall	0.15	2.5	1.15	14	6.04	8.69	2.25	3.86
	Plastering	0.04	2.5	1.15	23	2.645			
C-6	Wall	0.15	2.2	3.00	14	13.86	19.93	2.2	9.06
	Plastering	0.04	2.2	3.00	23	6.1			
C-7	Wall	0.15	3.37	1.15	14	8.14	11.7	2.75	4.25
	Plastering	0.04	3.37	1.15	23	3.56			
C-8	Wall	0.15	3.37	1.15	14	8.14	11.7	2.75	4.25
	Plastering	0.04	3.37	1.15	23	3.56			
C-9	Wall	0.15	4.41	3.00	14	27.78	39.95	4.41	9.05
	Plastering	0.04	4.41	3.00	23	12.17			
C-10	Wall	0.15	3.8	1.15	14	9.18	13.83	3.2	4.32
	Plastering	0.04	3.8	1.15	23	4.02			
C-11	Wall	0.15	3.8	1.15	14	9.18	13.83	3.2	4.32
	Plastering	0.04	3.8	1.15	23	4.02			
C-12	Wall	0.15	4.41	3.00	14	27.78	39.95	4.41	9.05
	Plastering	0.04	4.41	3.00	23	12.17			
C-13	Wall	0.15	3.37	1.15	14	8.14	11.7	2.75	4.25
	Plastering	0.04	3.37	1.15	23	3.56			

Total design factor load

Design load

Pd=1.35 DL +1.5LL

DL =self weight + partition wall

Table 3-9 total load calculation from 1 to 5 typical floors

Panel	Slab Self weight	Partition wall load	Total dead load	Total live load	pd=1.35DL+1.5LL
P-1	6.82	0.69	7.51	4	16.14
P-2	6.82	0.65	7.47	4	16.08
P-3	6.82	0.58	7.4	4	15.99
P-4	6.82	1.42	8.24	4	17.12
P-5	6.82	0	6.82	4	15.21
P-6	6.82	0	6.82	4	15.21
P-7	6.82	0	6.82	4	15.21
P-8	6.82	0	6.82	4	15.21
P-9	6.82	0	6.82	4	15.21
P-10	6.82	1.382	8.202	4	17.07
P-11	6.82	1.29	8.11	4	16.95
P-12	6.82	0	6.82	4	15.21
P-13	6.82	1.29	8.11	4	16.95
P-14	6.82	1.38	8.20	4	17.07
C-1	6.82	4.25	11.07	4	20.94
C-2	6.82	9.06	15.87	4	27.42
C-3	6.82	3.2	10.02	4	19.53
C-4	6.82	9.53	16.35	4	28.1
C-5	6.82	3.86	10.68	4	20.42
C-6	6.82	9.06	15.87	4	27.42
C-7	6.82	4.25	11.07	4	20.94
C-8	6.82	4.25	11.07	4	20.94
C-9	6.82	9.05	15.87	4	27.42
C-10	6.82	4.32	11.14	4	21.04
C-11	6.82	4.32	11.14	4	21.04
C-12	6.82	9.05	15.87	4	27.42
C-13	6.82	4.25	11.07	4	20.94

Moment Analysis of slabs for support & span moment

Slabs with the ratio of the longer to the shorter span, between 1& 2 transfer their load in two orthogonal directions. i.e. some portion of the load in the shorter direction and the remaining portion of the load in the long direction. These slabs are called two-way
Slabs and they deflect into a dish shaped curvature. This means that they have curvature in both directions and because moments are proportional to curvature, there are moments in both directions, which require reinforcement in the tension zone.
Individual panel moments with edges either simply supported or fully fixed are calculated as:

$$\mathbf{Mi = \alpha i P d L x^2}$$

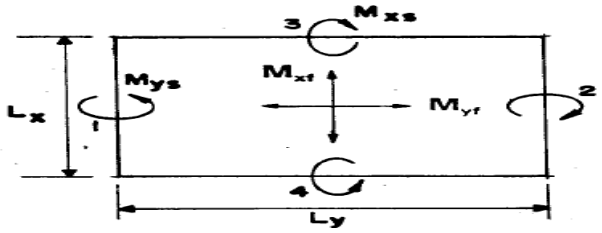


Figure 3-1**s**upport and span moment notations

Where:

Mi = design moment per unit width at the point of reference

α_i= the coefficient given in Table A-1 in EBCS-2 1995 as function of aspect ratio Ly/Lxand support condition.

Lx = shorter span of the panel
Ly = longer span of the panel

The Notation of Critical Moments

The subscripts for moments are

- s = support
- f = field (span)
- x = direction of shorter span
- y = direction of longer span.

Table 3-10 Computation of moments for all panels

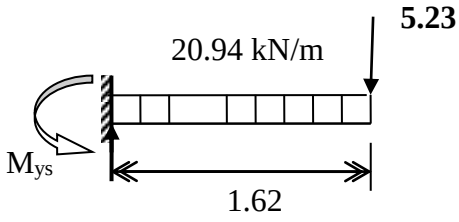
Panel	Type	Ly	Lx	Ly/Lx	Pd	α_{xs}	α_{xf}	α_{ys}	α_{yf}	Mxs	Mxf	Mys	Myf
P-1	4	6.1	4.3	1.42	16.14	0.075	0.056	0.045	0.034	22.38	16.71	13.43	10.15
P-2	4	6.1	4.6	1.33	16.08	0.071	0.052	0.045	0.034	24.16	17.69	15.31	11.57
P-3	4	6.1	4.6	1.33	15.99	0.071	0.052	0.045	0.034	24.02	17.59	15.23	11.50
P-4	4	6.1	4.3	1.42	17.21	0.075	0.056	0.045	0.034	23.87	17.82	14.32	10.82
P-6	1	4.6	2.8	1.64	15.21	0.056	0.042	0.032	0.024	6.678	5.008	3.816	2.862
P-7	1	3.6	2.8	1.28	15.21	0.045	0.034	0.032	0.024	5.366	4.054	3.816	2.862
P-8	1	4.6	2.8	1.64	15.21	0.056	0.042	0.032	0.024	6.678	5.008	3.816	2.862
P-10	4	6.1	4.3	1.42	17.07	0.075	0.056	0.045	0.034	23.67	17.67	14.20	10.73
P-11	2	6.1	4.6	1.33	16.95	0.053	0.039	0.037	0.028	19.01	13.99	13.27	10.04
P-12	2	6.1	3.6	1.69	15.21	0.062	0.046	0.037	0.028	12.22	9.067	7.293	5.519
P-13	2	6.1	4.6	1.33	16.95	0.053	0.039	0.037	0.028	19.01	13.99	13.27	10.04
P-14	4	6.1	4.3	1.42	17.07	0.075	0.056	0.045	0.034	23.67	17.67	14.20	10.73

Cantilever moment Analysis calculation for C1, C7.C8 and C13

There are also loads from external glass curtain wall which act as concentrated load on the strip.

Dead load from external curtain wall=1.35(3.37*1.15*1) =5.23KN

Cantilever moment: M_{ys} =20.94 * 1.62^2/2+5.23*1.62 =**35.95kNm**

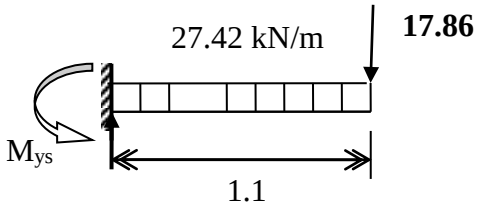


Cantilever moment Analysis calculation for C2, C9 and C12

There are also loads from external glass curtain wall which act as concentrated load on the strip.

Dead load from external curtain wall=1.35(4.41*3*1) =17.86KN

Cantilever moment: M_{ys} =27.42 * 1.1^2/2+17.86*1.1 =**36.24kNm**

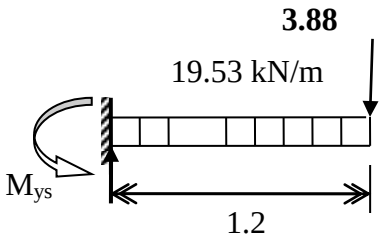


Cantilever moment Analysis calculation for C3 and C5

There are also loads from external glass curtain wall which act as concentrated load on the strip.

Dead load from external curtain wall=1.35(2.5*1.15*1) =3.88KN

Cantilever moment: M_{ys} =19.53 * 1^2/2+3.88*1 =**13.64kNm**

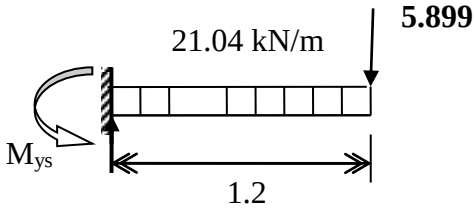


Cantilever moment Analysis calculation for C10 and C11

There are also loads from external glass curtain wall which act as concentrated load on the strip.

Dead load from external curtain wall=1.35(3.8*1.15*1) =5.899KN

Cantilever moment: M_{ys} =21.04 * 1.2^2/2+5.89*1.2 =**22.22kNm**

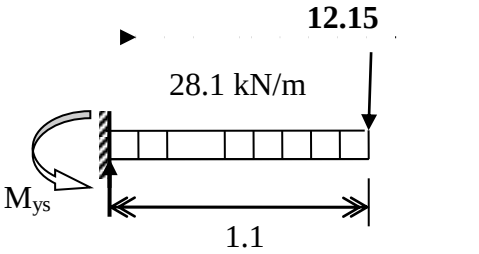


Cantilever moment Analysis calculation for C4 and C6

There are also loads from external glass curtain wall which act as concentrated load on the strip.

Dead load from external curtain wall=1.35(3*3*1) =12.15KN

Cantilever moment: M_{ys} =28.1 * 1.1^2/2+12.15*1.1 =**30.37kNm**



Strips in the slabs

For Panel 5 and 9

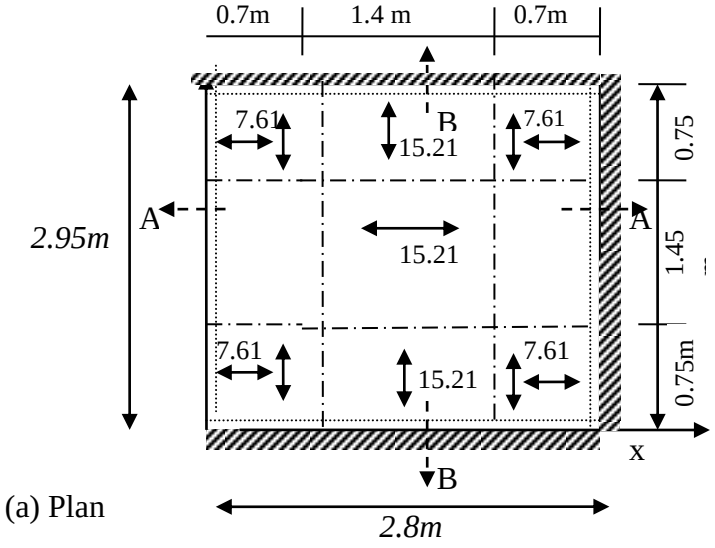
The discontinuity lines are selected as shown in the figure below.

Edge strip width = b/4 = 2.8/4 = 0.7 m

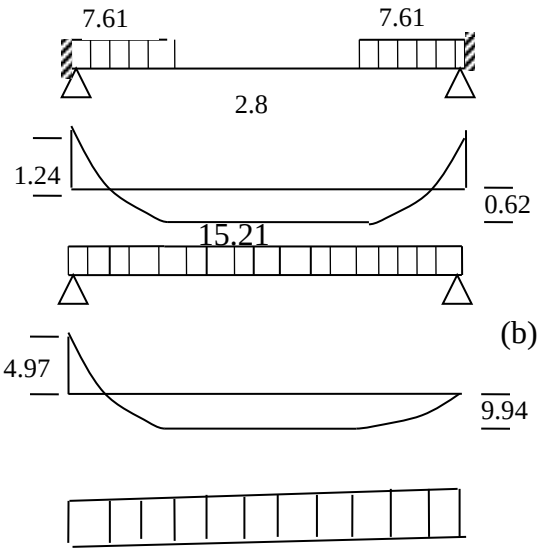
In the corners the load is divided equally in the two directions; elsewhere 100 percent of load is assigned to the direction indicated by the arrows.

A ratio of support moment to the span moment of 2 is used

W=15.21KN/M

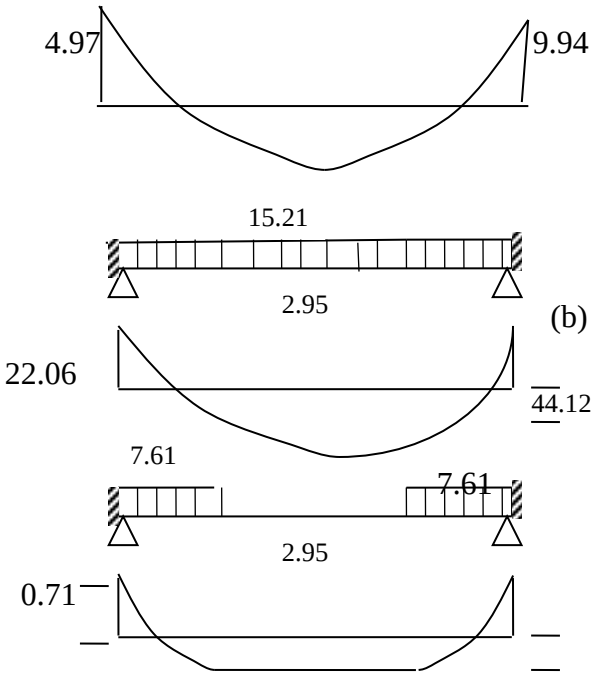


(a) Plan



(b) w_x and m_x along Edge strip-1

(b) w_x and m_x along A-A

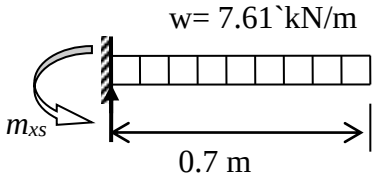


(b) w_x and m_x along Edge strip-B-B

(b) w_y and m_y along Edge strip

Calculation of moments

X direction edge strip -1:



Cantilever moment: $m_x = 7.61 * 0.7^2/2 = 1.86$kNm

Negative Moment: $m_{xs} = 1.86 * 2/3 = 1.24$kNm

Positive moment: $m_{xf} = 1.86 * 1/3 = 0.62$kNm

X direction middle strip along A-A-1:

Simply supported span moment: $m_x = 15.21 * 2.8^2/8 = 14.91$ kNm

Negative Moment: $m_{xs} = 14.91 * 2/3 = 9.94$ kNm

Positive moment: $m_{xf} = 14.91 * 1/3 = 4.97$ kNm

X direction middle strip along A-A-2:

Simply supported span moment: $m_x = 15.21 * 2.8^2/8 = 14.91$ kNm

Negative Moment: $m_{xs} = 14.91 * 2/3 = 9.94$ kNm

Positive moment: $m_{xf} = 14.984 * 1/3 = 4.97$ kNm

X direction Edge strip-2:

Simply supported span moment: $m_x = 7.61 * 2.8^2/2 = 29.83$ kNm

Negative Moment: $m_{xs} = 29.83 * 2/3 = 19.88$kNm

Positive moment: $m_{xf} = 29.83 * 1/3 = 9.94$kNm

Y direction middle strip along B-B:

Simply supported span moment: $m_y = 15.21 * 2.95^2/2 = 66.18kNm$

Negative Moment: $m_{ys} = 66.18* 2/3 = 44.12kNm$

Positive moment: $m_{yf} = 66.5 * 1/3 = 22.06kNm$

Y direction edgestrip:

Cantilever moment: $m_y= 7.61 * 0.75^2/2 = 2.14kNm$

Negative Moment: $m_{ys} = 2.14 * 2/3 = 1.42kNm$

Positive moment: $m_{yf} = 2.14 * 1/3 = 0.71kNm$

Governing moments obtained from strip method analysis

Direction	Moment	Value	Direction	Moment	value
X	Support	19.88	Y	Support	10.18
	Span	14.91		Span	7.21

Moment adjustment

i. <u>Support adjustment</u>

For a continuous support, there will be two supports which are different in magnitude. These moments are usually different in magnitude and must be adjusted to make only one moment. Therefore, the difference is distributed on either side of the support to equalize their moments. There are two cases moment adjustment EBCS-2, 1995.

A. If $\Delta M<10\%$ of the larger moment, the design moment is the average of the two.

B. If $\Delta M \geq 10\%$, then the unbalanced moment is distributed based on the stiffness.

$$M_d = M_R - \frac{K_R}{K_R + K_L} * \Delta M, \text{ taking the right} \quad \text{or} \quad M_d = M_L + \frac{K_L}{K_R + K_L} * \Delta M, \text{ taking the left}$$

Where: M_L-be moment in the left

M_R-be the moment in the right

K-the stiffness of the slabs, $K = \frac{I}{L_X}$

Support moment Adjustment for, P-3&P-4

Mxs (P-3) = 24.02KNM

Mxs (P-4) = 23.87 KNM

$100 * \frac{Mxs-Mxs}{Mxs} = 100 * \frac{24.02-23.87}{24.02} = 0.62\ \%$

0.62%<10% so, we use Average method.

Ms (adjusted)=$\frac{24.02+23.87}{2} = \mathbf{23.95}\boldsymbol{KNm}$

Adjustment for P-6 &P-7,P7&P8

Mys (P-6) = 3.81KNM

Mys (P-7) = 3.81 KNM

$100 * \frac{Mys-Mys}{Mys} = 100 * \frac{3.81-3.81}{3.81} = 0\ \%$

0%<10% so, we use Average method.

Ms (adjusted)=$\frac{3.81+3.81}{2} = \mathbf{3.81}\boldsymbol{KNm}$

Adjustment for P-5 & P-10, ,P-9&P-14,

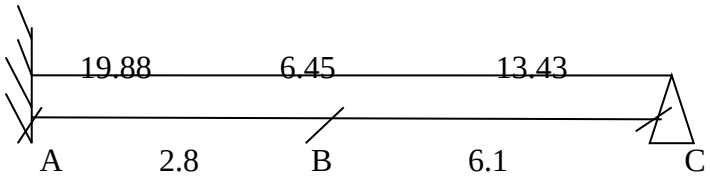
Mys (P-1 =P-10=P-14) = 13.43 KNM

Mys (P-5=P-9) = 19.88KNM

$100 * \frac{Mys-Mys}{Mys} = 100 * \frac{19.88-13.43}{19.88} = 32.445\%$

32.445%>10% so, we use moment Distribution method.

$\Delta M = Mys - Mys = 19.88 - 13.43 = 6.45$



Member	Relative stiffness(k)	Sum of k	DF
AB	KAB=$\frac{I}{L}=\frac{1}{2.8}$=**0.357**	**0.479**	DF=KAB/(KAB+KBC)=**0.745**
BC	KBC=$\frac{3I}{4L}=\frac{3}{4*6.1}$=**0.123**		DF=KBC/(KBC+KAB)=**0.256**

$M_d = M_R - \frac{K_R}{K_R + K_L} * \Delta M$, taking the right…….. $M_d = 19.88 - 0.745 * 6.45 = 15.075 KNm$

$$M_d = M_L + \frac{K_L}{K_R + K_L} * \Delta M$$, taking the left…… $M_d = 13.43 + 0.256 * 6.45 = 15.08 KNm$

Where: K = Relative stiffness DF = distribution factor

Therefore, the adjusted moment ForP-1= P-5=P-9=P-10=P-14ΔMs=**15.08KNm**

Table 3-11 support moment adjustement

Panel 6and 2 ,Panel 8and 3		
Joint	C	
Member	CB	CD
Length	2.8	6.1
Stiffness factor	0.123	0.357
Sum of k	0.479	
DF	0.256	0.745
FEM	6.678	15.31
Balance	2.21	6.431
Adjusted	8.889	8.889

Panel-5 and 6, Panel 9 & 8		
Joint	B	
Member	BA	BC
Length	2.95	4.6
Stiffness factor	0.254	0.167
Sum of k	0.421	
DF	0.603	0.397
FEM	10.18	3.81
Balance	3.841	2.53
Adjusted	6.34	6.34

Panel 6 and 11, Panel 8 and 13,		
Joint	C	
Member	CB	CD
Length	2.8	6.1
Stiffness factor	0.357	0.123
Sum of k	0.479	
DF	0.745	0.256
FEM	6.678	13.27
Balance	4.911	1.687
Adjusted	11.58	11.58

Panel 11and 12 ,Panel 12and 13		
Joint	C	
Member	CB	CD
Length	4.6	3.6
Stiffness factor	0.21	0.27
Sum of k	0.48	
DF	0.437	0.56
FEM	19.01	12.26
Balance	2.967	3.802
Adjusted	16.04	16.04

Panel-7 and 12		
Joint	B	
Member	BA	BC
Length	2.8	6.1
Stiffness factor	0.268	0.123
Sum of k	0.391	
DF	0.685	0.315
FEM	5.366	7.293
Balance	1.32	0.607
Adjusted	6.686	6.686

Panel 10 and 11, Panel 14 and 13,		
Joint	C	
Member	CB	CD
Length	4.3	4.6
Stiffness factor	0.174	0.167
Sum of k	0.341	
DF	0.51	0.489
FEM	23.67	19.01
Balance	2.377	2.278
Adjusted	21.28	21.28

Panel 1and 2 ,		
Joint	B	
Member	BA	BC
Length	4.3	4.6
Stiffness factor	0.174	0.163
Sum of k	0.337	
DF	0.52	0.48
FEM	22.38	24.16
Balance	0.93	0.85
Adjusted	23.31	23.31

Panel 5and 10,panel 9and 14		
Joint	C	
Member	CB	CD
Length	2.8	6.1
Stiffness factor k	0.357	0.123
Sum of k	0.479	
DF	0.745	0.256
FEM	19.88	14.2
Balance	4.232	1.454
Adjusted	15.65	15.65

Span /field/ moment adjustment for ground floor

Span moment is adjusted in the direction where maximum support moments exist. To adjust span moment, we add the maximum adjusted support moment and span moment in that direction then deduct the adjusted support moment

Span moment are adjusted only in the direction in which support moment is decreased during the support moment adjustment because the decreased moment must distributed to the field

Support Moment on Panel 1
Mxf= 16.71

Myf=10.15

Support Moment on Panel 2
Mxf=22.38+24.16-23.31=23.23

Myf=15.31+6.678-8.889=13.09

Support Moment on Panel 3
Mxf=24.02+23.87-23.95=23.94

Myf=15.31+6.678-8.889=13.09

Support Moment on Panel 4
Mxf= 17.82

Myf=10.82

Support Moment on Panel 5
Mxf=19.88+13.43-15.08=18.23.

Myf=10.18+3.81-6.34=7.65

Support Moment on Panel 6
Mxf=5.01

Myf=2.862

Support Moment on Panel 7
Mxf=4.054

Myf=2.862

Support Moment on Panel 8
Mxf= 5.01

Myf=2.862

Support Moment on Panel 9
Mxf=19.88+14.32-15.7=18.5

Myf=10.18+3.81-6.34=7.65

Support Moment on Panel 10
Mxf=23.67+19.01-21.28=21.4

Myf=10.73

Support Moment on Panel 11
Mxf=19.01+12.26-16.04=15.23

Myf=13.27+6.678-11.58=8.368

Support Moment on Panel 12
Mxf=9.07

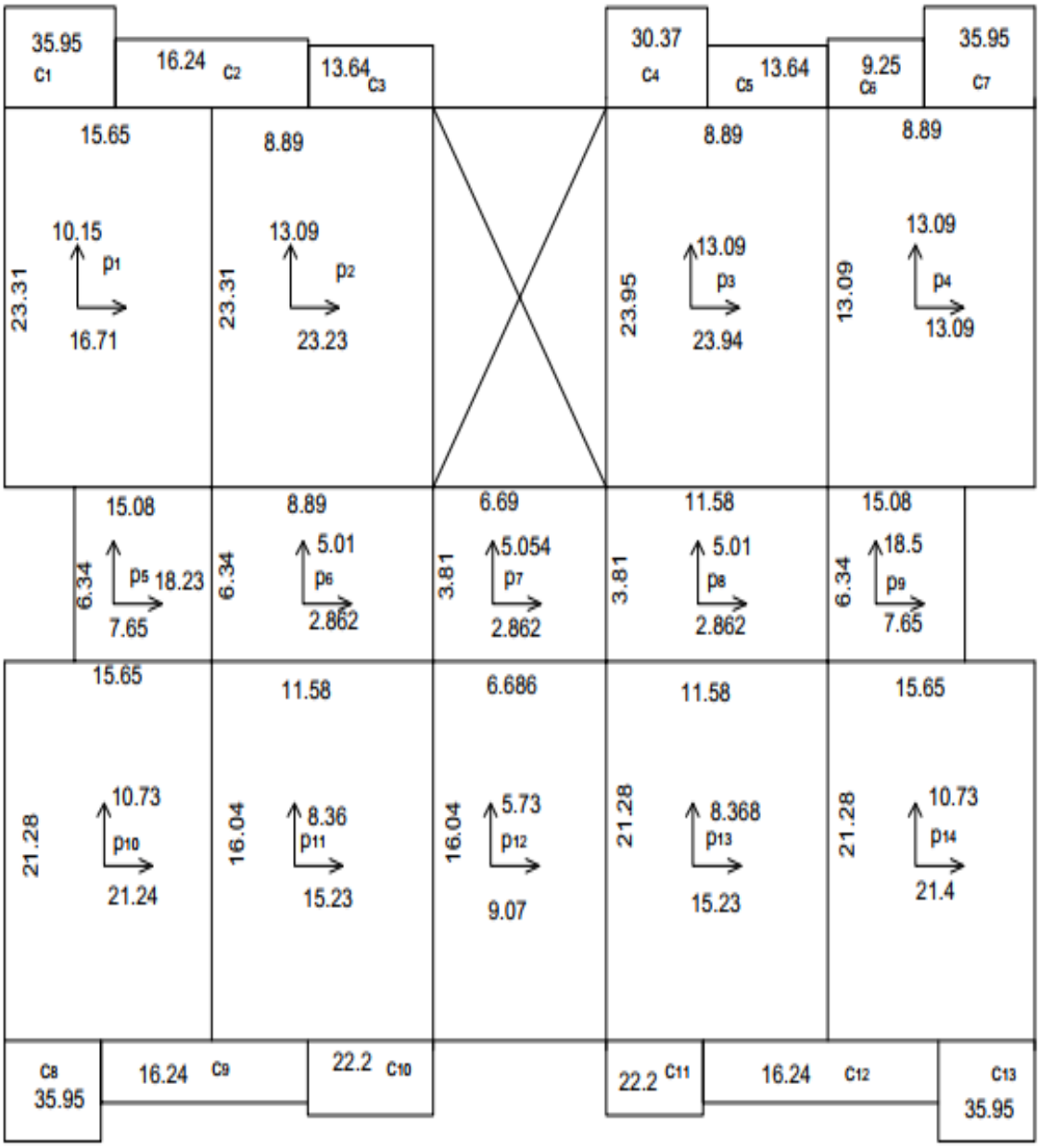
Myf=7.293+5.366-6.686=5.973

Support Moment on Panel 13
Mxf=19.01+12.22-16.04=15.23

Myf=13.27+6.678-11.58=8.368

Support Moment on Panel 14
Mxf=23.67+19.01-21.28=21.4

Myf=10.73



SLAB

Figure 3-2 adjusted span and support moment for slab

Flexural Reinforcement

Material data;

- Concrete c-25
- Steel grade s-400

Total depth of slab, **D = 210**

Effective depth, d: in x-direction; 210-22-12/2=182

In y-direction; 210-22-12-12/2=170

Width, **b=1000mm**

Check depth for flexure Maximum moment,

Mmax from all floor is 36.24KNm and take k = 0.15 ,

$d = \sqrt{\frac{Mmax}{0.295 * fcd * b}}$ for singly reinforced section

$d = \sqrt{\frac{36.24 \times 10^6}{0.295 * 11.33 * 1000}}$ = **101.21mm < 210mm (depth provide for serviceability)**

Therefore the depth provides for serviceability requirement is adequate for flexure and depth of 210mm satisfy flexural requirement for all floors. .

K=Msd/d2*b*fck

Z= d/2(1+ ($\sqrt{1.353} * k$)

As=Msd/fyd*z

as=$\frac{\pi d2}{4}$ S=b*as/As Maximum spacing is maximum of 400 & 3h Smax=400

Panel	moment notation	adjusted moment	D (mm)	As, min	As ,max	µsd	Z	As	No Of bar	Spacing	Spacing provide	AS Provide
	Mxs	23.31	182	261.5	8000	0.028	177.364	377.657	12	299.320	350	322.971
P-1	Mxf	16.71	182	261.5	8000	0.020	178.702	268.701	12	432.288	400	282.600
	Mys	15.65	170	244.3	8000	0.022	166.688	269.793	12	418.988	350	322.971
	Myf	10.15	170	244.3	8000	0.014	167.867	173.748	12	650.596	400	282.600
	Mxs	23.31	182	261.5	8000	0.028	177.364	377.657	12	299.320	350	322.971
P-2	Mxf	23.23	182	261.5	8000	0.028	177.381	376.326	12	300.378	400	282.600
	Mys	8.89	170	244.3	8000	0.012	168.135	151.920	12	744.076	350	322.971
	Myf	13.09	170	244.3	8000	0.018	167.239	224.917	12	502.585	400	282.600
	Mxs	23.95	182	261.5	8000	0.029	177.233	388.312	12	291.106	350	322.971
P-3	Mxf	23.94	182	261.5	8000	0.029	177.235	388.145	12	432.288	400	282.600
	Mys	8.89	142	204.0	8000	0.018	139.756	182.769	12	618.487	300	376.800
	Myf	13.09	142	204.0	8000	0.026	138.670	271.254	12	416.730	400	282.600
	Mxs	23.95	182	261.5	8000	0.029	177.233	388.312	12	291.106	300	376.800
P-4	Mxf	17.82	182	261.5	8000	0.022	178.478	286.908	12	393.993	400	282.600
	Mys	8.89	170	244.3	8000	0.012	168.135	151.920	12	744.076	200	565.200
	Myf	10.82	170	244.3	8000	0.015	167.724	185.375	12	609.790	400	282.600
	Mxs	15.08	182	261.5	8000	0.018	179.029	242.047	12	432.288	400	282.600
P-5	Mxf	18.23	182	261.5	8000	0.022	178.395	293.646	12	384.954	300	376.800
	Mys	6.34	170	244.3	8000	0.009	168.674	108.009	12	1046.576	150	753.600
	Myf	7.65	170	244.3	8000	0.011	168.398	130.541	12	462.803	400	282.600
	Mxs	8.89	182	261.5	8000	0.011	180.261	141.701	12	797.736	300	376.800
P-6	Mxf	5.01	182	261.5	8000	0.006	181.024	79.529	12	1421.377	350	322.971
	Mys	6.34	170	244.3	8000	0.009	168.674	108.009	12	1046.576	300	376.800
	Myf	2.86	170	244.3	8000	0.004	169.404	48.547	12	2328.443	300	376.800
	Mxs	6.69	182	261.5	8000	0.008	180.695	106.326	12	1063.141	350	322.971
P-7	Mxf	4.05	182	261.5	8000	0.005	181.211	64.287	12	1758.377	400	282.600
	Mys	3.81	170	244.3	8000	0.005	169.206	64.704	12	1747.035	350	322.971
	Myf	2.86	170	244.3	8000	0.004	169.404	48.513	12	2330.077	400	282.600
	Mxs	11.58	182	261.5	8000	0.014	179.727	185.146	12	610.544	350	322.971
P-8	Mxf	5.01	182	261.5	8000	0.006	181.024	79.529	12	1421.377	400	282.600
	Mys	3.81	170	244.3	8000	0.005	169.206	64.704	12	1747.035	350	322.971
	Myf	2.86	170	244.3	8000	0.004	169.404	48.547	12	2328.443	400	282.600
	Mxs	15.08	182	261.5	8000	0.018	179.029	242.047	12	467.017	350	322.971
P-9	Mxf	18.50	182	261.5	8000	0.022	178.341	298.086	12	432.288	400	282.600
	Mys	6.34	170	244.3	8000	0.009	168.674	108.009	12	1046.576	350	322.971
	Myf	7.65	170	244.3	8000	0.011	168.398	130.541	12	865.936	400	282.600
	Mxs	21.28	182	261.5	8000	0.026	177.778	343.966	12	328.638	350	322.971
P-10	Mxf	21.40	182	261.5	8000	0.026	177.753	345.953	12	432.288	400	282.600
	Mys	15.65	170	244.3	8000	0.022	166.688	269.793	12	418.988	300	376.800
	Myf	10.73	170	244.3	8000	0.015	167.744	183.812	12	614.976	400	282.600
	Mxs	16.04	182	261.5	8000	0.019	178.836	257.733	12	438.594	350	322.971
P-11	Mxf	15.23	182	261.5	8000	0.018	178.999	244.495	12	462.340	400	282.600
	Mys	11.58	170	244.3	8000	0.016	167.562	198.588	12	569.219	300	376.800
	Myf	8.36	170	244.3	8000	0.012	168.247	142.784	12	791.687	400	282.600
	Mxs	16.04	182	261.5	8000	0.019	178.836	257.733	12	438.594	350	322.971
P-12	Mxf	9.07	182	261.5	8000	0.011	180.225	144.615	12	781.662	400	282.600

	Mys	6.69	170	244.3	8000	0.009	168.601	113.953	12	991.986	300	376.800
	Myf	5.73	170	244.3	8000	0.008	168.803	97.543	12	1158.87	400	282.600
P-13	Mxs	21.28	182	261.5	8000	0.026	177.778	343.966	12	328.638	300	376.800
	Mxf	15.23	182	261.5	8000	0.018	178.999	244.495	12	462.340	400	282.600
	Mys	11.58	170	244.3	8000	0.016	167.562	198.588	12	569.219	300	376.800
	Myf	8.37	170	244.3	8000	0.012	168.246	142.922	12	790.922	400	282.600
P-14	Mxs	21.28	182	261.5	8000	0.026	177.778	343.966	12	328.638	400	282.600
	Mxf	21.40	182	261.5	8000	0.026	177.753	345.953	12	326.750	400	282.600
	Mys	15.65	170	244.3	8000	0.022	166.688	269.793	12	418.988	300	376.800
	Myf	10.73	170	244.3	8000	0.015	167.744	183.812	12	614.976	400	282.600
C-1,7,8,13	Mxs	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	300	376.800
	Mxf	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	400	282.600
	Mys	35.95	142	204.0	8000	0.071	132.424	780.104	12	144.904	300	376.800
	Myf	0.00	142	204.0	8000	0.000	142.000	0.000	12	0.000	400	282.600
C-2,9,12	Mxs	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	350	322.971
	Mxf	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	400	282.600
	Mys	36.24	142	204.0	8000	0.072	132.341	786.892	12	143.654	300	376.800
	Myf	0.00	142	204.0	8000	0.000	142.000	0.000	12	0.000	400	282.600
C-3,&5	Mxs	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	350	322.971
	Mxf	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	400	282.600
	Mys	13.64	142	204.0	8000	0.027	138.527	282.945	12	399.513	300	376.800
	Myf	0.00	142	204.0	8000	0.000	142.000	0.000	12	0.000	400	282.600
C-10&11	Mxs	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	350	322.971
	Mxf	0.00	154	221.3	8000	0.000	154.000	0.000	12	0.000	400	282.600
	Mys	22.22	142	204.0	8000	0.044	136.247	468.637	12	241.210	300	376.800
	Myf	0.00	142	204.0	8000	0.000	142.000	0.000	12	0.000	400	282.600

Table 3-12 reinforcment calculation of slab

Load transfer from floor slab to Intermediate beam

- **Sample calculation for P-1**

- **Shear coefficient for panel 1**

Lx=4.3m Ly=6.1 Ly/Lx=1.42

DL= 16.14KN/m^2 LL=4KN/m^2

Bvx = 0.524 Bvyc =0.4

Bvxd = 0.342 Bvyd = 0.26

- **Dead load and live load transferred for panel 1**

Dead load **Live load**

Vxc=0.524*7.51*4.3=**16.92KN** Vxc=0.524*4*4.3=**9.013KN**

Vxd = 0.324*7.51*4.3= **10.46KN** Vxd = 0.324*4*4.3= **5.573KN**

Vyc = 0.4*7.51*4.3= **12.92 KN** Vyc = 0.4*4*4.3= **6.88KN**

Vyd =0.26*7.51*4.3= **8.396 KN** Vyd =0.26*4*4.3= **4.472KN**

The rest of the panels are computed as above in tabular form as follow;

Table 3-13 Shear coefficient and load transferred value

panel	Shear coefficient		DL	LL	Pd	Ly	Lx	Ly/lx	Deadload transferred	Live load Transferred	pd Transferred
P-1	Bvxc	0.524	7.51	4	16.14	6.1	4.3	1.42	Vxc=16.92	11.27	36.37
	Bvxd	0.324	7.51	4	16.14	6.1	4.3	1.42	Vxd=10.46	6.97	22.48
	Bvyc	0.4	7.51	4	16.14	6.1	4.3	1.42	Vyc=12.92	8.6	27.76
	Bvyd	0.26	7.51	4	16.14	6.1	4.3	1.42	Vyd=8.396	5.59	18.04
P-2	Bvxc	0.506	7.47	4	16.08	6.1	4.6	1.33	Vxc=17.38	9.310	37.43
	Bvxd	0.33	7.47	4	16.08	6.1	4.6	1.33	Vxd=11.34	6.07	24.41
	Bvyc	0.4	7.47	4	16.08	6.1	4.6	1.33	Vyc=13.74	7.36	29.59
	Bvyd	0.26	7.47	4	16.08	6.1	4.6	1.33	Vyd=8.934	4.784	19.23
P-3	Bvxc	0.506	7.4	4	15.99	6.1	4.6	1.33	Vxc=17.22	9.31	37.22
	Bvxd	0.33	7.4	4	15.99	6.1	4.6	1.33	Vxd=11.23	6.07	24.27
	Bvyc	0.4	7.4	4	15.99	6.1	4.6	1.33	Vyc=13.62	7.36	29.422
	Bvyd	0.26	7.4	4	15.99	6.1	4.6	1.33	Vyd=8.85	4.784	19.12
P-4	Bvxc	0.524	8.24	4	17.12	6.1	4.3	1.42	Vxc=18.57	9.01	38.57
	Bvxd	0.324	8.24	4	17.12	6.1	4.3	1.42	Vxd=11.48	5.57	23.85

	Bvyc	0.4	8.24	4	17.12	6.1	4.3	1.42	Vyc=14.17	6.88	29.45
	Bvyd	0.26	8.24	4	17.12	6.1	4.3	1.42	Vyd=9.212	4.472	19.14
P-5	Bvxc	0.376	6.82	4	15.21	2.95	2.8	1.053	Vxc=7.18	4.21	16.01
	Bvxd	0	6.82	4	15.21	2.95	2.8	1.053	Vxd=0	0	0
	Bvyc	0.36	6.82	4	15.21	2.95	2.8	1.053	Vyc=6.87	4.03	15.33
	Bvyd	0.24	6.82	4	15.21	2.95	2.8	1.053	Vyd=4.58	2.69	10.22
P-6	Bvxc	0.467	6.82	4	15.21	4.6	2.8	1.643	Vxc=8.917	5.23	19.89
	Bvxd	0	6.82	4	15.21	4.6	2.8	1.643	Vxd=0	0	0
	Bvyc	0.33	6.82	4	15.21	4.6	2.8	1.643	Vyc=6.302	3.696	14.05
	Bvyd	0	6.82	4	15.21	4.6	2.8	1.643	Vyd=0	0	0
P-7	Bvxc	0.464	6.82	4	15.21	3.6	2.8	1.286	Vxc=8.86	5.197	19.76
	Bvxd	0.306	6.82	4	15.21	3.6	2.8	1.286	Vxd=5.84	3.43	13.03
	Bvyc	0.36	6.82	4	15.21	3.6	2.8	1.286	Vyc=10.16	5.96	22.66
	Bvyd	0	6.82	4	15.21	3.6	2.8	1.286	Vyd=0	0	0
P-8	Bvxc	0.467	6.82	4	15.21	4.6	2.8	1.643	Vxc=8.917	5.23	19.89
	Bvxd	0	6.82	4	15.21	4.6	2.8	1.643	Vxd=0	0	0
	Bvyc	0.33	6.82	4	15.21	4.6	2.8	1.643	Vyc=6.302	3.696	14.05
	Bvyd	0	6.82	4	15.21	4.6	2.8	1.643	Vyd=0	0	0
P-9	Bvxc	0.376	6.82	4	15.21	2.95	2.8	1.053	Vxc=7.18	4 .21	16.01
	Bvxd	0	6.82	4	15.21	2.95	2.8	1.053	Vxd=0	0	0
	Bvyc	0.36	6.82	4	15.21	2.95	2.8	1.053	Vyc=6.87	4.03	15.33
	Bvyd	0.24	6.82	4	15.21	2.95	2.8	1.053	Vyd=4.58	2.69	10.22
10	Bvxc	0.524	8.202	4	17.07	6.1	4.3	1.42	Vxc=18.48	9.013	38.46
	Bvxd	0.324	8.202	4	17.07	6.1	4.3	1.42	Vxd=11.43	5.573	23.78
	Bvyc	0.4	8.202	4	17.07	6.1	4.3	1.42	Vyc=14.11	6.88	29.36
	Bvyd	0.26	8.202	4	17.07	6.1	4.3	1.42	Vyd=9.17	4.472	19.08
11	Bvxc	0.437	8.11	4	16.95	6.1	4.6	1.33	Vxc=16.3	8.04	34.07
	Bvxd	0	8.11	4	16.95	6.1	4.6	1.33	Vxd=0	0	0
	Bvyc	0.36	8.11	4	16.95	6.1	4.6	1.33	Vyc=13.43	6.624	28.07
	Bvyd	0.24	8.11	4	16.95	6.1	4.6	1.33	Vyd=8.95	4.416	18.7
12	Bvxc	0.493	6.82	4	15.21	6.1	3.6	1.69	Vxc=12.10	7.09	26.99
	Bvxd	0	6.82	4	15.21	6.1	3.6	1.69	Vxd=0	0	0
	Bvyc	0.36	6.82	4	15.21	6.1	3.6	1.69	Vyc=8.84	4.008	15.24
	Bvyd	0.24	6.82	4	15.21	6.1	3.6	1.69	Vyd=5.89	3.456	13.14
13	Bvxc	0.437	8.11	4	16.95	6.1	4.6	1.33	Vxc=16.3	8.04	34.07
	Bvxd	0	8.11	4	16.95	6.1	4.6	1.33	Vxd=0	0	0
	Bvyc	0.36	8.11	4	16.95	6.1	4.6	1.33	Vyc=13.43	6.624	28.07
	Bvyd	0.24	8.11	4	16.95	6.1	4.6	1.33	Vyd=8.95	4.416	18.7
14	Bvxc	0.524	8.202	4	17.07	6.1	4.3	1.42	Vxc=18.48	9.013	38.46
	Bvxd	0.324	8.202	4	17.07	6.1	4.3	1.42	Vxd=11.43	5.573	23.78
	Bvyc	0.4	8.202	4	17.07	6.1	4.3	1.42	Vyc=14.11	6.88	29.36
	Bvyd	0.26	8.202	4	17.07	6.1	4.3	1.42	Vyd=9.17	4.472	19.08

Load transfer for cantilevers

Cantilever 1, 7,8,13

DL=6.82 *1.62m+3.875=14.92KN/m

LL=4 KN/m^2*1.62m=6.48 KN/m

PD=11.07 KN/m^2*1.62= 17.9KN/m

Cantilever 2, 9&12

DL=6.82KN/m^2*1.1m+13.2= 20.73KN/m

LL=4 KN/m^2*1.1m=4.4KN/m

PD=15.87KN/m^2*1.62= 25.71KN/m

Cantilever 3&5

DL=6.82KN/m^2*1m+2.875 = 9.69KN/m

LL=4 KN/m^2*1.m=4 KN/m

PD=10.02 KN/m^2*1= 10.02KN/m

Cantilever 4

DL=6.82KN/m^2*1.1m+9.00 = 16.51KN/m

LL=4 KN/m^2*1.1m=4.4 KN/m

PD=16.35 KN/m^2*1.1= 17.98KN/m

Cantilever 10&11

DL=6.82KN/m^2*1.2m+4.37 = 12.55KN/m

LL=4 KN/m^2*1.2m=4.8 KN/m

PD=11.14 KN/m^2*1.2= 13.36KN/m

Cantilever6

DL=6.82KN/m^2*1.1m+9.00 = 16.51KN/m

LL=4 KN/m^2*1.1m=4.4 KN/m

PD=15.87 KN/m^2*1.1= 17.46KN/m

Table 3-14 Shear load transfer from first to fifth typical floor slab to beam

		Total pd including plaster (KN/M)	cantilever load on beam	Beam self weight(250*350) 1.35(self weight of beam)	shear load	total load
P-1						
Axis(B, 1&2)	Vxc	16.14		11.8	36.37	64.31
Axis(A, 1&2)	Vxd	16.14	30.76	11.8	22.48	81.18
Axis(2, A&B)	Vyc	16.14	0	11.8	27.76	55.7
Axis(1, A&B)	Vyd	16.14	0	11.8	18.04	45.98
P-2						
Axis(B, 2&3)	Vxc	16.08	0	11.8	37.43	65.31
Axis(A, 2&3)	Vxd	16.08	22.89	11.8	24.48	75.24
Axis(3, A&B)	Vyc	16.08	0	11.8	29.59	57.47
Axis(2, A&B)	Vyd	16.08	0	11.8	19.23	47.11
P-3						
Axis(B, 4&5)	Vxc	15.99	0	11.8	37.22	74.25
Axis(A, 4&5)	Vxd	15.99	28.1	11.8	24.27	80.06
Axis(5, A&B)	Vyc	15.99	0	11.8	29.42	57.21
Axis(4, A&B)	Vyd	15.99	0	11.8	19.12	46.91
P-4						
Axis(B, 5&6)	Vxc	17.12	0	11.8	38.57	66.87
Axis(A, 5&6)	Vxd	17.12	34.95	11.8	24.82	76.89
Axis(5, A&B)	Vyc	17.12	0	11.8	29.45	58.37
Axis(6, A&B)	Vyd	17.12	0	11.8	19.14	48.06
P-5				11.8		
Axis(2, B&C)	Vxc	15.21	0	11.8	16.01	43.02
Axis(1, B&C)	Vxd	15.21	0	11.8	10.02	37.03
Axis(C, 1&2)	Vyc	15.21	0	11.8	15.33	42.34
Axis(B, 1&2)	Vyd	15.21	0	11.8	0	27.01
P-6				11.8		
Axis(3, B&C)	Vxc	15.21	0	11.8	19.89	46.9
Axis(2, B&C)	Vxd	15.21	0	11.8	0	27.01
Axis(C, 2&3)	Vyc	15.21	0	11.8	14.05	41.06
Axis(B, 2&3)	Vyd	15.21	0	11.8	0	27.01

P-7						
Axis(4, B&C)	Vxc	15.21	0	11.8	19.76	46.78
Axis(3, B&C)	Vxd	15.21	0	11.8	0	27.01
Axis(C, 3&4)	Vyc	15.21	0	11.8	22.6	49.61
Axis(B, 3&4)	Vyd	15.21	0	11.8	13.03	40.04
P-8						
Axis(5, B&C)	Vxc	15.21	0	11.8	19.89	46.9
Axis(4, B&C)	Vxd	15.21	0	11.8	0	27.01
Axis(C, 4&5)	Vyc	15.21	0	11.8	14.05	41.06
Axis(B, 3&4)	Vyd	15.21	0	11.8	0	27.01
P-9						
Axis(5, B&C)	Vxc	15.21	0	11.8	16.01	43.02
Axis(6, B&C)	Vxd	15.21	0	11.8	0	27.01
Axis(C, 5&6)	Vyc	15.21	0	11.8	15.33	42.34
Axis(B, 6&5)	Vyd	15.21	0	11.8	10.02	37.03
P-10						
Axis(C, 1&2)	Vxc	17.07	30.76	11.8	38.46	99.09
Axis(D, 1&2)	Vxd	17.07	0	11.8	23.78	52.65
Axis(2, C&D)	Vyc	17.07	0	11.8	29.36	58.23
Axis(1, C&D)	Vyd	17.07		11.8	19.08	47.95
P-11						
Axis(C, 2&3)	Vxc	16.95		11.8	34.07	62.82
Axis(D, 1&2)	Vxd	16.95	16.21	11.8	0	44.96
Axis(2, C&D)	Vyc	16.95	0	11.8	28.07	56.82
Axis(3, C&D)	Vyd	16.95	0	11.8	18.07	46.82
P-12						
Axis(C, 3&4)	Vxc	15.21	0	11.8	26.99	54
Axis(D, 3&4)	Vxd	15.21	0	11.8	0	27.01
Axis(4, C&D)	Vyc	15.21	0	11.8	15.24	42.25
Axis(3, C&D)	Vyd	15.21	0	11.8	13.14	40.15
P-13				11.8		
Axis(C, 4&5)	Vxc	16.95	0	11.8	34.07	62.82
Axis(D, 4&5)	Vxd	16.95	26.22	11.8	0	54.97
Axis(4, C&D)	Vyc	16.95	0	11.8	28.07	56.82
Axis(5, C&D)	Vyd	16.95	0	11.8	18.7	47.45
P-14						
Axis(C, 5&6)	Vxc	17.07	0	11.8	38.07	66.94
Axis(D, 5&6)	Vxd	17.07	30.78	11.8	23.78	83.43
Axis(5, C&D)	Vyc	17.07	0	11.8	29.36	71.36
Axis(6, C&D)	Vyd	17.07	0	11.8	19.08	47.95

CHAPTER 4

4. STAIR CASE ANALYSIS AND DESIGN

Design Analysis of Stair case

For the purpose of easy and comfortable passage over the different floor levels of the building, there is a staircase provision for the building.

Design procedure

- Determination of depth for deflection: This 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: This 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.
- Reinforcement Detailing: The arrangement of reinforcement.

Staircase layout

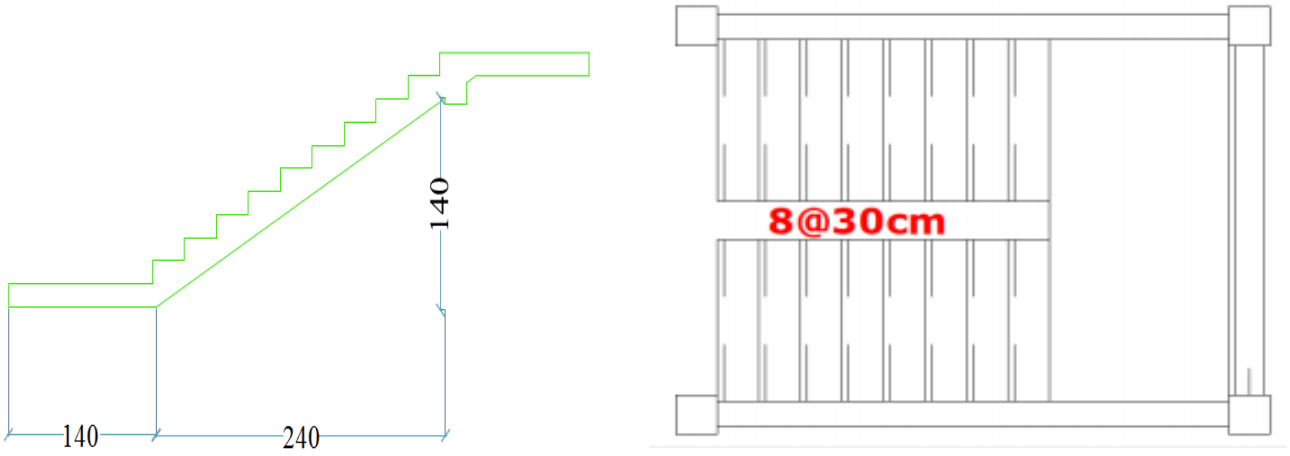


Figure 4-1 Lay Out Of Stair

Material Data

- Unit weight of cement screed = 23 KN/m^3
- Unit weight of concrete = 25KN/m^3
- Unit weight of plastering = 23 KN/m^3
- Thickness of cement screed = 2cm
- Thickness of plastering = 2cm
- fck = 20MPa
- fcd = 11.33MPa
- fyk=500MPa
- Take 1m width strip

Concrete cover design

- Concrete cover 22mm similar procedure with the slab concrete cover.

Design of stair

- No of riser in one flight = 9
- No trade =8
- Width of trade=30cm
- Riser height=15cm
- tan $\theta=\frac{1.4}{2.4}\Rightarrow\theta=\tan^{-1}(\frac{1.4}{2.4})$,$\theta=30^\circ$
- Inclined length of each step or breadth of step along slope(B) $=\sqrt{R^2+T^2}$
  $=\sqrt{0.15^2+0.3^2}$ = **0.34m**

Determination of minimum depth for deflection

According to EBCS-2 2015,(page 133) slab determination can be calculated by using the following formula and consideration.

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

Where;

$\frac{l}{d}$:- is the limit span/depth

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

$$N = \left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \text{ if } \rho > \rho_o$$

$$N = \left[11 + 1.5\sqrt{f_{ck}}\right] \text{ if } \rho = \rho_o$$

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

ρ_o:-is the reference reinforcement ratio

$$= \frac{\sqrt{f_{ck}}}{1000}$$

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)

$$F1 = \frac{500}{f_{yk}}$$

F2=1

*F*3=1

Table 4-1 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 two-way spanning simply supported slab	1.1	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1.13	18	26
Interior span of beam or one-way or two-way spanning slab	1.15	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1.12	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

Calculation of minimum depth for deflection

A. ρ_o:-is the reference reinforcement ratio

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

B. $N = \left[11 + 1.5\sqrt{f_{ck}}\right]$ =$N = \left[11 + 1.5\sqrt{20}\right]$ = **17.708**

C. fyk - Grade of steel, S-500

D. $F1 = \frac{500}{f_{yk}} = \frac{500}{500} = \mathbf{1}$

E. F2= 1, F3= 1

Depth determination: Serviceability Requirement For end panel (L=Lx=3400mm)

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

K= 1.13, N=17.708, F1= 1, F2=1, F3=1

$$\frac{l}{d} = 17.708 * 1.13 * 1 * 1 * 1 = \mathbf{23.0204}$$

$$d = \frac{3400mm}{23.0204} = \mathbf{147.69mm}$$

Depth determination: Serviceability Requirement For Interior panel (L=Lx=3400mm)

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

K= 1.5, N=17.708, F1= 1, F2=1, F3=1

$$\frac{l}{d} = 17.708 * 1.5 * 1 * 1 * 1 = \mathbf{26.562}$$

$$d = \frac{3400mm}{26.562} = \mathbf{128.00mm}$$

Governing effective depth of slab

$$D = dmin + cover + \frac{\Phi}{2}$$

Depth determination: Serviceability Requirement For end panel

Using Φ12mm bar and clear cover of 22mm

- $D = 147.69 + \frac{12}{2} + 22 = \mathbf{175.69 use\ 180mm}$

Depth determination: Serviceability Requirement For interior panel
Using Φ12mm bar and clear cover of 22mm

Total depth

- $D = 128.00 + \frac{12}{2} + 22 = \mathbf{156.00mm\ use\ 160mm}$

Therefore, the depth provide for serviceability requirement for slab depth for deflection is **180mm**

- Depth in shorter direction
  - $dx = 180 - 12/2 - 22 = \mathbf{152mm}$
- Depth in the longer direction
  - $dy = 180 - \frac{12}{2} - 12 - 22 = \mathbf{140mm}$

Load Determinations
Table Materials data used in stair

Material	Unit weight(KN/M^3)	Thickness(m)
Marbel	27	0.03
Cement screed	23	0.03
Concrete	25	0.18
Pastering	23	0.02

Live load

The live loads for design of staircase are to be taken from table 2.10 EBCS-1 on loading standards. The value used is category A.

qk =5kN/m^2total live load on stair case = 5kN/m2*1m = **5kN/m**

The live load given in codes assumes that it is acting along the horizontal span and directly used in calculation.

Load Computation on the stair

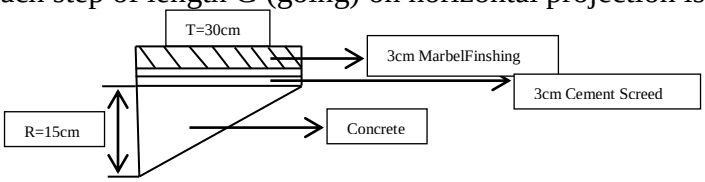
1. **Dead load for flight (inclined slab)**

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

Take 1m width strip

A. Step dead load

Dead load on each step of length G (going) on horizontal projection is given by



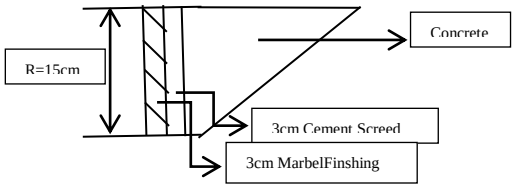
Dead load of cement screed=$t_{cement\ screed}$*$\delta_{cement\ screed}$= 0.03*23 KN/m^3=**0.69 KN/m**

Dead load of finishing=$t_{finshing}$*$\delta_{finshing}$=0.03*27 KN/m^3=**0.81 KN/m**

Dead load of Concrete= 1/2*h*δ_{cement} =1/2*0.15m*25 KN/m^3 =**1.875 KN/m**

Therefore DL of step= 0.69 KN/m + 0.81 KN/m + 1.875 KN/m=**3.375 KN/m**

B. Riser Dead Load

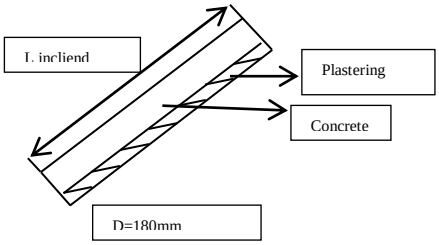


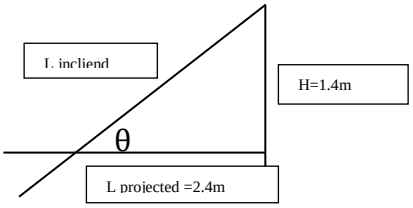
Riser Finishing=$\frac{\text{No of riser}(h_{cs}*t_{cs}*\delta_{cs})}{\text{Projected Length}} = \frac{9(0.15*0.03*27)}{8*0.3}$ = **0.4556 KN/m**

Riser cement screed =$\frac{\text{No of riser}(R_{cs}*t_{cs}*\delta_{cs})}{\text{Projected Length}} = \frac{9(0.15*0.03*23)}{8*0.3}$ = **0.388 KN/m**

Therefore D.L of riser (15cm) = 0.4556 KN/m + 0.388 KN/m = **0.8437KN/m**

C. Wasit Dead Load





Tan θ=$\frac{1.4}{2.4}$⇨θ = Tan^{-1}($\frac{1.4}{2.4}$) ,θ=**30.26°**

Sin $\theta = \frac{1.4}{L_{incliend}}$

$L_{incliend}$= $\frac{1.4}{\text{Sin }\theta} = \frac{1.4}{\text{Sin } 30.26} = \mathbf{2.778m}$

Dead load of concrete = $\frac{D*L_{inc}*\delta_{concrete}}{\text{Length of projected}} = \frac{0.18*2.778*25}{2.4}$ = **5.21KN/m**

Dead load of plastering = $\frac{t_{plastering}*L_{inc}*\delta_{plastering}}{\text{Length of projected}} = \frac{0.02*2.778*23}{2.4}$ = **0.53KN/m**

Therefore Dead load of waist = 5.21KN/m + 0.53K/m = **5.74KN/m**

Total Dead Load and design load for the inclined slab (steps)

Total dead load = Dead load of Step + dead load of riser + dead load of waist

= 3.375 KN/m +0.8437KN/m + 5.74 KN/m= **9.96 KN/m**

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

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

= 1.35*9.96KN/m + 1.5*5 KN/m

Pd = **20.946KN/m**

2. Dead load for Landing

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

=(tfin*γfin)+(tcs*γcs)+(tc$*\gamma$c)+(tpl*γpl)

= (0.03m*27 KN/m^3)+(0.03*23 KN/m^3) +(0.18m*25 KN/m^3)+(0.02m*23 KN/m^3)

=**6.46 KN/m**

Therefore D.L of Landing=**6.46 KN/m**

Total Dead Load and design load for the landing

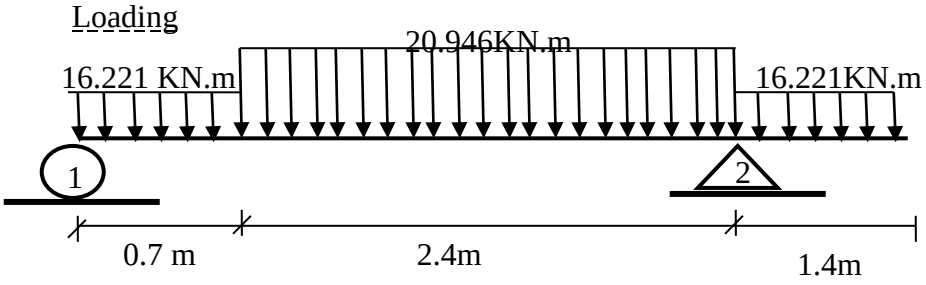
D.L=6.46KN/m L.L=5 KN/m2 * 1m= 5KN/m

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

= 1.35*6.46 KN/m + 1.5*5 KN/m Pd

=**16.221 KN/m**

Moment Analysis



Moment Analysis

+ΣM at 1=0

20.946*2.4m*(0.7+2.4/2)+16.22*1.4*(3.1+1.4/2)-R_2*3.1m+16.221*(0.7)*0.7/2=0

3.1R_2=185.78 KN.m

R_2=59.93 KN

ΣFy=0 (↓)

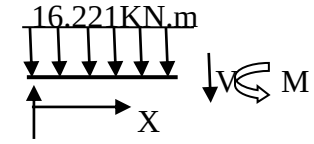
R_1+R_2=16.22 KN/m*0.7 m+20.946 KN/m*2.4 m+16.221 KN/m*1.4m

R_1=84.33KN-59.93KN

R_1=24.403KN

Analyzing using the method of section,

For Z=0, to Z=0.7m (Z in measured from support 1)



24.4KN

$M(x)+16.221KN/m* x^2/2-24.4x=0$

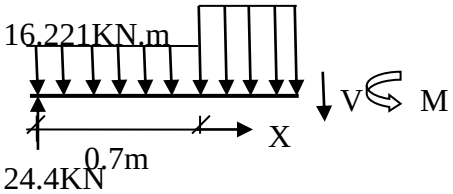
$M(x)= 24.4x-16.221/2\ x^2$

M(0)=0

At Z=0.7, x=0.7

M(0.8)=13.1KN.m

For Z=0.7 to Z=3.1m



$M(x)+20.946\ KN/m*x^2/2+16.221\ KN/m*0.7\ m*(x+0.7/2)=24.4KN\ (x+0.8)$

$M(x)=-20.946*x^2/2+35.75x+9.15$

At Z=0.7m, x=0

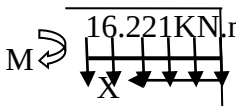
M(0)=9.15KN.m

At Z=3.1m, X=2.4m

M(2.4)= -60.32KN.m+85.8KN.m+9.15

M(2.4)=34.63KN.m

For Z=3.1 to Z=4.5m



$M(x)=-16.221* x^2/2$

At Z=4.9m, x=0

M(0)=0

At Z=3.1m, x=1.4m

M(1.4)= -15.965KN.m

Mmax (+ve) is b/n Z=0.7 to Z=3.1

$M(x)= -20.941x^2/2+35.75x+9.15$

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

$dM(x)/dx=d/dx(-20.946x^2+35.75x+9.15)=0$

$=-20.946x+35.75=0$, x=0.853m , Z=0.853m

M(0.853m)=17.88KN.m

Shear force computation

V(x)=dM(x)/dx

For Z=0 to Z=0.8m

$M(x)=24.4x-16.221x^2/2$

$dM(x)/dx=V(x)=-16.221x+24.4$

at Z=0, x=0

V(0)=24.4 KN

At Z=0.7m, x=0.7m

V(0.7)=13.04KN

For Z=0.7m to Z=3.1m

$M(x)=-20.946x^2/2+35.75x+9.15$

$dM(x)/dx=V(x)=-20.946x+35.75$

at Z=0.7m, x=0

V(0)=35.75 KN

At Z=3.1m, x=2.4m

V(2.4)= -14.52KN

For Z=3.1m to Z=4.5m

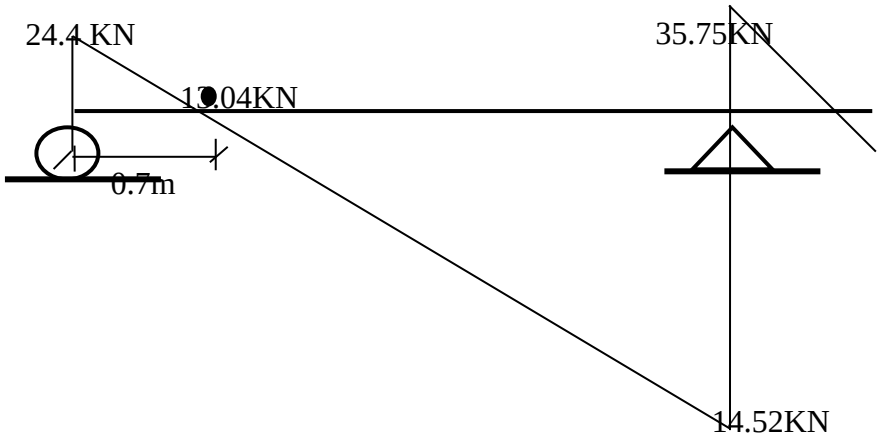
$M(x)= -16.22x^2/2$

$dM(x)/dx= -16.22x$

at Z=3.1m, x=1.4m

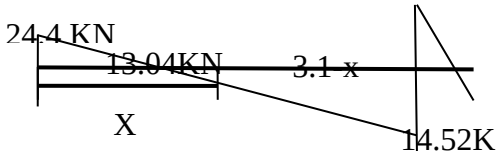
V(1.4)= -22.7 KN

Shear force diagram



Bending moment diagram

35.75



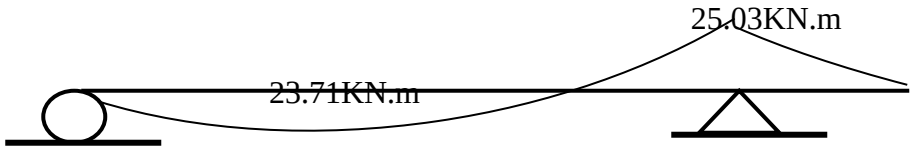
$\frac{24.4}{x}=\frac{14.52}{3.1-x}$

24.4(3.1-x)=14.52x

X=1.94m

½*24.4*1.94=23.71 M_{ma}

Bending moment diagram



Reinforcement

Material Data, C-25,

S-400

Effective depth, d=151mm

Width, bs=1000mm

Using M = 25.03KN/m, calculate reinforcement,

μsd = $M/fcd*b*d^2$= $25.03*10^6/11.33*1000*151^2$=0.097

usingμsd =0.097

Kz=0.91

Z=Kz*d =0.91*151 = 137.41mm

As=M/(fyd*z) =25.03*10^6/(137.41*347.82) =523.7mm^2

To calculate spacing by selecting diameter of bar as

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

As=calculated area of steel

S =1000∗153.9/523.7=293.9 there for use 300mm

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

Asmin= 0.26*2.2Mpa/400Mpa*1000mm*151mm

= 216mm^2

With d = 180-22-7=151mm, for ϕ14main rebar

Asmin=216mm^2<Ascal=523.77mm^2ok

smax ≤ {$2D$= 2∗180=360mm

$400mm$,
Scalc=300mm
Smax = 300mm

Provide ϕ14c/c300mm

$A_{sprovided}$=1000*153.9/300=512.87mm^2

B) Transverse reinforcement

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

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

As, ≥ 0.2* 216=43.2mm^2<$Asmin$

Therefore take the minimum reinforcement

Using Ø8 main transverse rebar

S =ba_s /As= $1000 * 50.26/ 43.2=1163.4$mm

S_{max}=2D=2*180=360
400
S_{calc}=1163.4

Therefore provide ϕ8c/c360mm

CHAPTER FIVE

5. LATERAL LOAD

General

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

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

Design Basis:

- ✓ Ground condition: (Assume sub soil class B)
- ✓ Seismic zone: (Shewa Robit, Zone 4)

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 Debere berhan, which is located in Longitude 39.5565-E and Latitude 9.6235-N. it is categorized seismic hazard zone four.

- ➢ 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, the procedures in the story shear force distribution include:

- ✓ Determination of center of mass for each floor
- ✓ Determination of center of stiffness for each floor
- ✓ Determination of eccentricity for each floor
- ✓ Calculation of the direct shear force for each frame system
- ✓ Weight and Center of mass calculation
- ✓ Computation of weight of structure along with center of mass

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

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = III

Wind Direction = 0;90 degrees

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

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

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

Air Density, ρ $\rho = 1.25$

Top Story = ROOF

Bottom Story = Base

Include Parapet = No

Factors and Coefficients

Structural Factor, $c_{s<c}$ [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_I [EC 4.4(1)] $k_I = 1$

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

Turbulence Intensity, I_v [EC 4.4(1)]

$$I_v = \frac{k_I}{c_o(z)\, ln\left(\frac{z}{z_0}\right)} \text{ for } z_{min} \le z \le z_{max}$$

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

Terrain Factor, k_r [EC 4.3.2(1) Eq. 4.5]

$$k_r = 0.19 \left(\frac{z_0}{0.07}\right)^{0.05}$$

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

$$c_r(z) = k_r\, ln\left(\frac{z}{z_0}\right) \text{ for } z_{min} \le z \le z_{max}$$

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

Lateral Loading

Peak Velocity Pressure, $<q_{p>(z)}$ [EC 4.5(1) Eq. 4.8]

$$q_p(z) = [1 + 7\, I_v(z)]\frac{1}{2}\rho\, [c_r(z)\, c_o(z)\, v_b]^2$$

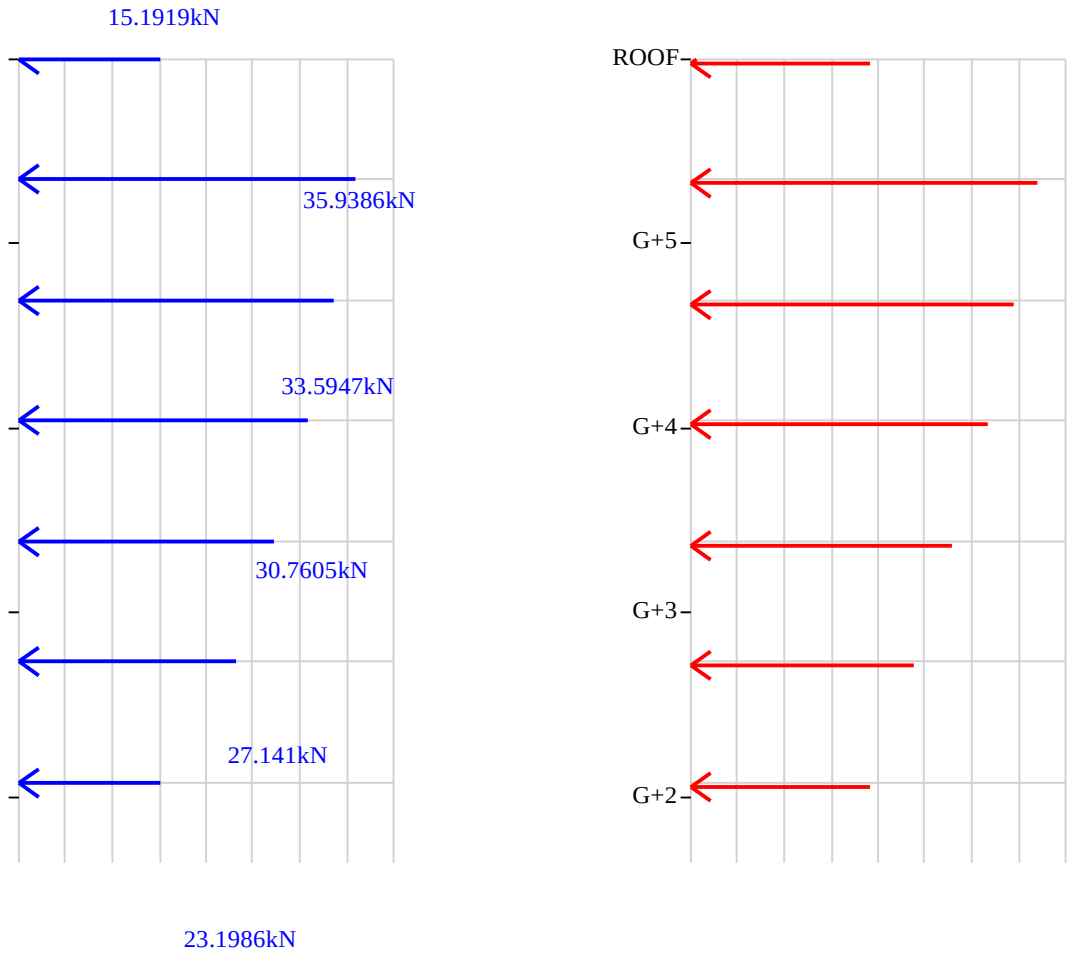
Wind Pressure, w [EC 5.2(1) Eq. 5.1]

$$w = q_p(z)\, c_s\, c_d\, (c_{p,wind} + c_{p,lee})$$

Applied Story Forces

ROOF 23.0165kN

Lateral Load to Stories - 0°



Story	Elevation m	X-Dir kN	Y-Dir kN
ROOF	20	15.1919	0
G+5	17	35.9386	0
G+4	14	33.5947	0
G+3	11	30.7605	0
G+2	8	27.141	0
G+1	5	23.1986	0
ground	2	15.1188	0
Base	0	0	0

23.0165
44.3277
41.4366
37.9409
33.4765
28.6138
22.9057

Story	Elevation m	X-Dir kN	Y-Dir kN
ROOF	20	0	
G+5	17	0	
G+4	14	0	
G+3	11	0	
G+2	8	0	
G+1	5	0	
ground	2	0	
Base	0	0	0

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

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = III

Wind Direction = 0 degrees

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

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

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

Air Density, ρ $\rho = 1.25$

Top Story = ROOF

Bottom Story = Base

Include Parapet = No

Factors and Coefficients

Structural Factor, $c_{s<c}$ [EC 6.2(1)]

Elevation, z_0

Minimum Elevation, z_{min}

Maximum Elevation, z_{max} $c_s c_d = 0.85$

Turbulence Factor, k_l [EC 4.4(1)] $z_0 = 0.3$

Orography Factor, c_o [EC 4.3.3] $z_{min} = 5$

$z_{max} = 200$

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

$= I_v(z_{min})$ for $z < z_{min}$ $c_o = 1$

Terrain Factor, k_r [EC 4.3.2(1) Eq. 4.5] $k_r = 0.19 \left(\frac{z_0}{0.07}\right)^{0.05}$

Roughness Factor, $<c_{r(z)}$ [EC 4.3.2(1) Eq. 4.4] $c_r(z) = k_r\, ln \left(\frac{z}{z_0}\right)$ for $z_{min} \le z \le z_{max}$

$= c_r(z_{min})$ for $z < z_{min}$

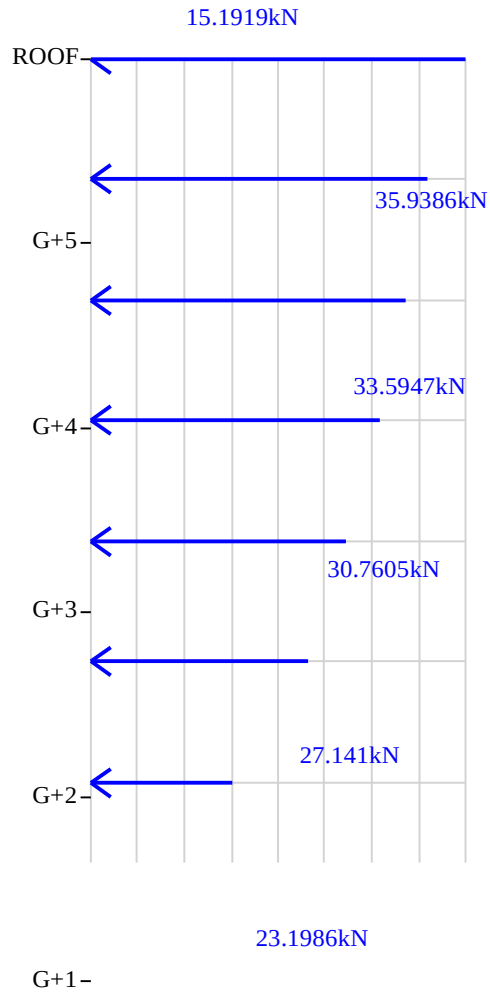
Lateral Loading

Peak Velocity Pressure, $<q_{p>(z)}$ [EC 4.5(1) Eq. 4.8]

Wind Pressure, w [EC 5.2(1) Eq. 5.1] $q_p(z) = [1 + 7\, I_v(z)] \frac{1}{2} \rho\, [c_r(z)\, c_o(z)\, v_b]^2$

Applied Story Forces

Lateral Load to Stories - 0°



Story	Elevation m	X-Dir kN	Y-Dir kN
ROOF	20	15.1919	0
G+5	17	35.9386	0
G+4	14	33.5947	0
G+3	11	30.7605	0
G+2	8	27.141	0
G+1	5	23.1986	0
ground	2	15.1188	0
Base	0	0	0

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

Direction and Eccentricity

Direction = X - Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

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

Structure Height Above Base, H $H = 20\ m$

Factors and Coefficients

Country = Other

Design Ground Acceleration, a_g

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] $T_B = 0.2\ sec$

Constant Acceleration Period Limit, T_C [EC Table 3.3] $T_C = 0.8\ sec$

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, $S_{d(T}$ [EC 3.2.2.5(4) Eq. 3.13]

$$\left[\frac{2}{3}\ \frac{T}{T_B}\left(\frac{2.5}{q}\ \frac{2}{3}\right)\right]$$

$$2.5$$

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

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

Equivalent Lateral Forces

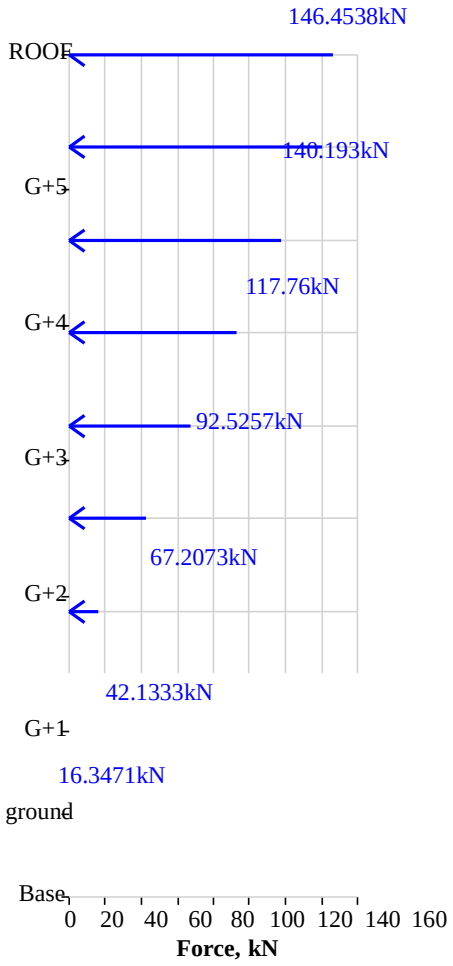
Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F (kN)
X - Ecc. Y	1.048	16911.8633	622.6201

Applied Story Forces

Lateral Load to Stories - X



Story	Elevation m	X-Dir kN	Y-Dir kN
ROOF	20	146.4538	0
G+5	17	140.193	0
G+4	14	117.76	0
G+3	11	92.5257	0
G+2	8	67.2073	0
G+1	5	42.1333	0
ground	2	16.3471	0
Base	0	0	0

Direction and Eccentricity

Direction = X + Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

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

Structure Height Above Base, H $H = 20\ m$

Factors and Coefficients

Country = Other

Design Ground Acceleration, a_g

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] $T_B = 0.2\ sec$

Constant Acceleration Period Limit, T_C [EC Table 3.3] $T_C = 0.8\ sec$

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)] $q = 4.9$

$$\left[\frac{2}{3}\frac{T}{T_B}\left(\frac{2.5}{q}\frac{2}{3}\right)\right]$$

Seismic Response

Spectral Response Acceleration, $S_{d(T}$
[EC 3.2.2.5(4) Eq. 3.13]

$$2.5$$

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

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

Equivalent Lateral Forces

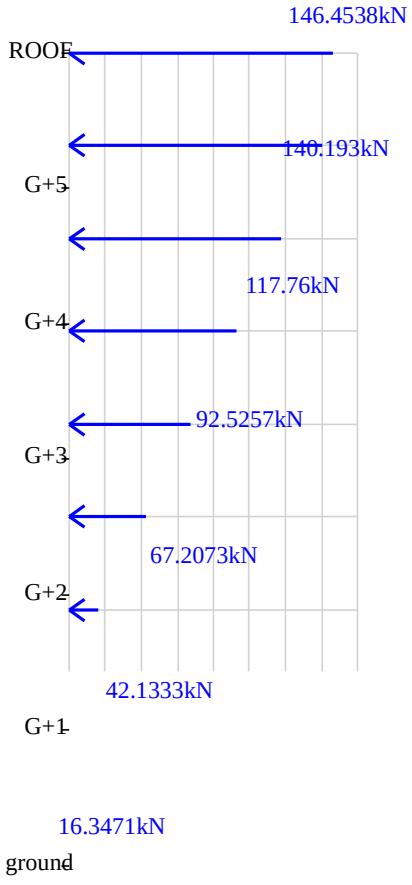
Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\,\lambda$

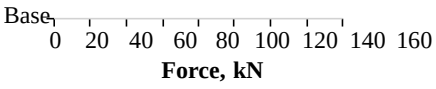
Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F (kN)
X + Ecc. Y	1.048	16911.8633	622.6201

Applied Story Forces

Lateral Load to Stories - X





Story	Elevation m	X-Dir kN	Y-Dir kN
ROOF	20	146.4538	0
G+5	17	140.193	0
G+4	14	117.76	0
G+3	11	92.5257	0
G+2	8	67.2073	0
G+1	5	42.1333	0
ground	2	16.3471	0
Base	0	0	0

Direction and Eccentricity

Direction = Y - Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

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

Structure Height Above Base, H $H = 20\ m$

Factors and Coefficients

Country = Other

Design Ground Acceleration, a_g

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] $T_B = 0.2\ sec$

Constant Acceleration Period Limit, T_C [EC Table 3.3] $T_C = 0.8\ sec$

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, $S_{d(T}$ [EC 3.2.2.5(4) Eq. 3.13] $\left[\frac{2}{3}\ \frac{T}{T_B}\left(\frac{2.5}{q}\ \frac{2}{3}\right)\right]^{2.5}$

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

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

Equivalent Lateral Forces

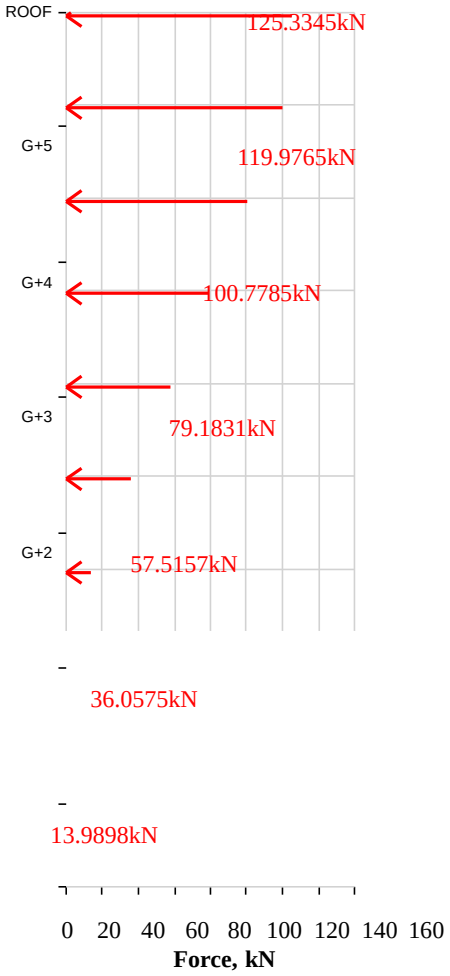
Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1)\,\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F (kN)
Y - Ecc. X	1.224	16911.8633	532.8357

Applied Story Forces

Lateral Load to Stories - Y



Story	Elevation m	X-Dir kN	Y-Dir kN
ROOF	20	0	125.3345
G+5	17	0	119.9765
G+4	14	0	100.7785
G+3	11	0	79.1831
G+2	8	0	57.5157
G+1	5	0	36.0575
ground	2	0	13.9898
Base	0	0	0

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

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

Structure Height Above Base, H $H = 20\ m$

Factors and Coefficients

Country = Other

Design Ground Acceleration, a_g

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] $T_B = 0.2\ sec$

Constant Acceleration Period Limit, T_C [EC Table 3.3] $T_C = 0.8\ sec$

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)] $q = 4.9$

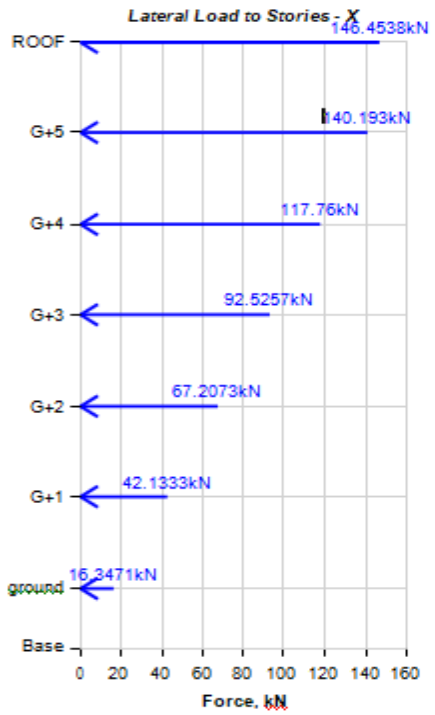
Seismic Response

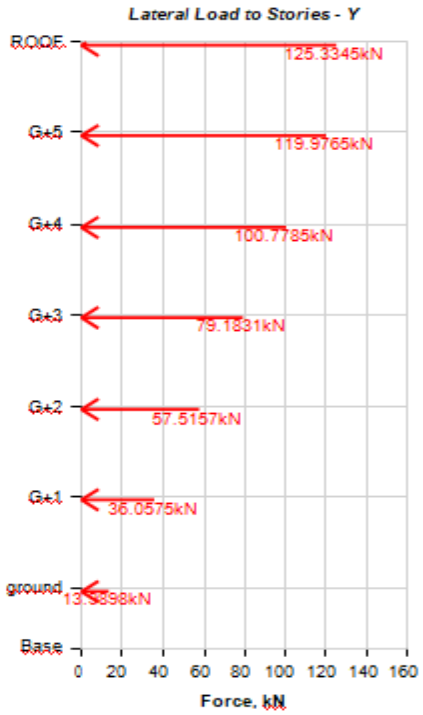
Spectral Response Acceleration, $S_{d(T}$ [EC 3.2.2.5(4) Eq. 3.13]

$$\left[\frac{2}{3}\ \frac{T}{T_B}\left(\frac{2.5}{q}\ \frac{2}{3}\right)\right]^{2.5}$$

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

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





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)
X	1.048	16911.8633	622.6201
Y	1.224	16911.8633	532.8357
X + Ecc. Y	1.048	16911.8633	622.6201
Y + Ecc. X	1.224	16911.8633	532.8357
X - Ecc. Y	1.048	16911.8633	622.6201
Y - Ecc. X	1.224	16911.8633	532.8357

Story	Elevation m	X-Dir kN	Y-Dir kN
ROOF	20	146.4538	0
G+5	17	140.193	0
G+4	14	117.76	0
G+3	11	92.5257	0
G+2	8	67.2073	0
G+1	5	42.1333	0
ground	2	16.3471	0
Base	0	0	0

Story	Elevation m	X-Dir kN	Y-Dir kN
ROOF	20	0	125.3345
G+5	17	0	119.9765
G+4	14	0	100.7785
G+3	11	0	79.1831
G+2	8	0	57.5157
G+1	5	0	36.0575
ground	2	0	13.9898
Base	0	0	0

Table 5.2 - Centers of Mass and Rigidity (continued)

Story	Diaphragm	Mass X kg	Mass Y kg	XCM m	YCM m	Cumulative X kg	Cumulative Y kg	XCCM m	YCCM m	XCR m	YCR m
G+2	D1	252427.17	252427.17	10.1998	8.6477	1225734	1225734	10.2788	8.6243		
G+1	D1	253200.73	253200.73	10.2039	8.676	1478934.73	1478934.73	10.266	8.6331		
ground	D1	245595.36	245595.36	10.5773	8.7539	1724530.09	1724530.09	10.3103	8.6504		

Table 5.2 - Centers 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	XCR m	YCR m
ROOF	D1	220029.05	220029.05	10.6895	8.6602	220029.05	220029.05	10.6895	8.6602		
G+5	D1	247791.71	247791.71	10.1825	8.5194	467820.77	467820.77	10.421	8.5857		
G+4	D1	252743.03	252743.03	10.1867	8.6484	720563.8	720563.8	10.3388	8.6077		
G+3	D1	252743.03	252743.03	10.1867	8.6484	973306.83	973306.83	10.2993	8.6182		

CHAPTER SIX

6. FRAME ANALYSIS

Frame Analysis Procedure using ETABS Our frame was analyzed using ETABS 2016. The procedure for modeling the frame will be as follows:

STEP 1: Plot Grid Coordinates

Plot grid coordinates that represent the given structural layout.

STEP 2: Define Material

Our building is made of reinforced concrete structures. Therefore, the materials which we define are concrete & steel rebar.

A. Concrete

- ✓ Grade: C-25
- ✓ Symmetry type: Isotropic
- ✓ Modulus of elasticity: 29GPA
- ✓ Poisson's ratio: 0.2
- ✓ Shear modulus (G): 12916.67MPa
- ✓ Coefficient of thermal expansion: 10 E-6
- ✓ Unit weight: 25KN/m3

B. Rebar

- ➢ Grade: S-500
- ➢ Unit weight: 78.49KN/m3
- ➢ Modulus of elasticity: 210000000MPa
- ➢ Poisson's ratio: 0.3
- ➢ Shear modulus (G): 76903069
- ➢ Minimum yield stress, fy: 435MPa
- ➢ Minimum tensile stress, fu: 500MPa
- ➢ Directional symmetry type: Uniaxial

STEP 3: Define Load and Frame Section .

We define three types of Frame Section those are Square Column (55x55 cm), Beam (40*35cm) and top tie beam (35*25cm).These frame section has the C-25 material and S-500 rebar defined in step 1.

Define the following loads in the load pattern.

- Dead load

Live load

- Wind load (X+)
- Wind load (X-)
- Wind load (Y+)
- Wind load (Y-)
- Seismic (X+)
- Seismic (X-)
- Seismic (Y+)
- Seismic (Y-)

Then, we have used the default load combinations in the ETABS itself.

Beam Cross section

- ✓ Top tie beam; 350x4250cm
- ✓ Intermediate beam: 400x350
- ✓ Grid beam; 400x300

Column

Cross section: 400x400cm

Those materials are defined in which their composition is C-25 and S-500 defined previously.

STEP 4: Draw the structure

Use the previously defined frame sections to draw the structure and it also includes assigning of restraints.

The restraints are assumed to be fixed joints.

STEP 5: Assigning loads

Then we assigned Fi (story shear) as a joint load on each frame joint and we analyze the 3D frame separately for each EQ coming from x and y direction. Finally, we have assigned all the transferred loads which are the dead and live load from slab and wall to the beams.

STEP 6: Analysis

Finally run the model and check for any errors.

ETABS Software Output

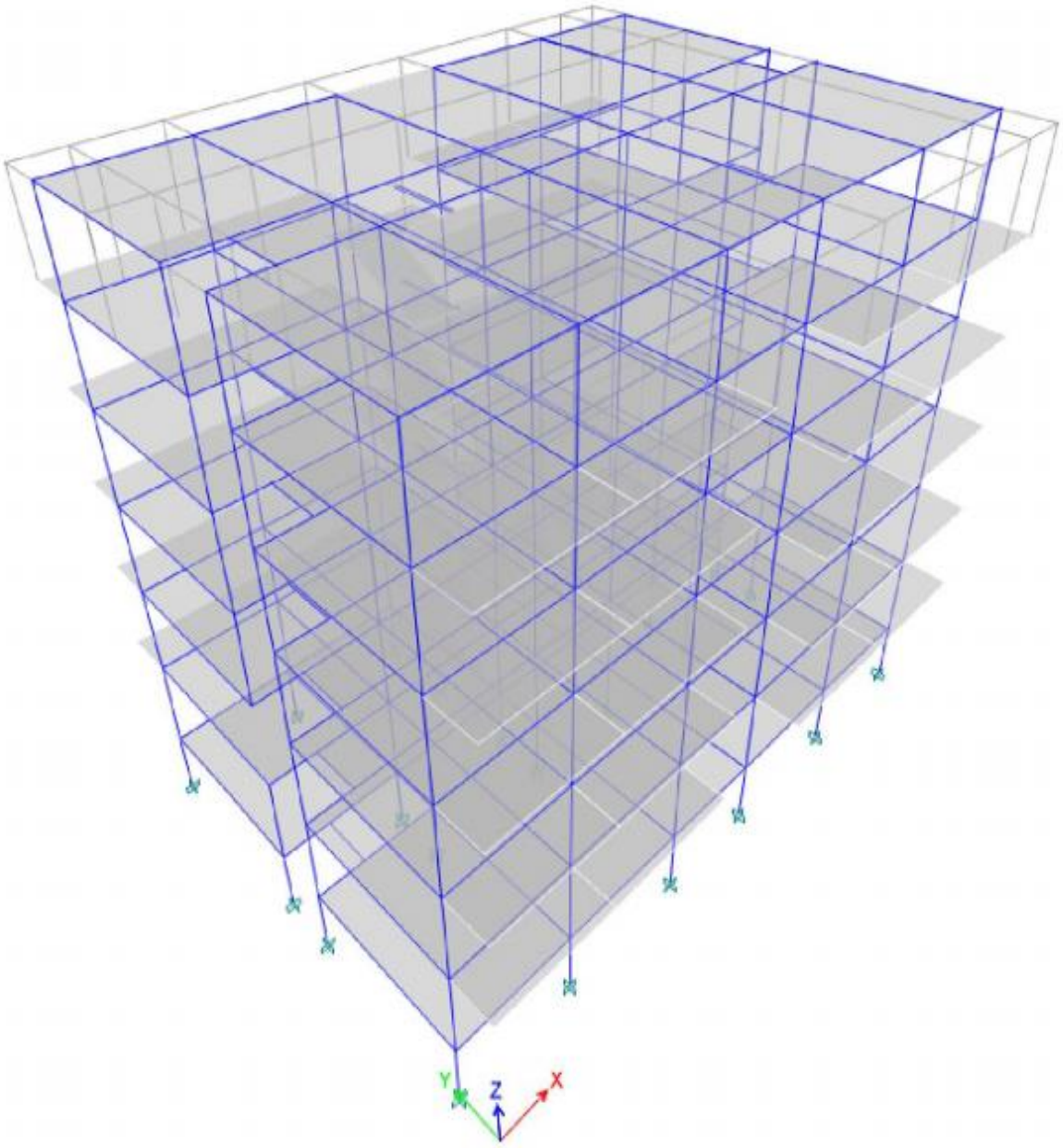


Figure 6-1Model of the building

AXIS A

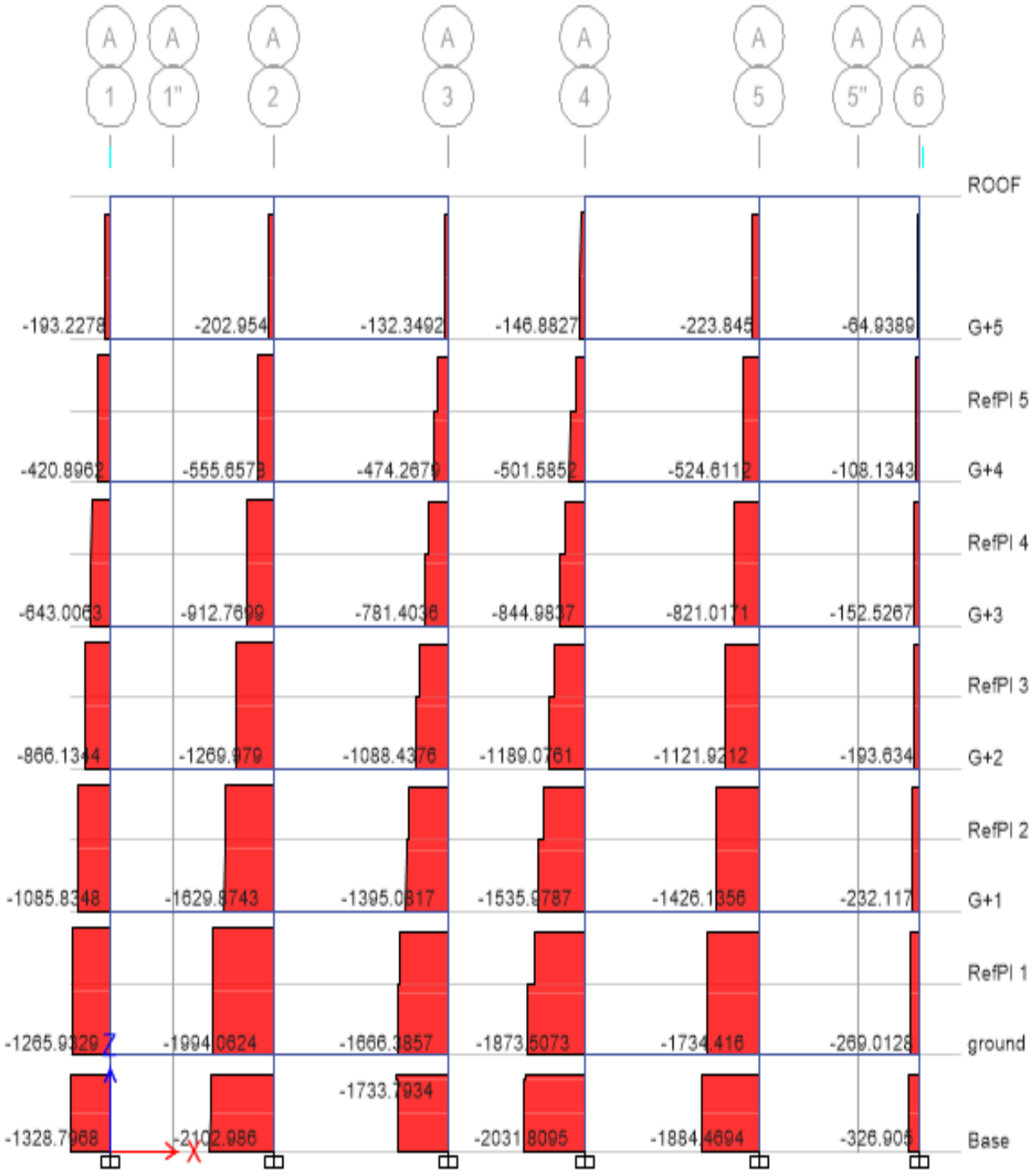


Figure 6-2 axial load

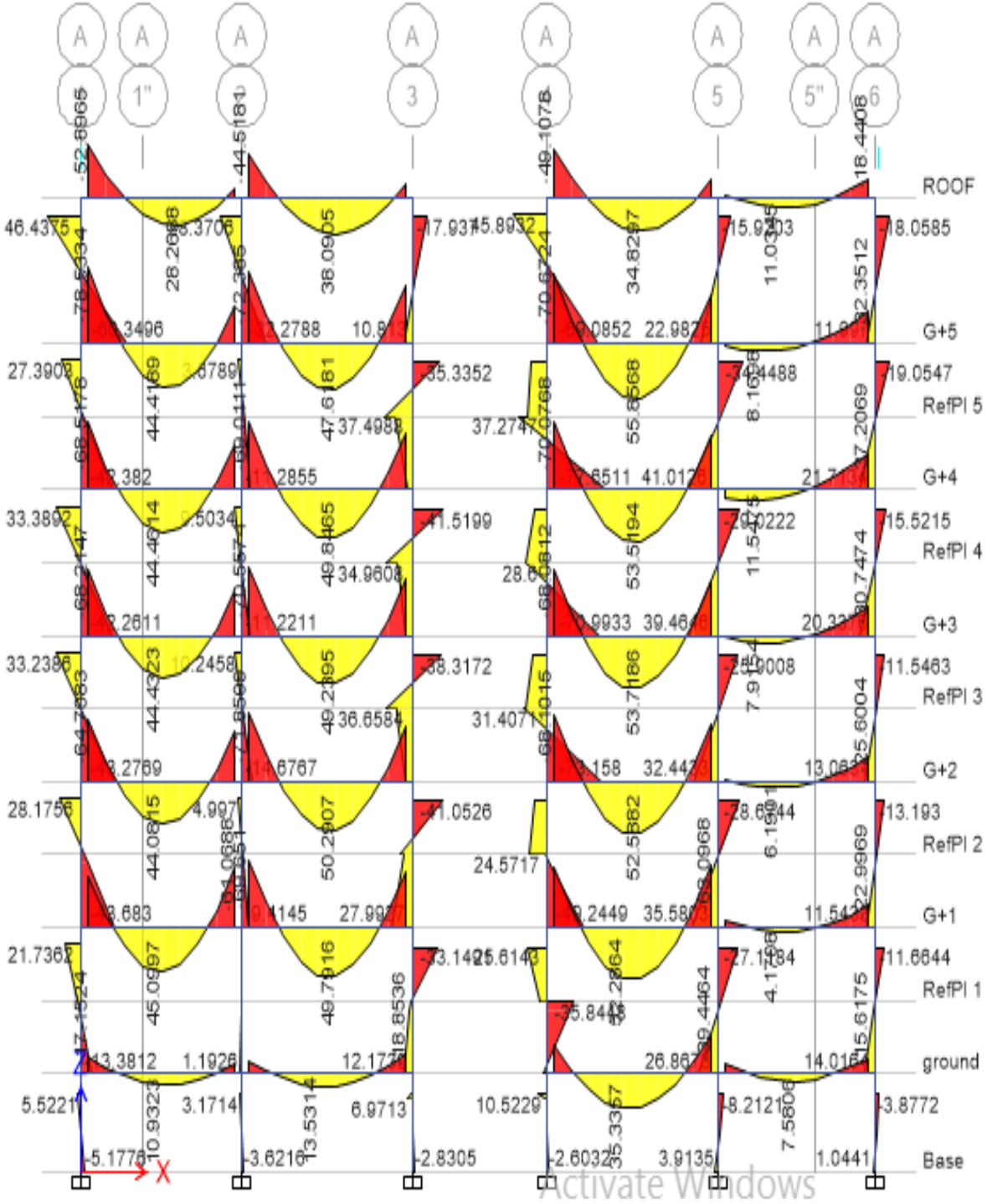


Figure 6-3 Bending moment

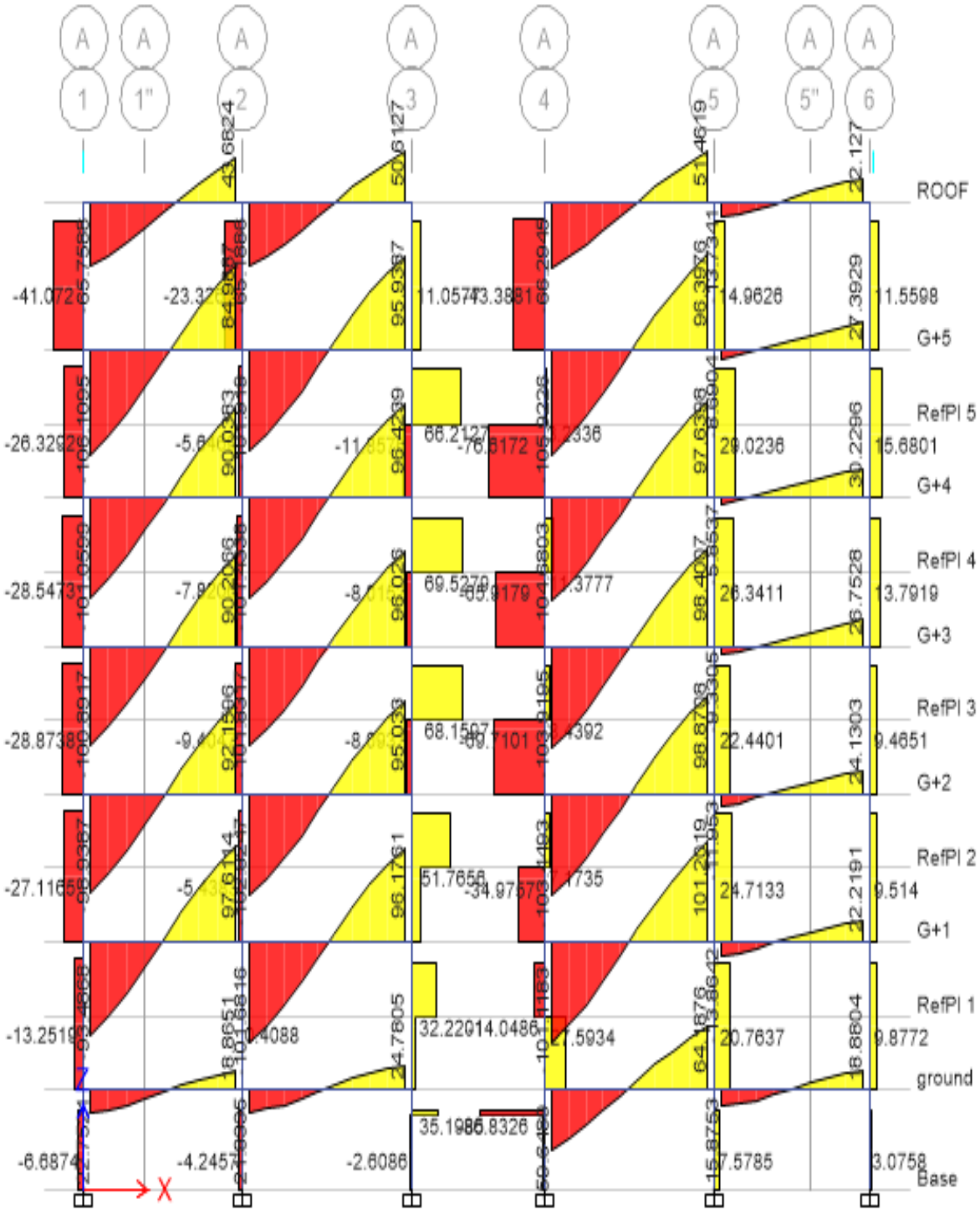


Figure 6-4 Shear force

CHAPTER SEVEN

7. BEAM DESIGN

General

Beam is a horizontal structural member which offers 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.

TYPES OF BEAM

Table 7-1 Types beams

Types of beam		
According to support condition	According to amount of reinforcement	According to location of reinforcement
Simply supported	Balanced section	Singly reinforced section
Cantilever	Under reinforced section	Doubly reinforced section
Propped	Over reinforced section	
Overhanging		
Continuous		

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 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.

Flexural Design

STEP 1: Design Data

- Dimension – 350x250mm
- Material used

A. Concrete

- Grade of concrete C-25
- Concrete cube strength, f_{cu}=25MPA
- Concrete characteristic strength, f_{ck} =0.8* f_{cu}=0.8*25=20MPA
- Concrete design strength, fcd =$0.85 * \frac{fck}{\gamma c}$=0.85*20/1.5=11.33MPA

B. Steel S-500

- Steel tensile strength, f $fyk = 500MPA$
- Steel tensile design strength, $fyd = \frac{fyk}{\gamma s} = \frac{500}{1.15} =$ 434.78$\boldsymbol{Mpa}$

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.

$$C_{nom}=C_{min}+ \triangle C_{dev}$$

Where,

Cmin, Minimum cover

C_{dev} allowance in design for devation

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 Cmin satisfying the requirements for both bond and environmental conditions shall be used.

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

A. Cmin, b minimum cover due to bond requirement

Table 7-2 Minimum cover due to bond requirement

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.	

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

B. Cmin, dur minimum cover due to environmental conditions

Environmental Requirement for $C_{min,dur}$ (mm) Structural Class Exposure Class according to Table

Table 7-3 minimum bond due to environmental condition

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	**Exposure Cla`ss 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

Therefore, As the structural class is S1 & Exposure class is X0, the minimum cover due to environmental requirement, Cmin, dur is 10mm.

C. ΔCdur, γ additive safety element

The concrete cover should be increased by the additive safety element $\Delta c_{dur\gamma}$.

Note: The value of $\Delta c_{dur,\gamma}$ 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. ΔCdur,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 $\Delta c_{dur,\gamma}$. 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. ΔCdur, 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 Δcdur,add.

Note: The recommended value for Δcdur, add without further specification, is 0 mm. Therefore, the reduction of minimum cover for use of additional protection, ΔCdur,add is 0mm.

Finally, $c_{min} = \max \{c_{min,b}; c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 12 \text{ mm}$

$c_{min} = \max \{20; 12_r + 0 - 0 - 0; 12 \text{ mm} = 22\text{mm}$

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 Δcdev may be reduced:

$$10 \text{ mm} \geq \Delta c_{dev} \square 5 \text{ mm}$$

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

$$10 \text{ mm} \geq \Delta c_{dev} \geq 0 \text{ mm}$$

Therefore, we have taken Δcdev = 5mm

STEP 2: Check depth for deflection

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

Where;

$$N = \left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \quad \text{if } \rho \leq \rho_o$$

$$N = \left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho-\rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \quad \text{if } \rho > \rho_o$$

$$N = \left[11 + 1.5\sqrt{f_{ck}}\right] \quad \text{if } \rho = \rho_o$$

Table 7-4 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 two-way spanning simply supported slab	1.1	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1.13	18	26
Interior span of beam or one-way or two-way spanning slab	1.15	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1.12	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

For Top-Tie beam

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

$N = \left[11 + 1.5\sqrt{f_{ck}}\right]$ if $\rho = \rho_o$, $N = \left[11 + 1.5\sqrt{20}\right]$ =**17.71**

$$\frac{l}{d} = 17.71 * 1.3 * 1 * 1 * 1 = \mathbf{23.02} \qquad ,where \quad l = lx$$

$$d \geq \frac{3500mm}{23.02} = \mathbf{152.02mm}$$

For intermediate beam

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

$N = \left[11 + 1.5\sqrt{f_{ck}}\right]$ if $\rho = \rho_o$, $N = \left[11 + 1.5\sqrt{20}\right]$ =**17.71**

$$\frac{l}{d} = 17.71 * 1.5 * 1.* 1 * 1 = 26.565 \qquad ,where \quad l = lx$$

$$d \geq \frac{4000mm}{26.565} = \mathbf{150.57mm}$$

$D_{required} = d_{required} + stirrup + \frac{dia\ of\ ba}{2} = 152.02 + 8 + \frac{20}{2} = 170.02$

Therefore $D_{provided} = 400mm \geq D_{required} = 170.02$ OK!!!

STEP 3: Check whether the beam is singly or doubly 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 $K = \frac{Msd}{bd2*fck} = \frac{100.98KNm}{0.35*0.4^2*20} = 0.126KNm$ <0.167 ---**Singly reinforced beam.**

STEP 4: Provide reinforcement

$Z = \frac{d}{2}(1+\sqrt{1-3.53k}) = \frac{357}{2}(1+\sqrt{1-3.53*0.126})$

$= 311.93 < 0.95d = 339.15$

$As_{calcu} = \frac{Msd}{fyd*Z} = \frac{100.98}{434.78*311.93} = 744.54mm^2$

STEP 5: Check for minimum & maximum reinforcement area

$As_{min} = \frac{0.26*fctm*bt*d}{fyk} = 0.0013btd = 156mm^2$

$A_{smax} = 0.04bd = 0.04*0.25*0.4 = 4000mm^2$ Therefore $As_{min} \leq As_{calcu} \leq As_{smax}$

$As_{provided} = As_{calcu} = 744.54\ mm^2$

STEP 6: No of steel, n

$a_s = \frac{\pi d^2}{4} = \frac{3.14*20^2}{4} = 314\ mm^2$ $n = \frac{Ascalcu}{as} = \frac{744.54}{314} = 2.37 \approx 3Ø20$

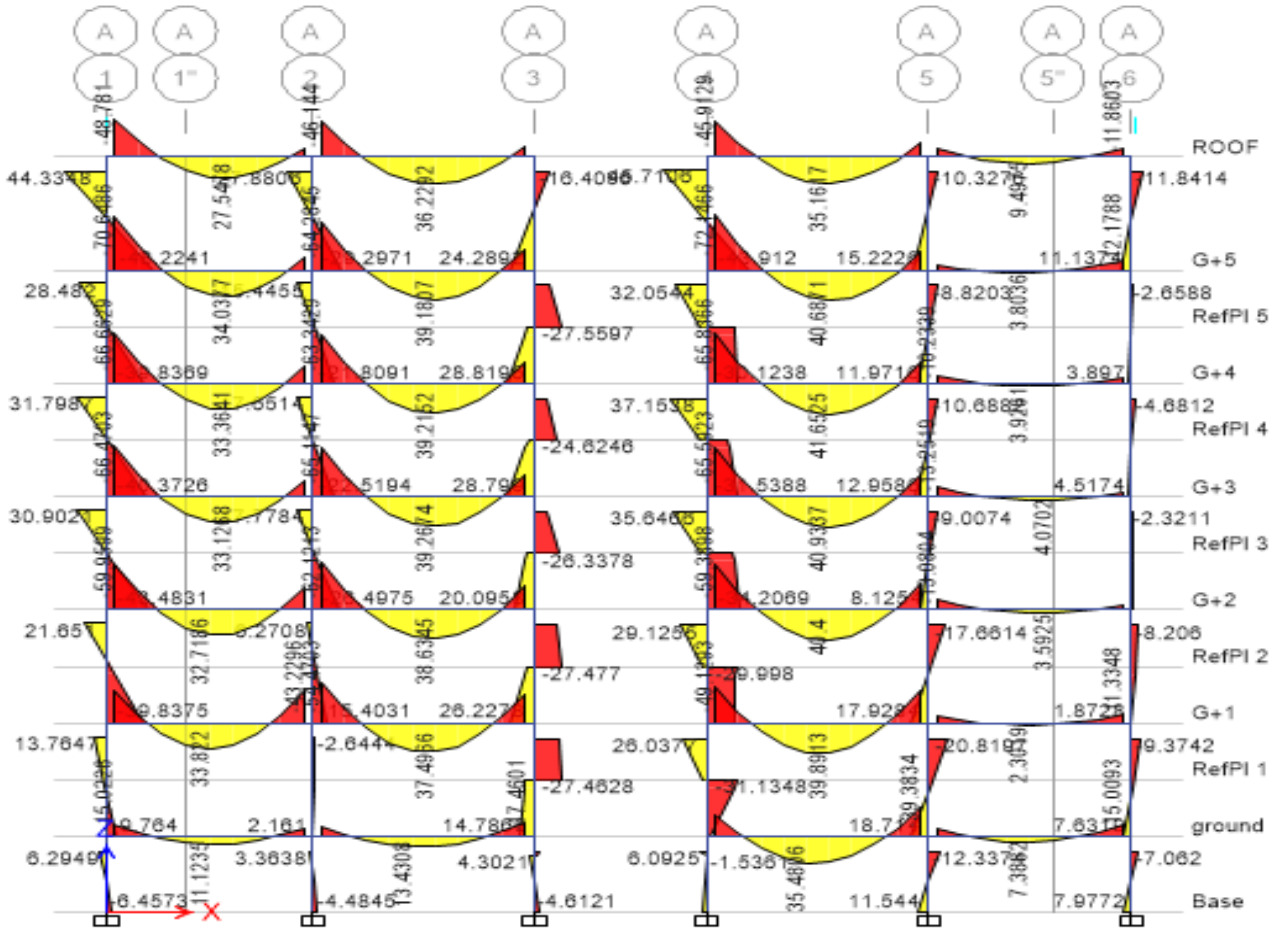


Figure 7-1 Bending moment diagram of beam in axisA

Table 7-5 Reinforcement on axis A

Axis-A roof

type	Location	moment (KNm)	b	d	K	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
support	1	48.78	250	307	0.0508	292.55	single	383.50	9	3500	383.50	1.22	2Ø20
Span	1 to 2	27.54	250	307	0.0287	299.02	single	211.83	97.5	3500	211.83	0.6778	2Ø20
support	2	46.14	250	307	0.0481	293.37	single	361.74	97.5	3500	361.74	1.15	2Ø20
Span	2to3	36.23	250	307	0.0377	296.41	single	281.13	97.5	3500	281.13	0.895	2Ø20
support	4	45.91	250	307	0.0478	293.44	single	359.84	97.5	3500	359.84	1.145	2Ø20
Span	4 to 5	35.167	250	307	0.0366	296.73	single	272.58	97.5	3500	272.58	0.867	2Ø20
support	5	0	250	307	0	307	single	0	97.5	3500	97.5	0.31	2Ø20
Span	5 to 6	9.49	250	307	0.0099	304.3	single	71.72	97.5	3500	97.5	0.31	2Ø20
support	6	11.86	250	307	0.0124	303.62	single	89.84	97.5	3500	97.5	0.31	2Ø20

Axis A Story 5

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	1	70.64	350	357	0.0526	339.59	single	478.436	162.44	5600	478.436	1.5237	2Ø20
Span	1 to 2	34.03	350	357	0.0253	348.84	single	224.373	162.44	5600	224.373	0.7146	2Ø20
Support	2	64.28	350	357	0.0478	341.24	single	433.263	162.44	5600	433.263	1.3798	2Ø20
Span	2 to 3	39.18	350	357	0.0292	347.57	single	259.273	162.44	5600	259.273	0.8257	2Ø20
Support	4	72.15	350	357	0.0537	339.2	single	489.228	162.44	5600	489.228	1.5581	2Ø20
Span	4to 5	40.68	350	357	0.0303	347.19	single	269.487	162.44	5600	269.487	0.8582	2Ø20
Support	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
Span	5 to 6	3.8	350	357	0.0028	356.11	single	162.44	162.44	5600	162.44	0.517	2Ø20

Axis A story 4

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	66.66	350	357	0.0496	340.62	single	450.114	162.44	5600	450.114	1.4335	2Ø20
span	1 to 2	33.36	350	357	0.0248	349	single	219.852	162.44	5600	219.852	0.7002	2Ø20
suport	2	63.34	350	357	0.0471	341.48	single	426.625	162.44	5600	426.625	1.3587	2Ø20
span	2 to 3	39.22	350	357	0.0292	347.56	single	259.545	162.44	5600	259.545	0.8266	2Ø20
suport	4	65.83	350	357	0.049	340.84	single	444.23	162.44	5600	444.23	1.4147	2Ø20
span	4to 5	41.65	350	357	0.031	346.95	single	276.105	162.44	5600	276.105	0.8793	2Ø20
suport	5	10.23	350	357	0.0076	354.59	single	66.357	162.44	5600	162.44	0.517	2Ø20
span	5 to 6	3.92	350	357	0.0029	356.08	single	25.321	162.44	5600	162.44	0.517	2Ø20

Axis A Story 3

type	Location	moment (KNm)	B	D	K	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	66.47	350	357	0.0495	340.67	single	448.766	162.44	5600	448.766	1.4292	2Ø20
Span	1 to 2	33.13	350	357	0.0247	349.06	single	218.301	162.44	5600	218.301	0.6952	2Ø20
suport	2	66.147	350	357	0.0492	340.75	single	446.476	162.44	5600	446.476	1.4219	2Ø20
Span	2 to 3	39.27	350	357	0.0292	347.54	single	259.885	162.44	5600	259.885	0.8277	2Ø20
Span	3	24.62	350	357	0.0183	351.13	single	161.268	162.44	5600	162.44	0.517	2Ø20
suport	4	65.54	350	357	0.0488	340.91	single	442.176	162.44	5600	442.176	1.4082	2Ø20
Span	4to 5	40.933	350	357	0.0305	347.13	single	271.212	162.44	5600	271.212	0.8637	2Ø20
suport	5	13.25	350	357	0.0099	353.87	single	86.1206	162.44	5600	162.44	0.517	2Ø20
Span	5 to 6	4.07	350	357	0.003	356.04	single	26.2919	162.44	5600	162.44	0.517	2Ø20
suport	6	4.52	350	357	0.0034	355.94	single	29.2076	162.44	5600	162.44	0.517	2Ø20

Axis A story 2

Type	Location	moment (KNm)	B	d	K	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	1	59.95	350	357	0.0446	342.35	single	402.768	162.44	5600	402.768	1.2827	2Ø20
Span	1 to 2	32.72	350	357	0.0243	349.16	single	215.537	162.44	5600	215.537	0.6864	2Ø20
Support	2	62.12	350	357	0.0462	341.79	single	418.025	162.44	5600	418.025	1.3313	2Ø20
Span	2 to 3	38.63	350	357	0.0287	347.7	single	255.533	162.44	5600	255.533	0.8138	2Ø20
Span	3	28.34	350	357	0.0211	350.23	single	186.114	162.44	5600	186.114	0.5927	2Ø20
Support	4	59.39	350	357	0.0442	342.49	single	398.839	162.44	5600	398.839	1.2702	2Ø20
Span	4to 5	40.4	350	357	0.0301	347.26	single	267.579	162.44	5600	267.579	0.8522	2Ø20
Support	5	13.08	350	357	0.0097	353.91	single	85.0059	162.44	5600	162.440	0.517	2Ø20
Span	5 to 6	3.59	350	357	0.0027	356.16	single	23.1838	162.44	5600	162.440	0.517	2Ø20
suport	6	0	350	357	0	357	single	0	162.44	5600	162.440	0.517	2Ø20

Axis A story 1

Type	Location	moment (KNm)	B	D	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	58.43	350	357	0.0435	342.73	single	392.112	162.44	5600	392.112	1.2488	2Ø20
Span	1 to 2	33.88	350	357	0.0252	348.87	single	223.361	162.44	5600	223.361	0.7113	2Ø20
suport	2	54.47	350	357	0.0405	343.74	single	364.468	162.44	5600	364.468	1.1607	2Ø20
Span	2 to 3	37.49	350	357	0.0279	347.98	single	247.792	162.44	5600	247.792	0.7891	2Ø20
Span	3	27.47	350	357	0.0204	350.44	single	180.292	162.44	5600	180.292	0.5742	2Ø20
suport	4	49.13	350	357	0.0366	345.09	single	327.454	162.44	5600	327.454	1.0428	2Ø20
Span	4to 5	39.89	350	357	0.0297	347.39	single	264.105	162.44	5600	264.105	0.8411	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
Span	5 to 6	2.3	350	357	0.0017	356.46	single	14.8405	162.44	5600	162.44	0.517	2Ø20
suport	6	1.335	350	357	0.001	356.69	single	8.60845	162.44	5600	162.44	0.517	2Ø20

Axis A Ground

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	15.023	300	357	0.013	352.84	single	97.9277	139.23	4800	139.23	0.4434	2Ø20
span	1 to 2	11.124	300	357	0.0097	353.93	single	72.289	139.23	4800	139.23	0.4434	2Ø20
suport	2	3.38	300	357	0.0029	356.07	single	21.8327	139.23	4800	139.23	0.4434	2Ø20
span	2 to 3	13.43	300	357	0.0117	353.29	single	87.4333	139.23	4800	139.23	0.4434	2Ø20
support	3	27.4	300	357	0.0238	349.34	single	180.397	139.23	4800	180.397	0.5745	2Ø20
suport	4	31.13	300	357	0.027	348.27	single	205.584	139.23	4800	205.584	0.6547	2Ø20
span	4to 5	35.48	300	357	0.0308	347.02	single	235.159	139.23	4800	235.159	0.7489	2Ø20
suport	5	39.5	300	357	0.0343	345.85	single	262.688	139.23	4800	262.688	0.8366	2Ø20
span	5 to 6	7.39	300	357	0.0064	354.97	single	47.8836	139.23	4800	139.23	0.4434	2Ø20
suport	6	5.01	300	357	0.0043	355.62	single	32.4023	139.23	4800	139.23	0.4434	2Ø20

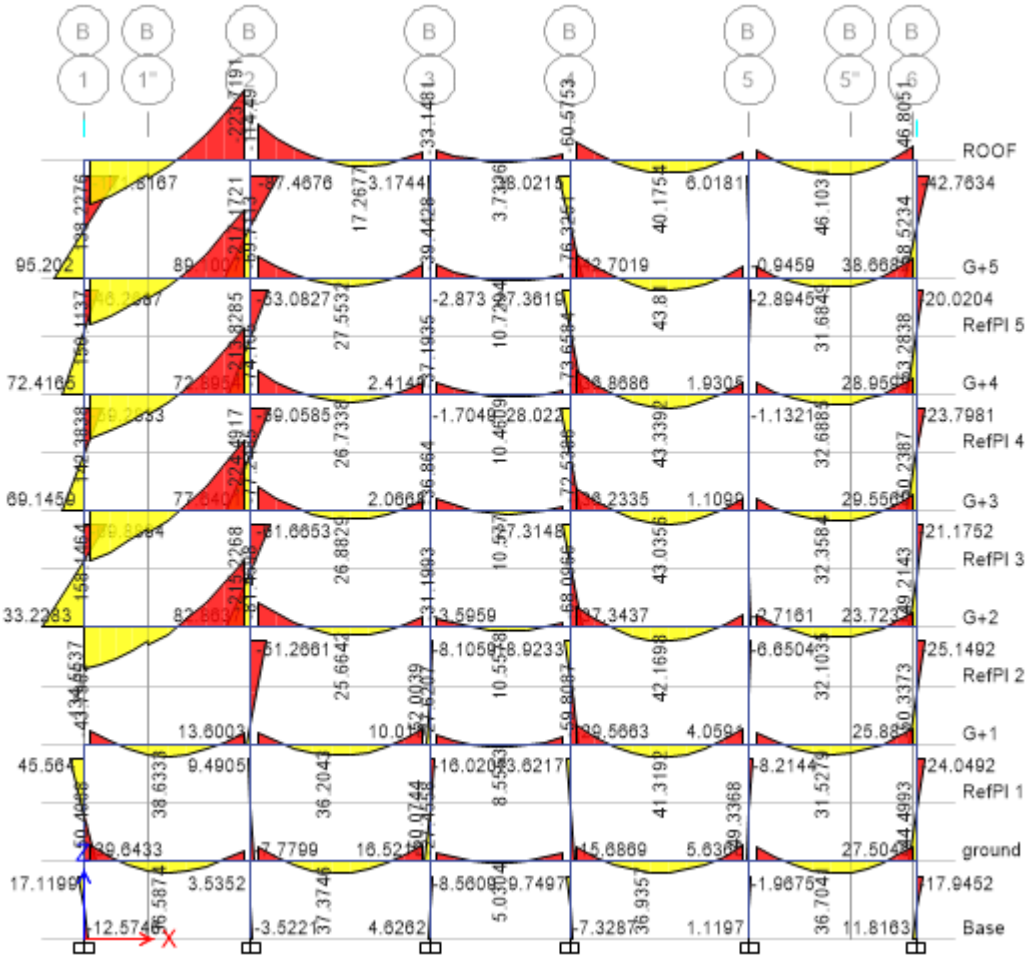


Figure Bending moment diagram of beam in axis B

Table 7-6 Reinforcement on axis B

Axis –B Roof

Type	Location	moment (KNm)	B	d	K	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
Support	1	0	250	307	0	307	single	0	99.775	3500	99.775	0.3177	2Ø20
Span	1 to 2	0	250	307	0	307	single	0	99.775	3500	99.775	0.3177	2Ø20
Support	2	223.19	250	307	0.2325	218.5	single	2349.38	99.775	3500	2349.38	7.4821	8Ø20
Span	2 to 3	17.27	250	307	0.018	302.05	single	131.507	99.775	3500	131.507	0.4188	2Ø20
Support	3	33.15	250	307	0.0345	297.34	single	256.425	99.775	3500	256.425	0.8166	2Ø20
Span	3 to 4	3.73	250	307	0.0039	305.94	single	28.0413	99.775	3500	99.775	0.3177	2Ø20
Support	4	60.57	250	307	0.0631	288.83	single	482.33	99.775	3500	482.33	1.5361	2Ø20
Span	4to 5	40.17	250	307	0.0418	295.21	single	312.968	99.775	3500	312.968	0.9967	2Ø20
Support	5	0	250	307	0	307	single	0	99.775	3500	99.775	0.3177	2Ø20
Span	5 to 6	46.103	250	307	0.048	293.39	single	361.428	99.775	3500	361.428	1.151	2Ø20
Support	6	46.805	250	307	0.0488	293.17	single	367.203	99.775	3500	367.203	1.1694	2Ø20

Axis –B story 5

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	0	350	357	0	320.5	Single	0	136.5	5600	136.5	0.43	2Ø20
span	1 to 2	0	350	357	0	357	Single	0	136.5	5600	136.5	0.43	2Ø20
suport	2	69.71	350	357	0.0519	339.83	Single	471.808	136.5	5600	471.808	1.50	2Ø20
span	2to3	27.55	350	357	0.0205	350.42	Single	180.846	136.5	5600	180.846	0.56	2Ø20
suport	3	39.44	350	357	0.0293	347.5	Single	261.060	136.5	5600	261.060	0.83	2Ø20
suport	3 to 4	10.72	350	357	0.008	354.47	Single	69.5579	136.5	5600	136.5	0.43	2Ø20
suport	4	76.32	350	357	0.0568	338.11	Single	519.214	136.5	5600	519.214	1.65	2Ø20
span	4 to 5	43.81	350	357	0.0326	346.42	Single	290.874	136.5	5600	290.874	0.93	2Ø20
suport	5	0	350	357	0	357	Single	0	136.5	5600	136.5	0.43	2Ø20
span	5 to 6	31.68	350	357	0.0236	349.41	Single	208.53	136.5	5600	208.53	0.66	2Ø20
suport	6	68.52	350	357	0.051	340.14	Single	463.348	136.5	3000	463.348	1.48	2Ø20

Axis –B story 4

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	0	350	357	0	317.42	single	0	136.5	5600	136.5	0.43	2Ø20
span	1 to 2	0	350	357	0	357	single	0	136.5	5600	136.5	0.43	2Ø20
suport	2	74.186	350	357	0.0552	338.67	single	503.822	136.5	5600	503.822	1.63	2Ø20
span	2to3	26.733	350	357	0.0199	350.62	single	175.364	136.5	5600	175.364	0.56	2Ø20
suport	3	37.1935	350	357	0.0277	348.06	single	245.780	136.5	5600	245.780	0.78	2Ø20
Span	3 to 4	10.46	350	357	0.0078	354.53	single	67.859	136.5	5600	136.5	0.43	2Ø20
suport	4	73.65	350	357	0.0548	338.81	single	499.975	136.5	5600	499.975	1.59	2Ø20
span	4 to 5	43.339	350	357	0.0322	346.53	single	287.649	136.5	5600	287.649	0.92	2Ø20
suport	5	0	350	357	0	357	single	0	136.5	5600	136.5	0.43	2Ø20
span	5 to 6	32.6883	350	357	0.0243	349.17	single	215.323	136.5	5600	215.323	0.69	2Ø20
Support	6	60.2387	350	357	0.0448	342.27	single	404.795	136.5	3000	404.795	1.29	2Ø20

Axis –B story 3

Type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
Support	1	0	350	357	0	357	single	0	136.5	5600	136.5	0.43	2Ø20
Span	1 to 2	0	350	357	0	357	single	0	136.5	5600	136.5	0.43	2Ø20
Support	2	77.2485	350	357	0.0575	337.87	single	525.866	136.5	5600	525.866	1.67	2Ø20
Span	2to3	26.882	350	357	0.02	350.58	single	176.360	136.5	5600	176.360	0.56	2Ø20
Support	3	36.864	350	357	0.0274	348.14	single	243.545	136.5	5600	243.545	0.78	2Ø20
Support	3 to 4	10.537	350	357	0.0078	354.51	single	68.3621	136.5	5600	136.5	0.43	2Ø20
Support	4	72.53	350	357	0.054	339.1	single	491.948	136.5	5600	491.948	1.57	2Ø20
Span	4 to 5	43.035	350	357	0.032	346.61	single	285.569	136.5	5600	285.569	0.91	2Ø20
Support	5	0	350	357	0	357	single	0	136.5	5600	136.5	0.43	2Ø20
Span	5 to 6	32.3564	350	357	0.0241	349.25	single	213.087	136.5	5600	213.087	0.68	2Ø20
Support	6	60.2387	350	357	0.0448	342.27	single	404.795	136.5	3000	404.795	1.29	2Ø20

Axis –B story 2

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
Support	1	0	350	357	0.1177	314.98	single	0	136.5	5600	136.50	0.43	2Ø20
Span	1 to 2	0	350	357	0	357	single	0	136.5	5600	136.50	0.43	2Ø20
Support	2	81.43	350	357	0.0606	336.76	single	556.145	136.5	5600	556.145	1.77	2Ø20
Span	2to3	25.664	350	357	0.0191	350.88	single	168.227	136.5	5600	168.227	0.54	2Ø20
Support	3	31.199	350	357	0.0232	349.53	single	205.298	136.5	5600	205.298	0.65	2Ø20
Support	3 to 4	10.55	350	357	0.0078	354.51	single	68.4470	136.5	5600	136.50	0.43	2Ø20
Support	4	68.0966	350	357	0.0507	340.25	single	460.315	136.5	5600	460.315	1.47	2Ø20
Span	4 to 5	42.169	350	357	0.0314	346.82	single	279.649	136.5	5600	279.649	0.89	2Ø20
Support	5	0	350	357	0	357	single	0	136.5	5600	136.50	0.43	2Ø20
Span	5 to 6	32.1	350	357	0.0239	349.31	single	211.361	136.5	5600	211.361	0.67	2Ø20
Support	6	49.21	350	357	0.0366	345.07	single	328.006	136.5	3000	328.006	1.04	2Ø20

Axis –B story 1

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	43.56	350	357	0.0324	346.48	single	289.162	136.5	5600	289.162	0.92	2Ø20
span	1 to 2	38.6338	350	357	0.0287	347.7	single	255.559	136.5	5600	255.559	0.83	2Ø20
suport	2	0	350	357	0	357	single	0	136.5	5600	136.50	0.43	2Ø20
span	2to3	36.204	350	357	0.0269	348.3	single	239.073	136.5	5600	239.073	0.76	2Ø20
suport	3	27.6207	350	357	0.0206	350.4	single	181.299	136.5	5600	181.299	0.58	2Ø20
suport	3 to 4	8.55	350	357	0.0064	354.98	single	55.3971	136.5	5600	136.50	0.43	2Ø20
suport	4	59.8087	350	357	0.0445	342.38	single	401.776	136.5	5600	401.776	1.28	2Ø20
span	4 to 5	41.32	350	357	0.0307	347.04	single	273.852	136.5	5600	273.852	0.87	2Ø20
suport	5	0	350	357	0	357	single	0	136.5	5600	136.50	0.43	2Ø20
span	5 to 6	31.5279	350	357	0.0235	349.45	single	207.510	136.5	5600	207.510	0.66	2Ø20
suport	6	50.3373	350	357	0.0375	344.78	single	335.796	136.5	3000	335.796	1.07	2Ø20

Axis –B Ground

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	50.4	300	357	0.0438	342.64	single	338.3175463	139.23	4800	338.318	1.0774	2Ø20
span	1 to 2	96.58	300	357	0.0838	328.28	single	676.6727447	139.23	4800	676.673	2.155	2Ø20
suport	2	0	300	357	0	357	single	0	139.23	4800	139.230	0.4434	2Ø20
span	2 to 3	37.37	300	357	0.0324	346.47	single	248.078298	139.23	4800	248.078	0.7901	2Ø20
support	3	69.97	300	357	0.0607	336.71	single	477.9522785	139.23	4800	477.952	1.5221	2Ø20
span	3 to 4	5.03	300	357	0.0044	355.62	single	32.53219926	139.23	4800	139.230	0.4434	2Ø20
suport	4	0	300	357	0	357	single	0	139.23	4800	139.230	0.4434	2Ø20
span	4to 5	36.94	300	357	0.0321	346.59	single	245.1353845	139.23	4800	245.135	0.7807	2Ø20
suport	5	29.334	300	357	0.0255	348.79	single	193.4368529	139.23	4800	193.437	0.616	2Ø20
span	5 to 6	36.7	300	357	0.0319	346.66	single	243.4937787	139.23	4800	243.494	0.7755	2Ø20
suport	6	44.493	300	357	0.0386	344.39	single	297.1504147	139.23	4800	297.15	0.9463	2Ø20

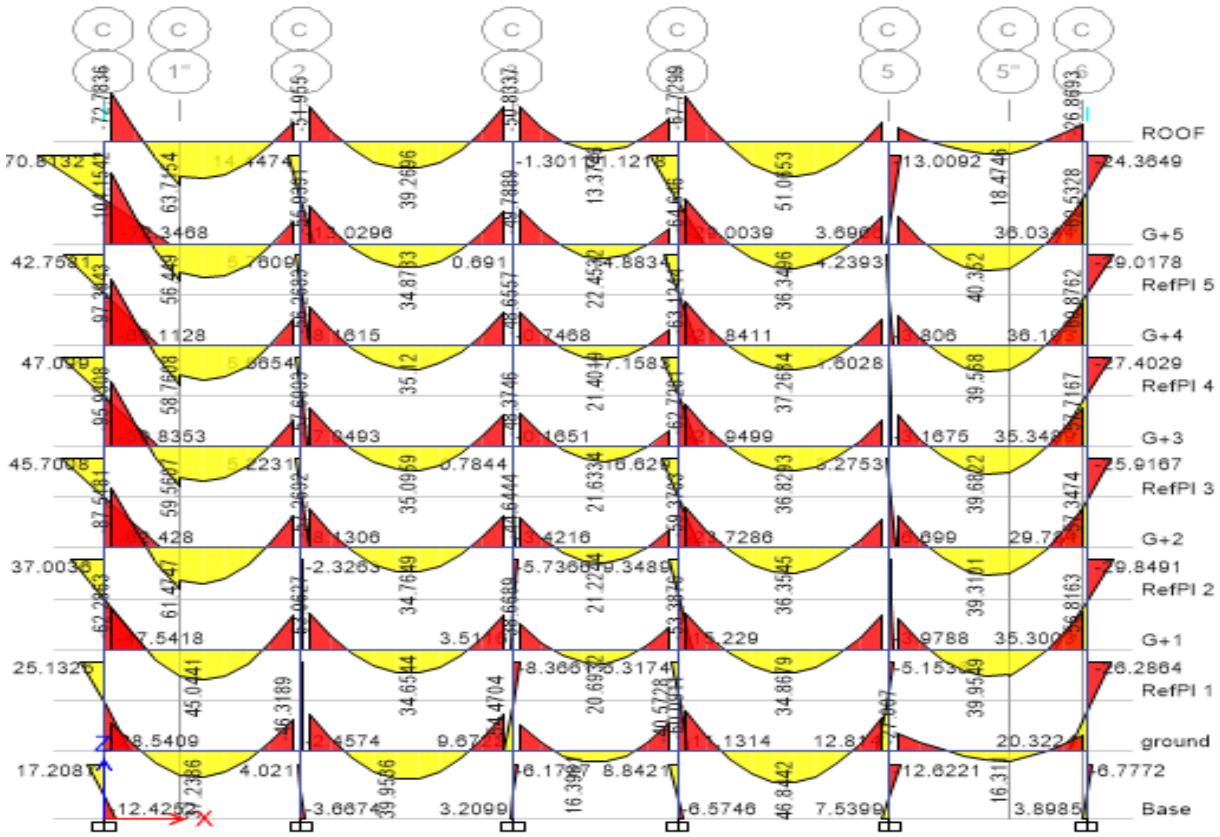


Figure 7-2 Bending moment diagram of beam in axisC

Table 7-7 Reinforcement on axis C

Axis –C Roof

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	72.78	250	307	0.0758	284.86	single	587.631	99.775	3500	587.631	2.9241	3Ø20
span	1 to 2	63.72	250	307	0.0664	287.82	single	509.199	99.775	3500	509.199	2.5338	3Ø20
suport	2	51.955	250	307	0.0541	291.56	single	409.853	99.775	3500	409.853	2.0395	3Ø20
span	2to3	39.27	250	307	0.0409	295.49	single	305.672	99.775	3500	305.672	1.5211	2Ø20
suport	3	50.83	250	307	0.0529	291.91	single	400.494	99.775	3500	400.494	1.9929	2Ø20
span	3 to4	13.37	250	307	0.0139	303.18	single	101.429	99.775	3500	101.429	0.5047	2Ø20
suport	4	67.72	250	307	0.0705	286.52	single	543.612	99.775	3500	543.612	2.7051	3Ø20
span	4 to 5	51.05	250	307	0.0532	291.84	single	402.323	99.775	3500	402.323	2.002	3Ø20
suport	5	0	250	307	0	307	single	0	99.775	3500	99.775	0.3177	2Ø20
span	5 to 6	36.704	250	307	0.0382	296.27	single	284.945	99.775	3500	284.945	1.4179	2Ø20
suport	6	26.86	250	307	0.028	299.22	single	206.463	99.775	3500	206.463	1.0274	2Ø20

Axis –C story 5

type	Location	moment (KNm)	B	D	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	104.105	350	357	0.0775	330.65	single	724.155	162.44	5600	724.155	2.3062	3Ø20
span	1 to 2	56.449	350	357	0.042	343.24	single	378.262	162.44	5600	378.262	1.2047	2Ø20
suport	2	55.03	350	357	0.0409	343.6	single	368.367	162.44	5600	368.367	1.1731	2Ø20
span	2to3	34.87	350	357	0.0259	348.63	single	230.048	162.44	5600	230.048	0.7326	2Ø20
suport	3	49.78	350	357	0.037	344.92	single	331.943	162.44	5600	331.943	1.0571	2Ø20
suport	3 to 4	22.45	350	357	0.0167	351.66	single	146.834	162.44	5600	162.44	0.517	2Ø20
suport	4	64.64	350	357	0.0481	341.14	single	435.808	162.44	5600	435.808	1.3879	2Ø20
span	4 to 5	36.34	350	357	0.027	348.27	single	239.995	162.44	5600	239.995	0.7643	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
span	5 to 6	40.352	350	357	0.03	347.28	single	267.252	162.44	5600	267.252	0.8511	2Ø20
suport	6	62.53	350	357	0.0465	341.69	single	420.913	162.44	5600	420.913	1.3405	2Ø20

Axis –C story 4

type	Location	moment (KNm)	B	D	K	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	97.39	350	357	0.0725	332.49	single	673.705	162.44	5600	673.705	2.1456	3Ø20
span	1 to 2	58.76	350	357	0.0437	342.65	single	394.424	162.44	5600	394.424	1.2561	2Ø20
suport	2	55.26	350	357	0.0411	343.54	single	369.969	162.44	5600	369.969	1.1782	2Ø20
span	2to3	35.12	350	357	0.0261	348.57	single	231.738	162.44	5600	231.738	0.738	2Ø20
suport	3	48.65	350	357	0.0362	345.21	single	324.142	162.44	5600	324.142	1.0323	2Ø20
suport	3 to 4	21.4	350	357	0.0159	351.91	single	139.866	162.44	5600	162.44	0.517	2Ø20
suport	4	63.12	350	357	0.047	341.53	single	425.073	162.44	5600	425.073	1.3537	2Ø20
span	4 to 5	37.26	350	357	0.0277	348.04	single	246.231	162.44	5600	246.231	0.7842	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
span	5 to 6	39.56	350	357	0.0294	347.47	single	261.858	162.44	5600	261.858	0.8339	2Ø20
suport	6	60.876	350	357	0.0453	342.11	single	409.272	162.44	5600	409.272	1.3034	2Ø20

Axis –C Story 3

type	Location	moment (KNm)	B	D	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	95.93	350	357	0.0714	332.88	single	662.815	162.44	5600	662.815	2.1109	3Ø20
Span	1 to 2	59.56	350	357	0.0443	342.44	single	400.032	162.44	5600	400.032	1.274	2Ø20
suport	2	57.6	350	357	0.0429	342.94	single	386.304	162.44	5600	386.304	1.2303	2Ø20
Span	2to3	35.09	350	357	0.0261	348.58	single	231.535	162.44	5600	231.535	0.7374	2Ø20
suport	3	48.37	350	357	0.036	345.28	single	322.210	162.44	5600	322.21	1.0261	2Ø20
suport	3 to 4	21.63	350	357	0.0161	351.86	single	141.391	162.44	5600	162.44	0.517	2Ø20
suport	4	62.73	350	357	0.0467	341.63	single	422.323	162.44	5600	422.323	1.345	2Ø20
Span	4 to 5	36.83	350	357	0.0274	348.15	single	243.315	162.44	5600	243.315	0.7749	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
Span	5 to 6	39.68	350	357	0.0295	347.44	single	262.675	162.44	5600	262.675	0.8365	2Ø20
suport	6	57.72	350	357	0.0429	342.91	single	387.143	162.44	5600	387.143	1.2329	2Ø20

Axis –C Story 2

Type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	87.52	350	357	0.0651	335.15	single	600.625	162.44	5600	600.625	1.9128	2Ø20
Span	1 to 2	61.47	350	357	0.0457	341.96	single	413.45	162.44	5600	413.45	1.3167	2Ø20
suport	2	57.26	350	357	0.0426	343.03	single	383.927	162.44	5600	383.927	1.2227	2Ø20
span	2to3	34.7	350	357	0.0258	348.67	single	228.899	162.44	5600	228.899	0.729	2Ø20
suport	3	44.6	350	357	0.0332	346.22	single	296.288	162.44	5600	296.288	0.9436	2Ø20
suport	3 to 4	21.2	350	357	0.0158	351.96	single	138.54	162.44	5600	162.44	0.517	2Ø20
suport	4	59.37	350	357	0.0442	342.49	single	398.699	162.44	5600	398.699	1.2697	2Ø20
span	4 to 5	36.35	350	357	0.027	348.27	single	240.063	162.44	5600	240.063	0.7645	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
span	5 to 6	39.3	350	357	0.0292	347.54	single	260.089	162.44	5600	260.089	0.8283	2Ø20
suport	6	57.3	350	357	0.0426	343.02	single	384.207	162.44	5600	384.207	1.2236	2Ø20

Axis –C Story 1

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	1	62.28	350	357	0.0463	341.75	single	419.152	162.44	5600	419.152	1.3349	2Ø20
span	1 to 2	45.01	350	357	0.0335	346.12	single	299.1	162.44	5600	299.1	0.9525	2Ø20
suport	2	53.06	350	357	0.0395	344.1	single	354.665	162.44	5600	354.665	1.1295	2Ø20
span	2to3	34.65	350	357	0.0258	348.68	single	228.561	162.44	5600	228.561	0.7279	2Ø20
suport	3	38.66	350	357	0.0288	347.7	single	255.737	162.44	5600	255.737	0.8144	2Ø20
suport	3 to 4	20.69	350	357	0.0154	352.08	single	135.16	162.44	5600	162.44	0.517	2Ø20
suport	4	53.38	350	357	0.0397	344.01	single	356.888	162.44	5600	356.888	1.1366	2Ø20
span	4 to 5	34.87	350	357	0.0259	348.63	single	230.0484	162.44	5600	230.048	0.7326	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
span	5 to 6	39.95	350	357	0.0297	347.38	single	264.513	162.44	5600	264.513	0.8424	2Ø20
suport	6	56.82	350	357	0.0423	343.14	single	380.853	162.44	5600	380.853	1.2129	2Ø20

Axis –C Ground

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	0	300	357	0	357	single	0	139.23	4800	139.23	0.4434	2Ø20
span	1 to 2	31.23	300	357	0.0271	348.24	single	206.262	139.23	4800	206.262	1.0264	2Ø20
suport	2	46.32	300	357	0.0402	343.85	single	309.837	139.23	4800	309.837	1.5418	2Ø20
span	2to3	39.96	300	357	0.0347	345.71	single	265.851	139.23	4800	265.851	1.3229	2Ø20
suport	3	54.47	300	357	0.0473	341.42	single	366.939	139.23	4800	366.939	1.8259	2Ø20
span	3 to4	16.39	300	357	0.0142	352.46	single	106.955	139.23	4800	139.23	0.4434	2Ø20
suport	4	60.09	300	357	0.0522	339.73	single	406.815	139.23	4800	406.815	2.0244	3Ø20
span	4 to 5	48.84	300	357	0.0424	343.1	single	327.403	139.23	4800	327.403	1.6292	2Ø20
suport	5	71.07	300	357	0.0617	336.37	single	485.957	139.23	4800	485.957	2.4182	3Ø20
span	5 to 6	16.31	300	357	0.0142	352.48	single	106.426	139.23	4800	139.23	0.4434	2Ø20
suport	6	0	300	357	0	357	single	0	139.23	4800	139.23	0.4434	2Ø20

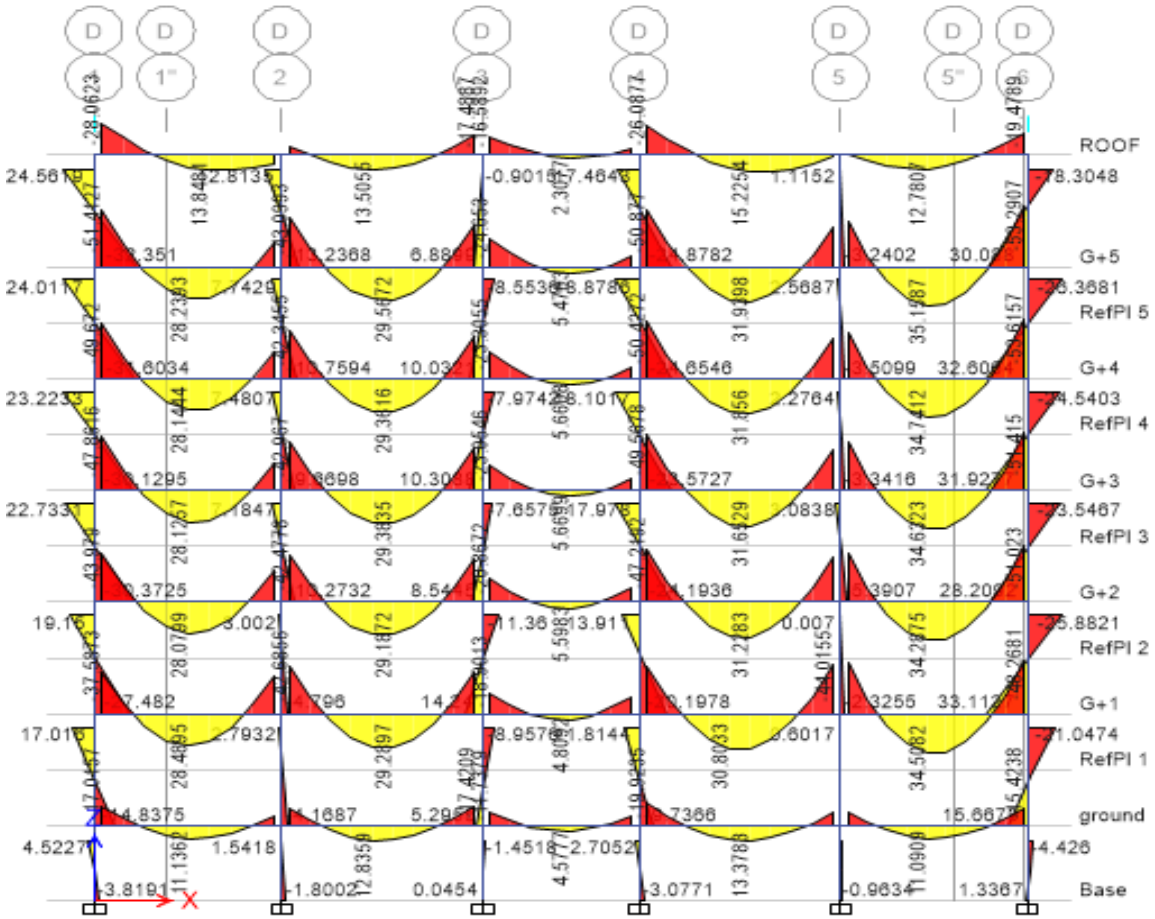


Figure 7-3 Bending moment diagram of beam in axis D

Table Reinforcement on axis D

Axis –D Roof

type	Location	moment (KNm)	B	D	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	28.06	250	307	0.0292	298.87	single	215.945	99.775	3500	215.945	1.0746	2Ø20
span	1 to 2	13.84	250	307	0.0144	303.04	single	105.042	99.775	3500	105.042	0.5227	2Ø20
suport	2	0	250	307	0	307	single	0	99.775	3500	99.775	0.3177	2Ø20
span	2to3	13.5	250	307	0.0141	303.14	single	102.428	99.775	3500	102.428	0.5097	2Ø20
suport	3	16.58	250	307	0.0173	302.25	single	126.169	99.775	3500	126.169	0.6278	2Ø20
span	3 to4	2.35	250	307	0.0024	306.34	single	17.644	99.775	3500	99.775	0.3177	2Ø20
suport	4	26.08	250	307	0.0272	299.45	single	200.312	99.775	3500	200.312	0.9968	2Ø20
span	4 to 5	15.23	250	307	0.0159	302.64	single	115.75	99.775	3500	115.746	0.576	2Ø20
suport	5	0	250	307	0	307	single	0	99.775	3500	99.775	0.3177	2Ø20
span	5 to 6	12.78	250	307	0.0133	303.35	single	96.899	99.775	3500	99.775	0.3177	2Ø20
suport	6	19.48	250	307	0.0203	301.4	single	148.65	99.775	3500	148.654	0.7397	2Ø20

Axis –D story 5

type	Location	moment (KNm)	b	D	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	51.41	350	357	0.0383	344.51	single	343.221	162.44	5600	343.221	1.7079	2Ø20
span	1 to 2	28.24	350	357	0.021	350.25	single	185.444	162.44	5600	185.444	0.9228	2Ø20
suport	2	43.09	350	357	0.0321	346.6	single	285.946	162.44	5600	285.946	1.4229	2Ø20
span	2to3	29.56	350	357	0.022	349.93	single	194.291	162.44	5600	194.291	0.9668	2Ø20
suport	3	24.65	350	357	0.0183	351.13	single	161.468	162.44	5600	161.468	0.8035	2Ø20
span	3 to4	5.47	350	357	0.0041	355.71	single	35.3686	162.44	5600	162.44	0.517	2Ø20
suport	4	50.87	350	357	0.0378	344.65	single	339.482	162.44	5600	339.482	1.6893	2Ø20
span	4 to 5	31.94	350	357	0.0238	349.35	single	210.284	162.44	5600	210.284	1.0464	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
span	5 to 6	35.15	350	357	0.0262	348.56	single	231.941	162.44	5600	231.941	1.1542	2Ø20
suport	6	53.29	350	357	0.0397	344.04	single	356.263	162.44	5600	356.263	1.7728	2Ø20

Axis –D story 4

type	Location	moment (KNm)	b	D	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	49.67	350	357	0.037	344.95	single	331.183	162.44	5600	331.183	1.648	2Ø20
span	1 to 2	28.14	350	357	0.0209	350.28	single	184.775	162.44	5600	184.775	0.9195	2Ø20
suport	2	42.34	350	357	0.0315	346.78	single	280.817	162.44	5600	280.817	1.3974	2Ø20
span	2to3	29.36	350	357	0.0218	349.98	single	192.95	162.44	5600	192.95	0.9601	2Ø20
suport	3	25.3	350	357	0.0188	350.97	single	165.8	162.44	5600	165.8	0.825	2Ø20
span	3 to4	5.66	350	357	0.0042	355.67	single	36.6017	162.44	5600	162.440	0.517	2Ø20
suport	4	50.4	350	357	0.0375	344.77	single	336.23	162.44	5600	336.23	1.6731	2Ø20
span	4 to 5	31.86	350	357	0.0237	349.37	single	209.745	162.44	5600	209.745	1.0437	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.440	0.517	2Ø20
span	5 to 6	34.74	350	357	0.0258	348.66	single	229.169	162.44	5600	229.169	1.1404	2Ø20
suport	6	53.62	350	357	0.0399	343.95	single	358.556	162.44	5600	358.556	1.7842	2Ø20

Axis –D story 3

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	47.86	350	357	0.0356	345.4	single	318.695	162.44	5600	318.695	1.5859	2Ø20
span	1 to 2	28.13	350	357	0.0209	350.28	single	184.708	162.44	5600	184.708	0.9191	2Ø20
suport	2	42.96	350	357	0.032	346.63	single	285.056	162.44	5600	285.056	1.4185	2Ø20
span	2to3	29.38	350	357	0.0219	349.97	single	193.084	162.44	5600	193.084	0.9608	2Ø20
suport	3	25.05	350	357	0.0186	351.03	single	164.133	162.44	5600	164.133	0.8167	2Ø20
span	3 to4	5.66	350	357	0.0042	355.67	single	36.6017	162.44	5600	162.44	0.517	2Ø20
suport	4	49.56	350	357	0.0369	344.98	single	330.423	162.44	5600	330.423	1.6442	2Ø20
span	4 to 5	31.65	350	357	0.0235	349.42	single	208.332	162.44	5600	208.332	1.0367	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
span	5 to 6	34.62	350	357	0.0258	348.69	single	228.358	162.44	5600	228.358	1.1363	2Ø20
suport	6	51.42	350	357	0.0383	344.51	single	343.29	162.44	5600	343.29	1.7083	2Ø20

Axis –D story 2

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	43.98	350	357	0.0327	346.37	single	292.039	162.44	5600	292.039	1.4532	2Ø20
span	1 to 2	28.07	350	357	0.0209	350.29	single	184.306	162.44	5600	184.306	0.9171	2Ø20
suport	2	42.47	350	357	0.0316	346.75	single	281.706	162.44	5600	281.706	1.4018	2Ø20
span	2to3	29.19	350	357	0.0217	350.02	single	191.81	162.44	5600	191.81	0.9545	2Ø20
suport	3	28.87	350	357	0.0215	350.1	single	189.665	162.44	5600	189.665	0.9438	2Ø20
span	3 to4	5.59	350	357	0.0042	355.68	single	36.1474	162.44	5600	162.44	0.517	2Ø20
suport	4	47.21	350	357	0.0351	345.57	single	314.219	162.44	5600	314.219	1.5636	2Ø20
span	4 to 5	31.2	350	357	0.0232	349.53	single	205.306	162.44	5600	205.306	1.0216	2Ø20
suport	5	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
span	5 to 6	34.28	350	357	0.0255	348.77	single	226.061	162.44	5600	226.061	1.1249	2Ø20
suport	6	51.023	350	357	0.038	344.61	single	340.541	162.44	5600	340.541	1.6946	2Ø20

Axis –D story 1

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	37.58	350	357	0.028	347.96	single	248.402	162.44	5600	248.402	1.2361	2Ø20
span	1 to 2	28.49	350	357	0.0212	350.19	single	187.119	162.44	5600	187.119	0.9311	2Ø20
suport	2	42.68	350	357	0.0318	346.7	single	283.142	162.44	5600	283.142	1.4089	2Ø20
span	2to3	29.29	350	357	0.0218	350	single	192.48	162.44	5600	192.48	0.9578	2Ø20
suport	3	18.9	350	357	0.0141	352.51	single	123.315	162.44	5600	162.440	0.517	2Ø20
span	3 to4	4.8	350	357	0.0036	355.87	single	31.0226	162.44	5600	162.440	0.517	2Ø20
suport	4	0	350	357	0	357	single	0	162.44	5600	162.440	0.517	2Ø20
span	4 to 5	30.8	350	357	0.0229	349.63	single	202.617	162.44	5600	202.617	1.0082	2Ø20
suport	5	44.01	350	357	0.0327	346.37	single	292.244	162.44	5600	292.244	1.4542	2Ø20
span	5 to 6	34.51	350	357	0.0257	348.72	single	227.615	162.44	5600	227.615	1.1326	2Ø20
suport	6	48.27	350	357	0.0359	345.3	single	321.521	162.44	5600	321.521	1.5999	2Ø20

Axis –D ground

type	Location	moment (KNm)	b	D	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	1	17.04	300	357	0.0148	352.28	single	111.254	139.23	4800	111.254	0.3543	2Ø20
span	1 to 2	11.136	300	357	0.0097	353.93	single	72.3677	139.23	4800	139.23	0.44	2Ø20
suport	2	0	300	357	0	357	single	0	139.23	4800	139.23	0.44	2Ø20
span	2to3	12.84	300	357	0.0111	353.45	single	83.553	139.23	4800	139.23	0.44	2Ø20
suport	3	11.71	300	357	0.0102	353.77	single	76.1322	139.23	4800	139.23	0.44	2Ø20
suport	3 to 4	4.58	300	357	0.004	355.74	single	29.6114	139.23	4800	139.23	0.44	2Ø20
suport	4	19.92	300	357	0.0173	351.47	single	130.357	139.23	4800	130.357	0.4152	2Ø20
span	4 to 5	13.38	300	357	0.0116	353.3	single	87.1043	139.23	4800	139.23	0.44	2Ø20
suport	5	0	300	357	0	357	single	0	139.23	4800	139.23	0.44	2Ø20
span	5 to 6	11.09	300	357	0.0096	353.94	single	72.0661	139.23	4800	139.23	0.44	2Ø20
suport	6	15.424	300	357	0.0134	352.73	single	100.574	139.23	4800	139.23	0.44	2Ø20

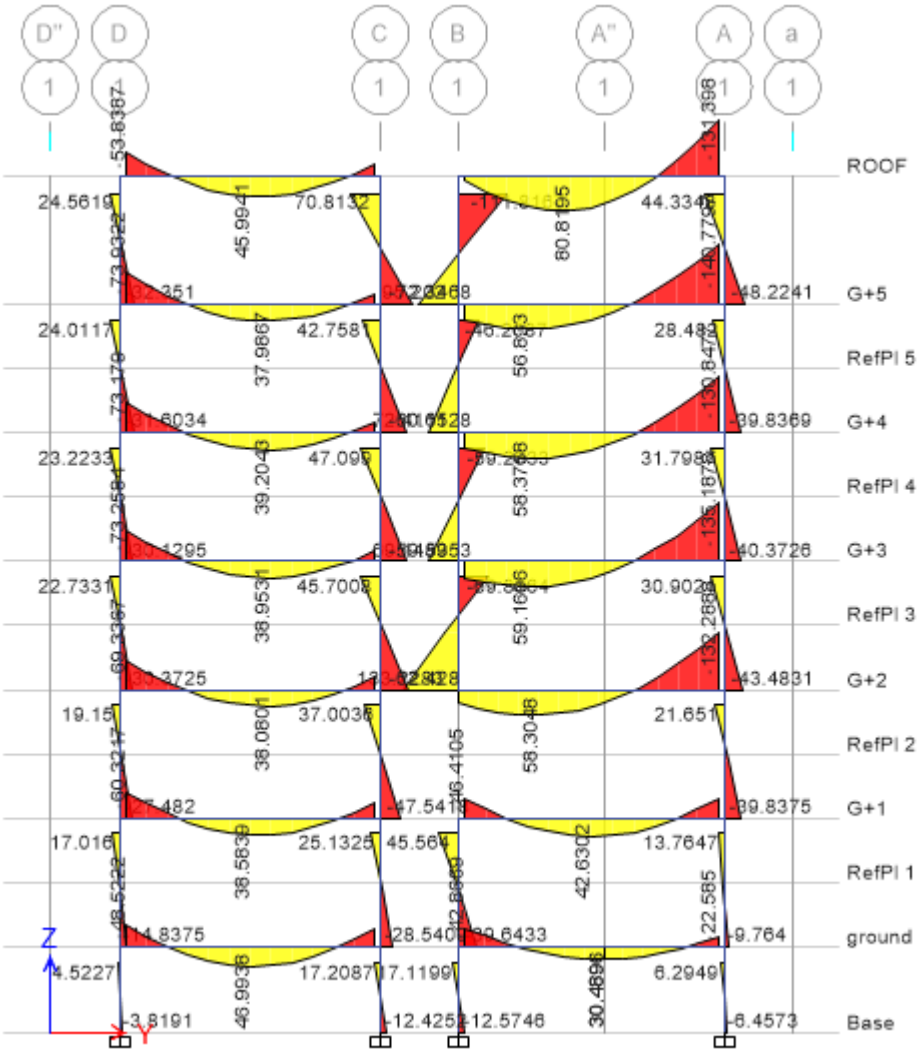


Figure 7-4 Bending moment diagram of beam in axis 1

Table 7-8 Reinforcement on axis 1

Axis 1 Roof

Type	Location	moment (KNm)	b	d	K	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	53.84	250	307	0.0561	290.97	single	425.588	99.775	3500	425.588	1.3554	2Ø20
Span	d to c	45.9	250	307	0.0478	293.45	single	359.759	99.775	3500	359.759	1.1457	2Ø20
Support	C	0	250	307	0	307	single	0	99.775	3500	99.7750	0.317	2Ø20
Span	c to b	0	250	307	0	307	single	0	99.775	3500	99.7750	0.317	2Ø20
Support	B	0	250	307	0	307	single	0	99.775	3500	99.7750	0.317	2Ø20
Support	b to a	80.82	250	307	0.0842	282.19	single	658.741	99.775	3500	658.741	2.0979	3Ø20
Support	A	131.4	250	307	0.1369	263.85	single	1145.42	99.775	3500	1145.42	3.6478	4Ø20

Axis 1 Story 5

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	73.93	350	357	0.055	338.74	single	501.985	162.44	5600	501.985	2.4979	3Ø20
Span	d to c	37.98	350	357	0.0283	347.86	single	251.118	162.44	5600	251.118	1.2496	2Ø20
Support	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.5173	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.5173	2Ø20
Support	B	0	350	357	0	357	single	0	162.44	5600	162.440	0.5173	2Ø20
Span	b to a	56.58	350	357	0.0421	343.2	single	379.177	162.44	5600	379.177	1.8868	2Ø20
Support	A	140.77	350	357	0.1047	320.21	single	1011.13	162.44	5600	1011.13	5.0315	6Ø20

Axis 1 Story 4

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	73.17	350	357	0.0544	338.93	single	496.534	162.44	5600	496.534	2.4708	3Ø20
Span	d to c	39.2	350	357	0.0292	347.56	single	259.409	162.44	5600	259.409	1.2908	2Ø20
Support	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Support	B	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Span	b to a	58.37	350	357	0.0434	342.75	single	391.69	162.44	5600	391.692	1.9491	2Ø20
Support	A	130.84	350	357	0.0974	323.11	single	931.359	162.44	5600	931.359	4.6345	5Ø20

Axis 1 Story 3

Type	Location	moment (KNm)	b	D	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	73.26	350	357	0.0545	338.91	single	497.179	162.44	5600	497.179	2.474	3Ø20
Span	d to c	38.95	350	357	0.029	347.62	single	257.709	162.44	5600	257.709	1.2824	2Ø20
Support	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Support	B	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Span	b to a	59.16	350	357	0.044	342.55	single	397.227	162.44	5600	397.227	1.9766	2Ø20
Support	A	135.2	350	357	0.1006	321.85	single	966.184	162.44	5600	966.184	4.8078	5Ø20

Axis 1 Story 2

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	69.34	350	357	0.0516	339.93	single	469.1649	162.44	5600	469.165	2.3346	3Ø20
Span	d to c	38.07	350	357	0.0283	347.84	single	251.7287	162.44	5600	251.729	1.2526	2Ø20
Support	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Support	B	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Span	b to a	58.3	350	357	0.0434	342.77	single	391.202	162.44	5600	391.202	1.9467	2Ø20
Support	A	132.28	350	357	0.0984	322.7	single	942.827	162.44	5600	942.827	4.6916	5Ø20

Axis 1 Story 1

Type	Location	moment (KNm)	b	d	K	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	60.32	350	357	0.0449	342.25	single	405.366	162.44	5600	405.366	2.0171	2Ø20
Span	d to c	38.58	350	357	0.0287	347.71	single	255.193	162.44	5600	255.193	1.2699	2Ø20
Support	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20
Support	B	46.41	350	357	0.0345	345.77	single	308.715	162.44	5600	308.715	1.5362	2Ø20
Span	b to a	42.63	350	357	0.0317	346.71	single	282.799	162.44	5600	282.8	1.4072	2Ø20
Support	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.51730	2Ø20

Axis 1 Ground

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	47.52	300	357	0.0413	343.49	single	318.192	139.23	4800	318.192	1.5834	2Ø20
Span	d to c	45.99	300	357	0.0399	343.95	single	307.542	139.23	4800	307.542	1.5304	2Ø20
Support	C	0	300	357	0	357	single	0	139.23	4800	139.230	0.44	2Ø20
Span	c to b	0	300	357	0	357	single	0	139.23	4800	139.230	0.44	2Ø20
Support	B	44.88	300	357	0.039	344.27	single	299.834	139.23	4800	299.834	1.492	2Ø20
Span	b to a	30.49	300	357	0.0265	348.46	single	201.251	139.23	4800	201.251	1.0014	2Ø20
Support	A	0	300	357	0	357	single	0	139.23	4800	139.230	0.44	2Ø20

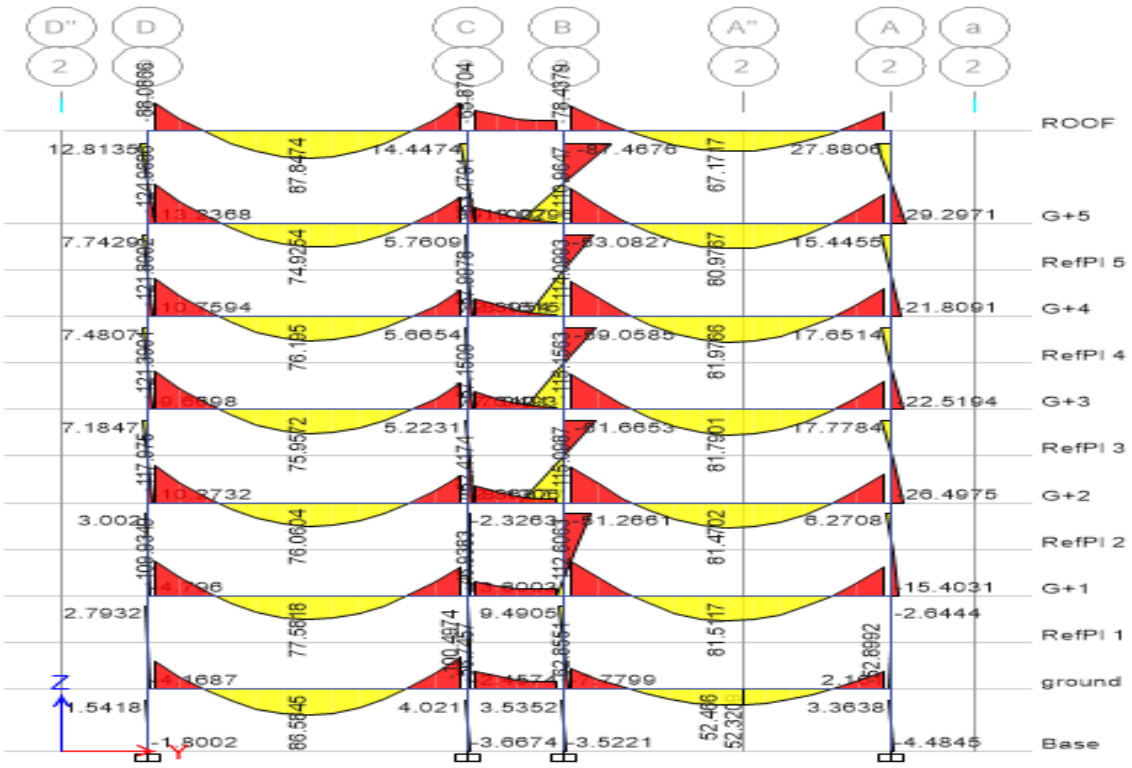


Figure 7-5 Bending moment diagram of beam in axis 2

Table 7-9Reinforcement on axis 2

Axis 2 Roof

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	D	88.08	250	307	0.0918	279.72	single	724.24821	99.775	3500	724.248	3.6039	2Ø20
span	d to c	87.85	250	307	0.0915	279.8	single	722.15327	99.775	3500	722.153	3.5935	2Ø20
suport	C	65.87	250	307	0.0686	287.12	single	527.65409	99.775	3500	527.654	2.6257	2Ø20
span	c to b	0	250	307	0	307	single	0	99.775	3500	0	0	2Ø20
suport	B	78.44	250	307	0.0817	282.98	single	637.53746	99.775	3500	637.537	3.1725	2Ø20
span	b to a	67.179	250	307	0.07	286.7	single	538.93771	99.775	3500	538.938	2.6818	2Ø20
suport	A	0	250	307	0	307	single	0	99.775	3500	0	0	2Ø20

Axis 2 Story 5

Type	Location	moment (KNm)	b	d	K	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	D	124.878	350	357	0.0929	324.83	single	884.227	162.44	5600	884.227	2.816	3Ø20
Span	d to c	74.93	350	357	0.0558	338.47	single	509.168	162.44	5600	509.168	1.6216	2Ø20
suport	C	63.47	350	357	0.0472	341.44	single	427.543	162.44	5600	427.543	1.3616	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
suport	B	110.94	350	357	0.0825	328.76	single	776.139	162.44	5600	776.139	2.4718	3Ø20
suport	b to a	80.98	350	357	0.0603	336.88	single	552.877	162.44	5600	552.877	1.7608	2Ø20
suport	A	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20

Axis 2 Story 4

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
Support	D	121.8	350	357	0.0906	325.71	single	860.109	162.44	5600	860.109	2.7392	3Ø20
Span	d to c	76.19	350	357	0.0567	338.14	single	518.235	162.44	5600	518.235	1.6504	2Ø20
Support	C	61.29	350	357	0.0456	342	single	412.183	162.44	5600	412.183	1.3127	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20
Support	B	114.06	350	357	0.0849	327.89	single	800.087	162.44	5600	800.087	2.548	3Ø20
Support	b to a	81.98	350	357	0.061	336.62	single	560.144	162.44	5600	560.144	1.7839	2Ø20
Support	A	0	350	357	0	357	single	0	162.44	5600	162.44	0.517	2Ø20

Axis 2 Story 3

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	121.3	350	357	0.0903	325.85	single	856.204	162.44	5600	856.204	2.726	32Ø20
Span	d to c	75.955	350	357	0.0565	338.21	single	516.542	162.44	5600	516.542	1.645	2Ø20
Support	C	56.15	350	357	0.0418	343.31	single	376.175	162.44	5600	376.175	1.198	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.517	2Ø20
suport	B	113.16	350	357	0.0842	328.14	single	793.164	162.44	5600	793.164	2.526	3Ø20
suport	b to a	81.79	350	357	0.0609	336.67	single	558.762	162.44	5600	558.762	1.779	2Ø20
suport	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.517	2Ø20

Axis 2 Story 2

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	D	117.97	350	357	0.0878	326.79	single	830.297	162.44	5600	830.297	2.644	3Ø20
span	d to c	76.06	350	357	0.0566	338.18	single	517.298	162.44	5600	517.299	1.647	2Ø20
suport	C	52.4	350	357	0.039	344.26	single	350.084	162.44	5600	350.084	1.114	2Ø20
span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.517	2Ø20
suport	B	115.09	350	357	0.0856	327.6	single	808.023	162.44	5600	808.023	2.573	3Ø20
suport	b to a	81.47	350	357	0.0606	336.75	single	556.436	162.44	5600	556.436	1.772	2Ø20
suport	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.517	2Ø20

Axis 2 Story 1

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	109.83	350	357	0.0817	329.07	single	767.652	162.44	5600	767.652	2.4448	3Ø20
Span	d to c	77.58	350	357	0.0577	337.78	single	528.259	162.44	5600	528.259	1.6824	2Ø20
Support	C	46.93	350	357	0.0349	345.64	single	312.292	162.44	5600	312.292	0.9946	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.517	2Ø20
Support	B	112.6	350	357	0.0838	328.3	single	788.862	162.44	5600	788.863	2.5123	3Ø20
Support	b to a	81.51	350	357	0.0606	336.74	single	556.727	162.44	5600	556.727	1.773	2Ø20
Support	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.517	2Ø20

Axis 2 Ground

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	0	300	357	0	357	Single	0	139.23	4800	139.230	0.440	2Ø20
Span	d to c	86.58	300	357	0.0752	331.5	Single	600.709	139.23	4800	600.709	1.9131	2Ø20
Support	C	50.74	300	357	0.044	342.54	Single	340.701	139.23	4800	340.7	1.085	2Ø20
Span	c to b	0	300	357	0	357	Single	0	139.23	4800	139.230	0.440	2Ø20
Support	B	62.85	300	357	0.0546	338.89	Single	426.553	139.23	4800	426.553	1.3584	2Ø20
suport	b to a	52.46	300	357	0.0455	342.02	Single	352.778	139.23	4800	352.778	1.1235	2Ø20
suport	A	52.89	300	357	0.0459	341.9	Single	355.803	139.23	4800	355.803	1.1331	2Ø20

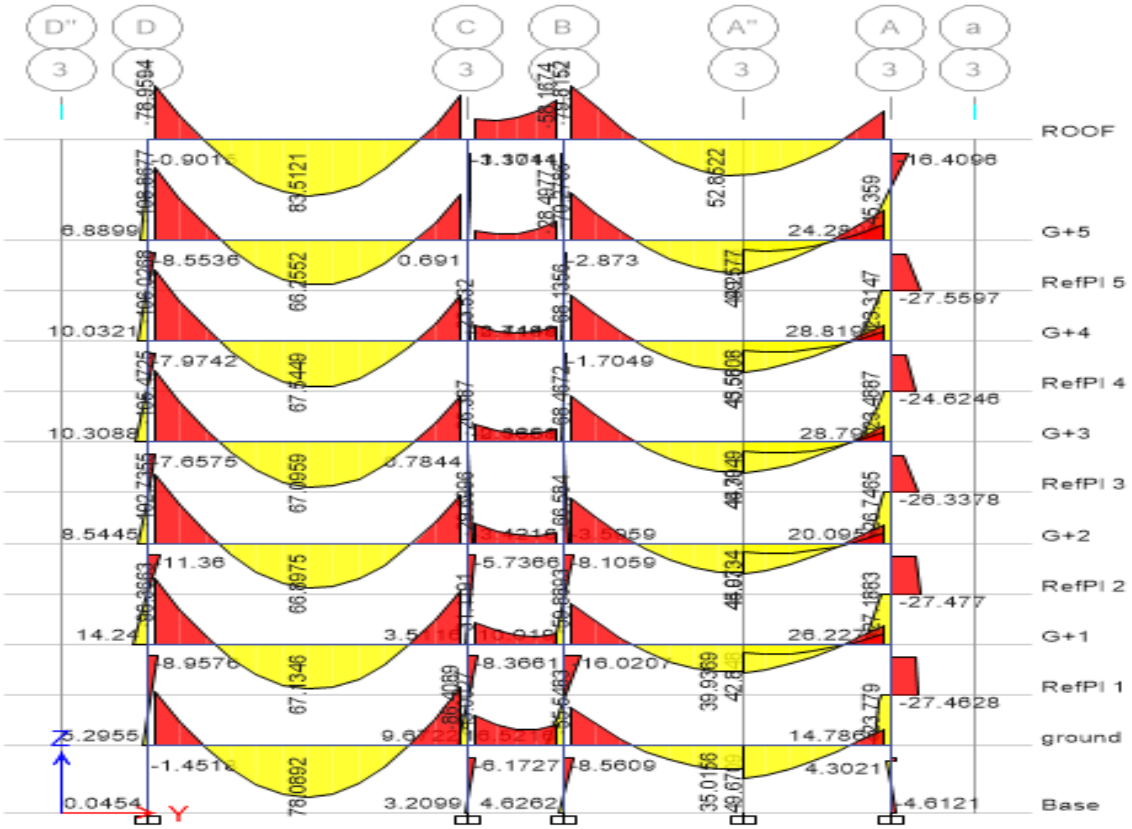


Figure 7-6 Bending moment diagram of beam in axis 3

Table 7-10 Reinforcement on axis 3

Axis 3 Roof

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	78.96	250	307	0.0823	282.81	Single	642.151	99.775	3500	642.159	2.0451	3Ø20
Span	d to c	83.51	250	307	0.087	281.28	Single	682.8	99.775	3500	682.865	2.1747	3Ø20
Support	C	0	250	307	0	307	Single	0	99.775	3500	99.77	0.317	2Ø20
Span	c to b	0	250	307	0	307	Single	0	99.775	3500	99.77	0.317	2Ø20
Support	B	78.82	250	307	0.0821	282.86	Single	640.914	99.775	3500	640.914	2.0411	3Ø20
Support	b to a	52.89	250	307	0.0551	291.27	Single	417.645	99.775	3500	417.65	1.3301	2Ø20
Support	A	0	250	307	0	307	Single	0	99.775	3500	99.77	0.317	2Ø20

Axis 3 Story 5

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
Support	D	108.87	350	357	0.081	329.34	Single	760.326	162.44	5600	760.326	2.4214	3Ø20
Span	d to c	66.25	350	357	0.0493	340.73	Single	447.206	162.44	5600	447.206	1.4242	2Ø20
Support	C	0	350	357	0	357	Single	0	162.44	5600	162.440	0.52	2Ø20
Span	c to b	0	350	357	0	357	Single	0	162.44	5600	162.440	0.52	2Ø20
suport	B	70.27	350	357	0.0523	339.69	Single	475.795	162.44	5600	475.795	1.5153	2Ø20
suport	b to a	49.25	350	357	0.0366	345.06	Single	328.282	162.44	5600	328.283	1.0455	2Ø20
suport	A	0	350	357	0	357	Single	0	162.44	5600	162.440	0.52	2Ø20

Axis 3 Story 4

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	106.47	350	357	0.0792	330	single	742.068	162.44	5600	742.068	2.3633	3Ø20
Span	d to c	67.09	350	357	0.0499	340.51	single	453.165	162.44	5600	453.165	1.4432	2Ø20
Support	C	23.53	350	357	0.0175	351.4	single	154.012	162.44	5600	162.440	0.52	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Support	B	68.53	350	357	0.051	340.14	single	463.399	162.44	5600	463.398	1.4758	2Ø20
Support	b to a	45.58	350	357	0.0339	345.97	single	303.012	162.44	5600	303.012	0.965	2Ø20
Support	A	0	350	357	0	345.06	single	0	162.44	5600	162.440	0.52	2Ø20

Axis 3 Story 3

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	106.47	350	357	0.0792	330	single	742.068	162.44	5600	742.068	2.3633	2Ø20
Span	d to c	67.09	350	357	0.0499	340.51	single	453.165	162.44	5600	453.165	1.4432	2Ø20
Support	C	20.87	350	357	0.0155	352.04	single	136.352	162.44	5600	162.440	0.52	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Support	B	68.46	350	357	0.0509	340.16	single	462.900	162.44	5600	462.9	1.4742	2Ø20
Support	b to a	44.39	350	357	0.033	346.27	single	294.848	162.44	5600	294.848	0.939	3Ø20
Support	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	4Ø20

Axis 3 Story 2

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	D	102.73	350	357	0.0764	331.03	single	713.774	162.44	5600	713.775	2.2732	3Ø20
span	d to c	66.89	350	357	0.0498	340.56	single	451.746	162.44	5600	451.745	1.4387	2Ø20
suport	C	20.86	350	357	0.0155	352.04	single	136.286	162.44	5600	162.440	0.52	2Ø20
span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
suport	B	66.58	350	357	0.0495	340.64	single	449.547	162.44	5600	449.546	1.4317	2Ø20
suport	b to a	44.93	350	357	0.0334	346.14	single	298.551	162.44	5600	298.551	0.9508	2Ø20
suport	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20

Axis 3 Story 1

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	D	96.36	350	357	0.0717	332.77	single	666.02	162.44	5600	666.02	2.1211	3Ø20
Span	d to c	67.13	350	357	0.0499	340.5	single	453.445	162.44	5600	453.449	1.4441	2Ø20
suport	C	31.19	350	357	0.0232	349.53	single	205.239	162.44	5600	205.238	0.6536	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
suport	B	58.88	350	357	0.0438	342.62	single	395.265	162.44	5600	395.264	1.2588	2Ø20
suport	b to a	39.94	350	357	0.0297	347.38	single	264.446	162.44	5600	264.445	0.8422	2Ø20
suport	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20

Axis 3 Ground

type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	D	0	300	357	0	357	single	0	139.23	4800	139.230	0.44	2Ø20
span	d to c	78.08	300	357	0.0678	334.19	single	537.376	139.23	4800	537.376	1.7114	2Ø20
suport	C	43.02	300	357	0.0373	344.82	single	286.952	139.23	4800	286.952	0.9139	2Ø20
span	c to b	0	300	357	0	357	single	0	139.23	4800	139.230	0.44	2Ø20
suport	B	55.64	300	357	0.0483	341.07	single	375.207	139.23	4800	375.207	1.1949	2Ø20
suport	b to a	35.01	300	357	0.0304	347.15	single	231.953	139.23	4800	231.953	0.7387	2Ø20
suport	A	0	300	357	0	357	single	0	139.23	4800	139.230	0.44	2Ø20

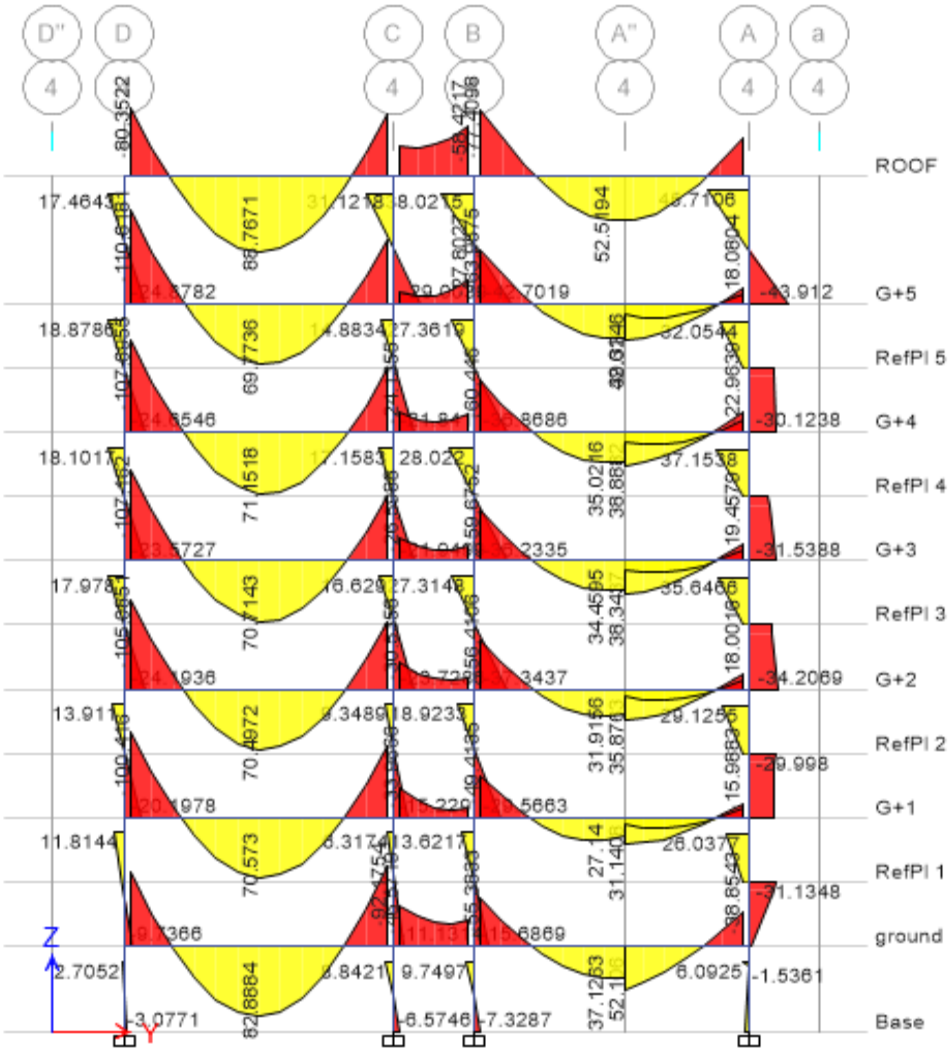


Figure 7-7 Bending moment diagram of beam in axis 4

Table 7-11 Reinforcement on axis 4

Axis 4 Roof

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	89.35	250	307	0.0931	279.28	single	735.84	99.775	3500	735.84	2.3434	3Ø20
Span	d to c	86.76	250	307	0.0904	280.17	single	712.243	99.775	3500	712.243	2.2683	3Ø20
Support	C	0	250	307	0	307	single	0	99.775	3500	99.7750	0.32	2Ø20
Span	c to b	0	250	307	0	307	single	0	99.775	3500	99.7750	0.32	2Ø20
Support	B	71.41	250	307	0.0744	285.32	single	575.658	99.775	3500	575.658	1.8333	2Ø20
Support	b to a	52.52	250	307	0.0547	291.38	single	414.563	99.775	3500	414.562	1.3203	2Ø20
Support	A	0	250	307	0	307	single	0	99.775	3500	99.7750	0.32	2Ø20

Axis 4 Story 5

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	110.6	350	357	0.0823	328.85	single	773.537	162.44	5600	773.537	2.4635	3Ø20
Span	d to c	66.7	350	357	0.0496	340.61	single	450.397	162.44	5600	450.397	1.4344	2Ø20
Support	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Support	B	63.57	350	357	0.0473	341.42	single	428.249	162.44	5600	428.249	1.3638	2Ø20
Support	b to a	48.62	350	357	0.0362	345.21	single	323.935	162.44	5600	323.935	1.0316	3Ø20
Support	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20

Axis 4 Story 4

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	107.8	350	357	0.0802	329.63	single	752.176	162.44	5600	752.176	2.3955	3Ø20
Span	d to c	71.15	350	357	0.0529	339.46	single	482.078	162.44	5600	482.078	1.5353	2Ø20
Support	C	21.13	350	357	0.0157	351.98	single	138.076	162.44	5600	162.440	0.52	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Support	B	60.44	350	357	0.045	342.22	single	406.209	162.44	5600	406.209	1.2937	2Ø20
Support	b to a	38.88	350	357	0.0289	347.64	single	257.233	162.44	5600	257.233	0.8192	2Ø20
Support	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20

Axis 4 Story 3

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	107.46	350	357	0.08	329.73	single	749.591	162.44	5600	749.59	2.3872	3Ø20
Span	d to c	70.71	350	357	0.0526	339.57	single	478.935	162.44	5600	478.935	1.5253	2Ø20
Support	C	26.5	350	357	0.0197	350.68	single	173.808	162.44	5600	173.808	0.5535	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Support	B	59.67	350	357	0.0444	342.42	single	400.803	162.44	5600	400.803	1.2764	2Ø20
Support	b to a	38.34	350	357	0.0285	347.77	single	253.562	162.44	5600	253.563	0.8075	2Ø20
Support	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20

Axis 4 Story 2

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	105.06	350	357	0.0782	330.39	single	731.379	162.44	5600	731.379	2.3292	3Ø20
Span	d to c	70.49	350	357	0.0524	339.63	single	477.365	162.44	5600	477.365	1.5203	2Ø20
Support	C	30.01	350	357	0.0223	349.82	single	197.311	162.44	5600	197.311	0.6284	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Support	B	56.4	350	357	0.042	343.25	single	377.919	162.44	5600	377.92	1.2036	2Ø20
Support	b to a	35.87	350	357	0.0267	348.38	single	236.812	162.44	5600	236.812	0.7542	3Ø20
Support	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20

Axis 4 Story 1

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	100.42	350	357	0.0747	331.66	single	696.394	162.44	5600	696.394	2.2178	3Ø20
Span	d to c	70.57	350	357	0.0525	339.61	single	477.935	162.44	5600	477.936	1.5221	2Ø20
Support	C	33.16	350	357	0.0247	349.05	single	218.503	162.44	5600	218.503	0.6959	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Support	B	49.4	350	357	0.0368	345.02	single	329.318	162.44	5600	329.318	1.0488	2Ø20
Support	b to a	31.14	350	357	0.0232	349.54	single	204.902	162.44	5600	204.902	0.6526	2Ø20
Support	A	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20

Axis 4 Ground

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
Support	D	0	300	357	0	357	single	0	139.23	4800	139.230	0.44	2Ø20
Span	d to c	82.88	300	357	0.0719	332.68	single	573.005	139.23	4800	573.005	1.8249	2Ø20
Support	C	46.55	300	357	0.0404	343.78	single	311.437	139.23	4800	311.437	0.9918	2Ø20
Span	c to b	0	300	357	0	357	single	0	139.23	4800	139.230	0.44	2Ø20
Support	B	55.3	300	357	0.048	341.17	single	372.802	139.23	4800	372.802	1.1873	2Ø20
Support	b to a	52.1	300	357	0.0452	342.13	single	350.247	139.23	4800	350.247	1.1154	2Ø20
Support	A	0	300	357	0	357	single	0	139.23	4800	139.230	0.44	2Ø20

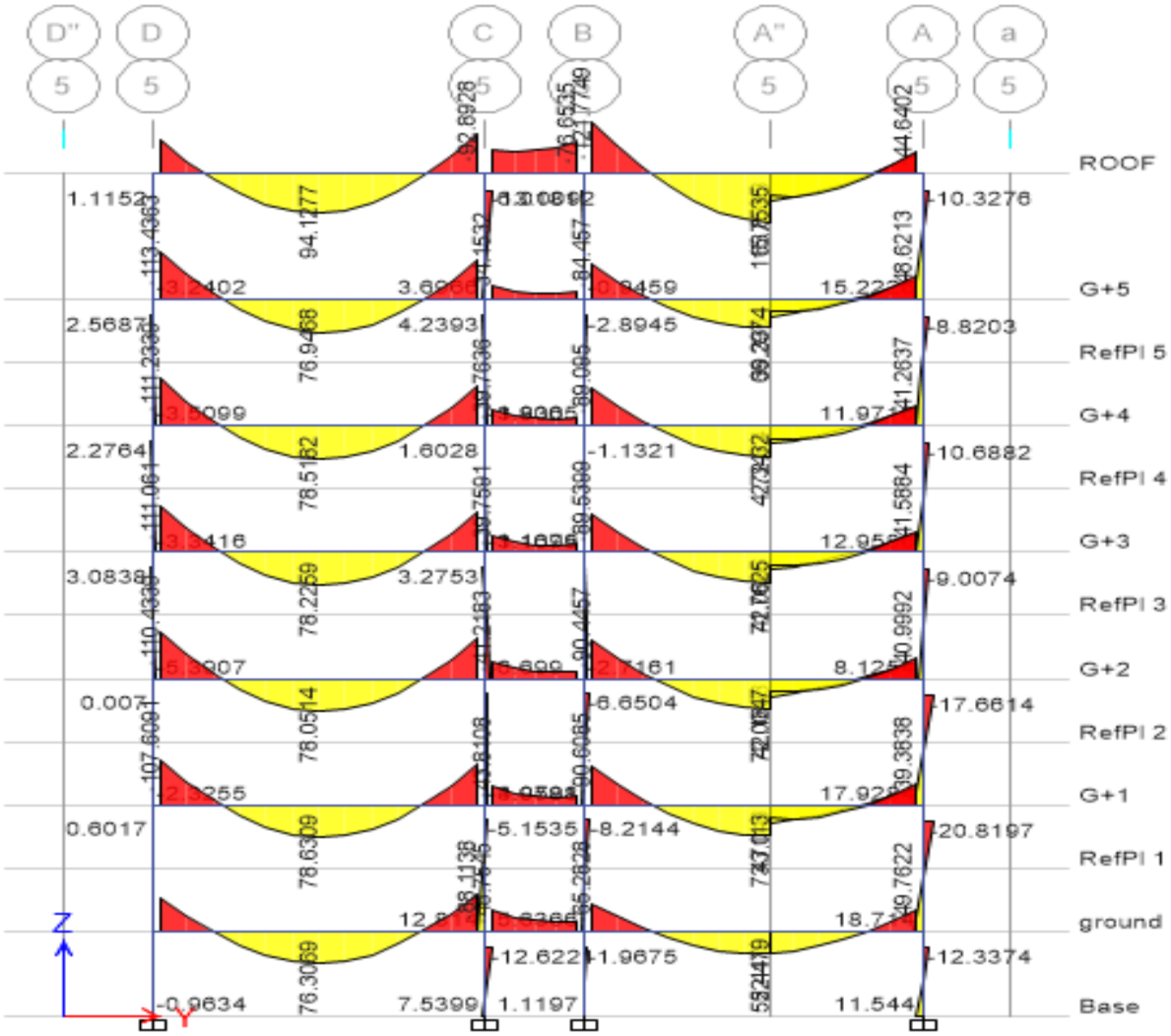


Figure 7-8 Bending moment diagram of beam in axis 5

Table 7-12 Reinforcement on axis 5

Axis 5 Roof

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	0	250	307	0	307	single	0	99.775	3500	99.7750	0.32	2Ø20
Span	d to c	94.13	250	307	0.0981	277.62	single	779.8325	99.775	3500	779.832	2.4835	3Ø20
Support	C	92.89	250	307	0.0968	278.06	single	768.3639	99.775	3500	768.364	2.447	3Ø20
Span	c to b	0	250	307	0	307	single	0	99.775	3500	99.7750	0.32	2Ø20
Support	B	72.77	250	307	0.0758	284.87	single	587.5433	99.775	3500	587.543	1.8712	2Ø20
Support	b to a	166.76	250	307	0.1737	248.97	single	1540.562	99.775	3500	1540.56	4.9062	5Ø20
Support	A	44.64	250	307	0.0465	293.84	single	349.4197	99.775	3500	349.42	1.1128	2Ø20

Axis 5 Story 5

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
Support	D	113.53	350	357	0.0845	328.04	single	796.00	162.44	5600	796.009	2.5351	3Ø20
Span	d to c	76.94	350	357	0.0572	337.95	single	523.64	162.44	5600	523.641	1.6676	2Ø20
Support	C	39.75	350	357	0.0604	336.84	single	554.11	162.44	5600	554.111	1.7647	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Support	B	84.45	350	357	0.0628	335.96	single	578.14	162.44	5600	578.145	1.8412	2Ø20
Support	b to a	88.23	350	357	0.0656	334.96	single	605.84	162.44	5600	605.84	1.9294	2Ø20
Support	A	48.62	350	357	0.0362	345.21	single	323.93	162.44	5600	323.935	1.0316	2Ø20

Axis 5 Story 4

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	111.23	350	357	0.0828	328.68	single	778.359	162.44	5600	778.359	2.4788	3Ø20
Span	d to c	78.52	350	357	0.0584	337.53	single	535.051	162.44	5600	535.051	1.704	2Ø20
Support	C	39.76	350	357	0.0296	347.42	single	263.22	162.44	5600	263.22	0.8383	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.44	0.52	2Ø20
Support	B	89.54	350	357	0.0666	334.61	single	615.480	162.44	5600	615.48	1.9601	2Ø20
Support	b to a	42.7	350	357	0.0318	346.69	single	283.278	162.44	5600	283.278	0.9022	2Ø20
Support	A	41.26	350	357	0.0307	347.05	single	273.443	162.44	5600	273.443	0.8708	2Ø20

Axis 5 Story 3

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	111.06	350	357	0.0826	328.73	single	777.057	162.44	5600	777.057	2.4747	3Ø20
Span	d to c	78.23	350	357	0.0582	337.61	single	532.955	162.44	5600	532.955	1.6973	2Ø20
Support	C	39.75	350	357	0.0296	347.43	single	263.152	162.44	5600	263.152	0.8381	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.44	0.52	2Ø20
Support	B	89.54	350	357	0.0666	334.61	single	615.480	162.44	5600	615.48	1.9601	2Ø20
Support	b to a	42.06	350	357	0.0313	346.85	single	278.904	162.44	5600	278.904	0.8882	2Ø20
Support	A	41.58	350	357	0.0309	346.97	single	275.627	162.44	5600	275.627	0.8778	2Ø20

Axis 5 Story 2

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	110.43	350	357	0.0822	328.9	single	772.237	162.44	5600	772.237	2.4594	3Ø20
Span	d to c	78.05	350	357	0.0581	337.66	single	531.654	162.44	5600	531.654	1.6932	2Ø20
Support	C	41.22	350	357	0.0307	347.06	single	273.169	162.44	5600	273.17	0.87	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.44	0.52	2Ø20
Support	B	90.44	350	357	0.0673	334.36	single	622.115	162.44	5600	622.115	1.9813	2Ø20
Support	b to a	42.08	350	357	0.0313	346.85	single	279.041	162.44	5600	279.041	0.8887	2Ø20
Support	A	40.99	350	357	0.0305	347.12	single	271.601	162.44	5600	271.601	0.865	2Ø20

Axis 5 Story 1

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	107.61	350	357	0.0801	329.68	single	750.731	162.44	5600	750.731	2.3909	3Ø20
Span	d to c	78.63	350	357	0.0585	337.5	single	535.847	162.44	5600	535.847	1.7065	2Ø20
Support	C	41.81	350	357	0.0311	346.91	single	277.197	162.44	5600	277.197	0.8828	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.44	0.52	2Ø20
Support	B	90.6	350	357	0.0674	334.32	single	623.296	162.44	5600	623.296	1.985	2Ø20
Support	b to a	43.01	350	357	0.032	346.62	single	285.398	162.44	5600	285.398	0.9089	2Ø20
Support	A	39.38	350	357	0.0293	347.52	single	260.633	162.44	5600	260.634	0.83	2Ø20

Axis 5 Ground

Type	Location	moment (KNm)	b	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	0	300	357	0	357	single	0	139.23	4800	139.23	0.44	2Ø20
Span	d to c	76.31	300	357	0.0662	334.7	single	524.325	139.23	4800	524.325	1.6698	2Ø20
Support	C	38.56	300	357	0.0335	346.1	single	256.234	139.23	4800	256.234	0.816	2Ø20
Span	c to b	0	300	357	0	357	single	0	139.23	4800	139.23	0.44	2Ø20
Support	B	65.28	300	357	0.0567	338.1	single	444.016	139.23	4800	444.016	1.4141	2Ø20
Support	b to a	59.24	300	357	0.0514	339.9	single	400.757	139.23	4800	400.757	1.2763	2Ø20
Support	A	49.76	300	357	0.0432	342.8	single	333.836	139.23	4800	333.836	1.0632	2Ø20

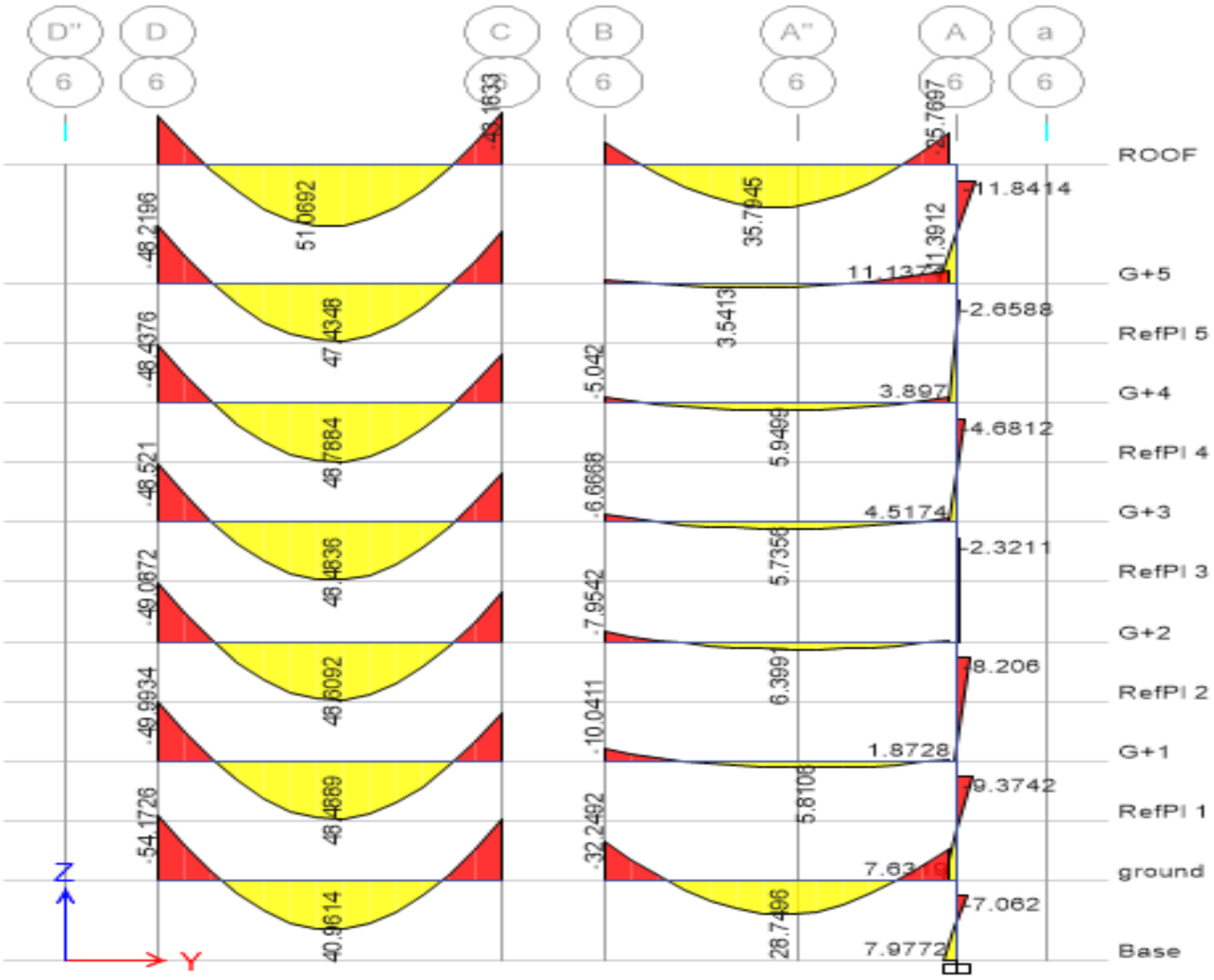


Figure 7-9 Bending moment diagram of beam in axis 6

Table 7-13 Reinforcement on axis 6

Axis 6 Roof

Type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	0	250	307	0	307	single	0	99.775	3500	99.7750	0.32	2Ø20
Span	d to c	51.06	250	307	0.0532	291.84	single	402.406	99.775	3500	402.406	1.281	2Ø20
Support	C	43.18	250	307	0.045	294.29	single	337.474	99.775	3500	337.475	1.074	2Ø20
Span	c to b	0	250	307	0	307	single	0	99.775	3500	99.7750	0.32	2Ø20
Support	B	0	250	307	0	307	single	0	99.775	3500	99.7750	0.32	2Ø20
suport	b to a	35.79	250	307	0.0373	296.54	single	277.591	99.775	3500	277.59	0.884	2Ø20
suport	A	25.76	250	307	0.0268	299.55	single	197.792	99.775	3500	197.792	0.629	2Ø20

Axis 6 Story 5

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	D	48.22	350	357	0.0359	345.31	single	321.176	162.44	5600	321.176	1.0229	2Ø20
span	d to c	47.43	350	357	0.0353	345.51	single	315.733	162.44	5600	315.733	1.0055	2Ø20
suport	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
suport	B	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
suport	b to a	3.54	350	357	0.0026	356.17	single	22.860	162.44	5600	162.440	0.52	2Ø20
suport	A	11.39	350	357	0.0085	354.31	single	73.939	162.44	5600	162.440	0.52	2Ø20

Axis 6 Story 4

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
suport	D	48.44	350	357	0.036	345.26	single	322.69	162.44	5600	322.693	1.0277	2Ø20
span	d to c	48.78	350	357	0.0363	345.17	single	325.039	162.44	5600	325.038	1.0352	2Ø20
suport	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
suport	B	5.04	350	357	0.0038	355.81	single	32.5789	162.44	5600	162.440	0.52	2Ø20
suport	b to a	5.95	350	357	0.0044	355.6	single	38.485	162.44	5600	162.440	0.52	2Ø20
suport	A	2.66	350	357	0.002	356.38	single	17.167	162.44	5600	162.440	0.52	2Ø20

Axis 6 Story 3

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	D	48.52	350	357	0.0361	345.24	single	323.245	162.44	5600	323.245	1.0294	2Ø20
span	d to c	48.48	350	357	0.0361	345.25	single	322.97	162.44	5600	322.969	1.0286	2Ø20
suport	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
suport	B	6.66	350	357	0.005	355.43	single	43.097	162.44	5600	162.440	0.52	2Ø20
suport	b to a	5.74	350	357	0.0043	355.65	single	37.121	162.44	5600	162.440	0.52	2Ø20
suport	A	4.68	350	357	0.0035	355.9	single	30.245	162.44	5600	162.440	0.52	2Ø20

Axis 6 Story 2

type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
suport	D	49.08	350	357	0.0365	345.1	single	327.109	162.44	5600	327.109	1.0417	2Ø20
span	d to c	48.31	350	357	0.0359	345.29	single	321.797	162.44	5600	321.797	1.0248	2Ø20
suport	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
suport	B	7.95	350	357	0.0059	355.13	single	51.489	162.44	5600	162.440	0.52	2Ø20
suport	b to a	6.39	350	357	0.0048	355.5	single	41.343	162.44	5600	162.440	0.52	2Ø20
suport	A	2.32	350	357	0.0017	356.46	single	14.969	162.44	5600	162.440	0.52	2Ø20

Axis 6 Story 1

Type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	remark
Support	D	49.99	350	357	0.0372	344.87	single	333.395	162.44	5600	333.395	1.0618	2Ø20
Span	d to c	48.48	350	357	0.0361	345.25	single	322.969	162.44	5600	322.969	1.0286	2Ø20
Support	C	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Span	c to b	0	350	357	0	357	single	0	162.44	5600	162.440	0.52	2Ø20
Support	B	10.04	350	357	0.0075	354.63	single	65.1160	162.44	5600	162.440	0.52	2Ø20
Support	b to a	5.81	350	357	0.0043	355.63	single	37.5755	162.44	5600	162.440	0.52	2Ø20
Support	A	1.87	350	357	0.0014	356.56	single	12.063	162.44	5600	162.440	0.52	2Ø20

Axis 6 Ground

Type	Location	moment (KNm)	B	d	k	Z	beam type	As,cal	As,min	Asmax	As,prov	No,bar	Remark
Support	D	54.17	300	357	0.047	341.51	single	364.822	139.23	5600	364.822	1.1619	2Ø20
Span	d to c	40.96	300	357	0.0356	345.42	single	272.734	139.23	5600	272.734	0.8686	2Ø20
Support	C	0	300	357	0	357	single	0	139.23	5600	139.23	0.44	2Ø20
Span	c to b	0	300	357	0	357	single	0	139.23	5600	139.23	0.44	2Ø20
Support	B	32.24	300	357	0.028	347.95	single	213.110	139.23	5600	213.11	0.6787	2Ø20
Support	b to a	28.74	300	357	0.0249	348.96	single	189.427	139.23	5600	189.427	0.6033	2Ø20
Support	A	9.37	300	357	0.0081	354.42	single	60.8067	139.23	5600	139.23	0.44	2Ø20

Shear Design

The concrete itself can resist shear by a combination of un-cracked concrete in the compression zone. The dowelling action of the bending reinforcement and aggregate interlocking across tension crack but, because concrete is weak in tension, the shear reinforcement is designed to resist all the tensile stress caused by the shear force.

Even where the shear force is small near the Centre span of beam a minimum amount of reinforcement in the form of links must be provided in order to form a cage supporting the longitudinal reinforcement and to resist any tensile stresses due to factors such as thermal movement and shrinkage of concrete.

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.

In regions of the member where $V_{Ed} \leq V_{Rd,c,}$ no calculated shear reinforcement is necessary. V_{Ed} is the design shear force in the section considered resulting from external loading and pre stressing (bonded or unbounded). When, on the basis of the design shear calculation, no shear reinforcement is required, minimum shear reinforcement should nevertheless be provided according to 9.2.2.

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

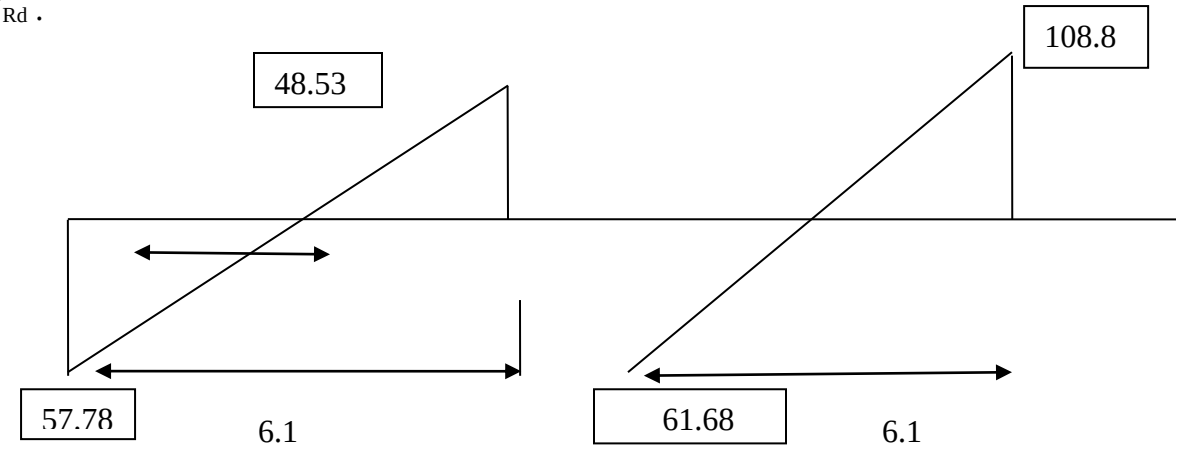


figure Sample Beam Design for Shear 1

$$\frac{108.82}{3.34} = \frac{Ved}{3.34-0.35} = Ved\,\frac{108.82(3.34-0.35)}{3.34} = \quad 97.42\text{KN}$$

A. Concrete capacity

The design value for the shear resistance VRd, c is given by:

$V_{Rd,c}= [C_{Rd,ck}(100\ \rho l\ f_{ck})^{1/3} + k1\ \sigma cp]\ b_w d$

with a minimum of $V_{Rd,c}= (V_{min}+ k1\ \sigma\ cp)\ bwd$

Where;

- fck = 20MPa
- $k=1+\ \sqrt{\frac{200}{d}}\ =1+\ \sqrt{\frac{200}{357}}\ =1.748\leq 2.0$
- $\rho_l =\frac{As1}{bwd}\ =\frac{314}{350*357}\ =0.0025\leq 0.02$

Therefore ρl=0.02

- $\sigma_{cp}\ =\frac{NED}{AC}\ =\frac{0}{350*400}=0$

NEd is the axial force in the cross-section due to loading or prestressing [in N] (NEd> 0 for compression). The influence of imposed deformations on NEd may be ignored.

AC is the area of concrete cross section [mm2].

- VRd,c is [N]
- Crdc=0.18/ γc=0.18/1.5=0.12

K1=0.15

- $V_{Rd,c}= [C_{Rd,ck}(100\ \rho l f_{ck})^{1/3} + k1\ \sigma cp]\ b_w d$

$= [0.12*1.748(100*\ 0.02*20)^{1/3} + 0.15\ *0]\ 350*350$

= 89.64KN≤ 103.34KN ---Shear reinforcement is required.

B. Diagonal compression check of concrete

Where v is a strength reduction factor for concrete cracked in shear.

$V=0.64f_{ck} \leq 60MPA$ from ESEN 1992/1;2015

$V_{rdmax} =\alpha_{cw} b_w Z v_1\ f_{cd}(\frac{1}{cot\theta+tan\theta})$

Where,

- α_{cw}=1 and $cot\theta = 2.5$
- b_w=300mm
- Z=0.9d=0.9*357=321.3
- f_{cd}=11.3MPA
- $v_1=0.6(1-\frac{fck}{250})\ =0.6(1-\frac{20}{250})=0.552$

- $V_{rdmax} = \alpha_{cw} b_w Z v_1 f_{cd} \left(\frac{1}{cot\theta + tan\theta}\right)$

$$= 0.25 * 0.9 * 0.29575 * 0.6 * 13.33 \left(\frac{1}{2.5+0.4}\right) =$$

168.84KN>V_{ED}=108.8KN

C. Calculate the required shear reinforcement

- The ratio of shear reinforcement is given by;

$P_w = A_{sw} / (S * b_w - \sin\alpha)$

Where:

- ✓ P_w is the shear reinforcement ratio.
- ✓ P_w should not be less than P_{wmin}
- ✓ Asw 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.

$$Pw\ min = \frac{0.08\sqrt{(fck)}}{fyk} = \frac{0.08\sqrt{(20)}}{400} = 0.00089$$

- The maximum longitudinal spacing between shear assemblies should not exceed S_{lmax}

$$Sl,max = 0.75d\ (1 + \cot\alpha) = 0.75 * 357\ (1 + \cot 90) = 267.75$$

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

- The maximum longitudinal spacing of bent-up bars should not exceed Sb,max.

$$Sb,max = 0.6d\ (1 + \cot\alpha) = 0.6 * 357\ (1 + \cot 90) = 214.2$$

The transverse spacing of the legs in a series of shear links should not exceed St,max

$$S_{tmax} = 0.75d = 267.75mm, \leq 600mm$$

$$Smin = \frac{Asw}{Pwbw \sin\alpha} = \frac{50.24}{0.00089 * 350 * 1} = 161.28mm \quad .$$

- Required shear reinforcement

$$\frac{Asw}{S} = \frac{VED}{0.78d * fykcot\theta}$$

$$S_{calc} = \frac{Asw * 0.78 * d * fyk * cot\theta}{Ved} = \frac{50.24 * 0.78 * 357 * 400 * 2.5}{97.42} = 143.6mm$$

$S_{prov} = \min\{Sl_{max}, Sb_{max}, S_{cal}\}$ but $S_{prov} \geq S_{min}$

S_{prov}= min{267.75,214.2,143.6} but $S_{prov} \geq 161.28$

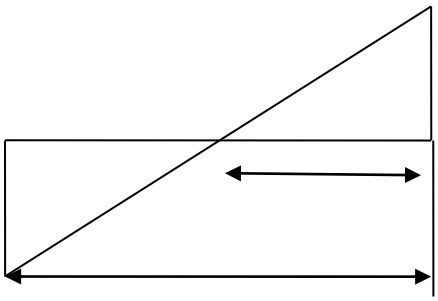
$S_{prov} \geq 143.6mm \geq S_{min}$ =161.28 - - - OK!!!

Therefore, Use dia of 8mm c/c 160mm.

Sample Beam Design for Shear 2

48.0

57.78



$\frac{48.08}{2.21} = \frac{Ved}{2.21-0.35} = Ved\ \frac{48.08(2.21-0.35)}{2.21} =$ 40.46KN

The design value for the shear resistance VRd, c is given by:

- $V_{Rd,c}= [C_{Rd,ck}(100\ \rho\ l\ f_{ck})^{1/3} + k1\ \sigma cp]\ b_w d$

with a minimum of $V_{Rd,c}= (V_{min}+ k1\ \sigma\ cp)\ bwd$

Where;

- fck = 20MPa
- $k=1+\ \sqrt{\frac{200}{d}}\ =1+\ \sqrt{\frac{200}{357}}\ =1.748 \leq 2.0$
- $\rho_l = \frac{As1}{bwd} = \frac{314}{350*357} = 0.0025 \leq 0.02$

Therefore ρl=0.02

- $\sigma_{cp} = \frac{NED}{AC} = \frac{0}{350*400} = 0.$

AC is the area of concrete cross section [mm2].

- VRd,c is [N]
- Crdc=0.18/ γc=0.18/1.5=0.12

K1=0.15

- $V_{Rd,c}= [C_{Rd,ck}(100\ \rho l f_{ck})^{1/3} + k1\ \sigma cp]\ b_w d$

$= [0.12*1.748(100*\ 0.02*20)^{1/3} + 0.15\ *0]\ 350*357$

= 89.64KN≥38.41KN ---Shear reinforcement is not required

Any compression longitudinal reinforcement (diameter φ) which is included in the resistance calculation should be held by transverse reinforcement with spacing not greater than 15φ.

Therefore, Use dia of 8mm c/c 300mm

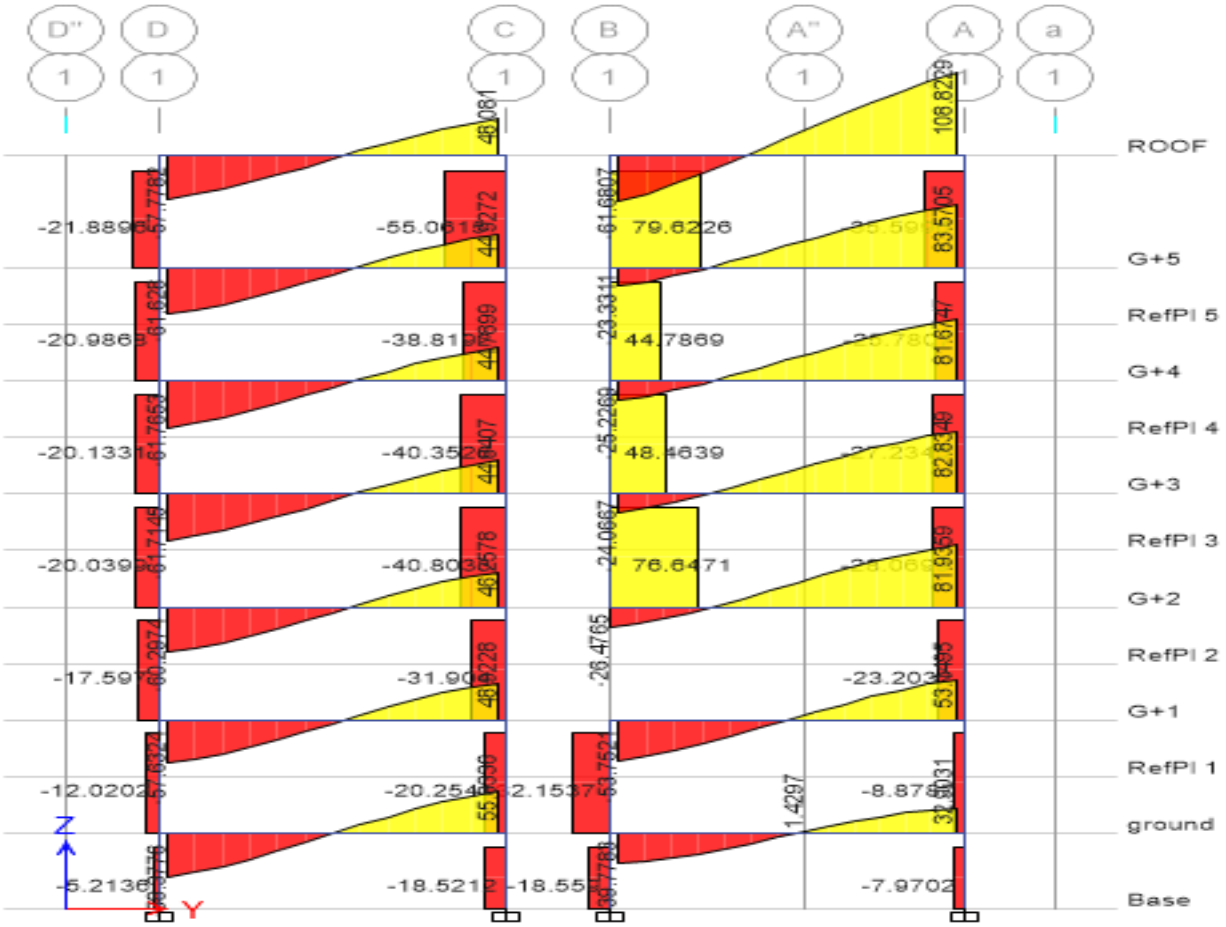


Figure 7-10 shear force diagram of beam axis 1

Axis 1 Roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	57.78	48.11	230.25	184.2	250.06	225.80	184.2	Φ 8 cc 210mm
	Near C	48.08	38.41	230.25	184.2	313.21	225.80	184.2	Φ 8 cc 210mm
B—A	Near B	61.68	51.5	230.25	184.2	233.60	225.80	184.2	Φ 8 cc 210mm
	NearA	108.82	38.07	230.25	184.2	316.01	225.80	184.2	Φ 8 cc 210mm

Axis 1 Story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	61.63	51.03	267.75	214.2	274.15	161.28	214.2	Φ 8 cc 210mm
	Near C	44.93	34.33	267.75	214.2	407.51	161.28	214.2	Φ 8 cc 210mm
B—A	Near B	23.33	12.693	267.75	214.2	1102.17	161.28	214.2	Φ 8 cc 210mm
	NearA	83.57	72.93	267.75	214.2	191.83	161.28	191.8	Φ 8 cc 210mm

Axis 1 Story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	61.765	51.16	267.75	214.2	273.45	161.28	214.2	Φ 8 cc 210mm
	Near C	44.79	34.19	267.75	214.2	409.18	161.28	214.2	Φ 8 cc 210mm
B—A	Near B	25.22	14.58	267.75	214.2	959.52	161.28	214.2	Φ 8 cc 210mm
	NearA	81.67	71.03	267.75	214.2	196.96	161.28	196.9566	Φ 8 cc 210mm

Axis 1 Story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	61.71	51.11	267.75	214.2	273.72	161.28	214.2	Φ 8 cc 210mm
	Near C	44.84	34.24	267.75	214.2	408.58	161.28	214.2	Φ 8 cc 210mm
B—A	Near B	24.066	13.53	267.75	214.2	1033.99	161.28	214.2	Φ 8 cc 210mm
	NearA	82.834	72.19	267.75	214.2	193.79	161.28	193.7918	Φ 8 cc 210mm

Axis 1 Story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	60.297	49.69	267.75	214.2	281.54	161.28	214.2	Φ 8 cc 210mm
	Near C	46.257	35.65	267.75	214.2	392.42	161.28	214.2	Φ 8 cc 210mm
B—A	Near B	26.48	15.69	267.75	214.2	891.64	161.28	214.2	Φ 8 cc 210mm
	NearA	81.93	71.14	267.75	214.2	196.65	161.28	196.6521	Φ 8 cc 210mm

Axis 1 Story 1

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	57.632	48.03	267.75	214.2	291.27	161.28	214.2	Φ 8 cc 210mm
	Near C	48.92	38.32	267.75	214.2	365.08	161.28	214.2	Φ 8 cc 210mm
B—A	Near B	53.75	43.04	267.75	214.2	325.04	161.28	214.2	Φ 8 cc 210mm
	NearA	53.84	43.13	267.75	214.2	324.36	161.28	214.2	Φ 8 cc 210mm

Axis 1 Ground

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	50.97	40.33	267.75	214.2	346.88	188.16	214.2	Φ 8 cc 210mm
	Near C	55.99	45.35	267.75	214.2	308.49	188.16	214.2	Φ 8 cc 210mm
B—A	Near B	39.77	32.55	267.75	214.2	429.80	188.16	214.2	Φ 8 cc 210mm
	NearA	32.8	25.58	267.75	214.2	546.91	188.16	214.2	Φ 8 cc 210mm

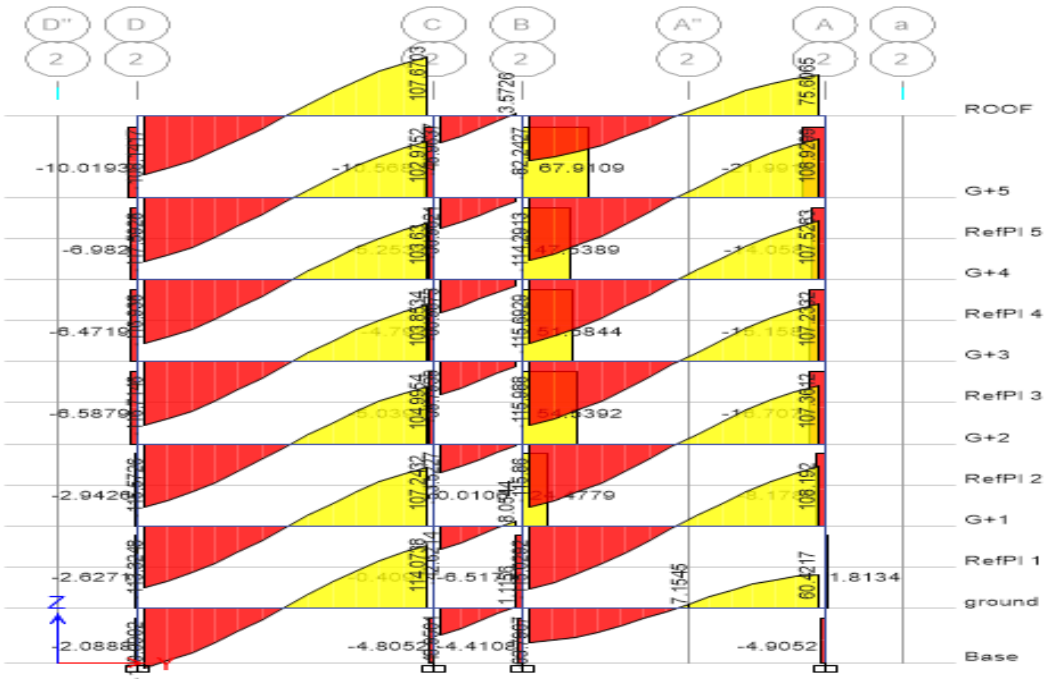


Figure 7-11 shear force diagram of beam axis 2

Axis 2 roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	108.14	88.43	230.25	184.2	136.05	225.80	136.0451	Φ 8 cc 210mm
	Near C	107.67	87.96	230.25	184.2	136.77	225.80	136.7721	Φ 8 cc 210mm
B—A	Near B	82.24	68.37	230.25	184.2	175.96	225.80	175.9612	Φ 8 cc 210mm
	Near A	75.61	61.14	230.25	184.2	196.77	225.80	184.2	Φ 8 cc 210mm

Axis 2 story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	117.58	95.63	267.75	214.2	146.29	161.28	146.2912	Φ 8 cc 210mm
	Near C	102.98	81.03	267.75	214.2	172.65	161.28	172.65	Φ 8 cc 210mm
B—A	Near B	114.29	92.08	267.75	214.2	151.93	161.28	151.9313	Φ 8 cc 210mm
	Near A	108.92	86.71	267.75	214.2	161.34	161.28	161.3404	Φ 8 cc 210mm

Axis 2 story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	116.99	95.04	267.75	214.2	147.20	161.28	147.1994	Φ 8 cc 210mm
	Near C	103.63	81.68	267.75	214.2	171.28	161.28	171.2761	Φ 8 cc 210mm
B—A	Near B	115.69	93.48	267.75	214.2	149.66	161.28	149.6559	Φ 8 cc 210mm
	Near A	107.53	85.32	267.75	214.2	163.97	161.28	163.9689	Φ 8 cc 210mm

Axis 2 story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	116.71	94.76	267.75	214.2	147.63	161.28	147.6343	Φ 8 cc 210mm
	Near C	103.85	81.9	267.75	214.2	170.82	161.28	170.816	Φ 8 cc 210mm
B—A	Near B	115.98	93.78	267.75	214.2	149.18	161.28	149.1771	Φ 8 cc 210mm
	Near A	107.23	85.02	267.75	214.2	164.55	161.28	164.5475	Φ 8 cc 210mm

Axis 2 story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	116.57	94.52	267.75	214.2	148.01	161.28	148.0092	Φ 8 cc 210mm
	Near C	104.99	82.94	267.75	214.2	168.67	161.28	168.6741	Φ 8 cc 210mm
B—A	Near B	115.86	93.65	267.75	214.2	149.38	161.28	149.3842	Φ 8 cc 210mm
	Near A	107.36	85.15	267.75	214.2	164.30	161.28	164.2963	Φ 8 cc 210mm

Axis 2 story 1

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	113.32	91.37	267.75	214.2	153.11	161.28	153.1119	Φ 8 cc 210mm
	Near C	107.24	85.29	267.75	214.2	164.03	161.28	164.0266	Φ 8 cc 210mm
B—A	Near B	110.282	88.54	267.75	214.2	158.01	161.28	158.0058	Φ 8 cc 210mm
	Near A	108.19	86.45	267.75	214.2	161.83	161.28	161.8257	Φ 8 cc 210mm

Axis 2 Ground

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	82.35	62.8	267.75	214.2	222.77	188.16	214.2	Φ 8 cc 210mm
	Near C	114.07	94.52	267.75	214.2	148.01	161.28	148.0092	Φ 8 cc 210mm
B—A	Near B	69.78	56.82	267.75	214.2	246.21	161.28	214.2	Φ 8 cc 210mm
	Near A	60.42	47.46	267.75	214.2	294.77	161.28	214.2	Φ 8 cc 210mm

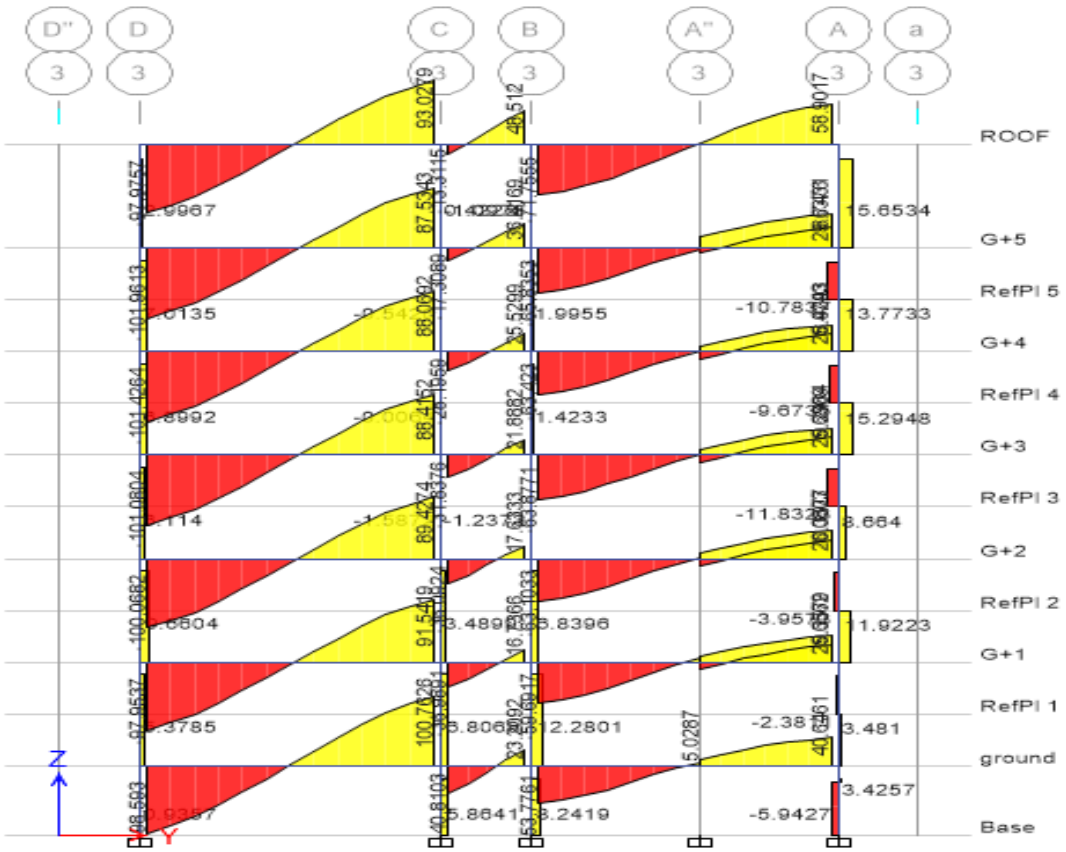


Figure 7-12 shear force diagram of beam axis 3

Axis 3 Roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	97.975	80.53	230.25	184.2	149.39	225.80	149.3912	Φ 8 cc 210mm
	Near C	93.02	75.58	230.25	184.2	159.18	225.80	159.1753	Φ 8 cc 210mm
C-B	Near C	15.31	2.61	267.75	214.2	5360.09	161.28	214.2	Φ 8 cc 210mm
	Near B	48.51	35.81	267.75	214.2	390.67	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	71.755	59.83	230.25	184.2	201.08	225.80	184.2	Φ 8 cc 210mm
	Near A	58.9	46.97	230.25	184.2	256.13	225.80	184.2	Φ 8 cc 210mm

Axis 3 story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	101.9	83.1	267.75	214.2	168.35	161.28	149.3912	Φ 8 cc 210mm
	Near C	87.53	68.67	267.75	214.2	203.73	161.28	159.1753	Φ 8 cc 210mm
C-B	Near C	17.31	5.57	267.75	214.2	2511.64	161.28	214.2	Φ 8 cc 210mm
	Near B	36.81	25.08	267.75	214.2	557.81	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	65.83	56.43	267.75	214.2	247.91	161.28	184.2	Φ 8 cc 210mm
	Near A	28.63	19.23	267.75	214.2	727.50	161.28	184.2	Φ 8 cc 210mm

Axis 3 story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	101.4	82.57	267.75	214.2	169.43	161.28	169.4299	Φ 8 cc 210mm
	Near C	88.06	69.21	267.75	214.2	202.14	161.28	202.136	Φ 8 cc 210mm
C-B	Near C	20.19	10.28	267.75	214.2	1360.88	161.28	214.2	Φ 8 cc 210mm
	Near B	25.53	15.62	267.75	214.2	895.64	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	63.42	54.23	267.75	214.2	257.97	161.28	214.2	Φ 8 cc 210mm
	Near A	28.92	19.73	267.75	214.2	709.06	161.28	214.2	Φ 8 cc 210mm

Axis 3 story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	101.08	82.22	267.75	214.2	170.15	161.28	170.1512	Φ 8 cc 210mm
	Near C	88.42	69.56	267.75	214.2	201.12	161.28	201.1189	Φ 8 cc 210mm
C-B	Near C	31.83	20.18	267.75	214.2	693.25	161.28	214.2	Φ 8 cc 210mm
	Near B	21.88	10.24	267.75	214.2	1366.19	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	63.87	54.56	267.75	214.2	256.41	161.28	214.2	Φ 8 cc 210mm
	Near A	29.69	20.38	267.75	214.2	686.45	161.28	214.2	Φ 8 cc 210mm

Axis 3 story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	109.06	89.31	267.75	214.2	156.64	161.28	156.6435	Φ 8 cc 210mm
	Near C	89.42	69.67	267.75	214.2	200.80	161.28	200.8014	Φ 8 cc 210mm
C-B	Near C	31.09	20.53	267.75	214.2	681.43	161.28	214.2	Φ 8 cc 210mm
	Near B	17.63	7.07	267.75	214.2	1978.76	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	63.1	54.23	267.75	214.2	257.97	161.28	214.2	Φ 8 cc 210mm
	Near A	26.05	17.18	267.75	214.2	814.31	161.28	214.2	Φ 8 cc 210mm

Axis 3 story 1

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	97.95	79.09	267.75	214.2	176.88	161.28	176.8849	Φ 8 cc 210mm
	Near C	91.54	72.68	267.75	214.2	192.49	161.28	192.4853	Φ 8 cc 210mm
C-B	Near C	30.9	20.68	267.75	214.2	676.49	161.28	214.2	Φ 8 cc 210mm
	Near B	16.25	6.03	267.75	214.2	2320.04	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	59.69	50.79	267.75	214.2	275.44	161.28	214.2	Φ 8 cc 210mm
	Near A	29.65	20.76	267.75	214.2	673.88	161.28	214.2	Φ 8 cc 210mm

Axis 3 ground

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	98.59	78.76	267.75	214.2	177.63	188.16	177.6261	Φ 8 cc 210mm
	Near C	100.7	80.87	267.75	214.2	172.99	188.16	172.9916	Φ 8 cc 210mm
C-B	Near C	40.81	26.93	267.75	214.2	519.49	188.16	214.2	Φ 8 cc 210mm
	Near B	23.2	9.32	267.75	214.2	1501.05	188.16	214.2	Φ 8 cc 210mm
B-A	Near B	53.77	44.37	267.75	214.2	315.30	188.16	214.2	Φ 8 cc 210mm
	Near A	40.65	31.25	267.75	214.2	447.67	188.16	214.2	Φ 8 cc 210mm

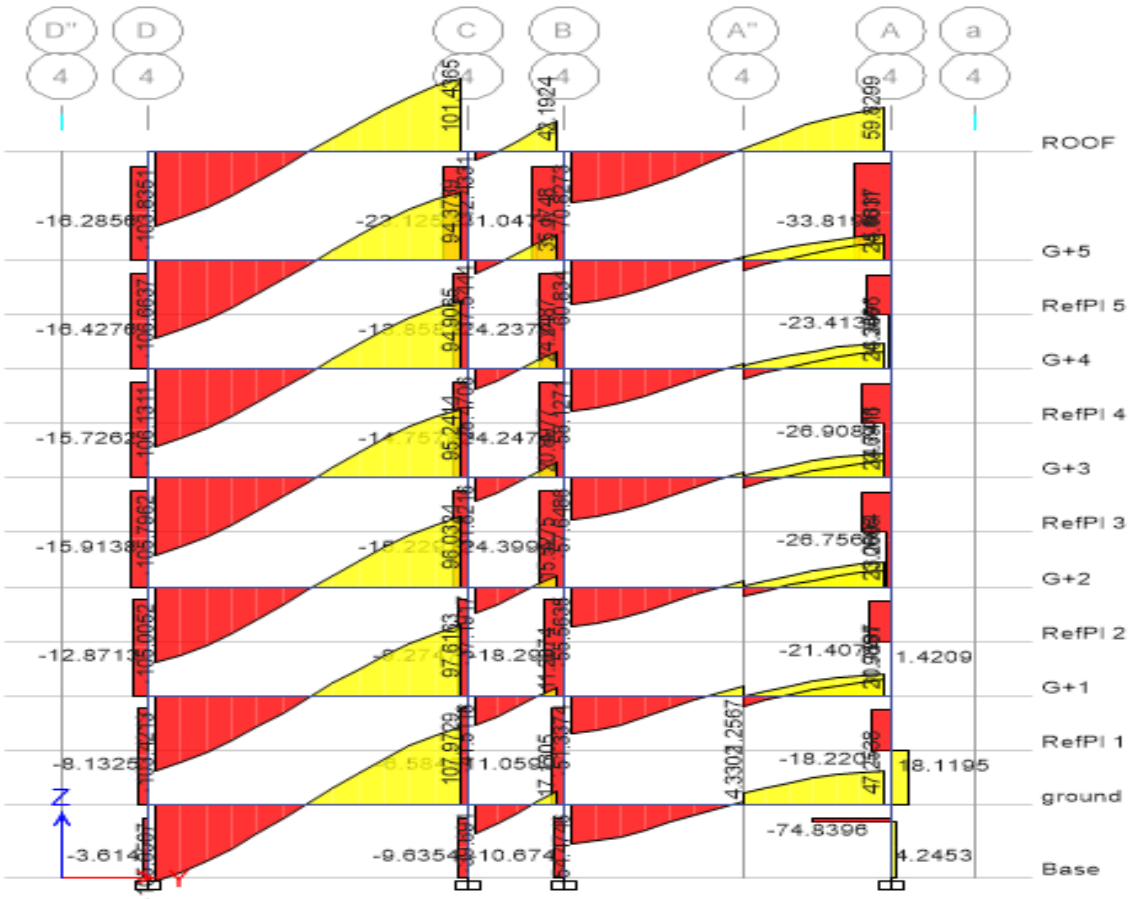


Figure 7-13 shear force diagram of beam axis 4

Axis 4 roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	103.84	85.09	230.25	184.2	141.39	225.80	141.3852	Φ 8 cc 210mm
	Near C	101.44	82.69	230.25	184.2	145.49	225.80	145.4888	Φ 8 cc 210mm
C-B	Near C	32.13	17.34	230.25	184.2	693.80	225.80	184.2	Φ 8 cc 210mm
	Near B	42.19	27.41	230.25	184.2	438.91	225.80	184.2	Φ 8 cc 210mm
B-A	Near B	70.83	58.89	230.25	184.2	204.29	225.80	184.2	Φ 8 cc 210mm
	Near A	59.83	47.89	230.25	184.2	251.21	225.80	184.2	Φ 8 cc 210mm

Axis 4 story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	106.66	86.65	267.75	214.2	161.45	161.28	161.4522	Φ 8 cc 210mm
	Near C	94.38	74.37	267.75	214.2	188.11	161.28	188.1112	Φ 8 cc 210mm
C-B	Near C	31.54	16.68	267.75	214.2	838.72	161.28	214.2	Φ 8 cc 210mm
	Near B	36.97	22.11	267.75	214.2	632.74	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	60.83	52.24	267.75	214.2	267.80	161.28	214.2	Φ 8 cc 210mm
	Near A	25.53	16.94	267.75	214.2	825.85	161.28	214.2	Φ 8 cc 210mm

Axis 4 story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	106.13	86.13	267.75	214.2	162.43	161.28	162.4269	Φ 8 cc 210mm
	Near C	94.9	74.89	267.75	214.2	186.81	161.28	186.8051	Φ 8 cc 210mm
C-B	Near C	20.47	10.78	267.75	214.2	1297.76	161.28	214.2	Φ 8 cc 210mm
	Near B	24.24	14.55	267.75	214.2	961.50	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	56.12	48.19	267.75	214.2	290.31	161.28	214.2	Φ 8 cc 210mm
	Near A	24.26	16.25	267.75	214.2	860.91	161.28	214.2	Φ 8 cc 210mm

Axis 4 story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	105.79	70.11	267.75	214.2	199.54	161.28	199.5412	Φ 8 cc 210mm
	Near C	95.24	58.87	267.75	214.2	237.64	161.28	214.2	Φ 8 cc 210mm
C-B	Near C	31.62	20.26	267.75	214.2	690.51	161.28	214.2	Φ 8 cc 210mm
	Near B	20.8	9.44	267.75	214.2	1481.97	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	57.65	49.66	267.75	214.2	281.71	161.28	214.2	Φ 8 cc 210mm
	Near A	22.68	14.68	267.75	214.2	952.99	161.28	214.2	Φ 8 cc 210mm

Axis 4 story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	105.01	85	267.75	214.2	164.59	161.28	164.5862	Φ 8 cc 210mm
	Near C	96.03	76.03	267.75	214.2	184.00	161.28	184.0041	Φ 8 cc 210mm
C-B	Near C	31.19	21.06	267.75	214.2	664.28	161.28	214.2	Φ 8 cc 210mm
	Near B	15.53	5.4	267.75	214.2	2590.71	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	65.66	56.83	267.75	214.2	246.17	161.28	214.2	Φ 8 cc 210mm
	Near A	23.06	14.23	267.75	214.2	983.12	161.28	214.2	Φ 8 cc 210mm

Axis 4 story 1

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	103.42	83.42	267.75	214.2	167.70	161.28	167.7036	Φ 8 cc 210mm
	Near C	97.6	77.59	267.75	214.2	180.30	161.28	180.3046	Φ 8 cc 210mm
C-B	Near C	41.07	29.71	267.75	214.2	470.88	161.28	214.2	Φ 8 cc 210mm
	Near B	11.2	3.5	267.75	214.2	3997.09	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	55.66	48.03	267.75	214.2	291.27	161.28	214.2	Φ 8 cc 210mm
	Near A	20.96	13.33	267.75	214.2	1049.50	161.28	214.2	Φ 8 cc 210mm

Axis 4 Ground

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	95.43	75.19	267.75	214.2	186.06	188.16	186.0597	Φ 8 cc 210mm
	Near C	107.97	87.73	267.75	214.2	159.46	188.16	159.4646	Φ 8 cc 210mm
C-B	Near C	39.69	27.34	267.75	214.2	511.70	188.16	214.2	Φ 8 cc 210mm
	Near B	17.28	4.95	267.75	214.2	2826.23	188.16	214.2	Φ 8 cc 210mm
B-A	Near B	54.47	44.35	267.75	214.2	315.44	188.16	214.2	Φ 8 cc 210mm
	Near A	47.25	37.13	267.75	214.2	376.78	188.16	214.2	Φ 8 cc 210mm

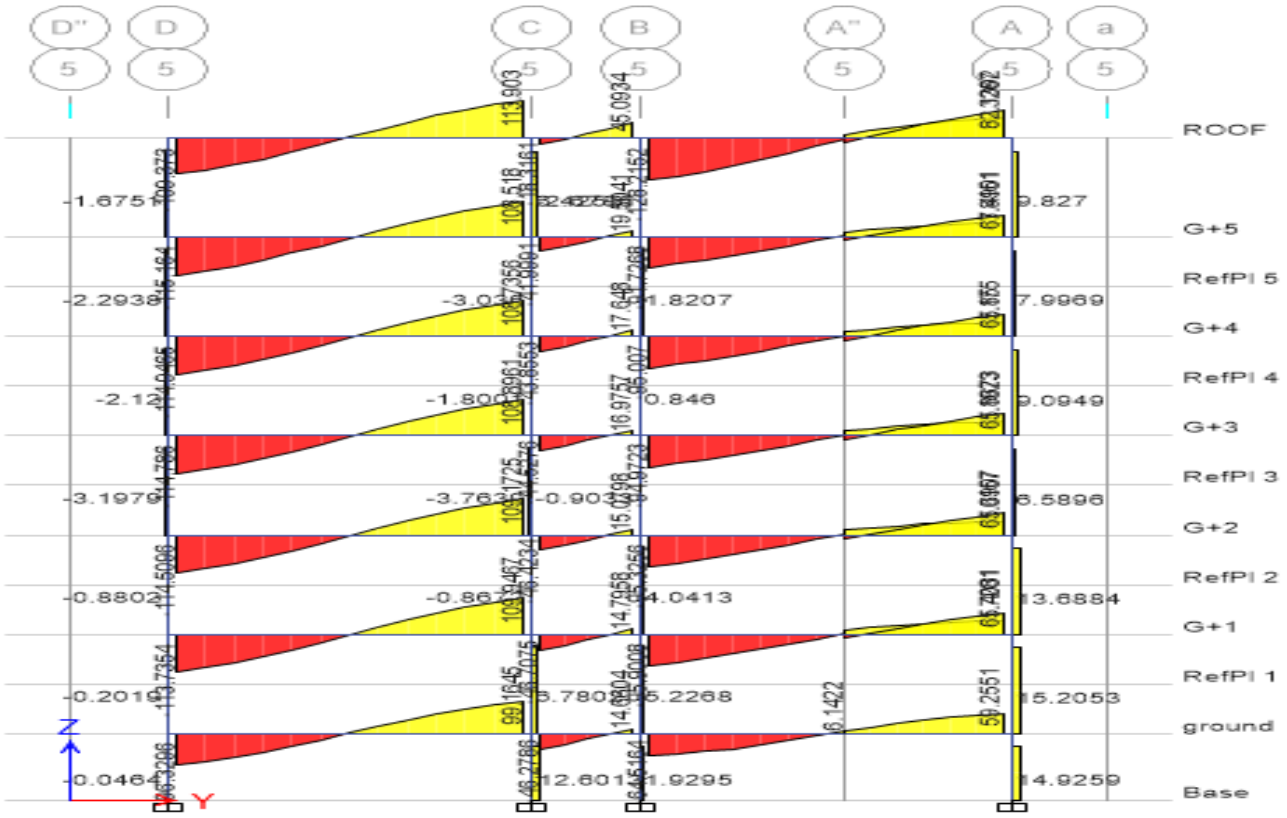


Figure 7-14 shear force diagram of beam axis 5

Axis 5 Roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	100.373	81.13	230.25	184.2	148.29	225.80	148.2863	Φ 8 cc 210mm
	Near C	113.9	94.3	230.25	184.2	127.58	225.80	127.5766	Φ 8 cc 210mm
C-B	Near C	18.32	5.71	230.25	184.2	2106.91	225.80	184.2	Φ 8 cc 210mm
	Near B	45.09	32.47	230.25	184.2	370.51	225.80	184.2	Φ 8 cc 210mm
B-A	Near B	128.22	108.99	230.25	184.2	110.38	225.80	110.3814	Φ 8 cc 210mm
	Near A	82.33	63.1	230.25	184.2	190.66	225.80	184.2	Φ 8 cc 210mm

Axis 5 story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	115.16	92.9	267.75	214.2	150.59	161.28	150.5902	Φ 8 cc 210mm
	Near C	108.52	86.26	267.75	214.2	162.18	161.28	162.1821	Φ 8 cc 210mm
C-B	Near C	41.99	28.72	267.75	214.2	487.11	161.28	214.2	Φ 8 cc 210mm
	Near B	19.2	5.94	267.75	214.2	2355.19	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	61.72	45.83	267.75	214.2	305.25	161.28	214.2	Φ 8 cc 210mm
	Near A	67.81	54.92	267.75	214.2	254.73	161.28	214.2	Φ 8 cc 210mm

Axis 5 story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	114.04	91.87	267.75	214.2	152.28	161.28	152.2786	Φ 8 cc 210mm
	Near C	108.73	86.56	267.75	214.2	161.62	161.28	161.62	Φ 8 cc 210mm
C-B	Near C	43.85	30.52	267.75	214.2	458.38	161.28	214.2	Φ 8 cc 210mm
	Near B	17.65	4.32	267.75	214.2	3238.39	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	95	78.99	267.75	214.2	177.11	161.28	177.1089	Φ 8 cc 210mm
	Near A	65.855	49.85	267.75	214.2	280.64	161.28	214.2	Φ 8 cc 210mm

Axis 5 story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	114.78	92.52	267.75	214.2	151.21	161.28	151.2087	Φ 8 cc 210mm
	Near C	108.89	86.63	267.75	214.2	161.49	161.28	161.4894	Φ 8 cc 210mm
C-B	Near C	41.02	28.44	267.75	214.2	491.91	161.28	214.2	Φ 8 cc 210mm
	Near B	16.97	4.39	267.75	214.2	3186.75	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	94.97	78.9	267.75	214.2	177.31	161.28	177.3109	Φ 8 cc 210mm
	Near A	65.88	49.87	267.75	214.2	280.53	161.28	214.2	Φ 8 cc 210mm

Axis 5 story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	114.51	92.65	267.75	214.2	151.00	161.28	150.9966	Φ 8 cc 210mm
	Near C	105.17	83.31	267.75	214.2	167.92	161.28	167.925	Φ 8 cc 210mm
C-B	Near C	46.42	33.09	267.75	214.2	422.78	161.28	214.2	Φ 8 cc 210mm
	Near B	15.03	1.71	267.75	214.2	8181.19	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	95.32	79.29	267.75	214.2	176.44	161.28	176.4388	Φ 8 cc 210mm
	Near A	65.69	49.67	267.75	214.2	281.66	161.28	214.2	Φ 8 cc 210mm

Axis 5 story 1

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	113.74	91.48	267.75	214.2	152.93	161.28	152.9277	Φ 8 cc 210mm
	Near C	109.94	87.68	267.75	214.2	159.56	161.28	159.5555	Φ 8 cc 210mm
C-B	Near C	43.7	31.02	267.75	214.2	450.99	161.28	214.2	Φ 8 cc 210mm
	Near B	14.79	2.11	267.75	214.2	6630.25	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	95.01	79.05	267.75	214.2	176.97	161.28	176.9745	Φ 8 cc 210mm
	Near A	65.41	49.45	267.75	214.2	282.91	161.28	214.2	Φ 8 cc 210mm

Axis 5 Ground

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	96.32	76.86	267.75	214.2	182.02	188.16	182.017	Φ 8 cc 210mm
	Near C	99.16	79.71	267.75	214.2	175.51	188.16	175.5091	Φ 8 cc 210mm
C-B	Near C	46.27	33.06	267.75	214.2	423.16	188.16	214.2	Φ 8 cc 210mm
	Near B	14.68	1.47	267.75	214.2	9516.89	188.16	214.2	Φ 8 cc 210mm
B-A	Near B	64.5	52.19	267.75	214.2	268.06	188.16	214.2	Φ 8 cc 210mm
	Near A	59.25	46.94	267.75	214.2	298.04	188.16	214.2	Φ 8 cc 210mm

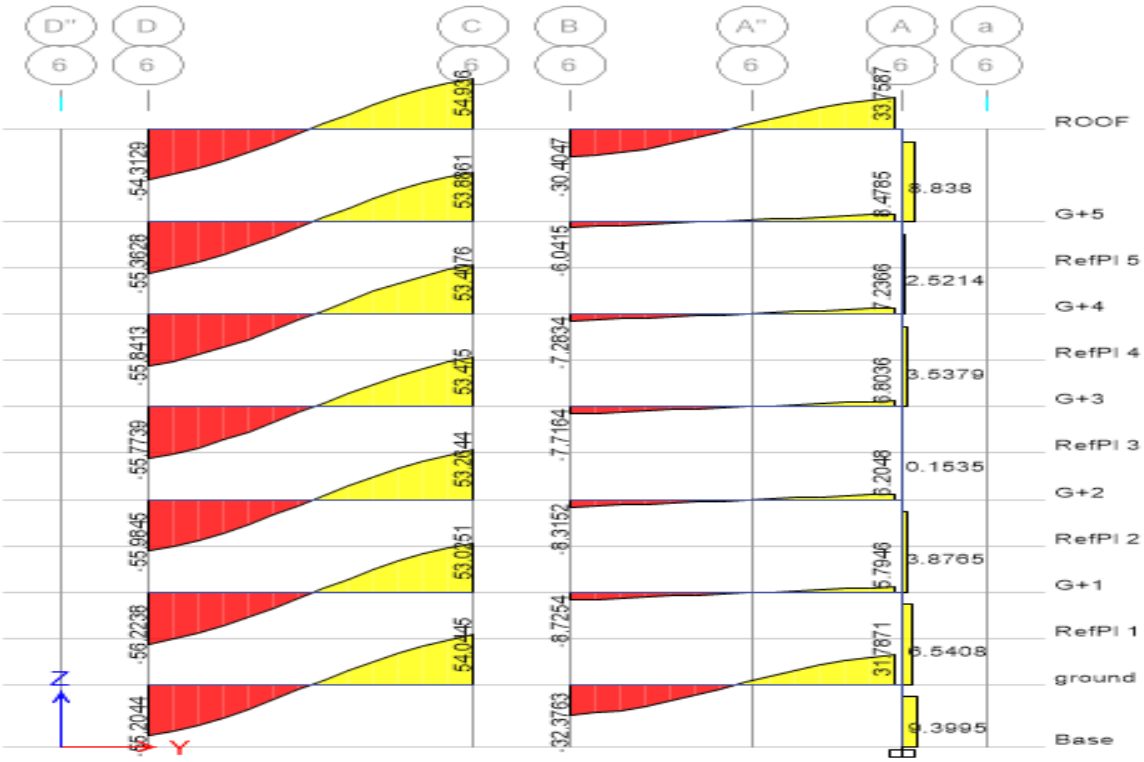


Figure 7-15 shear force diagram of beam axis 6

Axis 6 roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	54.31	44.33	230.25	184.2	271.38	225.80	184.2	Φ 8 cc 210mm
	Near C	54.94	44.96	230.25	184.2	267.58	225.80	184.2	Φ 8 cc 210mm
B-A	Near B	30.4	24.54	230.25	184.2	490.24	225.80	184.2	Φ 8 cc 210mm
	Near A	33.75	27.89	230.25	184.2	431.35	225.80	184.2	Φ 8 cc 210mm

Axis 6 Story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	55.36	44.49	267.75	214.2	314.45	161.28	214.2	Φ 8 cc 210mm
	Near C	53.88	43.01	267.75	214.2	325.27	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	6.04	4.59	267.75	214.2	3047.89	161.28	214.2	Φ 8 cc 210mm
	Near A	8.4785	7.03	267.75	214.2	1990.02	161.28	214.2	Φ 8 cc 210mm

Axis 6 Story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	55.84	44.49	267.75	214.2	314.45	161.28	214.2	Φ 8 cc 210mm
	Near C	53.48	42.6	267.75	214.2	328.40	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	7.28	5.84	267.75	214.2	2395.52	161.28	214.2	Φ 8 cc 210mm
	Near A	7.24	5.79	267.75	214.2	2416.21	161.28	214.2	Φ 8 cc 210mm

Axis 6 Story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	55.77	44.89	267.75	214.2	311.65	161.28	214.2	Φ 8 cc 210mm
	Near C	53.47	42.59	267.75	214.2	328.48	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	7.71	6.26	267.75	214.2	2234.80	161.28	214.2	Φ 8 cc 210mm
	Near A	6.81	5.36	267.75	214.2	2610.04	161.28	214.2	Φ 8 cc 210mm

Axis 6 Story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	55.98	45.11	267.75	214.2	310.13	161.28	214.2	Φ 8 cc 210mm
	Near C	53.26	42.39	267.75	214.2	330.03	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	8.32	6.88	267.75	214.2	2033.41	161.28	214.2	Φ 8 cc 210mm
	Near A	6.2	4.76	267.75	214.2	2939.04	161.28	214.2	Φ 8 cc 210mm

Axis 6 Story 1

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	56.22	45.35	267.75	214.2	308.49	161.28	214.2	Φ 8 cc 210mm
	Near C	53.01	42.14	267.75	214.2	331.98	161.28	214.2	Φ 8 cc 210mm
B-A	Near B	8.72	7.18	267.75	214.2	1948.44	161.28	214.2	Φ 8 cc 210mm
	Near A	6.79	5.25	267.75	214.2	2664.73	161.28	214.2	Φ 8 cc 210mm

Axis 6 Ground

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
D-C	Near D	55.204	44.33	267.75	214.2	315.58	188.16	214.2	Φ 8 cc 210mm
	Near C	54.04	43.17	267.75	214.2	324.06	188.16	214.2	Φ 8 cc 210mm
B-A	Near B	32.38	25.99	267.75	214.2	538.28	188.16	214.2	Φ 8 cc 210mm
	Near A	31.78	25.39	267.75	214.2	551.00	188.16	214.2	Φ 8 cc 210mm

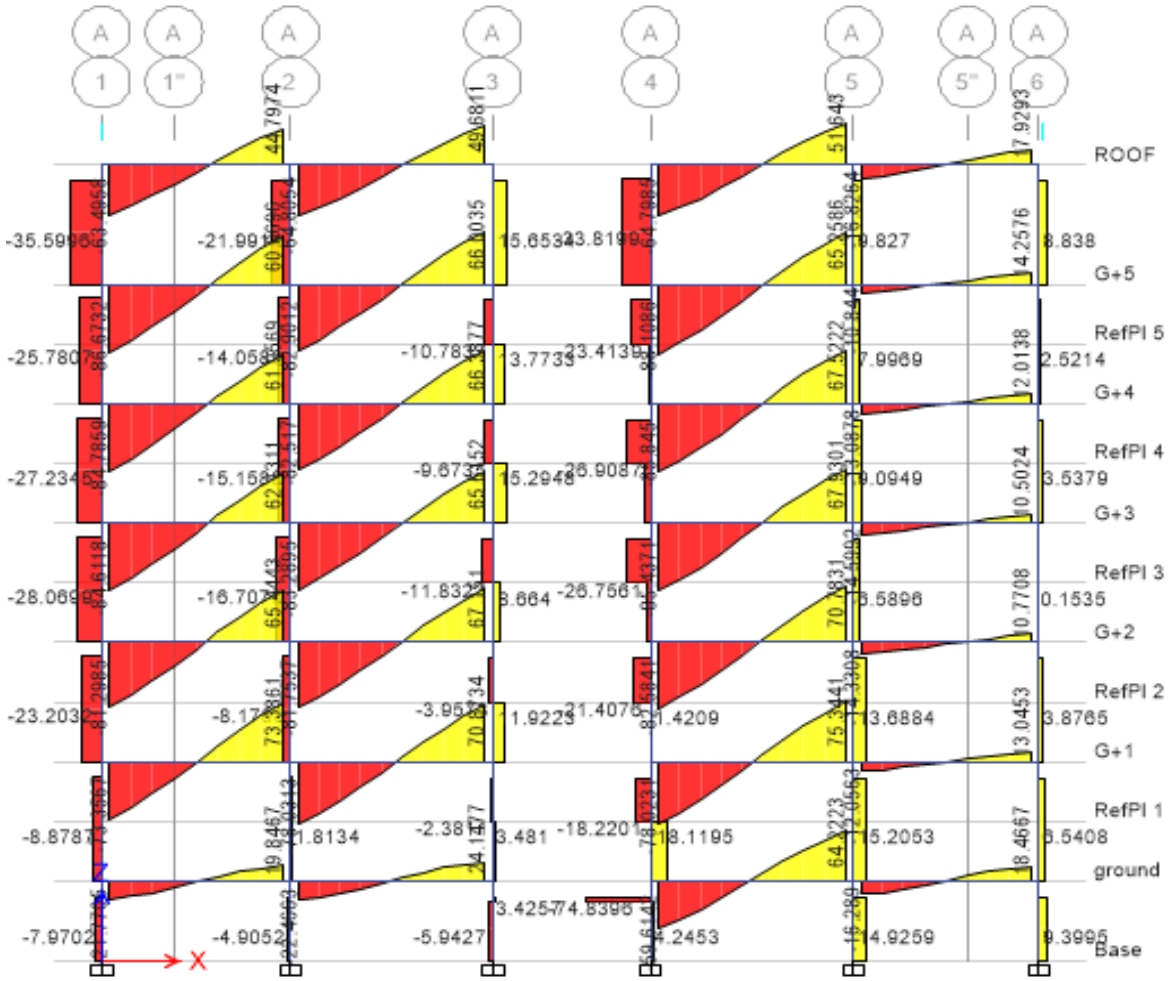


Figure 7-16 shear force diagram of beam axis A

Axis A Roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	63.49	49.46	230.25	184.2	243.24	225.80	184.2	Φ 8 cc 210mm
	Near 2	44.79	30.76	230.25	184.2	391.11	225.80	184.2	Φ 8 cc 210mm
2—3	Near 2	64.81	50.92	230.25	184.2	236.26	225.80	184.2	Φ 8 cc 210mm
	Near 3	49.68	35.99	230.25	184.2	334.27	225.80	184.2	Φ 8 cc 210mm
4—5	Near 4	64.7	50.61	230.25	184.2	237.71	225.80	184.2	Φ 8 cc 210mm
	Near 5	51.64	37.55	230.25	184.2	320.39	225.80	184.2	Φ 8 cc 210mm
5—6	Near 5	9.83	6.28	230.25	184.2	1915.68	225.80	184.2	Φ 8 cc 210mm
	Near 6	17.93	14.3	230.25	184.2	841.29	225.80	184.2	Φ 8 cc 210mm

Axis A Story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	86.67	65.87	267.75	214.2	212.39	225.80	212.3855	Φ 8 cc 210mm
	Near 2	60.7	39.89	267.75	214.2	350.71	225.80	214.2	Φ 8 cc 210mm
2—3	Near 2	82.9	63.21	267.75	214.2	221.32	225.80	214.2	Φ 8 cc 210mm
	Near 3	66.03	46.61	267.75	214.2	300.15	225.80	214.2	Φ 8 cc 210mm
4—5	Near 4	88.1	67.86	267.75	214.2	206.16	225.80	206.1572	Φ 8 cc 210mm
	Near 5	65.25	45.01	267.75	214.2	310.82	225.80	214.2	Φ 8 cc 210mm
5—6	Near 5	7.99	4.85	267.75	214.2	2884.50	225.80	214.2	Φ 8 cc 210mm
	Near 6	14.25	11.11	267.75	214.2	1259.21	225.80	214.2	Φ 8 cc 210mm

Axis A Story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	84.78	64.16	267.75	214.2	218.05	225.80	214.2	Φ 8 cc 210mm
	Near 2	61.26	40.64	267.75	214.2	344.24	225.80	214.2	Φ 8 cc 210mm
2—3	Near 2	82.51	62.86	267.75	214.2	222.56	225.80	214.2	Φ 8 cc 210mm
	Near 3	66.37	46.72	267.75	214.2	299.44	225.80	214.2	Φ 8 cc 210mm
4—5	Near 4	88.84	68.21	267.75	214.2	205.10	225.80	205.0994	Φ 8 cc 210mm
	Near 5	67.53	46.89	267.75	214.2	298.35	225.80	214.2	Φ 8 cc 210mm
5—6	Near 5	9.09	6.11	267.75	214.2	2289.66	225.80	214.2	Φ 8 cc 210mm
	Near 6	12.01	9.03	267.75	214.2	1549.26	225.80	214.2	Φ 8 cc 210mm

Axis A Story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	84.61	63.88	267.75	214.2	219.00	225.80	214.2	Φ 8 cc 210mm
	Near 2	62.23	41.5	267.75	214.2	337.10	225.80	214.2	Φ 8 cc 210mm
2—3	Near 2	83.26	63.62	267.75	214.2	219.90	225.80	214.2	Φ 8 cc 210mm
	Near 3	65.63	45.98	267.75	214.2	304.26	225.80	214.2	Φ 8 cc 210mm
4—5	Near 4	85.13	64.93	267.75	214.2	215.46	225.80	214.2	Φ 8 cc 210mm
	Near 5	67.93	47.73	267.75	214.2	293.10	225.80	214.2	Φ 8 cc 210mm
5—6	Near 5	8.59	5.83	267.75	214.2	2399.63	225.80	214.2	Φ 8 cc 210mm
	Near 6	10.5	8.19	267.75	214.2	1708.16	225.80	214.2	Φ 8 cc 210mm

Axis A Story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	87.29	65.73	267.75	214.2	212.84	225.80	212.8378	Φ 8 cc 210mm
	Near 2	65.44	43.88	267.75	214.2	318.82	225.80	214.2	Φ 8 cc 210mm
2—3	Near 2	81.75	62.1	267.75	214.2	225.28	225.80	214.2	Φ 8 cc 210mm
	Near 3	67.15	47.5	267.75	214.2	294.52	225.80	214.2	Φ 8 cc 210mm
4—5	Near 4	82.58	62.34	267.75	214.2	224.41	225.80	214.2	Φ 8 cc 210mm
	Near 5	70.78	50.54	267.75	214.2	276.81	225.80	214.2	Φ 8 cc 210mm
5—6	Near 5	13.68	10.23	267.75	214.2	1367.53	225.80	214.2	Φ 8 cc 210mm
	Near 6	10.77	7.31	267.75	214.2	1913.79	225.80	214.2	Φ 8 cc 210mm

Axis A Story 1

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	73.36	52.65	267.75	214.2	265.71	225.80	214.2	Φ 8 cc 210mm
	Near 2	73.38	52.67	267.75	214.2	265.61	225.80	214.2	Φ 8 cc 210mm
2—3	Near 2	78.03	58.39	267.75	214.2	239.59	225.80	214.2	Φ 8 cc 210mm
	Near 3	70.83	51.19	267.75	214.2	273.29	225.80	214.2	Φ 8 cc 210mm
4—5	Near 4	78.02	57.78	267.75	214.2	242.12	225.80	214.2	Φ 8 cc 210mm
	Near 5	75.34	55.1	267.75	214.2	253.90	225.80	214.2	Φ 8 cc 210mm
5—6	Near 5	15.21	11.1	267.75	214.2	1260.35	225.80	214.2	Φ 8 cc 210mm
	Near 6	13.9	9.79	267.75	214.2	1428.99	225.80	214.2	Φ 8 cc 210mm

Axis A Ground

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	21.56	15.72	267.75	214.2	889.94	188.16	214.2	Φ 8 cc 210mm
	Near 2	19.84	13.99	267.75	214.2	999.99	188.16	214.2	Φ 8 cc 210mm
2—3	Near 2	22.4	16.26	267.75	214.2	860.38	188.16	214.2	Φ 8 cc 210mm
	Near 3	24.14	17.99	267.75	214.2	777.64	188.16	214.2	Φ 8 cc 210mm
4—5	Near 4	59.61	43.27	267.75	214.2	323.31	188.16	214.2	Φ 8 cc 210mm
	Near 5	64.22	47.88	267.75	214.2	292.19	188.16	214.2	Φ 8 cc 210mm
5—6	Near 5	14.92	10.21	267.75	214.2	1370.21	188.16	214.2	Φ 8 cc 210mm
	Near 6	18.47	13.76	267.75	214.2	1016.70	188.16	214.2	Φ 8 cc 210mm

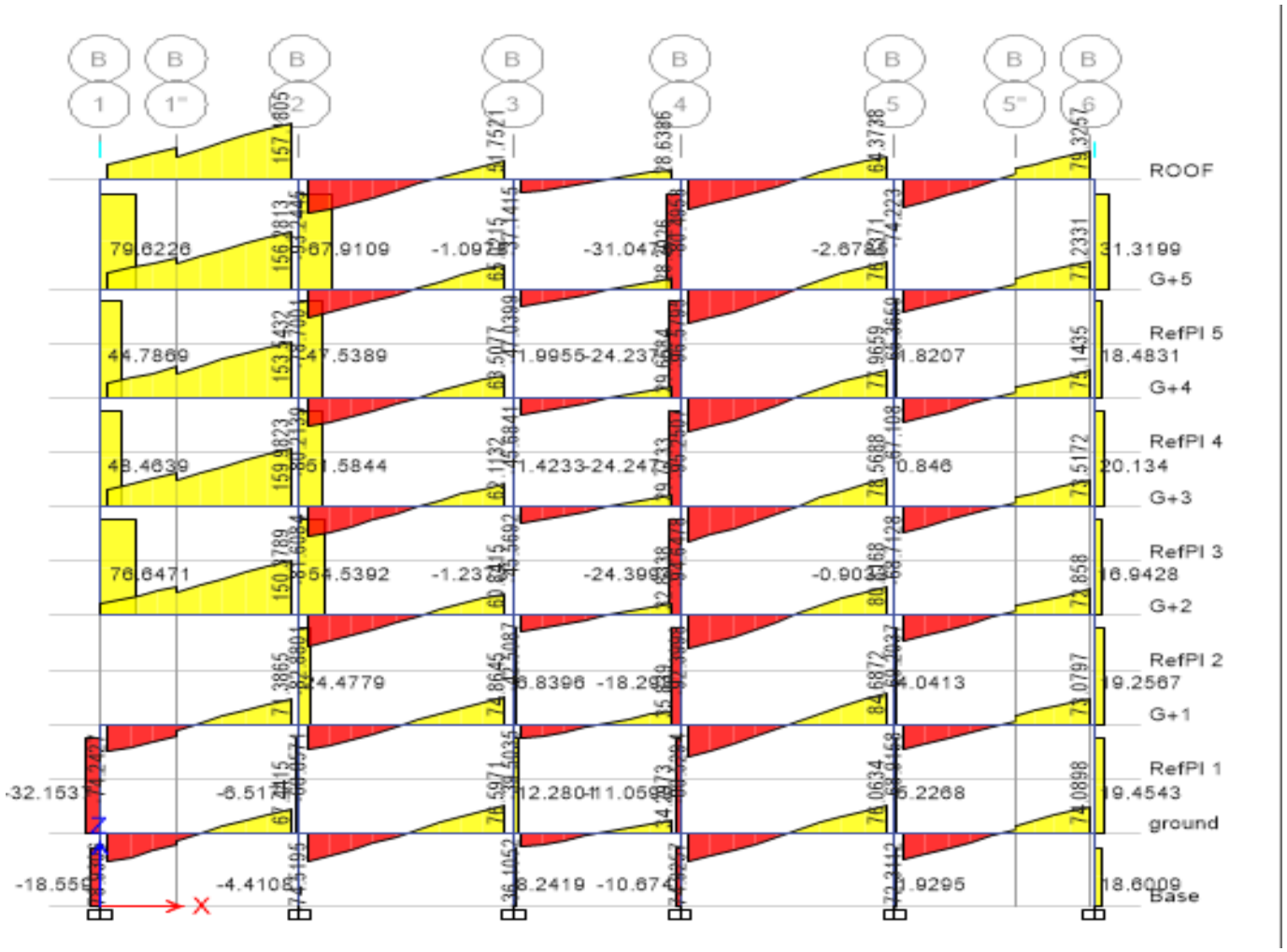


Figure 7-17 shear force diagram of beam axis B

Axis B Roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	93.24	75.68	230.25	184.2	158.96	225.80	158.965	Φ 8 cc 210mm
	Near 2	51.75	34.19	230.25	184.2	351.87	225.80	184.2	Φ 8 cc 210mm
2—3	Near 2	37.14	26.96	230.25	184.2	446.23	225.80	184.2	Φ 8 cc 210mm
	Near 3	28.63	18.45	230.25	184.2	652.06	225.80	184.2	Φ 8 cc 210mm
4—5	Near 4	80.49	62.95	230.25	184.2	191.11	225.80	184.2	Φ 8 cc 210mm
	Near 5	64.38	46.84	230.25	184.2	256.84	225.80	184.2	Φ 8 cc 210mm
5—6	Near 5	74.23	54.35	230.25	184.2	221.35	225.80	184.2	Φ 8 cc 210mm
	Near 6	79.32	59.35	230.25	184.2	202.70	225.80	184.2	Φ 8 cc 210mm

Axis B Story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	78.7	59.74	267.75	214.2	234.18	161.28	214.2	Φ 8 cc 210mm
	Near 2	65.02	40.05	267.75	214.2	349.31	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	47.03	34.35	267.75	214.2	407.27	161.28	214.2	Φ 8 cc 210mm
	Near 3	28.2	15.52	267.75	214.2	901.41	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	96.57	73.79	267.75	214.2	189.59	161.28	189.5898	Φ 8 cc 210mm
	Near 5	76.03	53.25	267.75	214.2	262.72	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	66.36	42.15	267.75	214.2	331.91	161.28	214.2	Φ 8 cc 210mm
	Near 6	77.233	53.02	267.75	214.2	263.86	161.28	214.2	Φ 8 cc 210mm

Axis B Story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
2—3	Near 2	80.21	61.25	267.75	214.2	228.41	161.28	214.2	Φ 8 cc 210mm
	Near 3	63.5	44.54	267.75	214.2	314.10	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	65.68	41.17	267.75	214.2	339.81	161.28	214.2	Φ 8 cc 210mm
	Near 4	79.66	55.15	267.75	214.2	253.67	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	95.25	72.39	267.75	214.2	193.26	161.28	193.2564	Φ 8 cc 210mm
	Near 5	77.96	55.1	267.75	214.2	253.90	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	67.1	47.02	267.75	214.2	297.53	161.28	214.2	Φ 8 cc 210mm
	Near 6	75.14	55.06	267.75	214.2	254.08	161.28	214.2	Φ 8 cc 210mm

Axis B Story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	81.6	62.64	267.75	214.2	223.34	161.28	214.2	Φ 8 cc 210mm
	Near 2	62.1	43.14	267.75	214.2	324.29	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	45.57	24.47	267.75	214.2	571.71	161.28	214.2	Φ 8 cc 210mm
	Near 3	79.56	58.46	267.75	214.2	239.31	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	94.65	71.79	267.75	214.2	194.87	161.28	194.8716	Φ 8 cc 210mm
	Near 5	78.57	55.71	267.75	214.2	251.12	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	68.71	48.63	267.75	214.2	287.68	161.28	214.2	Φ 8 cc 210mm
	Near 6	73.52	53.44	267.75	214.2	261.79	161.28	214.2	Φ 8 cc 210mm

Axis B Story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	82.88	63.96	267.75	214.2	218.73	161.28	214.2	Φ 8 cc 210mm
	Near 2	60.45	41.54	267.75	214.2	336.78	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	42.51	23.07	267.75	214.2	606.41	161.28	214.2	Φ 8 cc 210mm
	Near 3	72.8	53.36	267.75	214.2	262.18	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	92.15	69.38	267.75	214.2	201.64	161.28	201.6407	Φ 8 cc 210mm
	Near 5	80.36	57.59	267.75	214.2	242.92	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	69.2	49.15	267.75	214.2	284.64	161.28	214.2	Φ 8 cc 210mm
	Near 6	18.47	13.76	267.75	214.2	1016.70	188.16	214.2	Φ 8 cc 210mm

Axis B Story 1

1—2	Near 1	74.24	53.68	267.75	214.2	260.62	161.28	214.2	Remark
	Near 2	71.38	50.82	267.75	214.2	275.28	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	80.85	60.3	267.75	214.2	232.00	161.28	214.2	Φ 8 cc 210mm
	Near 3	74.86	54.3	267.75	214.2	257.64	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	39.5	20.13	267.75	214.2	694.97	161.28	214.2	Φ 8 cc 210mm
	Near 4	75.36	55.99	267.75	214.2	249.86	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	89.33	66.38	267.75	214.2	210.75	161.28	210.7537	Φ 8 cc 210mm
	Near 5	84.69	61.7	267.75	214.2	226.74	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	68.91	48.87	267.75	214.2	286.27	161.28	214.2	Φ 8 cc 210mm
	Near 6	73.07	53.03	267.75	214.2	263.81	161.28	214.2	Φ 8 cc 210mm
1—2	Near 1	74.24	53.68	267.75	214.2	260.62	161.28	214.2	Φ 8 cc 210mm

Axis B Ground

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	78.9	58.25	267.75	214.2	240.17	188.16	214.2	Φ 8 cc 210mm
	Near 2	67.4	46.75	267.75	214.2	299.25	188.16	214.2	Φ 8 cc 210mm
2—3	Near 2	74.5	54.57	267.75	214.2	256.36	188.16	214.2	Φ 8 cc 210mm
	Near 3	76.59	56.65	267.75	214.2	246.95	188.16	214.2	Φ 8 cc 210mm
3—4	Near 3	36.1	24.29	267.75	214.2	575.95	188.16	214.2	Φ 8 cc 210mm
	Near 4	34.38	22.48	267.75	214.2	622.32	188.16	214.2	Φ 8 cc 210mm
4—5	Near 4	71.93	52.4	267.75	214.2	266.98	188.16	214.2	Φ 8 cc 210mm
	Near 5	76.06	56.53	267.75	214.2	247.48	188.16	214.2	Φ 8 cc 210mm
5—6	Near 5	72.31	51.64	267.75	214.2	270.91	188.16	214.2	Φ 8 cc 210mm
	Near 6	74.09	53.4	267.75	214.2	261.98	188.16	214.2	Φ 8 cc 210mm

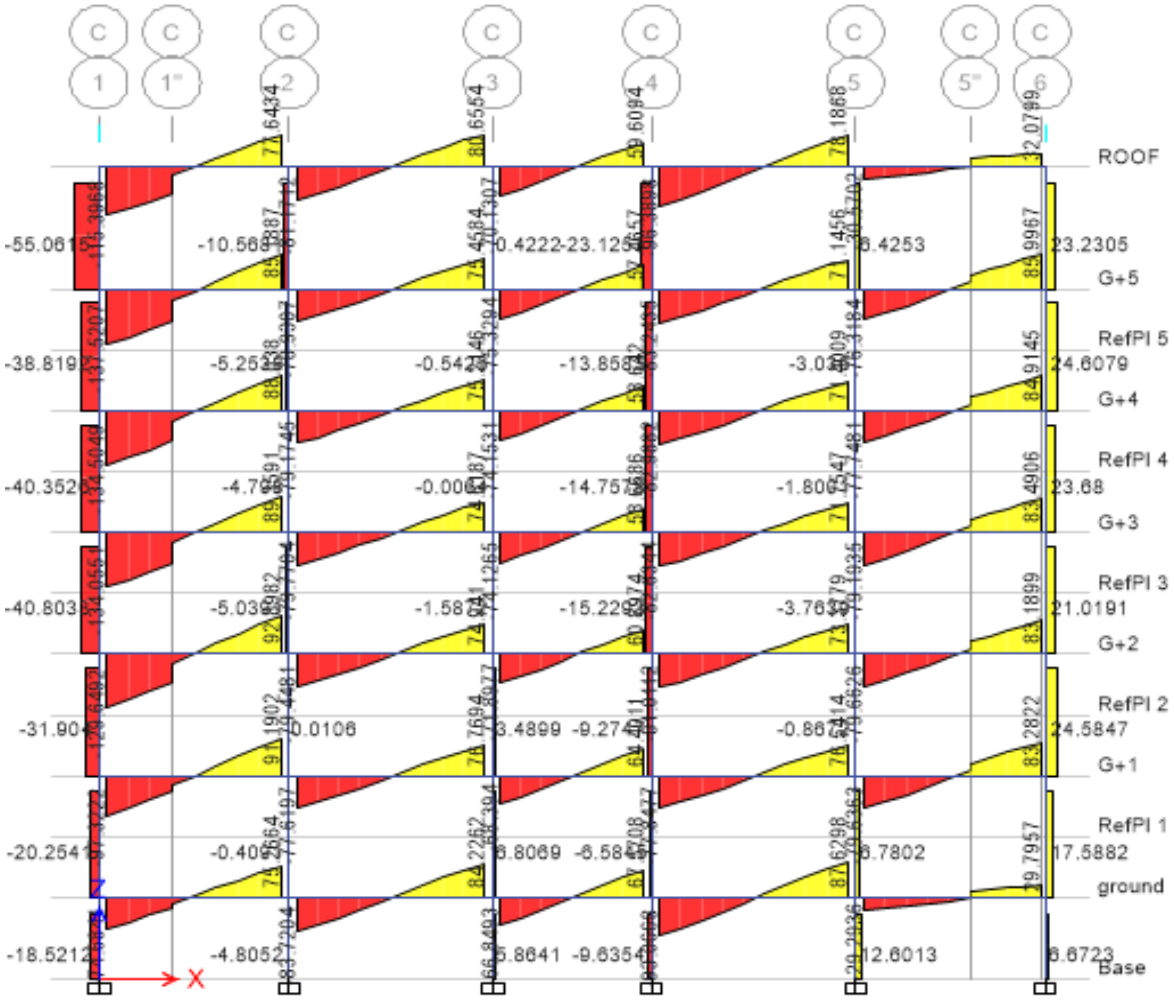


Figure 7-18 shear force diagram of beam axis C

Axis C Roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	115.39	90.39	230.25	184.2	133.10	225.80	133.0951	Φ 8 cc 210mm
	Near 2	77.64	52.64	230.25	184.2	228.54	225.80	184.2	Φ 8 cc 210mm
2—3	Near 2	85.78	65.63	230.25	184.2	183.31	225.80	183.3075	Φ 8 cc 210mm
	Near 3	80.65	60.49	230.25	184.2	198.88	225.80	184.2	Φ 8 cc 210mm
3—4	Near 3	70.13	50.06	230.25	184.2	240.32	225.80	184.2	Φ 8 cc 210mm
	Near 4	59.6	39.53	230.25	184.2	304.34	225.80	184.2	Φ 8 cc 210mm
4—5	Near 4	96.38	75.24	230.25	184.2	159.89	225.80	159.8946	Φ 8 cc 210mm
	Near 5	78.18	57.04	230.25	184.2	210.91	225.80	184.2	Φ 8 cc 210mm
5—6	Near 5	30.57	22.45	230.25	184.2	535.88	225.80	184.2	Φ 8 cc 210mm
	Near 6	32.08	23.96	230.25	184.2	502.11	225.80	184.2	Φ 8 cc 210mm

Axis C Story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	137.52	105.9	267.75	214.2	132.10	161.28	132.1042	Φ 8 cc 210mm
	Near 2	85.98	54.43	267.75	214.2	257.02	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	70.99	51.67	267.75	214.2	270.75	161.28	214.2	Φ 8 cc 210mm
	Near 3	75.45	51.13	267.75	214.2	273.61	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	75.32	52.9	267.75	214.2	264.46	161.28	214.2	Φ 8 cc 210mm
	Near 4	57.48	35.09	267.75	214.2	398.68	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	65.35	47.33	267.75	214.2	295.58	161.28	214.2	Φ 8 cc 210mm
	Near 5	71.15	53.14	267.75	214.2	263.26	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	76.32	52.84	267.75	214.2	264.76	161.28	214.2	Φ 8 cc 210mm
	Near 6	89.99	66.51	267.75	214.2	210.34	161.28	210.3418	Φ 8 cc 210mm

Axis C Story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	134.5	103.08	267.75	214.2	135.72	161.28	135.7182	Φ 8 cc 210mm
	Near 2	88.1	56.68	267.75	214.2	246.82	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	79.17	58.75	267.75	214.2	238.12	161.28	214.2	Φ 8 cc 210mm
	Near 3	75.6	55.18	267.75	214.2	253.53	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	74.15	51.77	267.75	214.2	270.23	161.28	214.2	Φ 8 cc 210mm
	Near 4	58.6	36.22	267.75	214.2	386.25	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	62.8	45.04	267.75	214.2	310.61	161.28	214.2	Φ 8 cc 210mm
	Near 5	71.8	54.04	267.75	214.2	258.88	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	77.75	54.79	267.75	214.2	255.34	161.28	214.2	Φ 8 cc 210mm
	Near 6	84.92	61.95	267.75	214.2	225.82	161.28	214.2	Φ 8 cc 210mm

Axis C Story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	134.05	102.51	267.75	214.2	136.47	161.28	136.4728	Φ 8 cc 210mm
	Near 2	89.36	57.82	267.75	214.2	241.95	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	79.77	59.47	267.75	214.2	235.24	161.28	214.2	Φ 8 cc 210mm
	Near 3	74.08	53.78	267.75	214.2	260.13	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	74.12	51.73	267.75	214.2	270.44	161.28	214.2	Φ 8 cc 210mm
	Near 4	58.66	36.27	267.75	214.2	385.71	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	62.89	45.18	267.75	214.2	309.65	161.28	214.2	Φ 8 cc 210mm
	Near 5	71.35	53.64	267.75	214.2	260.81	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	79.19	56.84	267.75	214.2	246.13	161.28	214.2	Φ 8 cc 210mm
	Near 6	83.49	60.42	267.75	214.2	231.54	161.28	214.2	Φ 8 cc 210mm

Axis C Story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	129.65	98.24	267.75	214.2	142.40	161.28	142.4046	Φ 8 cc 210mm
	Near 2	92.89	61.48	267.75	214.2	227.55	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	79.45	59.18	267.75	214.2	236.39	161.28	214.2	Φ 8 cc 210mm
	Near 3	74.13	53.86	267.75	214.2	259.74	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	71.81	51.12	267.75	214.2	273.67	161.28	214.2	Φ 8 cc 210mm
	Near 4	50.89	30.2	267.75	214.2	463.24	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	61.45	43.6	267.75	214.2	320.87	161.28	214.2	Φ 8 cc 210mm
	Near 5	73.39	55.9	267.75	214.2	250.27	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	79.66	56.69	267.75	214.2	246.78	161.28	214.2	Φ 8 cc 210mm
	Near 6	83.19	60.19	267.75	214.2	232.43	161.28	214.2	Φ 8 cc 210mm

Axis C Story 1

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	91.32	65.46	267.75	214.2	213.72	161.28	213.7157	Φ 8 cc 210mm
	Near 2	91.19	66.04	267.75	214.2	211.84	161.28	211.8387	Φ 8 cc 210mm
2—3	Near 2	77.61	57.31	267.75	214.2	244.11	161.28	214.2	Φ 8 cc 210mm
	Near 3	76.26	55.96	267.75	214.2	250.00	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	68.39	46	267.75	214.2	304.13	161.28	214.2	Φ 8 cc 210mm
	Near 4	64.4	42.01	267.75	214.2	333.01	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	71.85	52.27	267.75	214.2	267.65	161.28	214.2	Φ 8 cc 210mm
	Near 5	76.54	56.95	267.75	214.2	245.65	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	79.63	56.63	267.75	214.2	247.04	161.28	214.2	Φ 8 cc 210mm
	Near 6	83.28	60.28	267.75	214.2	232.08	161.28	214.2	Φ 8 cc 210mm

Axis C Ground

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	74.68	53.51	267.75	214.2	261.44	188.16	214.2	Φ 8 cc 210mm
	Near 2	76	54.09	267.75	214.2	258.64	188.16	214.2	Φ 8 cc 210mm
2—3	Near 2	83.72	61.56	267.75	214.2	227.26	188.16	214.2	Φ 8 cc 210mm
	Near 3	84.22	62.06	267.75	214.2	225.42	188.16	214.2	Φ 8 cc 210mm
3—4	Near 3	66.85	44.16	267.75	214.2	316.80	188.16	214.2	Φ 8 cc 210mm
	Near 4	67.71	45.02	267.75	214.2	310.75	188.16	214.2	Φ 8 cc 210mm
4—5	Near 4	83.06	60.54	267.75	214.2	231.08	188.16	214.2	Φ 8 cc 210mm
	Near 5	87.63	65.11	267.75	214.2	214.86	188.16	214.2	Φ 8 cc 210mm
5—6	Near 5	29.29	20.95	267.75	214.2	667.77	188.16	214.2	Φ 8 cc 210mm
	Near 6	29.79	21.45	267.75	214.2	652.21	188.16	214.2	Φ 8 cc 210mm

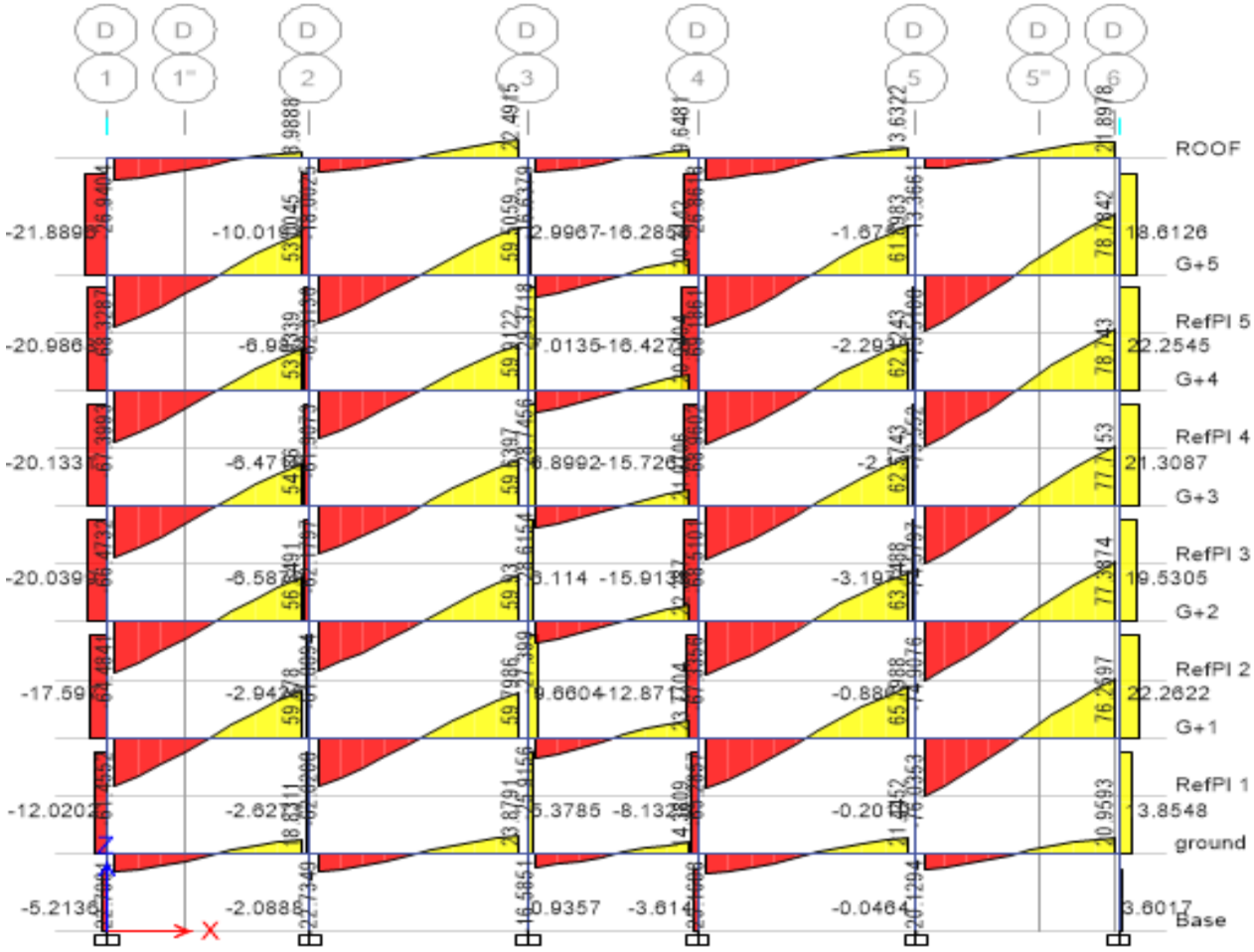


Figure 7-19 shear force diagram of beam axis D

Axis D Roof

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	26.94	22.29	230.25	184.2	539.73	225.80	184.2	Φ 8 cc 210mm
	Near 2	8.98	4.33	230.25	184.2	2778.40	225.80	184.2	Φ 8 cc 210mm
2—3	Near 2	18.01	13.11	230.25	184.2	917.66	225.80	184.2	Φ 8 cc 210mm
	Near 3	22.49	17.59	230.25	184.2	683.94	225.80	184.2	Φ 8 cc 210mm
3—4	Near 3	16.64	12.57	230.25	184.2	957.08	225.80	184.2	Φ 8 cc 210mm
	Near 4	9.65	5.58	230.25	184.2	2156.00	225.80	184.2	Φ 8 cc 210mm
4—5	Near 4	26.86	21.99	230.25	184.2	547.09	225.80	184.2	Φ 8 cc 210mm
	Near 5	13.63	8.46	230.25	184.2	1422.04	225.80	184.2	Φ 8 cc 210mm
5—6	Near 5	13.34	8.78	230.25	184.2	1370.21	225.80	184.2	Φ 8 cc 210mm
	Near 6	21.89	17.33	230.25	184.2	694.20	225.80	184.2	Φ 8 cc 210mm

Axis D Story 5

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	68.32	51.2	267.75	214.2	273.24	161.28	214.2	Φ 8 cc 210mm
	Near 2	53	35.9	267.75	214.2	389.69	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	62.31	46.24	267.75	214.2	302.55	161.28	214.2	Φ 8 cc 210mm
	Near 3	59.51	43.43	267.75	214.2	322.12	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	29.37	21.01	267.75	214.2	665.87	161.28	214.2	Φ 8 cc 210mm
	Near 4	20.23	11.87	267.75	214.2	1178.59	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	68.19	51.2	267.75	214.2	273.24	161.28	214.2	Φ 8 cc 210mm
	Near 5	61.89	44.73	267.75	214.2	312.76	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	73.51	52.01	267.75	214.2	268.98	161.28	214.2	Φ 8 cc 210mm
	Near 6	78.78	57.28	267.75	214.2	244.24	161.28	214.2	Φ 8 cc 210mm

Axis D Story 4

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	67.39	50.26	267.75	214.2	278.35	161.28	214.2	Φ 8 cc 210mm
	Near 2	53.93	36.8	267.75	214.2	380.16	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	61.9	45.92	267.75	214.2	304.66	161.28	214.2	Φ 8 cc 210mm
	Near 3	59.21	43.23	267.75	214.2	323.61	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	28.74	18.73	267.75	214.2	746.92	161.28	214.2	Φ 8 cc 210mm
	Near 4	30.61	20.6	267.75	214.2	679.12	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	68.96	51.61	267.75	214.2	271.07	161.28	214.2	Φ 8 cc 210mm
	Near 5	62.52	45.17	267.75	214.2	309.72	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	73.55	52.05	267.75	214.2	268.78	161.28	214.2	Φ 8 cc 210mm
	Near 6	78.74	57.24	267.75	214.2	244.41	161.28	214.2	Φ 8 cc 210mm

Axis D Story 3

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	66.47	49.55	267.75	214.2	282.34	161.28	214.2	Φ 8 cc 210mm
	Near 2	54.86	37.69	267.75	214.2	371.18	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	62.11	46.1	267.75	214.2	303.47	161.28	214.2	Φ 8 cc 210mm
	Near 3	59.53	43.5	267.75	214.2	321.61	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	28.61	18.71	267.75	214.2	747.72	161.28	214.2	Φ 8 cc 210mm
	Near 4	31.07	20.98	267.75	214.2	666.82	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	68.51	51.21	267.75	214.2	273.19	161.28	214.2	Φ 8 cc 210mm
	Near 5	62.57	45.27	267.75	214.2	309.03	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	74.51	53.02	267.75	214.2	263.86	161.28	214.2	Φ 8 cc 210mm
	Near 6	77.71	56.2	267.75	214.2	248.93	161.28	214.2	Φ 8 cc 210mm

Axis D Story 2

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1—2	Near 1	66.48	49.1	267.75	214.2	284.93	161.28	214.2	Φ 8 cc 210mm
	Near 2	56.64	39.3	267.75	214.2	355.98	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	61	45.13	267.75	214.2	309.99	161.28	214.2	Φ 8 cc 210mm
	Near 3	59.93	43.43	267.75	214.2	322.12	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	27.39	11.22	267.75	214.2	1246.87	161.28	214.2	Φ 8 cc 210mm
	Near 4	68.51	52.34	267.75	214.2	267.29	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	67.33	50.11	267.75	214.2	279.18	161.28	214.2	Φ 8 cc 210mm
	Near 5	63.14	45.92	267.75	214.2	304.66	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	74.9	53.4	267.75	214.2	261.98	161.28	214.2	Φ 8 cc 210mm
	Near 6	77.38	55.89	267.75	214.2	250.31	161.28	214.2	Φ 8 cc 210mm

Axis D Story 1

Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	61.45	46.88	267.75	214.2	298.42	161.28	214.2	Φ 8 cc 210mm
	Near 2	59.47	28.89	267.75	214.2	484.24	161.28	214.2	Φ 8 cc 210mm
2—3	Near 2	62.02	45.95	267.75	214.2	304.46	161.28	214.2	Φ 8 cc 210mm
	Near 3	59.79	43.72	267.75	214.2	319.99	161.28	214.2	Φ 8 cc 210mm
3—4	Near 3	25.92	17.54	267.75	214.2	797.60	161.28	214.2	Φ 8 cc 210mm
	Near 4	23.77	15.39	267.75	214.2	909.02	161.28	214.2	Φ 8 cc 210mm
4—5	Near 4	63.28	46.69	267.75	214.2	299.63	161.28	214.2	Φ 8 cc 210mm
	Near 5	65.79	48.69	267.75	214.2	287.32	161.28	214.2	Φ 8 cc 210mm
5—6	Near 5	76.03	54.54	267.75	214.2	256.51	161.28	214.2	Φ 8 cc 210mm
	Near 6	76.26	54.74	267.75	214.2	255.57	161.28	214.2	Φ 8 cc 210mm

Axis D Ground

span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	22.7	16.81	267.75	214.2	832.23	188.16	214.2	Φ 8 cc 210mm
	Near 2	19	13.11	267.75	214.2	1067.11	188.16	214.2	Φ 8 cc 210mm
2--3	Near 2	22.74	16.59	267.75	214.2	843.27	188.16	214.2	Φ 8 cc 210mm
	Near 3	23.88	17.73	267.75	214.2	789.05	188.16	214.2	Φ 8 cc 210mm
3--4	Near 3	16.58	6.39	267.75	214.2	2189.33	188.16	214.2	Φ 8 cc 210mm
	Near 4	43.8	33.62	267.75	214.2	416.12	188.16	214.2	Φ 8 cc 210mm
4—5	Near 4	25.1	18.97	267.75	214.2	737.47	188.16	214.2	Φ 8 cc 210mm
	Near 5	21.35	15.22	267.75	214.2	919.17	188.16	214.2	Φ 8 cc 210mm
5—6	Near 5	20.13	14.33	267.75	214.2	976.26	188.16	214.2	Φ 8 cc 210mm
	Near 6	20.96	15.16	267.75	214.2	922.81	188.16	214.2	Φ 8 cc 210mm

Lap Length

The design lap length is:

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

where:

- $l_{b,rqd}$ is basic anchorage length
- $l_{0,min} \geq \max \{0.3 \alpha_6 l_{b,rqd}; 15 \emptyset ; 200 \text{ mm}\}$
- values $\alpha_1 \ \alpha_2 \alpha_3$ and α_5 may be taken from table ;
- α_6 = Values of □ 6 are given in Table 8.3.

Table Values of the coefficient α_6

Percentage of lapped bars relative to the total cross-section area	< 25%	33%	50%	> 50%
α_6	1	1.15	1.4	1.5
Note: Intermediate values may be determined by interpolation.				

$l_0 = \alpha_1 \ \alpha_2 \alpha_3 \alpha_5 \ \alpha_6 \ l_{b,rqd}$ = 1x0.7x0.7x0.7x1.4x749.6 = 359.86mm ≥ l0,min

l0,min ≥ max {0.3 α_6 lb,rqd; 15 ∅ ; 200 mm}

≥max {0.3x1.4x749.6=314.83mm; 15x20=300mm; 200 mm}

Therefore, Take l0 = 360mm

` CHAPTER EIGHT

8. Column design and analysis

Introduction tocolumn design

Columns are upright members which carry the loads from beams and slabs dawn to the foundation and therefore they are primary compression members, although they may also have to resist bending forces due to some eccentricity or due to the continuity of the structure.

For the purpose of design calculations, structural members may be classified as Sway or Non-sway depending on their sensitivity to second order effect due to lateral displacements.

According to EBCS 2 1995 section 4.4.4.2

1) A frame may be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes.
2) Any other frame shall be classified as a sway frame and the effects of horizontal displacements of it nodes shall be taken in to account in its design

   A frame may be classi

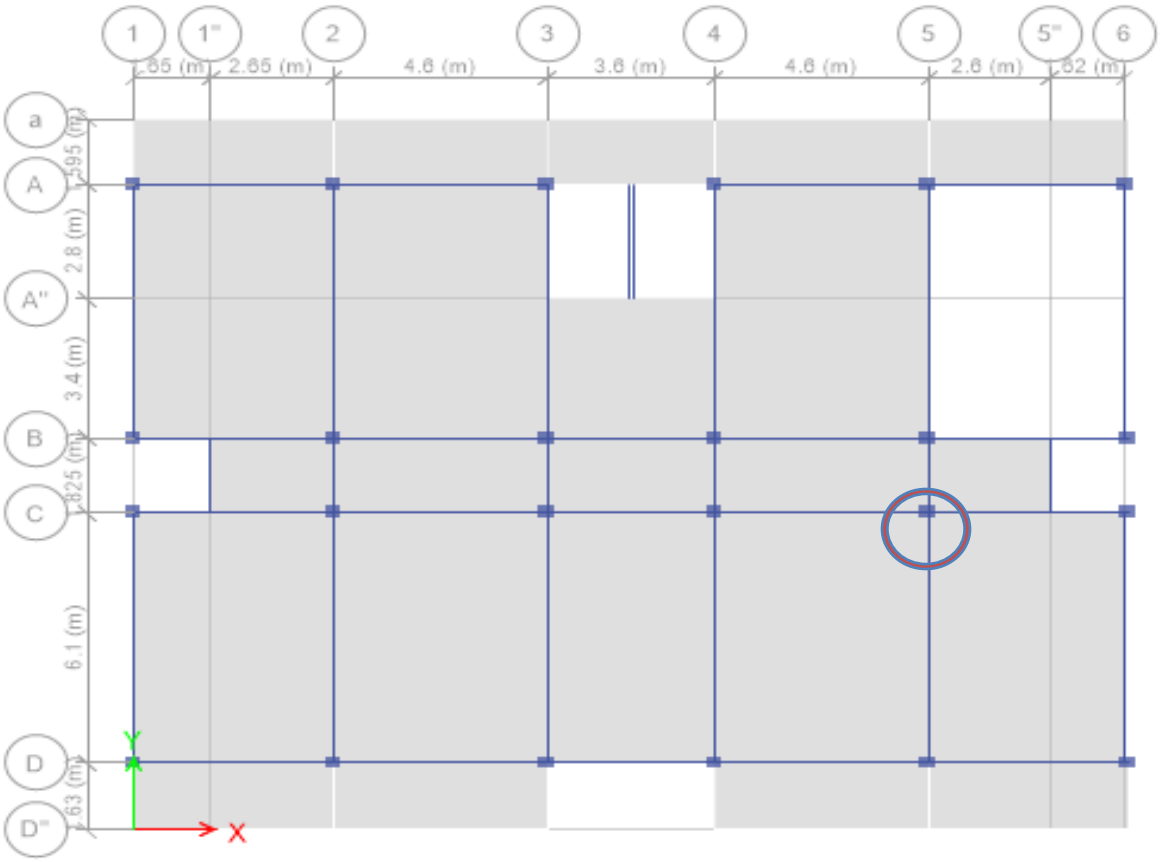
   Where N_{sd} is design value of the total vertical load

   N_{cr} is its critical value for failure in a sway mode

Column may be classified based the following criteria

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

Sample calculation



Effective length determination

The effective height (length) of a column is the distance between the two consecutive points of contra flexure or zero bending moments. The effective length, lo of member is defined as the length of a pin ended strut with constant normal forces having the same cross-section and buckling load. It depends on the deflected shape and the end restraints.

Let calculate for **C-6 on 5th floor** elevation

Given data

l_{bxl}= 4300mm l_{bxR}= 4000mm

l_{byl}= 2500mm l_{bxR}= 5800mm

l_c = 3000mm

$top\ tie\ Beam = 250\ X\ 350$

$intermideat\ beam = 350 \times 400$, Column = 300x300

Determination of effectivedepth

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

Stiffness in x direction

$$K_{1x} = \frac{\left(\frac{300*300^3}{12}{2800}\right)}{\left(\frac{250*350^3}{12}{4300}\right)+\left(\frac{250*350^3}{12}{4000}\right)} = \frac{241071.43}{207727.7+223307.29} = \mathbf{0.56}$$

Same for $K_{2x} = \frac{\left(\frac{300*300^3}{12}{2800}\right)}{\left(\frac{300*400^3}{12}{4400}\right)+\left(\frac{300*400^3}{12}{4000}\right)} = \frac{241071.43}{3633636.36+400000} = \mathbf{0.156}$

$$K_{1y} = \frac{\left(\frac{300*300^3}{12}{2800}\right)}{\left(\frac{250*350^3}{12}{2500}\right)+\left(\frac{250*350^3}{12}{5800}\right)} = \frac{241071.43}{357291.67+154005.03} = \mathbf{0.47}$$

Same for K_{2y}=$\frac{\left(\frac{300*300^3}{12}{2800}\right)}{\left(\frac{300*400^3}{12}{2500}\right)+\left(\frac{300*400^3}{12}{5800}\right)} = \frac{241071.43}{640000+275862.07}$=**0.26**

$$l_{ox} = 0.5*2800\sqrt{\left(1+\frac{0.56}{0.45+0.56}\right)\cdot\left(1+\frac{0.156}{0.45+0.156}\right)}$$=**2510**

$$l_{oy} = 0.5*2800\sqrt{\left(1+\frac{0.47}{0.45+0.47}\right)\cdot\left(1+\frac{0.26}{0.45+0.26}\right)}$$=**2540**

Check slenderness limit

Columns are broadly categorized into two as short and slender column. In short column second order effect are negligible, in this case it is not necessary to consider slenderness effect and compression members can be designed based on forces determined from first order analysis .

When the unsupported length of column is long, lateral deflection shall be so high that the moment shall increase and weaken the the column. such a column, whose axial load carrying capacity is significantly reduced by moments results from lateral deflection of column is referred to as slender/ long/ column.

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

$$i = \sqrt{\frac{I_c}{A}} = \sqrt{\frac{\frac{300*300^3}{12}}{300*300}} = 86.6$$

$$\lambda_x = \frac{l_{ox}}{i} = \frac{2510}{86.6} = 28.98$$

$$\lambda_y = \frac{l_{oy}}{i} = \frac{2540}{86.6} = 29.33$$

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

Determination of accidental eccentricity, equivalent first order eccentricity and second order effect.Accidental eccentricity if column is slender column (ʎ> ʎlim) ea is calculated as =max(lo/400)

$$e_{ix} = max\begin{Bmatrix}\frac{2510}{400} = 6.275\\ \frac{300}{30} = 10\\ 20mm\end{Bmatrix} = \mathbf{20\ mm}$$

$$e_{iy} = max\begin{Bmatrix}\frac{2540}{400} = 6.35\\ \frac{300}{30} = 10\\ 20mm\end{Bmatrix} = \mathbf{20\ mm}$$

$M01x = min\{|-41.72|, |55.397|\} + 0.02 * 257.02 = 36.6$
$M02x = max\{|-41.72|, |55.397|\} + 0.02 * 257.02 = 60.6$
$M01y = min\{|-15.29|, |4.35|\} + 0.02 * 257.02 = 9.49$
$M02y = min\{|-15.29|, |4.35|\} + 0.02 * 257.02 = 10.14$

$$n = \frac{NED}{A_c f_{cd}} = \frac{257.02 * 10^3}{90000 * 11.33} = 0.252$$

$$\lambda\text{xlim} = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{36.6}{60.6})}{\sqrt{0.252}} = \mathbf{33.62mm}$$

$$\lambda\text{ylim} = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{9.49}{10.14})}{\sqrt{0.252}} = \mathbf{23.44mm}$$

28.98 < 33.62, column is not **slender**

29.33 >23.44, column is **slender**

Calculation of reinforcement

Functions of lateral reinforcement

- they hold the longitudinal bars in position in the forms while the concrete is being placed

- they prevent the slender longitudinal bars from buckling out ward by bursting the thin concrete cover

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

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

$$V_{sd} = \frac{N_{sd}}{A_c f_{cd}} = \frac{257.02 * 1000}{90000 * 11.33} = 0.252$$

$$\mu_{sd,x} = \frac{M_{EDx}}{f_{cd}*A_c*h} = \frac{60.6*1000000}{11.33*90000*300} = 0.19$$

$$\mu_{sd,y} = \frac{M_{EDy}}{f_{cd}*A_c*h} = \frac{10.14*1000000}{11.33*90000*300} = 0.033$$

ω , is taken from biaxial chart on [appendix F] for this sample ω= 0.3

Minimum and maximum area of reinforcement

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

$$A_{s,min} = max\begin{Bmatrix}\left(0.1 * \frac{257.02}{\frac{400}{1.15}} * 1000\right) = 73.9mm^2\\ 0.002 * 300 * 300 = 180\end{Bmatrix}$$

$$A_{s,min} = 180mm^2$$

$A_{s,max} = 0.04Ac$ = 0.04*300*300= 3600

$$A_{s,tot} = \frac{\omega*A_c*f_{cd}}{f_{yd}} = \frac{0.3*90000*11.33}{\frac{400}{1.15}} = 879.49$$

Let us use ø 16 bar

$$\text{No of bar} = \frac{A_{st}}{a_{st}} = \frac{879.49}{\frac{3.14*16^2}{4}} = 3.97$$

Use 4ø16 bar

For shear reinforcement

Take ø 8mm

$$\text{Spacing} = min\begin{Bmatrix}20 * \text{ølong} = 20 * 16 = 320\\ lesser\ dimension\ of\ column = 300\\ 400mm\end{Bmatrix}$$

Use ø 8 c/c 300

loca	col	bc	hc	bb	Hb	lc	Lbx left	Lbx right	Lby left	Lby right	K1x	k2x	K1y	K2y	Lox	loy
ROOF	C1	300	300	250	350	2800	0	3095	0	5800	0.84	0.84	1.57	1.57	2309.84	2487.40
ROOF	C2	300	300	250	350	2800	3950	4300	0	5800	0.56	0.56	1.57	1.57	2173.53	2487.40
ROOF	C3	300	300	250	350	2800	4300	3300	0	5800	0.50	0.50	1.57	1.57	2139.56	2487.40
ROOF	C4	300	300	250	350	2800	3300	4300	0	5800	0.50	0.50	1.57	1.57	2139.56	2487.40
ROOF	C5	300	300	250	350	2800	4300	3950	0	5800	0.56	0.56	1.57	1.57	2173.53	2487.40
ROOF	C6	300	300	250	350	2800	3950	0	0	5800	1.07	1.07	1.57	1.57	2384.45	2487.40
ROOF	C7	300	300	250	350	2800	0	3950	5800	2500	1.07	1.07	0.47	0.47	2384.45	2116.32
ROOF	C8	300	300	250	350	2800	3950	4350	5800	2500	0.56	0.56	0.47	0.47	2175.44	2116.32
ROOF	C9	300	300	250	350	2800	4350	3300	5800	2500	0.51	0.51	0.47	0.47	2141.30	2116.32
ROOF	C10	300	300	250	350	2800	3300	4300	5800	2500	0.50	0.50	0.47	0.47	2139.56	2116.32
ROOF	C11	300	300	250	350	2800	4300	3950	5800	2500	0.56	0.56	0.47	0.47	2173.53	2116.32
ROOF	C12	300	300	300	350	2800	3950	0	5800	2500	0.89	0.89	0.39	0.39	2329.28	2052.59
ROOF	C13	300	300	300	350	2800	0	3950	2500	5800	0.89	0.89	0.39	0.39	2329.28	2052.59
ROOF	C14	300	300	300	350	2800	3950	4350	2500	5800	0.47	0.47	0.39	0.39	2111.92	2052.59
ROOF	C15	300	300	300	350	2800	4350	3300	2500	5800	0.42	0.42	0.39	0.39	2077.55	2052.59
ROOF	C16	300	300	300	350	2800	3300	4300	2500	5800	0.42	0.42	0.39	0.39	2075.80	2052.59
ROOF	C17	300	300	300	350	2800	4300	3950	2500	5800	0.46	0.46	0.39	0.39	2109.99	2052.59
ROOF	C18	300	300	300	350	2800	3950	0	2500	5800	0.89	0.89	0.39	0.39	2329.28	2052.59
ROOF	C19	300	300	300	350	2800	0	3950	5800	0	0.89	0.89	1.30	1.30	2329.28	2440.91
ROOF	C20	300	300	300	350	2800	3950	4350	5800	0	0.47	0.47	1.30	1.30	2111.92	2440.91
ROOF	C21	300	300	300	350	2800	4350	3300	5800	0	0.42	0.42	1.30	1.30	2077.55	2440.91
ROOF	C22	250	400	300	400	2800	3300	4300	5800	0	0.56	0.56	1.73	1.73	2173.56	2510.50
ROOF	C23	250	400	300	400	2800	4300	3950	5800	0	0.61	0.61	1.73	1.73	2207.19	2510.50
ROOF	C24	250	400	300	400	3200	3950	0	5800	0	1.03	1.03	1.51	1.51	2713.07	2832.73

loc	col	Mx top	Mx bot	My top	My bot	NED	Mo 1x	Mo 1y	Mo 2x	Mo 2y	λx	λy	λxlim	λylim	Slenderness (x)	Slenderness (y)
ROOF	C1	46.43	111.8	46.43	60.34	193.22	50.29	50.29	115.6	64.20	26.67	28.72	44.76	32.43	Short	short
ROOF	C2	57.46	76.91	28.37	32.27	202.95	61.52	32.43	80.97	36.33	25.10	28.72	32.46	27.87	Short	slender
ROOF	C3	40.02	46.35	17.93	10.81	132.34	42.67	13.46	49.00	20.58	24.71	28.72	35.45	44.71	Short	short
ROOF	C4	40.36	20.69	45.89	69.08	146.88	23.63	48.83	43.30	72.02	24.71	28.72	46.84	41.47	Short	short
ROOF	C5	95.25	90.78	15.92	22.98	223.84	95.26	20.40	99.73	27.46	25.10	28.72	24.48	31.46	slender	short
ROOF	C6	27.49	17.42	18.05	11.99	64.93	18.72	13.29	28.79	19.35	27.53	28.72	64.07	61.83	Short	short
ROOF	C7	12.33	0.93	12.33	104.2	18.51	1.30	12.70	12.70	104.57	27.53	24.44	182.6	180.4	Short	s
ROOF	C8	56.43	79.48	56.43	98.85	364.23	63.71	63.71	86.76	106.13	25.12	24.44	24.88	28.34	slender	short
ROOF	C9	20.97	30.43	0.25	4.16	231.76	25.61	4.89	35.07	8.80	24.73	24.44	31.33	36.97	Short	short
ROOF	C10	15.33	12.41	37.50	48.16	263.26	17.68	42.77	20.60	53.43	24.71	24.44	25.51	27.26	Short	short
ROOF	C11	42.63	45.65	3.40	0.00	342.36	49.48	6.85	52.50	10.25	25.10	24.44	20.13	27.42	slender	short
ROOF	C12	14.80	2.74	47.73	39.46	123.71	5.21	41.93	17.27	50.20	26.90	23.70	61.82	38.23	Short	short
ROOF	C13	8.84	14.62	75.74	83.09	184.68	12.53	79.43	18.31	86.78	26.90	23.70	36.75	28.40	Short	short
ROOF	C14	12.25	24.29	14.73	16.85	347.04	19.19	21.67	31.23	23.79	24.39	23.70	28.66	20.83	Short	slender
ROOF	C15	37.12	50.52	3.29	2.13	283.93	42.80	7.81	56.20	8.97	23.99	23.70	27.39	24.20	Short	short
ROOF	C16	44.73	50.09	31.94	35.11	284.56	50.42	37.63	55.78	40.80	23.97	23.70	23.21	22.67	slender	slender
ROOF	C17	47.72	55.39	15.29	0.00	257.02	52.86	5.14	60.53	20.43	24.36	23.70	25.36	44.43	Short	short
ROOF	C18	30.23	34.76	27.83	38.90	97.70	32.18	29.78	36.71	40.85	26.90	23.70	40.97	48.31	short	short
ROOF	C19	48.49	54.54	29.42	43.76	97.91	50.45	31.38	56.50	45.72	26.90	28.19	40.11	50.38	short	short
ROOF	C20	79.92	93.36	15.17	17.77	143.67	82.79	18.04	96.23	20.64	24.39	28.19	34.45	33.89	short	short
ROOF	C21	74.03	83.56	2.38	11.04	152.42	77.08	5.43	86.61	14.09	23.99	28.19	32.27	52.37	short	short
ROOF	C22	76.56	84.79	20.10	32.93	156.11	79.68	23.22	87.91	36.05	18.82	21.74	32.93	43.81	short	short
ROOF	C23	75.89	85.97	0.00	0.00	146.69	78.82	2.93	88.90	2.93	19.11	21.74	34.81	29.96	short	short
ROOF	C24	33.03	31.07	22.30	37.32	87.91	32.83	24.06	34.79	39.08	23.50	24.53	41.82	59.95	short	short

Table 8-1 slenderness

loc	col	medx	medy	vsd	μsdy	μsdx	ω	As	As1 min	AS1 max	As prov	Ø	No of bar	Longer bar	Shear reinfo	col
ROOF	C1	115.60	64.20	0.2	0.21	0.38	0.3	1034.74	180	3600	1034.74	16	5.15	4ø16	ø 8 c/c	300
ROOF	C2	80.97	36.33	0.2	0.12	0.26	1.3	4483.88	180	3600	4483.88	16	22.31	4ø16	ø 8 c/c	300
ROOF	C3	49.00	20.58	0.1	0.07	0.16	2.3	7933.02	180	3600	7933.02	16	39.48	4ø16	ø 8 c/c	300
ROOF	C4	43.30	72.02	0.1	0.24	0.14	0.1	344.914	180	3600	344.914	16	1.72	4ø16	ø 8 c/c	300
ROOF	C5	99.73	27.46	0.2	0.09	0.33	0.3	1034.74	180	3600	1034.74	16	5.15	4ø16	ø 8 c/c	300
ROOF	C6	28.79	19.35	0.1	0.06	0.09	0.15	517.371	180	3600	517.371	16	2.57	4ø16	ø 8 c/c	300
ROOF	C7	12.70	104.5	0.0	0.34	0.04	6.3	21729.6	180	3600	21729.6	16	108.13	4ø16	ø 8 c/c	300
ROOF	C8	86.76	106.	0.4	0.35	0.28	7.3	25178.7	180	3600	25178.7	16	125.29	4ø16	ø 8 c/c	300
ROOF	C9	35.07	8.80	0.2	0.03	0.11	8.3	28627.8	180	3600	28627.8	16	142.46	4ø16	ø 8 c/c	300
ROOF	C10	20.60	53.43	0.3	0.17	0.07	9.3	32077	180	3600	32077	24	70.94	6ø16	ø 8 c/c	300
ROOF	C11	52.50	10.25	0.3	0.03	0.17	10.3	35526.1	180	3600	35526.1	16	176.78	4ø16	ø 8 c/c	300
ROOF	C12	17.27	50.20	0.1	0.16	0.06	11.3	38975.3	180	3600	38975.3	16	193.95	4ø16	ø 8 c/c	300
ROOF	C13	18.31	86.78	0.2	0.28	0.06	12.3	42424.4	180	3600	42424.4	16	211.11	4ø16	ø 8 c/c	300
ROOF	C14	31.23	23.79	0.3	0.08	0.10	13.3	45873.5	180	3600	45873.5	16	228.27	4ø16	ø 8 c/c	300
ROOF	C15	56.20	8.97	0.3	0.03	0.18	14.3	49322.7	180	3600	49322.7	16	245.44	4ø16	ø 8 c/c	300
ROOF	C16	55.78	40.80	0.3	0.13	0.18	15.3	52771.8	180	3600	52771.8	16	262.60	4ø16	ø 8 c/c	300
ROOF	C17	60.53	20.43	0.3	0.07	0.20	16.3	56220.9	180	3600	56220.9	16	279.76	4ø16	ø 8 c/c	300
ROOF	C18	36.71	40.85	0.1	0.13	0.12	17.3	59670.1	180	3600	59670.1	16	296.93	4ø16	ø 8 c/c	300
ROOF	C19	56.50	45.72	0.1	0.15	0.18	18.3	63119.2	180	3600	63119.2	16	314.09	4ø16	ø 8 c/c	300
ROOF	C20	96.23	20.64	0.1	0.07	0.31	19.3	66568.4	180	3600	66568.4	16	331.25	4ø16	ø 8 c/c	300
ROOF	C21	86.61	14.09	0.1	0.05	0.28	20.3	70017.5	180	3600	70017.5	16	348.42	4ø16	ø 8 c/c	300
ROOF	C22	87.91	36.05	0.1	0.08	0.19	21.3	81629.6	200	4000	81629.6	16	406.20	4ø16	ø 8 c/c	250
ROOF	C23	88.90	2.93	0.1	0.01	0.20	22.3	85462	200	4000	85462	16	425.27	4ø16	ø 8 c/c	250
ROOF	C24	34.79	39.08	0.1	0.09	0.08	23.3	89294.3	200	4000	89294.3	16	444.34	4ø16	ø 8 c/c	250

Table 8-2 longtiudinal and shear reinforcement

CHAPTER EIGHT

9. FOUNDATION DESIGN

Introduction to Foundation

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

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

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

-Overall stability

-Bearing resistance failure

-Failure by sliding

Axially loaded footing at intersection of axis C-2(at Elevation-C) on structure two which has a compression force of **2772.7KN** is selected as a representative sample for isolated footing design. From ETABS analysis output results the following loading data are obtained.

P =2772.7KN

M_x = 20.1KN-m

M_y = 4.8KN-m

W=10%P=277.27KN

Foundation depth assumed to be 1.5m below the basement floor.

Material used

Steel

- Steel grade, S – 500
- Fyk = 500mpa, Fyd = 500/1.15 = 434.78MPa

Concrete

Concrete grade, C-30

Fck = 30 * 0.8= 24mpa,

Fctk= $0.21Fck^{2/3}$= 1.74MPa

Design square footing

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

Eccentricity By using ultimate limit state design (ULSD- method)

ex = my/p = 4.8/2772.7 = 0.0017m,
ey = mx/p = 20.1/2772.7 = 0.0072m

Area of footing required

$$A = \left(\frac{N+W}{\gamma soil}\right) = \frac{2772.7 + 277.27}{150} = 20.33m^2$$

Using square footing B*H*h= 2.5*2.5*0.7m, A=6.25m^2

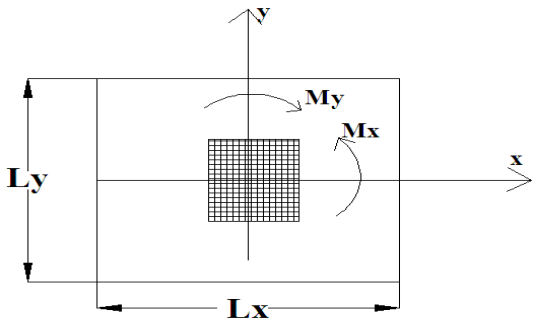
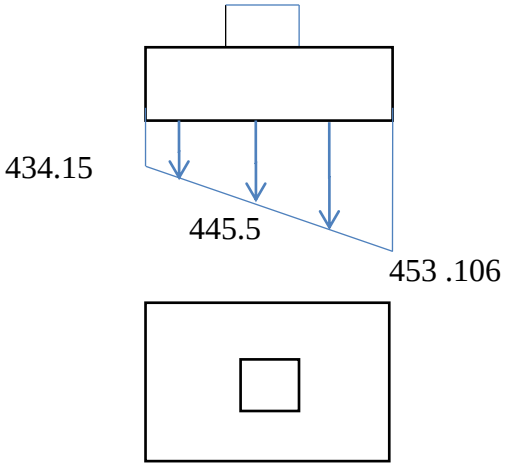


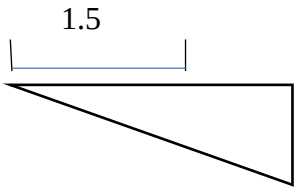
Figure 9-1 Isolated Footing Pad Layout

Actual contact pressure

$$\sigma = \frac{P}{A}\left(1 \pm \frac{6e_y}{b} \pm \frac{6e_x}{a}\right)$$

σ_{max} = 2772.7/6.25 (1 + 6(0.0072)/2.5+ 6(0.0017)/2.5), ρmax = 453.106KN/m^2
σ_{min} = 2772.7/6.25 (1 - 6(0.0072)/2.5- 6(0.0017)/2.5), ρmin =434.15KN/m^2
The column dimension is 50*50mm





18.96

By using similarity rule

$\frac{1.5}{x} = \frac{2.5}{18.96}$, x=11.376

$M_{xx}=\left(445.5*\frac{1^2}{2}\right)+(453.106-434.15)\left(\frac{1}{2}\right)*1*\frac{2}{3}=229.06\text{KN.m}$

$M_{yy}=\left(\frac{453.106+434.15}{2}\right)*\frac{1^2}{2}=221.8KN.m$

Effective depth determination

Before calculating the effective depth we need to find concrete cover

Cover design

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

exposure class-xc_3, EBCS-2 Table 9-1exposure classes

min cover regard to bond, c_{min}, b=12mm

min cover regard to durability, c_{min}, dur=25mm

} c_{min}=25mm

allowance is design for devation, Δc_{dev}=10mm

$c_{nom}=c_{min}+\Delta c_{dev}$=25+10=**35mm**

$dx=h-ccov-0.5\emptyset bar=700-35-(0.5*24)=653\text{mm}$

$dy=h-ccov-1.5\emptyset bar=700-35-(1.5*24)=629\text{mm}$

Determination of reinforcement

Main reinforcement-longitudinal bar

$$k=\frac{M_{xx}}{fck*b*d^2}=\frac{229.06}{24*2.5*70^2}=0.000779$$

$$z=d\left(0.25-\frac{k}{1.134}\right)=0.7\left(0.25-\frac{0.000779}{1.134}\right)=0.175$$

$$z=\frac{d}{2}\left(1+\sqrt{1-3.53k}\right)\leq 0.95d=0.665\rightarrow\frac{700}{2}\left(1+\sqrt{1-3.53*0.000779}\right)=0.698$$

$$\therefore z=0.665$$

$$Asreq=\frac{Mxx}{fyd*z}=\frac{229.06}{434.78*0.665}=79.22mm^2$$

Minimum and maximum area of reinforcement

$Asmin=0.26*\left(\frac{fctm}{fyk}\right)*bd=0.26*\left(\frac{2.5}{500}\right)*2.5*0.7=\mathbf{2271.45mm^2}$,

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

$$Asmax=0.04*Ac=0.04*2.5*0.7=\mathbf{70000mm^2}$$

Use the minimum of the two $\boldsymbol{Asmin=2271.45mm^2}$

Provide **6Ø24(As=2712.96mm^2)**

Main reinforcement-transvers bar

$$k=\frac{M_{yy}}{fck*b*d^2}=\frac{221.8}{24*2.5*629^2}=0.00934<Kbal=0.167$$

There for compression reinforcement is not required.

$$z = d\left(0.25 - \frac{k}{1.134}\right) = 0.23d < 0.95d$$

$$Asreq = \frac{Myy}{0.87fyk * z} = \frac{221.06 * 10^6}{0.87 * 500 * 0.239 * 629} = 3380.43mm^2$$

Minimum and maximum area of reinforcement

$$Asmin = 0.26 * \left(\frac{fctm}{fyk}\right) * bd = 0.26 * \left(\frac{2.5}{500}\right) * 2.5 * 629 = \mathbf{2044.25mm^2}$$

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

Since $A_s < A_{min}$ use $A_{smin} = \mathbf{2044.25mm^2}$

Provide **5Ø24(As=2260.8mm²)**

Vertical shear

Critical shear at 1.0d from face of column

Average pressure critical section

$=434.15+(\frac{2.2}{2.5})*18.96$

$=450.83$

Design shear force

$V\varepsilon_d = 450.83*0.35*2.5=394.47KN$

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

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

$Asl = 0mm^2$

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

$V_{RdC} = 0$

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

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

V_{min}= 398.41KN

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

Punching shear

Critical shear at 2.0d from face of column

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

$2d = 1329$

Control perimeter

$U = 2(500+500)+(2\pi*1329) = 10346.12mm$

Area within perimeter

$A = [0.5*0.5]+[2*0.5*1.329]+[2*0.5*1.329]+[\pi*1.329^2] = 8.454m^2$

Average punching shear stress

$V\varepsilon d = (\frac{453.106+434.15}{2})[(2.5*2.5)-8.454] =$ **977.75 KN**

Punching shear stress

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

$W1=0.5c1^2+c1c2+4c2d+16d^2+2\pi dc1$

$=0.5(500)^2+500*500+4*500*664.5+16*664.5^2+2*\pi*664.5*500$

$W1=$ $\underline{10855494mm^2}$

$B=1+0.65(\frac{20.1*10^6}{977.75*10^3})(\frac{10346.12}{10855494})=1.012$

$V_{\epsilon d}=\frac{1.012*977.75*10^3}{10346.12*700} =0.136N/mm^2$

Punching shear resistance

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

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

$=0.035(1.535)^{3/2}(24)^{1/2} =0.324$

$0.324>V\varepsilon d(0.136)$ **OK**

Maximum punching shear at column perimeter

Max punching shear force

Column perimeter $u_o =2(500+500) =2000$

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

$W1=0.5*500^2+500*500$

$W1=375000mm^2$

$B=1+0.65(\frac{20.1*10^6}{2772.7*10^3})(\frac{2000}{375000})=1.025$

$V\varepsilon_d= (\frac{1.025*2772.7*10^3}{2000*664.5}) =2.14N/mm^2$

Maximum shear resistance

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

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

$=4.34> V\varepsilon_d=1.025$ **OK**

CONCLUSION AND RECOMMENDATION

CONCLUSION

Throughout the whole process, we have understood that structural design has a key role for a specific construction project. This project titled “Structural design of institutional G+5 hotel building” is designed with the provisions of ES-EN 2015. The roof has a flat.

All the RC structures slab, beam, column etc. have been designed according to the ESEN code recommendation. All the beams were designed for shear and flexure as usual. The columns were checked whether they are sway or non-sway. Finally, the frame elements were provided with accurate and adequate reinforcement.

In this final semester project of structural design of G+5 without basement building, there were a lot of obstacles and try to take these in order to come up with the most realistic and well organized final document. Each part of the building had been analyzed and designed.

In design of this building, the three basic questions of design were being tried to address. These structural members are economical, safe and workable with good aesthetics as per the standard of our country building code. But we haven’t done much as we expected we tried our best

Generally, this final project is drawn the following conclusion:

Isolated footing type has been selected for the foundation because of it has Good bearing capacity of soil in Addis Ababa town.

The intended slab type in this project is solid slab. These type of slab has been used because span length of all panels are less than six meter.

The Frame analysis is done by ETABS (2016) Software and the output is obtained by using design manual method to design beams and columns.

Check Lateral load including wind load analysis by using manual method calculation or work compared with ETABS (2016) Software out put our result is stable.

Generally, this building is designed to answer most economical, safe and good aesthetics values.

RECOMMENDATION

Doing this project has brought a great benefit in making all of us fully qualified for the responsibilities we are expected to face ahead us. So, we would like to appreciate all the deeds and efforts that the university and the department of civil engineering has been doing.

However, there are a lot of work left to be done for all parties to obtain a qualified candidate as we all wish. The drawbacks that we saw during the project may be as follows: The department has a weak time management culture. So, it should follow a strict calendar for every task. The tasks may be exam schedule, project initiation/submission date and so on. Because overlapping of lot of works happens which brings a negative effect in both the students receiving the right knowledge and getting the desired result of grades for each course.

We hope that the purpose of learning this degree program is to be able to create a qualified civil engineer who build a better Ethiopia. So, in order to achieve this big dream, all the parties from the university dean to the students we should change our attitude.

Teachers (especially advisors in this project) should work closely with the students in assisting in every steps of the project to make all the works worthy. Last but not least, we have struggled with lack of knowledge in specific areas of the field. Therefore, we strongly recommend for the department of the civil engineering to focus on the following portions:

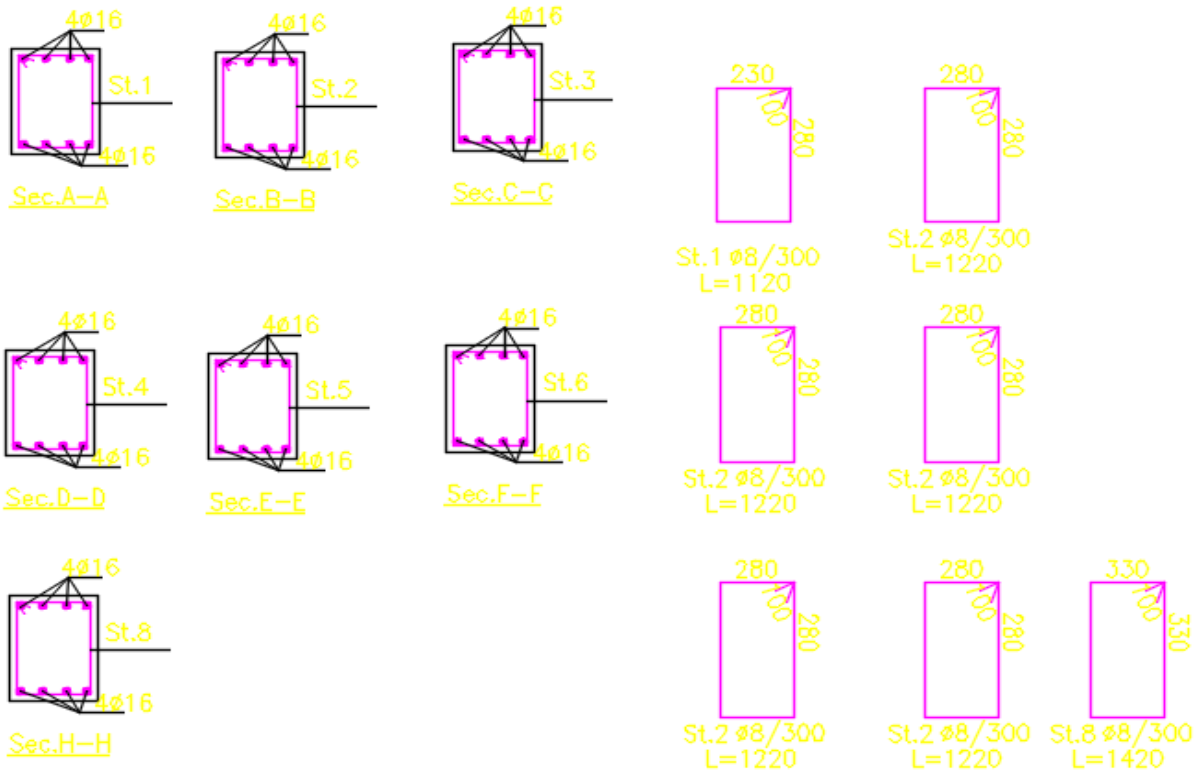
- ✓ Reinforcement detail of each member in a building with detail drawing
- ✓ Software lessons were much appreciated if they were given as course.
- ✓ Most courses need a routine practical visit outside the class room right after we grasped the theory to be familiar with the real world, like Reinforced concrete structure. We should have seen how those structures that we designed in the class rooms are constructed.
- ✓ It is better to know the title of the project at the end of first semester, accordingly, the necessary preparation can be handle in the most proper way.
- ✓ To make the students rich in information, there should be internet access in the computer room.
- ✓ There is no truss for roof design in this building so, it was better to do that to be knowledgeable.
- ✓ It is recommended that, it is better to come up with the design data which have the necessary information.
- ✓ It is better if the project incorporating all types of slab like ribbed and flat slab design. In order to take and understand all the actual design phenomena.
- ✓ In order to have general understanding about the procedures and preparation of the project, the school should give orientation for the students.

It is better to incorporate this type of project since it helps the student to develop the quality of working in teams, understanding with each other and develop good interaction with those who has better knowledge and experience than us.

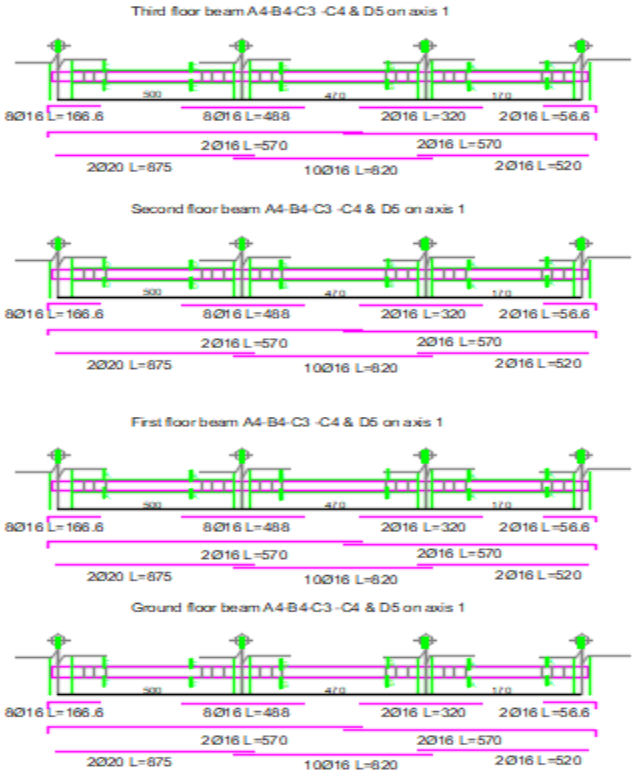
We have taken deferent kinds of assumptions which was unclear for as.

Apendix a

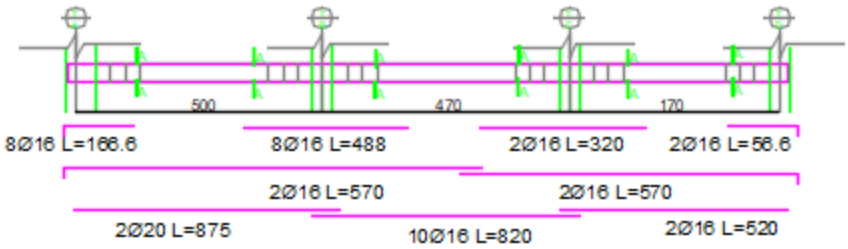
Beam section



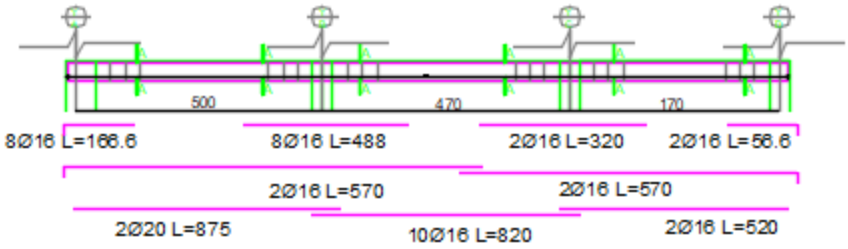
Appendix b beam detail



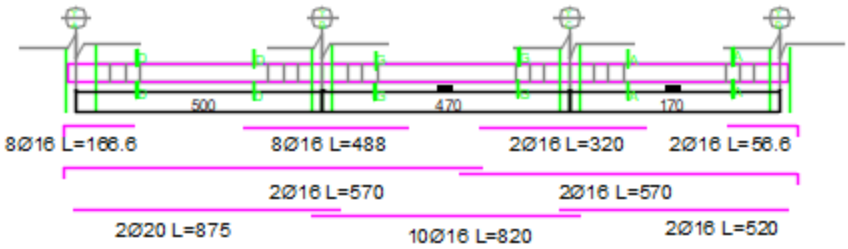
Roof floor beam A4-B4-C3 -C4 & D5 on axis 1



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

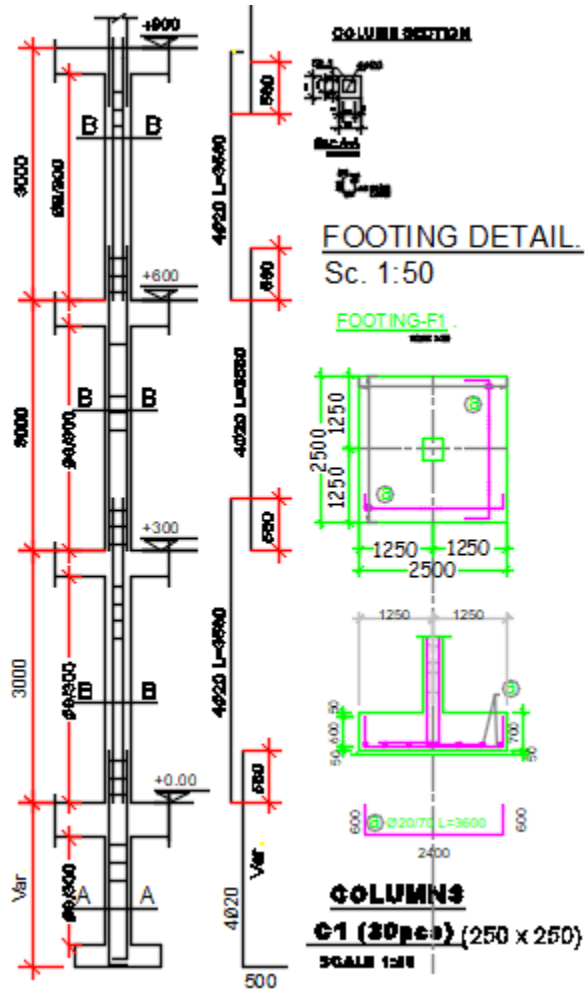


Fourth floor beam A4-B4-C3 -C4 & D5 on axis 1



Appendix c

Column and foundation detail



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