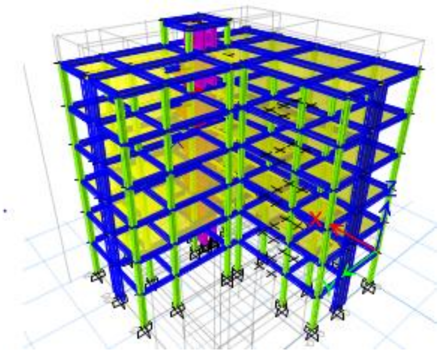
College of Engineering and Technology

Title: Structural design of a G+4 reinforced concrete residential building



Gedlu Bekele[1], Semenugus Gosaya[2], Sintayew Melese[3], Siraj Worake[4], Siraj Abdurehman[5],

Advisor/s

Mr. Yosef Gemesa

Mr. Debisa Getachew

August, 2021

[1]Wolkite University, Civil Engineering Department, ID Number Engr/388/09,

[2]Wolkite University, Civil Engineering Department, ID Number Engr/754/09,,

[3]Wolkite University, Civil Engineering Department, ID Number Engr/781/09,

[4]Wolkite University, Civil Engineering Department, ID Number Engr/783/09,

[5]Wolkite University, Civil Engineering Department, ID Number Engr/785/09,

Wolkite University, College of Engineering and Technology

Civil Engineering Department

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 Gedlu Bekele, Semenugus Gosaya, Sintayew Melese, Siraj Worake, and Siraj Abdurehman, entitled Structural design of a G+4 reinforced concrete residential building in Wolkite, and recommended for acceptance as a fulfillment of the requirement of **B.Sc. in Civil Engineering**

______________________	______________________	______________________
Chairman/Coordinator	Signature	Date
______________________	______________________	______________________
Advisor	Signature	Date
______________________	______________________	______________________
Internal Examiner	Signature	Date

Preface

This document is a report as a part of fulfillment for a Bachelor of Science in Civil Engineering. The project is conducted by X

Summary

This project is about the structural design and analysis of G+4 building considering all the effects the building receives in its service year. Structural design involves proportioning of structural members of the building and providing reinforcement which results safe and economical.
This thesis works on the structural analysis and designed of G+4 residential building which has L-shape plan layout with lift shaft and an opening in the slab. We design the building as a solid slab system. In the design process, we deal with the structural arrangement of the building.

Then, we estimate the load of the components', of the building. We analyze and design solid slab and the stair cases then after we compute the load transfer to the beams and also determine the lateral loads actions on the structure and the center of mass at which the lateral loads act at each floor of the building. After all we modeled the structure using ETABS software and loaded the gravity loads and lateral loads on the structure. Having analyzed the structure we obtained the analysis result. We check whether the center of mass and center of rigidity of the structure got close to each other in order to reduce the stresses to columns due to Torsional effect. Then we provide shear walls in the X and Y directions to get the CM and CR close to each other until satisfactory result is attained by trial and error. And then after we check the ultimate limit state and serviceability limit state of the structure.

We use Coefficient method and Hillerborg strip method for slab analysis & design. Finally we design the structural components such as column, beams and footings using the analysis result from ETABS output. In addition to design of solid slab structural system, we also analyze and design of flat slab and ribbed slab for the same plan layout of the building. At the end, we drew reinforcement detailing for the structural components.

The thesis will have an objective to develop new skill, strength our capacity on structural analysis and design as well as integrate different discipline for specified and justified problem. Beside this, it will create awareness for others to develop this project idea for civil engineering profession.

Acknowledgement

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

Our deepest gratitude and appreciation goes to our project advisors, Mr. Debisa Getachew, Mr. Bulo Lema, & Mr.Yosefe for they valuable advice and endless technical assistance which brought this project approach in the successful completion.

Also our acknowledge goes to all stuff of civil engineering department lectures. Finally we are very grateful to our families who gave a very outstanding support standing from the beginning of our university life until the end of this project.

In general, we want to pass grateful acknowledgement for those we have helped us in the process either in advice or in material until the project comes to completion.

Table of Contents

List of figures

List of Tables

LIST OF ABBREVIATIONS

- Reinforced concrete---RC
- Ultimate limit state ULS--ULS
- Ethiopian building code standard based on European norm--------------EBCS EN
- Working stress method--WSM
- Earth quake load coming from positive X direction----------------------------- EQX+
- Earth quake load coming from negative X direction----------------------------- EQX-
- Earth quake load coming from positive Y direction----------------------------- EQY+
- Earth quake load coming from negative Y direction----------------------------- EQY-
- Wind load on frames or roof sections---Fwe
- Design dead load---GK
- Design Live load--QK
- Factored Design load---Pd

Introduction

Structural design is a mixture of art and science, combining the engineers feeling for the behavior of structure with a sound knowledge of the principles of statics, dynamics, mechanics of material, and structure analysis, to produce a safe economical structure that will serve its intended purpose. It is the process of determining the dimensions and lay out of the load resisting (structural)components of a structure to satisfy the purpose of use ,to possess safety and durability and to be economical .

Structural analysis is the assessment of the performance of a given structure under given loads, and other effects such as support movements or temperature change.

Once the form and structural arrangement have been finalized the structural design procedure consists of the following:-

⇒ Idealization of the structure in to component parts

⇒ Load estimation the various structural components.

⇒ Analysis to determine the maximum internal stress and strains

⇒ Design of section and reinforcement arrangement

⇒ Detail drawings and bar schedule preparation

There are three methods of concrete design these are:-

1) Working stress method

2) Ultimate load method (ULM)

3) Limit states method (LSM)

Working stress method:- In this method, the section of reinforced concrete members are designed assuming straight line stress-strain relationships, i.e, the response and stress are elastic.

The internal bending moments and forces for a structure are calculated assuming linear elastic behavior. Because of elastic stress distribution is assumed in design, it is not really applicable to a semi-plastic material such as concrete, nor it is suitable when determinations are not proportional to the load, as in slender columns.

The ultimate strength design (USD) methods: - in this method, sections are design taking the actual inelastic strains into account. The design stresses used are the ultimate Strengths of materials and for safety the loads are magnified or scaled up by load factors As this method does not apply factors of safety to material stress, it cannot directly taken account of Variability of the materials and also cannot be used to calculate the deflections or cracking load.

The limit state Design (LSD) method: - More recently, it has been recognized that the design approach for reinforced concrete should ideally combine the best features of ultimate strength and working stress design. This is desirable because if sections are proportioned by ultimate strength requirements alone, there is a danger that although the load factors are adequate to ensure safety against strength failure, the cracking and deflections at service loads may be excessive. Thus to ensure a satisfactory design, the deflections and crack widths must be checked for service loads to make sure that they lie within reasonable limiting values dictated by functional requirement of the structure. This check requires the use of elastic theory. Therefore, in the LSD method, structures will be designed for strength at ultimate loads (ULS) and deflection and crack width checked at service loads (SLS). Ethiopian building code standard ES EN 1992:2015 is based on the LSD method. We used this design code to design our structural system. Since it considers not only the ultimate limit state (ULS) but also serviceability limit state therefore, it is possible to design structure which is both safe and serviceable.

1.1 Background of the project

Reinforced concrete is the most widely used construction material in the world in the construction industry. It is a composite structure of construction material concrete and steel reinforcement bars. It is a concrete with steel bars embedded in it. The universal nature of reinforced concrete construction stems from the wide availability of reinforcing bars and the constituents of concrete (gravel or crushed rock, sand, water, and cement), from the relatively simple skills required in concrete construction, and from the economy of reinforced concrete compared with other forms of construction. Plain concrete and reinforced concrete are used in buildings of all sorts, underground structures, water tanks, wind turbine foundations and towers, offshore oil exploration and production structures, dams and bridges.

In this project we design the structural member of G+4 residential building for living purpose Located at Gubre town.

1.2 Objective

This team has planned to work on different disciplines that have a good effort both in the development of new skill for our self and putting some tangible contribution for our profession. This thesis has wide objectives which are classified into two categories. Bearing in mind, it may have more objectives than we try to state here.

The main objective of this B.Sc. project is to

Since all the team members are the future responsible civil engineers;

As a structural engineer we are expected to convert architectural engineers' (given drawing) drawing in to real world building with safe and durable structural design with in architectural engineers' (given drawing) dimension so, our main objectives of design safe, durable and economical structural design. In case the structure fails, it must be in such a way it will minimize risks and casualty.

This study has the following sub objectives

The project also has an objective to create an important contribution in the civil engineering profession.

- ❖ Demonstrate structural analysis and design skill
- ❖ Understand the new Ethiopian Building Code for Structures (EBCS 2015)
- ❖ Creating an awareness for others to develop such kind of idea for either B.Sc or M.Sc thesis
- ❖ Relating theoretical knowledge to the actual site condition
- ❖ Develop ability to work in diverse team
- ❖ Development of new skills such as development of software and applying existing software

1.3 Structural Design Philosophy and Methods

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

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

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

1) Working Stress Design (WSD) or Elastic Design Method
2) Ultimate Strength Design (USD) Method, and

3) Limit State Design (LSD) Method

Methods for slab analysis & design

Coefficient method is based up on elastic theory but the reinforcement for slabs is calculated by strength methods the account for the actual inelastic behavior of structural members at the factored load stage. This method limits the structural usefulness of the material up to ascertain load at which the maximum stress in extreme fibers reaches the yield stress of the material in bending. The exactness of this method is restricted to square or rectangular slabs with symmetric supports and for uniform distribution loads. **Hillerborg strip methods** are the alternative plastic theories to the usual coefficient and finite element method of analysis and design method. Plastic analysis methods are derived from the general theory structural plasticity, which states that the collapse load of a structure lies between two limits, an upper and a lower bound of the true collapse load applied to the slabs of any shape, any support condition, uniformly or partially and none uniformly distributed loads, slabs with holes of any size and so on.

1.4 Design Specifications and Constants

- ✓ Purpose – Residensial Building
- ✓ Approach- Limit state design method

1.4.1 Material used and Properties

Material selected to design the slab structure

- ✓ Concrete grade of C20/25
- ✓ Steel grade of S-400
- ✓ Unit weight of concrete, $\gamma c = 25 \mathrm{KN/m^3}$

Design strength value

a) Concrete

$\gamma c = 1.5$

$fctd = 1.03mpa$

$fctm = 2.2$

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

Elastic modulus of concrete, $Ec = 29Gpa$

b) Steel

$\gamma s = 1.15$

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

Elastic modulus of steel, Es=200Gpa

1.4.2 Design Aids used for this paper

Method (Approach) = Limit state design method

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

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

The codes serve at least four distinct functions:

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

2 Roof Analysis and Deign

2.1 General

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

2.2 Wind load analysis

Structures deflect or stop the wind, converting the wind's kinetic energy into potential energy of pressure-creating load. The action of wind can be of the type of suction or pressure to our structures both externally or internally. We consider in our structural project, the design of G+4 residential building for living purpose located in Gubre.

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

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

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

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

Table 2.1 necessary data of wind load analysis

Roof type	Flat roof
Location	Gubre
Building height	18.9m
Terrain category	**Category III**: **-** for area with regular cover of vegetation or buildings with obstacles with separations of maximum 20 obstacle heights(such as villages, sub urban terrain, permanent forest)
Directional factor	1**:** recommended value
Seasonal factor	1: recommended value

Basic Wind velocity	22m/**s**: for Ethiopia
Z_0	0.3m: for terrain category III from table
$Z_{0.II}$	0.05m**:** for (terrain category II)
Minimum height	5m: for terrain category III from table
Maximum height	200m
Topography factor	1: recommended
Turbulence factor	1: recommended
Air density	1.25 kg/m^3: recommended
Reference height	15m: for h < b

Determination of basic wind velocity V_b

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

Where $C_{dir} = directional\ factor$

$C_{sea} = seasonal\ factor$

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

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

Roughness factor $C_r(Z)$

- The roughness factor accounts for the variability of the mean wind velocity at the site of the structure due to:

- The height above ground level

-The ground roughness of the terrain upwind of the structure in the wind direction considered

$$C_r(Z) = K_r \ln\left(\frac{Z}{Z_0}\right) \quad for\ Z_{min} \leq Z \leq Z_{max}$$

$$C_r(Z) = C_r(Z_{min}) \quad for\ Z \leq Z_{min}$$

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

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

Z_{min} = Minimum height

Z_{max} = Maximum height taken as 200m

Z_0, Z_{min} depend on the terrain category. Recommended values are given in ES EN: 2015 Table 4.1 depending on five representative terrain categories.

Table 2.2 Terrain category

Terrain category	Z_0 m	Z_{min} m
0 Sea or coastal area exposed to the open sea	0.003	1
I Lakes or flat and horizontal area with negligible vegetation and without obstacles	0.01	1
II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle height	0.05	2
III Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0.3	5
IV Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1.0	10
The terrain categories are illustrated in Annex A.1.		

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

$$C_r(Z) = K_r \ln\left(\frac{Z}{Z_0}\right) \qquad for\ 5\ \leq 18.9 \leq\ 200$$

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

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

$$= 0.215 ln\left(\frac{18.9}{0.3}\right)$$

$$C_r(Z) = \mathbf{0.891}$$

Mean wind velocity $V_m(Z) = C_r(Z) * C_0(Z) * V_b$

Where, $C_r(Z)$ = Roughness factor

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

$V_b = basic\ wind\ velocity$

$V_m(Z) = 0.891 * 1 * 22\ m/s$ = **19.60 m/s**

Wind turbulence $l_V(Z)$

The turbulence intensity $l_V(Z)$ at height Z is defined as the standard deviation of the turbulence divided by the mean wind velocity

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

Where, $K_l = turbulence\ factor$

$C_0(Z) = orography\ factor$

$$l_V(Z) = \frac{1}{1 * ln\left(\frac{18.9}{0.3}\right)} = \mathbf{0.24}$$

Peak velocity pressure $q_p(Z)$

The peak velocity pressure $q_p(Z)$ at height z, which includes mean and short-term velocity fluctuations, should be determined.

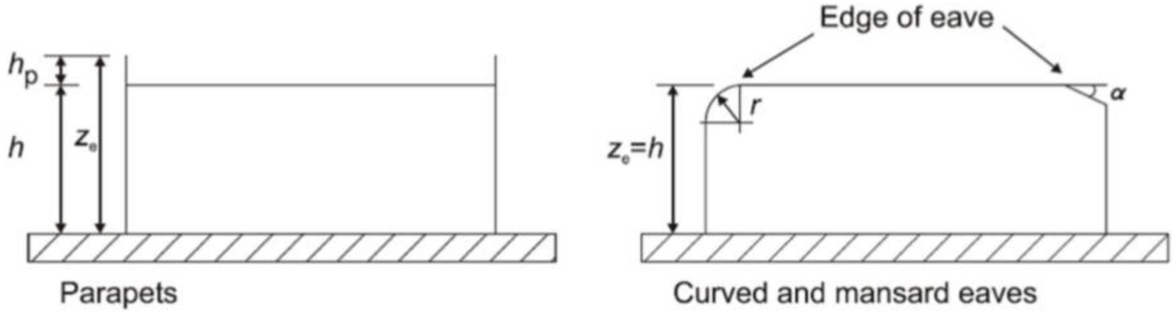
$$q_p(Z) = [1 + 7 * l_V(Z)] * \frac{1}{2} * \rho * {V_m}^2(Z)$$

$where\ \rho = air\ density$ The recommended value is 1.25 kg/m3.

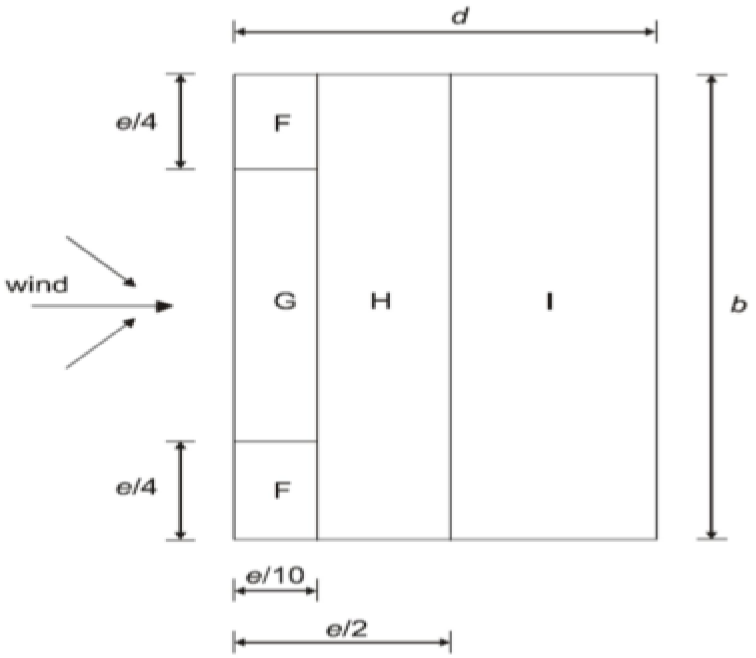
$$q_p(Z) = [1 + 7 * 0.24] * \frac{1}{2} * 1.25 * 19.60^2 = 643.47N = 0.643KN$$

Note:-The reference height for flat roof and roof with curved or mansard should be taken as h.
The reference height for flat roof and roof with parapet should be taken as $h + h_P$

Figure 2.1 Roof reference height



Case -



$e = b \; or \; 2 * h$

b = 10.475 or (2*18.9 = 37.80)

$e = 10.475$

Whichever is smaller

b –cross wind direction

d = 9.96

$\frac{e}{2} = 5.24$

$\frac{e}{4} = 2.62$

$\frac{e}{10} = 1.05$

$$h_P = 0.9, \quad hp/h = 0.05, \; \text{and} \; h = 18$$

Determination of area for case 1

Area of F $= 2.62 * 1.05 = 2.75 \; m^2$

Area of G $= 5.24 * 1.05 = 5.5 \; m^2$

Area of H $= 4.19 * 10.475 = 43.89 \; m^2$

Area of I $= 4.72 * 10.475 = 49.44 \; m^2$

Table 2.3 Roof type

Roof type		Zone							
		F		G		H		I	
		cpe,10	cpe,1	cpe,10	cpe,1	cpe,10	cpe,1	cpe,10	cpe,1
Sharp eaves		-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	+0,2 -0,2	
With Parapets	$h_p/h=0,025$	-1,6	-2,2	-1,1	-1,8	-0,7	-1,2	+0,2 -0,2	
	$h_p/h=0,05$	-1,4	-2,0	-0,9	-1,6	-0,7	-1,2	+0,2 -0,2	
	$h_p/h=0,10$	-1,2	-1,8	-0,8	-1,4	-0,7	-1,2	+0,2 -0,2	

External pressure coefficients for flat roofs

Wind pressure computation

Determination of External and Internal Pressure Coefficient

The wind pressure acting on the external surfaces, W_e

$$W_e = q_p(Z_e)C_{pe}$$

$C_{pe} = C_{pe},1$…………………………………..……….$A \leq 1m^2$

$C_{pe} = C_{pe},1 + (C_{pe},10 - C_{pe},1)\ln A$…….…........$1m^2 < A < 10m^2$

$C_{pe} = C_{pe},10$……………..…………..…………….$A \geq 10m^2$

Table 2.4 Region

Region	F		G		H		I	
C_{pe}	$C_{pe},10$	$C_{pe},1$	$C_{pe},10$	$C_{pe},1$	$C_{pe},10$	$C_{pe},1$	$C_{pe},10$	$C_{pe},1$
$W_e = 0.643 * C_{pe}$	-0.79		-0.98		-0.7		+0.2	
							-0.2	
	+0.0		+0.0		+0.0		+0.2	
	-0.8		-0.93		-0.74		-0.2	

Critical Pressures are -0.93

0.21

The wind pressure acting on the internal surfaces of a structure, W_i

$$W_i = q_p(Z_i)C_{pi}$$

Table 2.5 Region

Region	F	G	H	I
$W_i = 0.643 * C_{pe}$	C_{pe}	C_{pe}	C_{pe}	C_{pe}
	+0.8	+0.8	+0.8	+0.8
	-0.5	-0.5	-0.5	-0.5
	+0.51	+0.51	+0.51	+0.51
	-0.32	-0.32	-0.32	-0.32

Critical Pressures are -0.53

0.84

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

Zone	External pressure (W_e)	Internal pressure (W_i)	Net pressure $W_{net} = W_e - W_i$
G	-0.93	-0.32	-0.61
I	+0.21	+0.51	-0.30

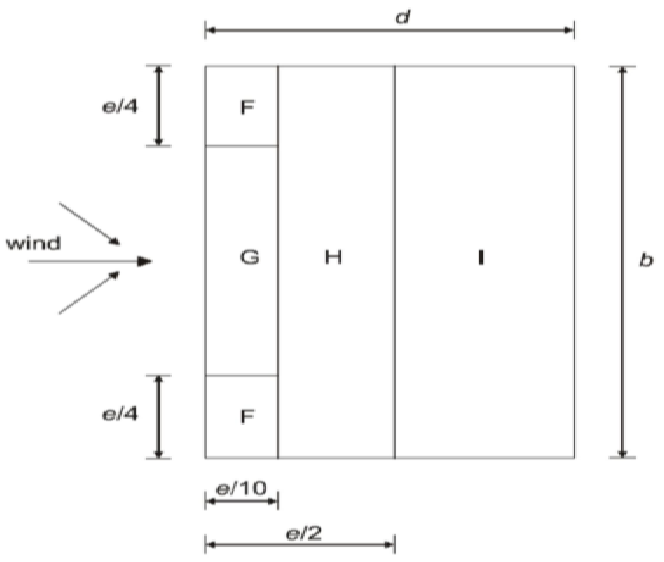
Hence, the net pressure with reference to the external pressure

Table 2.6 Region

Zone	Area	W_{net}	Wind Load
F	2.75	-0.63	-0.23
G	5.5	-0.63	-0.11
H	43.8	-0.63	-0.014
I	49.44	-0.63	-0.0127

g

Case -2



$hp = 0.9,$ $hp/h = 0.05$ $h = 18$

Determination of area for case 2

$$\text{Area of F} = 0.97 * 2.42 = 2.35\text{m}^2$$

$$\text{Area of G} = 4.84 * 0.97 = 4.69\text{m}^2$$

$$\text{Area of H} = 3.87 * 9.675 = 37.06\text{m}^2$$

$$\text{Area of I} = 16.91 * 9.675 = 163.2\text{m}^2$$

The wind pressure acting on the external surfaces, W_e

$\mathrm{W_e} = \mathrm{q_p}(\mathrm{Z_e})\mathrm{C_{pe}}$

$\mathrm{C_{pe}} = \mathrm{C_{pe}}, 1$……………………………….…….$A \leq 1m^2$

$\mathrm{C_{pe}} = \mathrm{C_{pe}}, 1 + (\mathrm{C_{pe}}, 10 - \mathrm{C_{pe}}, 1)\ln A$………..... $1m^2 < A < 10m^2$

$\mathrm{C_{pe}} = \mathrm{C_{pe}}, 10$…………..……………………….$A \geq 10m^2$

Region

Region	F		G		H		I	
C_{pe}	$C_{pe}, 10$	$C_{pe}, 1$	$C_{pe}, 10$	$C_{pe}, 1$	$C_{pe}, 10$	$C_{pe}, 1$	$C_{pe}, 10$	$C_{pe}, 1$
$\mathrm{W}_e = 0.643 * \mathrm{C_{pe}}$	-0.79 +0.0 -0.8		-0.98 +0.0 -0.93		-0.7 +0.0 -0.74		+0.2 -0.2 +0.2 -0.2	

Critical Pressures are -0.93

0.21

The wind pressure acting on the internal surfaces of a structure, W_i

$$W_i = q_p(Z_i)C_{pi}$$

Table 2.7 Cpe

Region	F	G	H	I
$\mathrm{W_i} = 0.643 * \mathrm{C_{pe}}$	C_{pe}	C_{pe}	C_{pe}	C_{pe}
	+0.8	+0.8	+0.8	+0.8
	-0.5	-0.5	-0.5	-0.5
	+0.84	+0.84	+0.84	+0.84
	-0.53	-0.53	-0.53	-0.53

Critical Pressures are -0.53

0.84

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

Zone	External pressure (W_e)	Internal pressure (W_i)	Net pressure $W_{net} = W_e - W_i$
G	-0.93	-0.32	-0.61
I	+0.21	+0.84	-0.63

Hence, the net pressure with reference to the external pressure

Table 2.8 Neat pressure

Zone	Area	W_{net}	Wind Load
F	2.35	-0.63	-0.21
G	4.69	-0.63	-0.3
H	37.06	-0.63	-0.017
I	163.2	-0.63	-0.0039

Total design load for roof slab

Dead load

Water proofing bitumen $= 0.02\text{m} * 17\text{KN/m3} = 0.28\text{KN/m}^2$

Cement screed load $=$ cement screed thickness $*$ unit weight $= 0.03\text{m} * 23\text{KN/m3}$
$= 0.69\text{KN/m}^2$

Weight of RC slab $=$ slab thickness $*$ unit weight $= 0.17\text{m} * 25\text{KN/m3} = 4.25\text{KN/m}^2$

Ceiling plaster load $=$ ceiling thickness $*$ unit weight $= 0.02\text{ m} * 23\text{ KN/m3} = 0.46\text{KN/m}^2$

Live load = 0.5 KN/m^2 for Category Terrace

2.3 Roof Load Combinations

Combination for F Pd =1.35DL+1.5LL+0.9WL

= (1.35*5.68) + (1.5*0.5) + (0.9*(-0.27)) = 8.17KN/m^2

Combination for G Pd = 1.35DL+1.5LL+0.9WL

= (1.35*5.68) + (1.5*0.5) + (0.9*(-0.13)) = 8.317KN/m^2

Combination for H Pd = 1.35DL+1.5LL+0.9WL

= (1.35*5.94) - (1.5*0.4+ (0.9*(-0.017)) = 8.417KN/m^2

Combination for I Pd = 1.35DL+1.5LL+0.9WL

$= (1.35 * 5.94) + (1.5 * 0.4 + (0.9 * (-0.0127))$ = 8.4117KN/m^2

Therefore, the governing combination is combination – **I**

Wind load = 8.41

Concrete cover determination

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

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

$c_{min,b}$ = diameter of bar which is Φ = 12mm

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

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

$\Delta c_{dur,st}$ = 0mm, EBCS EN: 1990: 2015 Recommended

$\Delta c_{dur,add}$ = 0mm, EBCS EN: 1990: 2015 Recommended

Δc_{dev} = 10mm, EBCS EN: 1990: 2014 Recommended

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

Therefore: c_{min} = max {12mm; 10mm+0 - 0 - 0; 10mm} take c_{min} = 12mm

$c_{nom} = c_{min} + \Delta c_{dev}$ = 10mm + 10mm = 20 mm

<u>Panel-1</u>

Example $3780/(17.71 * 1.3 * 1.25) = 131.3412\text{mm}$

D = $131.3412\text{mm} + 20\text{mm} + 10\text{mm}/2 = 156.3412\text{mm}$

Provide D = 160mm

Table 2.9 Depth

Panel	Lx	Ly	N	K	F1	F2&F3	d (mm)	D (mm)
P1	3780	4720	17.1	1.3	1.25	1	131.3412	156.3412
P2	3780	4790	17.1	1.3	1.25	1	131.3412	156.3412
P3	4720	6530	17.1	1.3	1.25	1	164.0028	189.0028
P4	4790	6530	17.1	1.3	1.25	1	166.435	191.435
P5	3780	3900	17.1	1.3	1.25	1	131.3412	156.3412
P6	3900	6520	17.1	1.3	1.25	1	135.5108	160.5108
P7	5390	5630	17.1	1.3	1.25	1	168.1513	193.1556
P8	3780	5620	17.1	1.3	1.25	1	131.3412	156.3412
P9	5620	6520	17.1	1.3	1.25	1	170.2745	195.2745
P10	5620	5630	17.1	1.3	1.25	1	170.2745	194.2745

P11	3880	6480	17.1	1.3	1.25	1	134.8158	159.8158
P12	3880	4530	17.1	1.3	1.25	1	131.3412	156.3412

$D_{max} = 195.2745$

$D_{provided} = 200$

Hence, panel -9 governs the effective depth of the solid slab.

Design load calculation

Design load for panel-1

$\text{Water proofing bitumen} = 0.02\text{m} * 14\text{KN/m3} = 0.28\text{KN/m}^2$

$\text{Cement screed load} = \text{cement screed thickness} * \text{unit weight}$

$= 0.03\text{m} * 23\text{KN/m3} = 0.69\text{KN/m}^2$

$\text{Weight of RC slab} = \text{slab thickness} * \text{unit weight}$

$= 0.17\text{m} * 25\text{KN/m3} = 4.25\text{KN/m}^2$

$\text{Ceiling plaster load} = \text{ceiling thickness} * \text{unit weight}$

$= 0.02\text{ m} * 23\text{ KN/m3} = 0.46\text{KN/m}^2$

Wind load = 8.41KN/m^2

$\text{Total dead load} = \text{water proofing bitumen load} + \text{cement screed load} + \text{RC slab load} + \text{ceiling plaster load}$

$= 0.28 + 0.69 + 4.25 + 0.46 + 8.41 = 14.09\text{KN/m}^2$

LL = 0.5 KN/m^2 for C 25 – Category Terrace

Similarly for other panels, in tabulated form in Apendax 2B

2.4 Moment and Shear force Analysis

The panels of the slabs are analyzed using the coefficient method of analysis according to the newly revised Ethiopian building code standard.

Two way slabs are analyzed using coefficient method if the following conditions are satisfied.

Let us do of for panel-1or sample and others are done by excel.

Panel -1

$a_{xs} = 0.066 \qquad L_x{}^2 = 14.29 \qquad Pd = 19.77\text{KN/m}^2 \qquad M_{xs} = 18.646\text{KNm/m}$

$a_{xf} = 0.049$ $L_x^2 = 14.29$ $Pd = 19.77KN/m^2$ $M_{xf} = 13.843KNm/m$

$a_{ys} = 0.047$ $L_x^2 = 14.29$ $Pd = 19.77KN/m^2$ $M_{ys} = 12.73KNm/m$

$a_{yf} = 0.036$ $L_x^2 = 14.2$ $Pd = 19.77KN/m^2$ $M_{yf} = 9.605KNm/m$

Roof solid slab moment determination

Similarly for other panel we summarize in table as follow
Apendax 2C

3 Slab Design

3.1 Introduction

A reinforced concrete slab is a broad, flat plate, with top and bottom surfaces parallel. It is used to provide flat surfaces mainly for roofs and floors of buildings, parking lots, roadways and so on. It may be supported by reinforced concrete beams, by masonry or reinforced concrete or continuously by the ground Beams (and is poured monolithically with such walls, by structural steel members, directly by column or continuously by the ground.

3.2 Overview of slab analysis and design procedures

Slabs support area loads. The loads carried by slabs are of two types. These are live load and dead load.

Live load

Live load or imposed loads are loads on the structure are those arising from occupancy. They depend on the particular function of the slab. To determine the live load on a slab, we first determine its occupancy or function. EBCS EN 1991-1-1 2015 page 12 Table 6.1 puts possible building occupancies in different Categories. EBCS EN 1991-1-1-2015 page 13 also gives the respective live load value for each category on page 13 Table 6.2 We will take this area loads and place them on each slab according to their category.

Table 3.1 Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m²]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

Table 3.2 Categories of use

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3**: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1**: Areas in general retail shops **D2**: Areas in department stores

Dead load

These loads are come from materials that are permanently attached to the slab. Some of the sources of dead load are shown on figure below and are the following

- Slab it-self
- Celling plaster (below slab)
- Cement screed (above slab)
- Finishing material (Above slab)
- Exterior and interior walls resting on the slab

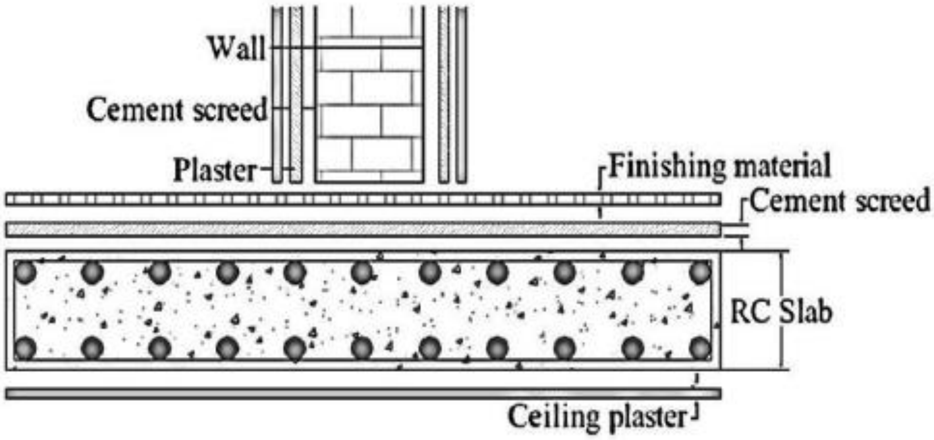


Figure 3.1: Slab section

For the walls, we first check whether the factored area load from the wall exceeds 20 % of the design load without wall load. If this wall load does not exceed 20% of the design load no need to check shear capacity of slab. However, if the wall load exceeds 20% of the design load checks shear capacity of the slab. After determining both dead load and live load, we will calculate the design load by applying safety factors.

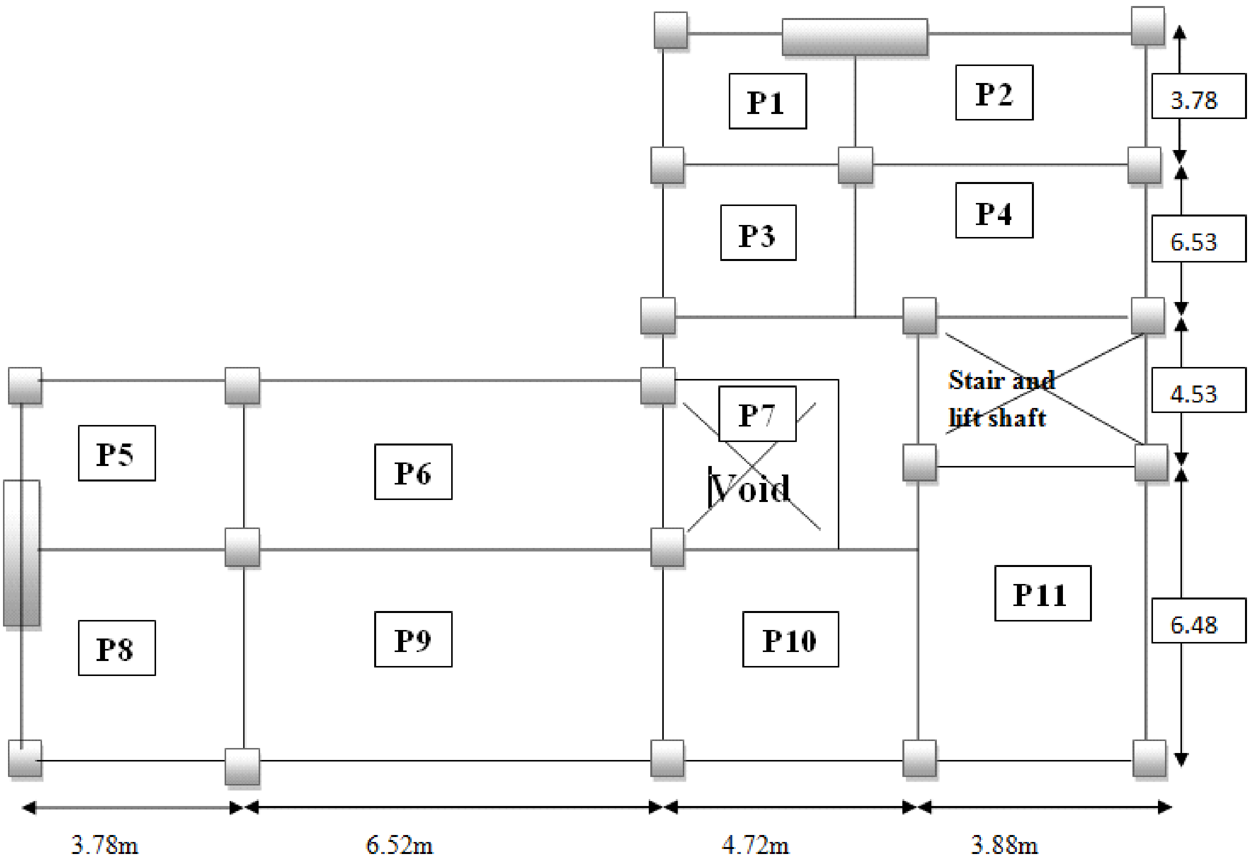


Figure 3.2 Structural layout

3.3 Depth determination (Deflection criteria)

Generally, it is not necessary to calculate the deflections explicitly as simple rules, for example limits to span/depth ratio may be formulated, which will be adequate for avoiding deflection problems in normal circumstances.

The limiting span/depth ratio may be estimated using Expressions shown below and multiplying this by correction factors to allow for the type of reinforcement used and other variables, from EBCS EN: 1992-1-1:2015, Sec.7.4

For initial sizing of slabs and beams the engineer has to rely on conservative guesswork, as the allowable span/depth ratio cannot be checked until the reinforcement design is complete.

The minimum depth of a slab for deflection requirement is computed by:

$$\frac{\mathrm{l}}{\mathrm{d}} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if \;\; \rho \leq \rho_o$$

$$\frac{\mathrm{l}}{\mathrm{d}} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if \;\; \rho \geq \rho_o$$

$\rho' = 0$, because in slab there is no negative reinforcement at mid span.

Where:

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

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

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

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

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

fck - is in MPa units

Equations 7.16.a and 7.16.b can only be applied at the end of the design, after the reinforcement has been designed, so for scheme design the engineer is forced to rely on guesswork.

Assume that $\rho = \rho o$ and equation

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

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

$$N = 11 + 1.5 * 20\text{^}0.5 = 17.71$$

$F1 = \frac{500}{fyk} = \frac{500}{400} = 1.25$

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

Basic ratios of span/effective depth for reinforced concrete members without axial compression table 7.4N the value of K for end span is 1.3, for interior span is 1.5, and for cantilever is 0.4.

End span ($K = 1.3$)

$$\frac{l}{d} = 17.71 * 1.3 * 1.25 * 1 * 1 = 28.78 \quad where\ l = lx - \text{short span}$$

Interior span ($K = 1.5$)

$$\frac{l}{d} = 17.71 * 1.5 * 1.25 * 1 * 1 = 33.21$$

Cantilever ($K = 0.4$)

$$\frac{l}{d} = 17.71 * 0.4 * 1.25 * 1 * 1 = 8.86\ and\ \ d = Lx/(K * N * F1 * F2 * F3)$$

$D = d_{min} + C_{nom} + \varphi l/2$ Where, d_{min} is governing effective depth.

Example

Panel-1

$\frac{l}{d} = 28.78mm$

$l = L_x = 378$ mm

$d = 3780/28.78 = 131.34mm$

$D = 131.34 + 20 + 10/2 = 156.34$

Provide D = 160

Table 3.3 Basic ratios of span/effective depth for reinforced concrete

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	1,0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1,3	18	26
Interior span of beam or one-way or two-way spanning slab	1,5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1,2	17	24
Cantilever	0,4	6	8

Concrete cover determination

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

Where,

c_{nom} - is the nominal cover

c_{min} - is the minimum cover

Δc_{dev} - is an allowance in design for deviations

Minimum cover, c_{min}

- Minimum concrete cover, c_{min}, shall be provided in order to ensure the safe transmission of bond forces
- Protections of the steel against corrosion (durability)
- An adequate fire resistance,
- It is a principle that the greater value for c_{min} satisfying the requirements for both bond and environmental conditions should be used.

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

Where

$c_{min,b}$ -is the minimum cover due to bond requirement.

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

$\Delta c_{dur,\gamma}$ - is additive safety element

$\Delta c_{dur,st}$ -is reduction of minimum cover for use of stainless steel $\Delta c_{dur,add}$ is reduction of minimum cover for use of additional protection

In order to transmit bond forces safely and to ensure adequate compaction of the concrete, the minimum cover should not be less than $c_{min,b}$ given in table 4.2 of ES-EN 1992-11:2015 page 32 & as shown below

Table 3.4 Minimum cover, Cmin, b, requirements with regard to bonding

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.	

Assuming separated arrangement of bars and for 20mm aggregate size

$$c_{min,b} = \text{diameter of bar which is } \Phi = 12\text{mm}$$

- Values of minimum cover, $c_{min\,,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

- The recommended Structural Class (for design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N ES-EN1992-1-1:2015. The recommended minimum Structural Class is S1.

Table 3-5: Values of minimum cover, $c_{min\,,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080

Table 3.5 Values of minimum cover, Cmin,dur, requirements

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

C_{dur}, dur = 10mm, with exposure class of Xo with S-4 for 50-year life time

Allowance in design for variation

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

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

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

$\Delta c_{dur,st}$ = 0mm, EBCS EN: 1990: 2015 Recommended

$\Delta c_{dur,add}$ = 0mm, EBCS EN: 1990: 2015 Recommended

Δc_{dev} = 10mm, EBCS EN: 1990: 2014 Recommended

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

Therefore: c_{min} = max {12mm; 10mm+0 - 0 - 0; 10mm} take c_{min} = 12mm

$c_{nom} = c_{min} + \Delta c_{dev}$ = 10mm + 10mm = 20 mm

Example

Panel-1

L/ d = 28.78mm

L = Lx = 378 mm

d = 3780/28.78 = 131.34mm

D = 131.34 + 20 + 10/2 = 156.34

Provide D = 160

Table 3.6 Effective Depth

Panel	Lx	Ly	N	K	F1	F2&F3	d (mm)	D (mm)
P1	3780	4720	17.1	1.3	1.25	1	131.3412	156.3412
P2	3780	4790	17.1	1.3	1.25	1	131.3412	156.3412
P3	4720	6530	17.1	1.3	1.25	1	164.0028	189.0028
P4	4790	6530	17.1	1.3	1.25	1	166.435	191.435
P5	3780	3900	17.1	1.3	1.25	1	131.3412	156.3412
P6	3900	6520	17.1	1.3	1.25	1	135.5108	160.5108

P7	5390	5630	17.1	1.3	1.25	1	168.1513	193.1556
P8	3780	5620	17.1	1.3	1.25	1	131.3412	156.3412
P9	5620	6520	17.1	1.3	1.25	1	170.2745	195.2745
P10	5620	5630	17.1	1.3	1.25	1	170.2745	194.2745
P11	3880	6480	17.1	1.3	1.25	1	134.8158	159.8158
P12	3880	4530	17.1	1.3	1.25	1	131.3412	156.3412

$D_{max} = 195.2745$

$D_{provided} = 200$

3.3.1 Analysis and Design of load

$$Pd = 1.35DL + 1.5LL$$

Where: P_d = total factored design load, DL = total dead load on slab, LL = total live load on slab

The dead load and live load are referred from EBCS EN 1991-1-1- 2014

Dead Load

The following table containing the unit weight of different construction material.

Table 3.7 Unit weight of material

Floor f.	Unit wt.	Thickness	Thickness of mortar	Load
Clay Tile	21	0.04	0.01	1.07
Marble	27	0.02	0.03	1.23
Parquet	9	0.01	0.04	1.01
PVC	16	0.002	0.048	1.136
Rubble	17	0.01	0.04	1.09
Terrazzo	23	0.03	0.02	1.15
Ceramic	23	0.01	0.04	1.15
Mortar	23		0	0
Porcelain	23	0.03	0.048	1.794
RC	25			0

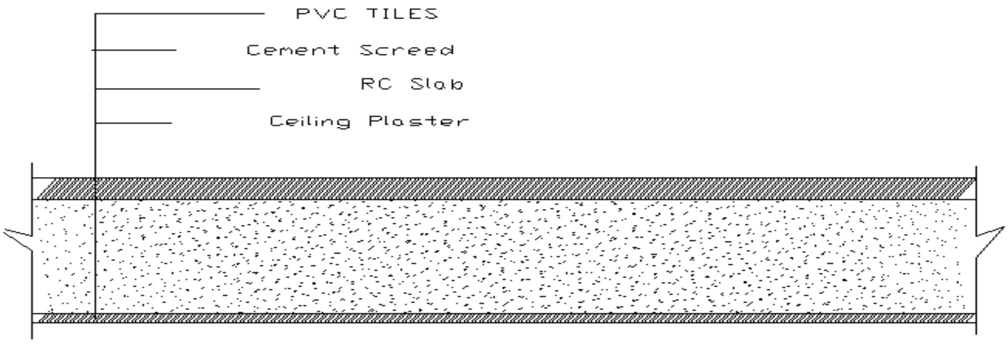


Figure 3.3 Sectional of floor slabs

Design load for panel-1

Ceramics load = ceramic thickness*unit weight of ceramics

= 0.02m*23m^2 = 0.46KN/m^2

Cement screed load = cement screed thickness*unit weight

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

Weight of RC slab = slab thickness*unit weight

= 0.125m*25KN/m^3 = 3.13 KN/m^2

Ceiling plaster load = ceiling thickness*unit weight

= 0.02 m*23 KN/m^3 = 0.46 KN/m^2

Partition load = thickness*length*height *unit weight of HCB/area of panel

= 0.15m*3.34m*3.05m*12(KN/m^3)/17.84 m^2 =1.023KN/m^2

Wall plaster load = 2* thickness*length*height *unit weight of plaster/area of panel

= 2*0.025m*3.34m*3.05m*23(KN/m^3)/17.84 m^2 = 0.65KN/m^2

= ∑ 7.62 KN/m^2

DL = 0.46 + 0.69+ 3.13 + 0.46 + 1.02+0.65 = 7.62 KN/m^2

Live load

Table 3.8 Live Load for different functions

Function	Category		Live Load
Kitchen	A	General	2 KN/m^2
Cafeteria	C	C1	3 KN/m^2
Shop	D	D1	5 KN/m^2
Corridor	C	C3	5 KN/m^2
Balcony	A	balconies	4 KN/m^2
Toilet	A	General	2 KN/m^2
Shower	A	General	2 KN/m^2
Landing	A	Stairs	3 KN/m^2
Office	B	Office	3 KN/m^2
Bed Room	A	General	2 KN/m^2

Category-A Bed Room

Live load (LL) = 2 KN/m^2

Design load, Pd = 1.35DL + 1.5LL

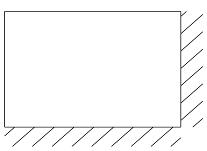
Panel -1

Pd = 1.35*7.62 + 1.5*2 = 13.29

Similarly for other panels, in tabulated form Appendaix 3B

3.3.2 Design Moment Analysis

Slab moment is calculated using the coefficient method from EBCS-2, 1995 as follow



⍰ $L_X = 3.78m;\ L_Y = 4.72m$

$$L_Y / L_X = 4.72/3.78 = 1.25$$

The moment equation is given by the following formula from [EBCS-2, 1995 section A.3.2]

$$M_i = \alpha_i PdLx^2$$ EBCS-2, 1995, Page 108

Where

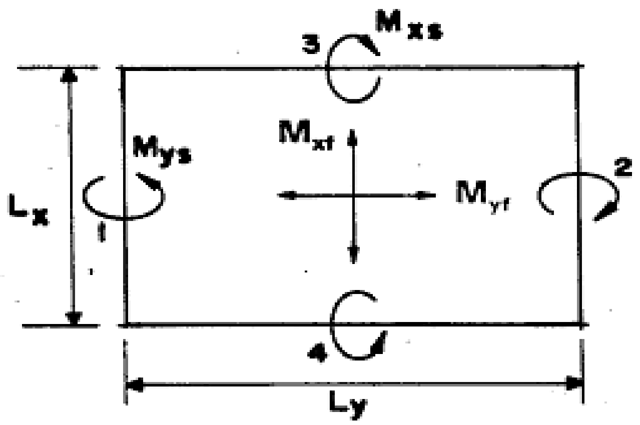
M_i = Design moment per unite width at a point of reference

Pd = Uniformly Distributed Design Load

α_i = Coefficient given in EBCS-2, 1995, section A.3.3 table A-1 as function of aspect ratio L_y/L_x and support condition.

L_x - is the shorter span of panel

L_y - is the longer span of the panel



Panel -1

$a_{xs} = 0.066$ $\quad L_x^2 = 14.29$ $\quad Pd = 13.29KN/m^2$ $\quad M_{xs} = 12.53\ KNm/m$

$a_{xf} = 0.049$ $\quad L_x^2 = 14.29$ $\quad Pd = 13.29KN/m^2$ $\quad M_{xf} = 9.31\ KNm/m$

$a_{ys} = 0.047$ $\quad L_x^2 = 14.29$ $\quad Pd = 13.29KN/m^2$ $\quad M_{ys} = 8.93\ KNm/m$

$a_{yf} = 0.036$ $\quad L_x^2 = 14.29$ $\quad Pd = 13.29KN/m^2$ $\quad M_{yf} = 6.84\ KNm/m$

Panel-10

According to EBCS-2, 1995 section A.3.3 $M_{xs,\ int}$ for this slab should be interpolated between fully continuous and fully discontinuous edges based on the ratio of the supported length to the unsupported length of the edge

Continuous

$a_{xs,\ cont} = 0.039$ $\quad M_{xs,\ cont} = 0.039 * 21.63 * 5.62^2 = 26.64\ KNm/m$

Discontinuous

$a_{xs,\ discont} = 0.00$ $\quad M_{xs,\ discont} = 0$

By interpolating between continuous and discontinuous the moment at support is calculated.

Ratio of discontinuous to continuous length is $= 1.82/3.81 = 0.478$

1……………….26.64	$(26.64 - M_{xs,\ int})/(1 - 0.478) = (26.64 - 0)/(1 - 0)$
0.478……..……..$M_{xs,\ int}$	$M_{xs,\ int} = 12.73KNm/m$
0……………..………0	

Similarly for other panel we summarize in table as follow Appendaix 3C

3.3.3 Adjustment of Moment

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

Two methods are used to perform the unbalanced support moment, according to EBCS-2, 1995 section A.3.3.

Method-I

When the difference between initial support moments is less than 20 percent of the large moment .In this method dimensioning is normally carried out either using:-

a) Initial moments directly, or

b) Based on the average initial moment at the moments.

Method-II

Procedure for applying moment-**II** is as follows:-

a) Support and span moments are first calculated for individual panel, by assuming each panel fully loaded. That is calculated above in our cases.

b) The unbalanced moment is distributed using moments distribution method. The relative stiffness of each panel shall be taken proportional to its gross moments of inertia divided by the smaller span.

c) If the support moment is decreased, the span moments M_{xf} and M_{yf} are then increased to allow for the changes of support moments. If a support moment is increased, no adjustment shall be made to the span moment

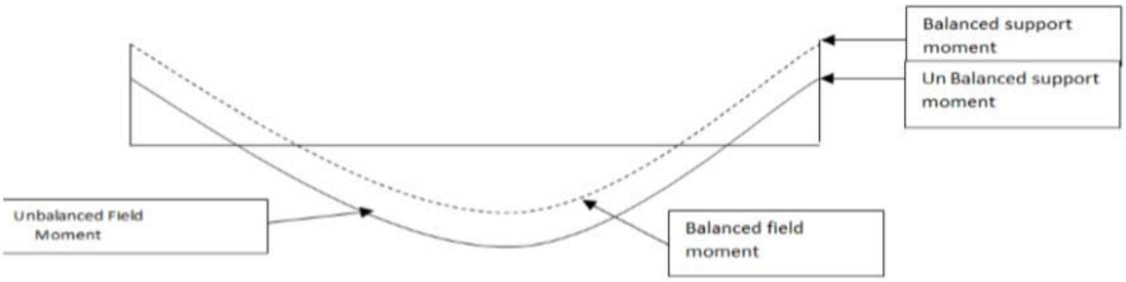


Fig.1.3 Notation of critical moments

$\Delta M_{xf} = C_x + \Delta M \longrightarrow M_{xf}^* = M_{xf} + \Delta M_{xf}$

$\Delta M_{yf} = C_y \Delta M \longrightarrow M_{yf}^* = M_{yf} + \Delta M_{yf}$

Where:-

ΔM_{xf} –change in field moment along short direction

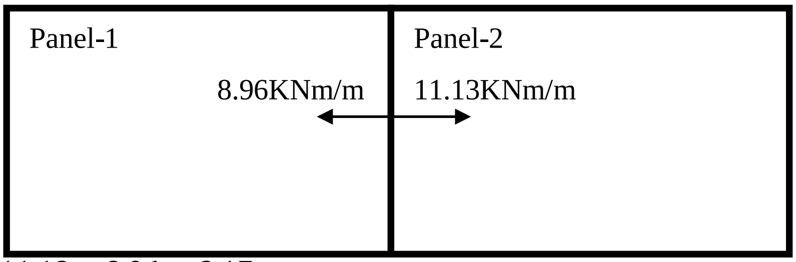
ΔM_{yf} –change in field moment along long direction

M_{xf}^* –adjusted field moment along short direction

M_{yf}^* –adjusted field moment along long direction

C_x & C_y –factor for adjusted span moment in the short and long direction respectively from EBCS-1995, section A.3.3 table A-2.

Panel-1 and panel-2

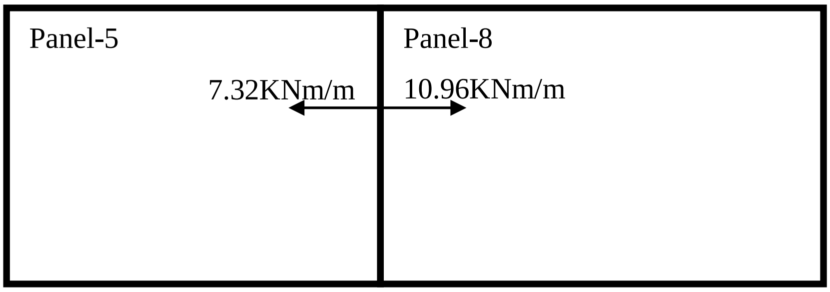


$\Delta M = 11.13 - 8.96 = 2.17$

$\Delta M / M_{large} = 2.17/11.13 = 0.195 * 100 = \mathbf{19.5}\% < 20\%$, hence averaging of moment is used Using Method-I

$$M_{average} = \frac{11.13 + 8.96}{2} = 10.045 \text{KNm/m}$$

Panel-5 and panel-8

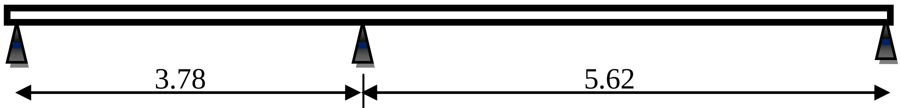


$M_L = 10.96 \qquad M_s = 7.32$

$\Delta M = 10.96 - 7.32 = 3.64$

$\Delta M/M_{large} = 3.64/10.96 = 0.33 * 100 = \mathbf{33}\% > 20\%,$

Hence adjustment by moment redistribution is required. Using Method-II



K	0.198	0.133
D.F	0.598	0.401
FEM	-7.32	10.96
	-2.177	-1.460
FM	-9.50	9.50

Balanced or adjusted support moment Since10.96 > 9.5 (balanced moment) field moment adjustment is required for panel-8

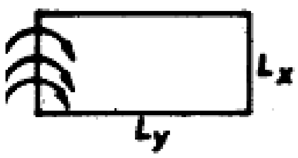
$$D.F = \frac{(K_{large})}{K_{large} + K_{small}}$$

$$\frac{\frac{3I}{4L}}{\frac{3I}{4L} + \frac{3I}{4L}} = \frac{\frac{3I}{4*5.62}}{\frac{3I}{4*5.62} + \frac{3I}{4*3.78}} = 0.401$$

$M_{adj} = M_{large} - D.F * \Delta M$

$10.96 - (0.401 * 3.64) = 9.5$

Field moment of Adjusted



L_y/L_x=5.62/3.78=1.487m

To calculate factor for adjusted span moment by interpolation $C_x = 0.306 \qquad C_y = 0.096$

$\Delta M = M_{large} - M_{balence}$

10.96-9.5= 1.46

$\Delta M_{x\,f} = C_x * \Delta M = 0.306 * 1.46 = 0.447$

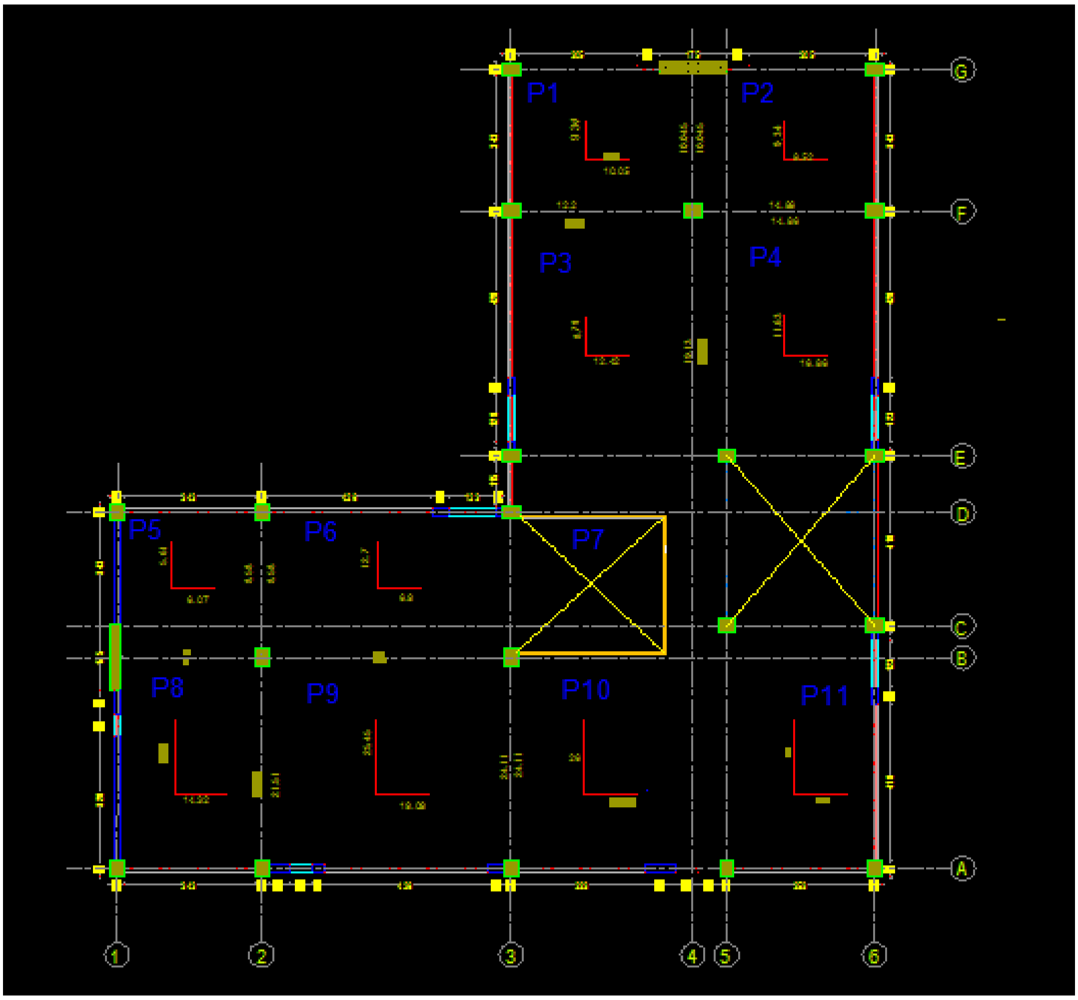
$\Delta\, M_{y\,f} = C_y * \Delta\, M = 0.096 * 1.46 = 0.140$

Adjusted field moment of panel-8

$M_{xf}^{*} = M_{x\,f} + \Delta\, M_{x\,f} = 13.76 + 0.56 = 14.32 \text{KNm/m}$

$M_{yf}^{*} = M_{y\,f} + \Delta\, M_{y\,f} = 8.40 + 0.140 = 8.54 \text{ K N m/m}$ Appendaix 3D

Adjusted moment



Strip Method of slab Analysis of (slab with opening, Panel-7)

This panel is slab with large opening and it has to be treated more rigorously. The strip method, offers a rational and safest basis for designing such cases. Integral load bearing beam are combined along the edge of the opening usually having the same depth as the remainder of the slab but with extra reinforcement to peak up the load the affected region and to transmit the load to the supports. These integral beams should be chosen so as to carry the loads most directly to be supported edge of the slab. The width of strong band should be selected so that the reinforcement ratio at or below the values required to produce a tension control member. Doing so, we ensure ductile behavior of the slab. This strategy makes the slab part at the unsupported edge to be costly, since we provide a large amount of reinforcement in the strong band. To optimize the cost, the strong band should have a minimum width.

Panel-7

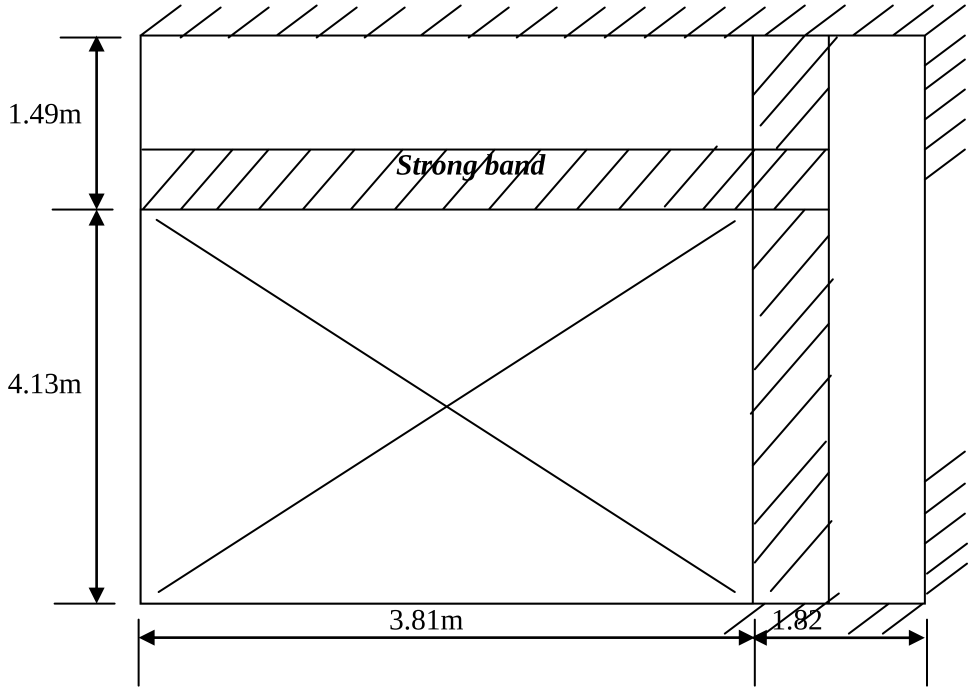
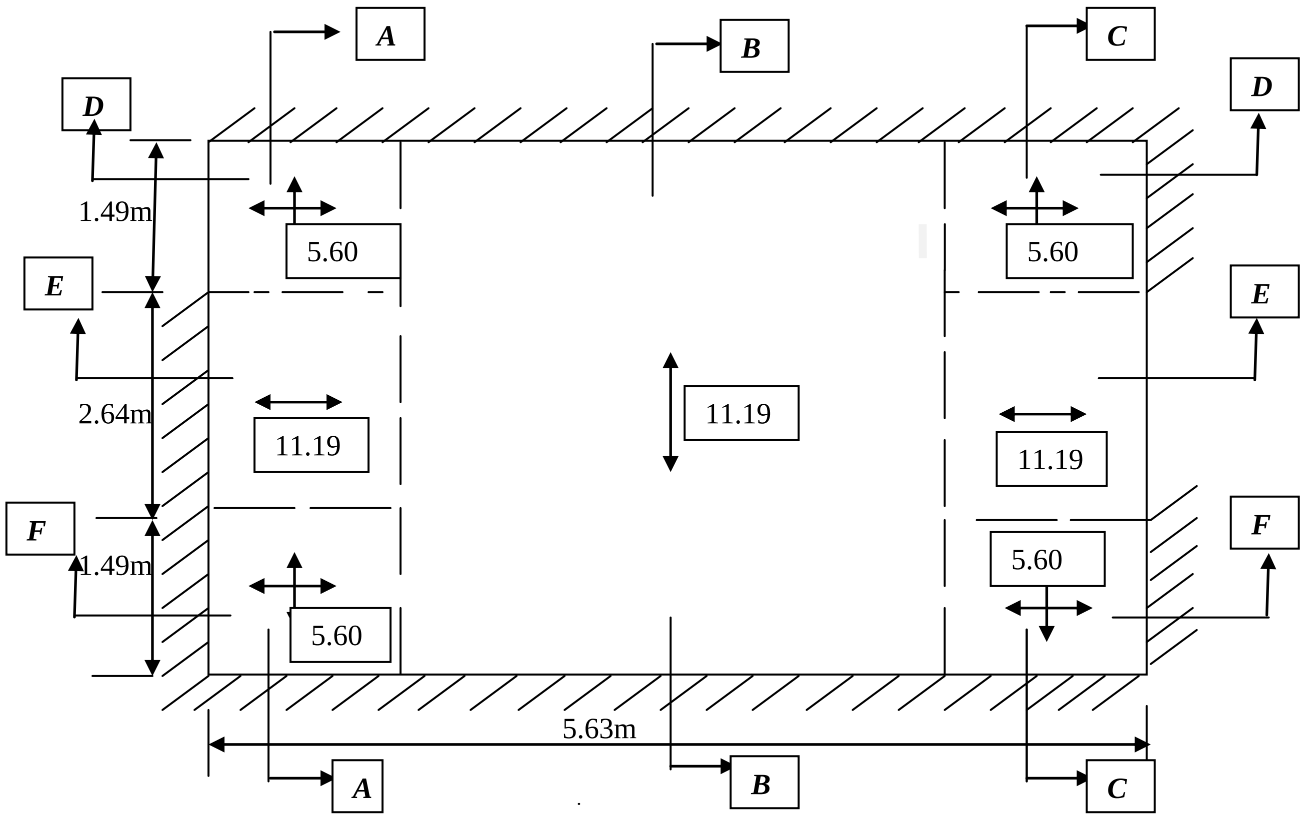


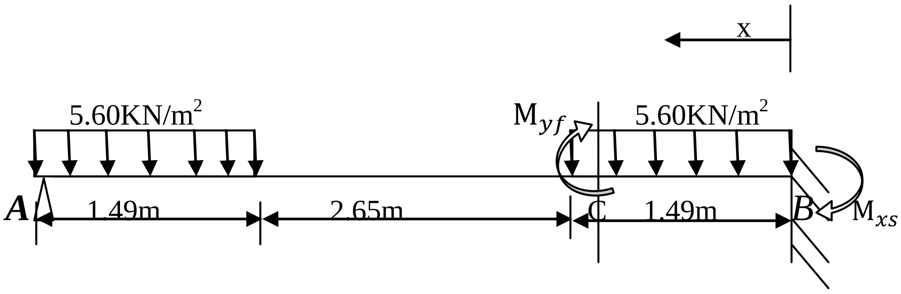
Figure 3.4 panel 7

Figure 3.5 panel 7



X-direction (edge)

Strip D-D



$$M_{xs} = 5.60 * 1.49m * 1.49m/2 + 5.60 * 1.49m * 4.885 = 47KNm/m$$

Reaction:

$\sum Fy = 0 \qquad RA + RB - 5.60 * 1.49m - 5.60 * 1.49m = 0$

$$RA + RB = 16.70KN/m$$

$\sum Ma = 0 \qquad M_{xs} + 5.60 * 1.49m * \frac{1.49m}{2} + 5.60 * 1.49m * \left(5.63 - \frac{1.49m}{2}\right) - 5.63 * RB = 0$

$$RB = 16.70KN/m \qquad RA = 0$$

$\sum Mc - c = 0 \qquad M_{xf} + M_{xs} + 5.60 * \frac{X^2}{2} - RB * X = 0$

$$M_{xf} + 47 + 5.60 * \frac{X^2}{2} - 16.70 * X = 0$$

$$M_{xf} = 16.70 * X - 5.60 * \frac{X^2}{2} - 47$$

differntiating the field moment with respect to X gives the position of maximum field moment

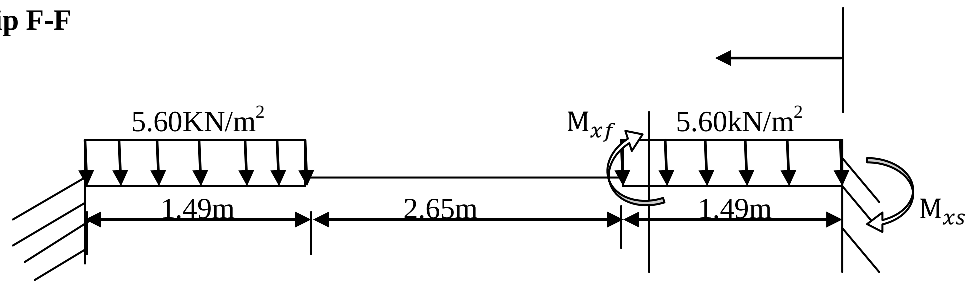
$\frac{dM_{xf}}{dx} = 0 \implies 16.70 - 5.60 * X - 0 = 0$

$$X = 16.70/5.60 = 2.982m$$

Subtituting X=2.98 in eqution above :-

$$M_{xf} = 16.70 * 2.982 - 5.60 * 2.982^2/2 - 47 = -22.10KNm/m$$

Strip F-F

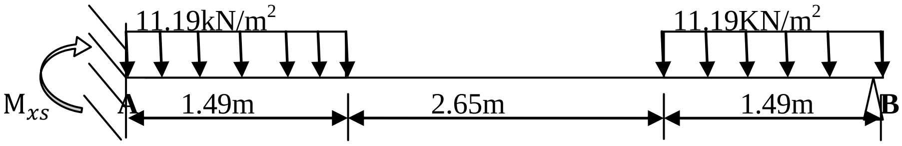


Cantilever moment ; $M_C = 5.60 * 1.49 * 1.49/2 = 6.22KNm/m$

Support moment; $M_{xs} = \frac{2}{3} * M_C = 2/3 * 6.06 = 4.14kNm/m$

Field moment ; $M_{xf} = \frac{1}{3} * M_C = 1/3 * 6.06 = 2.07kNm/m$

Strip E-E



$$M_{xs} = 11.19 * 1.49 * \frac{1.49}{2} + 11.19 * 1.49 * 4.885 = 93.87KNm/m$$

Reaction:-

$\sum Fy = 0$ $RA + RB - 11.19 * 1.49 - 11.19 * 1.49 = 0$

$RA + RB = 33.35KN/m$

$\sum Mb = 0$ $M_{xs} + 11.19 * 1.49 * \frac{1.49}{2} + 11.19 * 1.49 * \left(5.63 - \frac{1.49}{2}\right) - 5.63 * RA = 0$

$RA = 33.35KN/m$ $RB = 0$

$\sum Mc - c = 0$ $M_{xf} + Mxs + 11.19 * \frac{X^2}{2} - RA * X = 0$

$$M_{xf} + 93.87 + 11.192 * \frac{X^2}{2} - 33.35 * X = 0$$

$$M_{xf} = 33.35 * X - 11.19 * \frac{X^2}{2} - 93.87$$

differntiating the field moment with respect to X gives the position of maximum field moment

$\frac{dMxf}{dx} = 0 \Rightarrow$ $32.54 - 11.19 * X - 0 = 0$

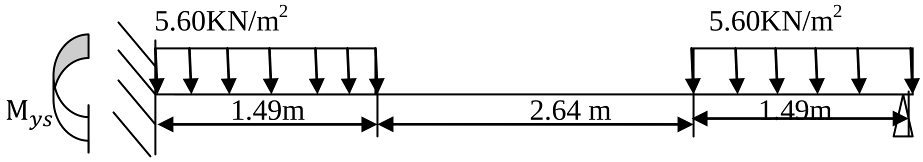
$$X = 33.35/11.19 = 2.98m$$

Subtituting $X = 2.98$ in eqution above : −

$$M_{xf} = 33.35 * 2.98 - 11.19 * 2.98/2 - 93.87 = 44.17KNm/m$$

Y-direction

Strip A-A and Strip C-C

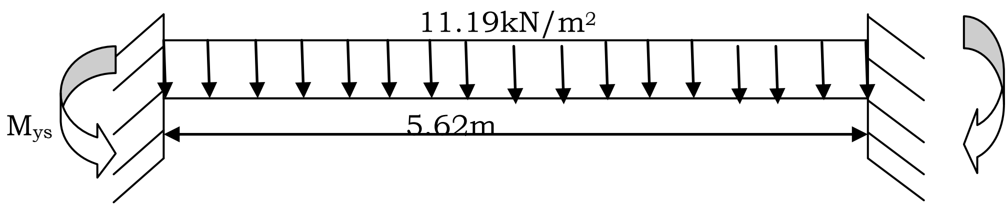


Cantilever moment ; $M_C = 5.60 * 1.49 * 1.49/2 = 6.22KNm/m$

Support moment; $M_{xs} = 2/3 * Mc = 2/3 * 6.22 = 4.14kNm/m$

Field moment ; $M_{xf} = 1/3 * Mc = 1/3 * 6.06 = 2.07kNm/m$

Strip B-B



Cantilever moment ; $M_C = \left(\frac{w*L^2}{8}\right) = 11.19 * 5.62 * \frac{5.62}{8} = 44.18KN/m$

Support moment; $M_{xs} = 2/3 * Mc = 2/3 * 44.18 = 29.45kNm/m$

Field moment ; $M_{xf} = 1/3 * Mc = 1/3 * 44.18 = 14.73kNm/m$

Because of the holes, some strip lacks support at one end. To supprt them, the strong band need to be provide

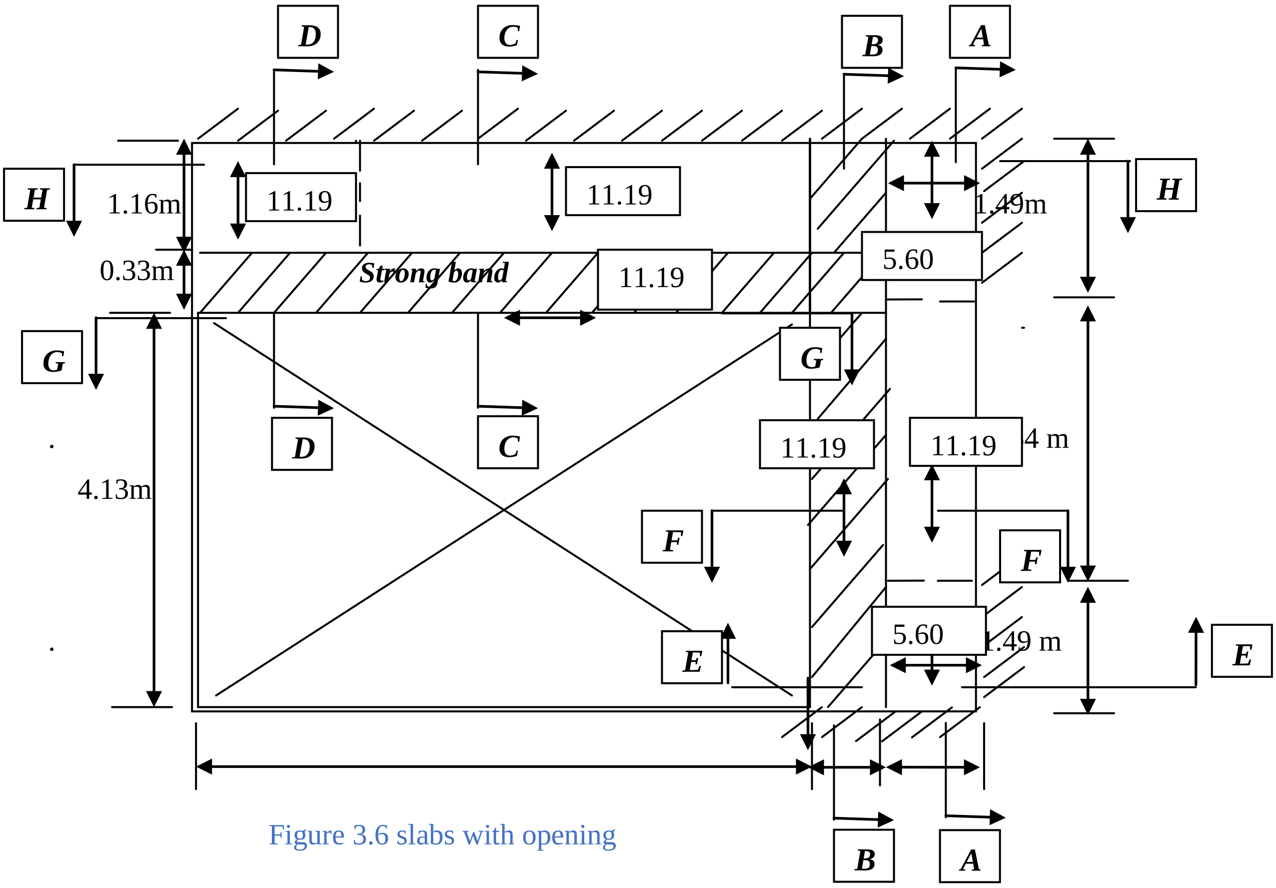
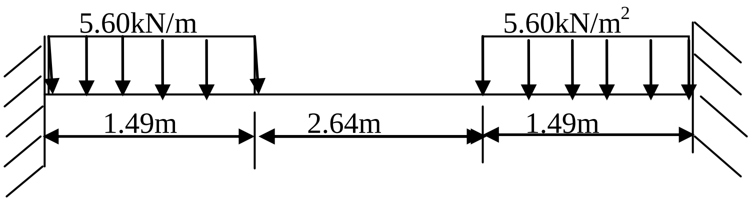


Figure 3.6 slabs with opening

Y-y direction

Strip A-A

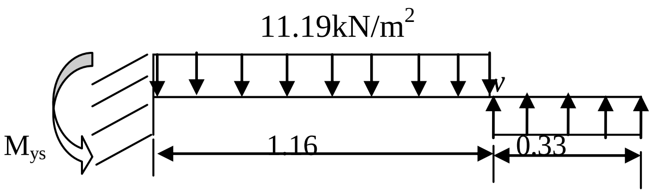


Cantilever moment ; $M_C = 5.60 * 1.49 * 1.49/2 = 6.22KNm/m$

Support moment; $M_{ys} = 2/3 * Mc = 2/3 * 6.22 = 4.14kNm/m$

Field moment ; $M_{yf} = 1/3 * Mc = 1/3 * 6.22 = 2.07kNm/m$

Strip C-C

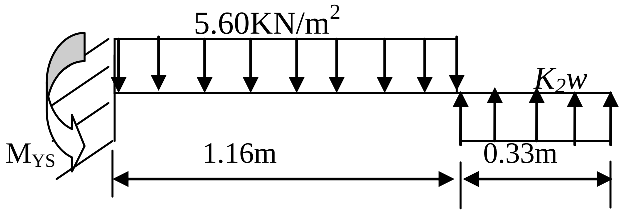


From the basic case; $M_{ys} = 29.45 KNm/m$

$29.45 - 11.19 * 1.16 * 1.16/2 + 11.19\, K_1 * 0.33(1.16 + 0.33/2) = 0$

K_1 = -4.48

Strip D-D

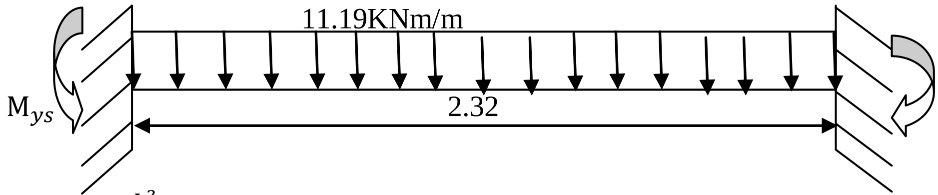


From the basic case; $M_{ys} = 4.14 kNm/m$

$$4.14 - 5.60 * 1.16 * 1.16/2 + 5.60 K_2 * 0.33(1.16 + 0.33/2) = 0$$

$$K_2 = -0.152$$

Strip G-G



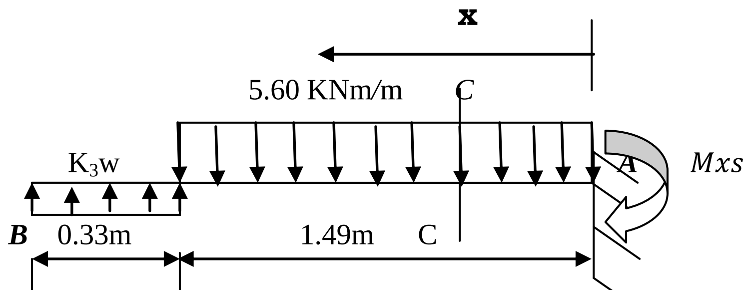
Cantilever moment ; $M_C = \frac{w*L^2}{8} = 11.19 * 2.32 * 2.32/8 = 7.53 KN/m$

Support moment; $M_{xs} = 2/3 * M_C = 2/3 * 7.53 = 5.02 kNm/m$

Field moment; $M_{xf} = 1/3 * M_C = 1/3 * 7.53 = 2.45 kNm/m$

x-x direction

Strip E-E



From the basic case; $M_{xs} = 4.14 kNm/m$

$$4.14 + 5.60\, K_3 * 0.33 * 1.655 - 5.60 * 1.49 * 1.49/2 = 0$$

$K_3 = 0.68$

Reaction: -

$\sum Fy = 0 \implies RB - 5.60 * 1.49 + 3.7 * 5.60 * 0.33 = 0$

$RB = 7.087KN/m$

$$\sum M_{C-C} = M_{xf} + M_{xs} + 5.60 * \frac{X^2}{2} - RB * X = 0$$

$$M_{xf} + 4.14 + 5.60 * \frac{X^2}{2} - 7.087 * X = 0$$

$$M_{xs} = 7.087 * X - 5.60 * \frac{X^2}{2} - 4.14$$

differntiating the field moment with respect to X gives the position of maximum field moment

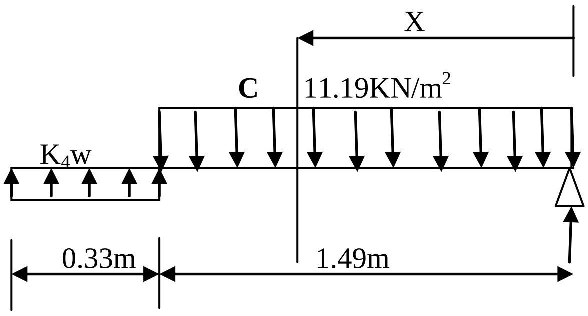
$$\frac{dM_{xf}}{dx} = 0 \implies 7.087 - 5.60 * X - 0 = 0$$

$$X = \frac{7.087}{5.60} = 1.266m$$

Subtituting $X = 1.26$ in eqution above :-

$$M_{xf} = 7.087 * 1.266 - 5.60 * 1.266 * \frac{1.266}{2} - 4.14 = 0.346KNm/m$$

Strips F-F



From the basic case; $M_{xs} = 0kNm/m$

$$0 + 11.19 * K_4 * 0.33 * 1.655 - 11.19 * 1.49 * 1.49/2 = 0$$

$K_4 = 2.03$

Reaction: -

$\sum Fy = 0 \implies RA - 11.19 * 1.49 + 2.03 * 11.19 * 0.33 = 0$

$RA = 9.17KN/m$

$$\sum Mc - c = M_{xf} + M_{xs} + 11.19 * \frac{X^2}{2} - RB * X = 0$$

$$M_{xf} + 0 + 11.19 * \frac{X^2}{2} - 9.17 * X = 0$$

$$M_{xf} = 9.17 * X - 11.19 * \frac{X^2}{2} - 0$$

differntiating the field moment with respect to X gives the position of maximum field moment

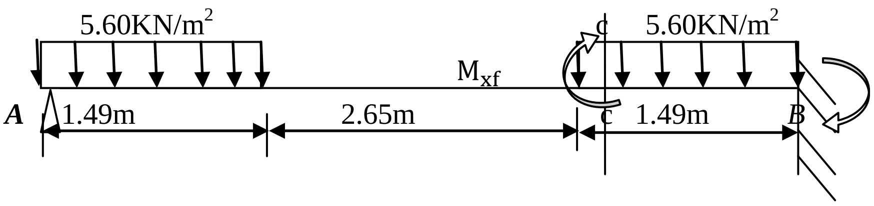
$\frac{dM_{xf}}{dx} = 0 \implies 9.17 - 11.19 * X - 0 = 0$

$$X = \frac{9.17}{11.19} = 0.82m$$

Subtituting $X = 0.82m$ in eqution above :-

$$M_{xf} = 9.17 * 0.82 - 5.60 * 0.82 * \frac{0.82}{2} - 0 = 5.64KNm/m$$

Strip H-H



$$M_{xs} = 5.60 * 1.49 * 1.49/2 \ + 5.60 * 1.49 * 4.885 = 47KNm/m$$

Reaction: -

$\sum Fy = 0 \implies RA + RB - 5.60 * 1.49 - 5.60 * 1.49 = 0$

$$RA + RB = 16.70KN/m$$

$\sum Ma = 0 \implies M_{xs} + 5.60 * 1.49 * \frac{1.49}{2} + 5.60 * 1.49 * \left(5.63 - \frac{1.49}{2}\right) - 5.63 * RB = 0$

$$RB = 16.70KN/m \qquad RA = 0$$

$\sum Mc - c = 0 \qquad M_{xf} + M_{xs} + 5.46 * \frac{X^2}{2} - RB * X = 0$

$$M_{xf} + 47 + 5.60 * \frac{X^2}{2} - 16.70 * X = 0$$

$$M_{xf} = 16.70 * X - 5.60 * \frac{X^2}{2} - 47$$

differntiating the field moment with respect to X gives the position of maximum field moment

$\frac{dM_{xf}}{dx} = 0 \qquad 16.70 - 5.60 * X - 0 = 0$

$$X = \frac{16.70}{5.60} = 2.98m$$

Subtituting $X = 2.98$ in eqution above :-

$$M_{xf} = 16.70 * 2.98 - 5.60 * 2.982/2 - 47 = -22.10KNm/m$$

3.3.4 Slab Reinforcement

Main Reinforcement

For the given

- Material Data, C-20/25, S-400
- Effective depth, d = 150 mm
- Width, b = 1000 mm
- Diameter of bar ǿ=10 and 12

Effective depth:

d_x = 200-20-0.5(10) = 175

d_y = 200-20-1.5(10) = 165

Flexural reinforcement

Minimum and maximum reinforcement area,

(1) The area of longitudinal tension reinforcement should not be taken as less than As,min.

Note 1: See also 7.3 for area of longitudinal tension reinforcement to control cracking.

(2) Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slabs. In areas near supports transverse reinforcement to principal top bars is not necessary where there is no transverse bending moment.

(3) Sections containing less reinforcement than As,min should be considered as unreinforced (see Section 12).

(4) The cross-sectional area of tension or compression reinforcement should not exceed 0.04 Ac outside lap locations.

Note: In addition to Note 2 of 9.2.1.1 (1), for slabs where the risk of brittle failure is small, As,min may be taken as 1.2 times the area required in ULS verification

(5) The spacing of bars should not exceed smax,slabs. Note: For the value of smax,slabs, refer to the National Annex. The recommended value is: - for the principal reinforcement, 3h ≤ 400 mm, where h is the total depth of the slab; - for the secondary reinforcement, 3.5h ≤ 450 mm .

Reinforcement in slabs near supports

(1) In simply supported slabs, half the calculated span reinforcement should continue up to the support and be anchored therein in accordance with 8.4.4. Note: Curtailment and anchorage of reinforcement may be carried out according to 9.2.1.3, 9.2.1.4 and 9.2.1.5. (2) Where partial fixity occurs along an edge of a slab, but is not taken into account in the analysis, the top reinforcement should be capable of resisting at least 25% of the maximum moment in the

adjacent span. This reinforcement should extend at least 0.2 times the length of the adjacent span, measured from the face of the support. It should be continuous across internal supports and anchored at end supports. At an end support the moment to be resisted may be reduced to 15% of the maximum moment in the adjacent span.

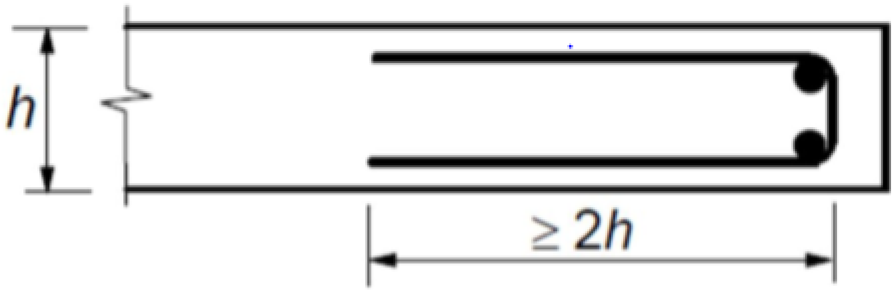
Corner reinforcement

(1) If the detailing arrangements at a support are such that lifting of the slab at a corner is restrained, suitable reinforcement should be provided.

Reinforcement at the free edges

(1) Along a free (unsupported) edge, a slab should normally contain longitudinal and transverse reinforcement, generally arranged as shown below

(2) The normal reinforcement provided for a slab may act as edge reinforcement.



Edge reinforcement for a slab

The spacing of bars should not exceed smax,slabs. Note: For the value of smax,slabs, refer to the National Annex. The recommended value is: - for the principal reinforcement, 3h ≤ 400 mm, where h is the total depth of the slab; - for the secondary reinforcement, 3.5h ≤ 450 mm ..

$$A_{s\,min} = \max\begin{cases} \dfrac{0.26 * fctm * b * d}{fyk} = \dfrac{0.26 * 2.2 * 1000 * 175}{400} = 250.25\text{mm}^2 \\ 0.0013 * b * d = 0.0013 * 1000 * 175 = 227.5\text{mm}^2 \end{cases}$$

$$A_{s\,max} = 0.04 * A_C = 0.04 * 1000 * 225 = 9000\text{mm}^2/\text{m}$$

But $A_{s\,min}$ varies as the depth varies

for primary reinforcement

$$S_{max,slab} = \min\begin{cases} 3h = 3*225 = 675\text{mm} \\ 400\text{mm} \end{cases}$$

$S_{max,slab}$ = 400mm

for secondary reinforcement

$$S_{max,slab} = \min \begin{cases} 3.5h = 3.5*225 = 700mm \\ 450mm \end{cases}$$

Sample reinforcement calculation

Panel 1

X direction

$$\mathbf{M_{xs} = 12.2kNm}$$

$$\mu sd,s = \frac{M_{xs} * 10^6}{fcd * b * d^2} = \frac{12.2 * 10^6}{11.33 * 1000 * 175^2} = 0.035$$

$$z = \frac{d}{2} * \left(1 + \sqrt{1 - 2 * \mu sd,s}\right) = \frac{175}{2} * \left(1 + \sqrt{1 - 2 * 0.035}\right) = 171.87mm$$

$$A_{st,cal} = \frac{M_{xs}}{z * fyd} = \frac{12.2 * 10^6}{171.87 * 347.8} = 204.08mm^2$$

$$A_{s\ min} = \max \begin{cases} \frac{0.26 * fctm * b * d}{fyk} = \frac{0.26 * 2.2 * 1000 * 175}{400} = 250.25mm^2 \\ 0.0013 * b * d = 0.0013 * 1000 * 200 = 227.5mm^2 \end{cases}$$

$$A_{st,min} = 250.25mm^2$$

$$A_{st,req} = \max \begin{cases} A_{s,cal} = 204.08mm^2 \\ A_{st,min} = 250.25mm^2 \end{cases}$$

$$A_{st,req} = 250.25mm^2$$

$$as = \frac{\pi d^2}{4} = \frac{\pi * 10^2}{4} = 78.54mm^2$$

$$S_{cal} = \frac{b * as}{A_{s,req}} = \frac{1000 * 78.54}{250.25} = 313.8mm$$

$$S_{provided} = 310mm$$

$$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 78.54}{310} = 253.35mm^2$$

Provide ф10 C/C 310

$$\mathbf{M_{xf} = 9.34kNm}$$

$$\mu sd,s = \frac{M_{xf} * 10^6}{fcd * b * d^2} = \frac{9.34 * 10^6}{11.33 * 1000 * 175^2} = 0.027$$

$$z = \frac{d}{2} * \left(1 + \sqrt{1 - 2 * \mu sd, s}\right) = \frac{175}{2} * \left(1 + \sqrt{1 - 2 * 0.027}\right) = 172.61mm$$

$$A_{st,cal} = \frac{M_{xf}}{z * fyd} = \frac{9.34 * 10^6}{1172.61 + 347.8} = 155.56mm^2$$

$$A_{s\,min} = \max\begin{cases} \frac{0.26 * fctm * b * d}{fyk} = \frac{0.26 * 2.2 * 1000 * 175}{400} = 250.25mm^2 \\ 0.0013 * b * d = 0.0013 * 1000 * 175 = 227.5mm^2 \end{cases}$$

$A_{st,min} = 250.25mm^2$

$$A_{st,req} = \max\begin{cases} A_{s,cal} = 155.56mm^2 \\ A_{s,min} = 250.25mm^2 \end{cases}$$

$A_{st,req} = 250.25mm^2$

$$as = \frac{\pi d^2}{4} = \frac{\pi * 10^2}{4} = 78.54mm^2$$

$$S_{cal} = \frac{b * as}{A_{s,req}} = \frac{1000 * 78.54}{250.25mm^2} = 313.8mm$$

$S_{provided} = 310mm$

$$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 78.54}{310} = 253.35mm^2$$

Provide ф10 C/C 270

Y direction

$\mathbf{M_{ys} = 10.05kNm}$

$$\mu sd, s = \frac{M_{ys} * 10^6}{fcd * b * d^2} = \frac{10.05 * 10^6}{11.33 * 1000 * 165^2} = 0.033$$

$$z = \frac{d}{2} * \left(1 + \sqrt{1 - 2 * \mu sd, s}\right) = \frac{165}{2} * \left(1 + \sqrt{1 - 2 * 0.033}\right) = 162.27mm$$

$$A_{st,cal} = \frac{M_{ys}}{z * fyd} = \frac{10.05 * 10^6}{162.27 * 347.8} = 177.97mm^2$$

$$A_{st,req} = \max\begin{cases} \frac{0.26 * fctm * b * d}{fyk} = \frac{0.26 * 2.2 * 1000 * 165}{400} = 235.95 \\ 0.0013 * b * d = 0.0013 * 1000 * 165 = 214.5 \end{cases}$$

$A_{st,min} = 235.95mm^2$

$$A_{st,req} = \max\begin{cases} A_{s,cal} = 177.97mm^2 \\ A_{s,min} = 235.95mm^2 \end{cases}$$

$A_{st,req} = 235.95mm^2$

$as = \frac{\pi d^2}{4} = \frac{\pi * 10^2}{4} = 78.54mm^2$

$S_{cal} = \frac{b * as}{A_{s,req}} = \frac{1000 * 78.54}{235.95} = 332.9mm$

$S_{provided} = 330mm$

$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 78.54}{330} = 238.21$

Provide ϕ10 C/C 330

$\mathbf{M_{yf} = 6.86kNm}$

$\mu sd, s = \frac{M_{yf} * 10^6}{fcd * b * d^2} = \frac{6.86 * 10^6}{11.33 * 1000 * 190^2} = 0.022$

$z = \frac{d}{2} * \left(1 + \sqrt{1 - 2 * \mu sd, s}\right) = \frac{190}{2} * \left(1 + \sqrt{1 - 2 * 0.022}\right) = 163.14mm$

$A_{st,cal} = \frac{M_{ys}}{z * fyd} = \frac{6.86 * 10^6}{163.14 * 347.8} = 120.89mm^2$

$$A_{st,req} = \max\begin{cases} \frac{0.26 * fctm * b * d}{fyk} = \frac{0.26 * 2.2 * 1000 * 165}{400} = 235.95 \\ 0.0013 * b * d = 0.0013 * 1000 * 165 = 214.5 \end{cases}$$

$A_{st,min} = 235.95mm^2$

$$A_{st,req} = \max\begin{cases} A_{s,cal} = 120.89mm^2 \\ A_{s,min} = 235.95mm^2 \end{cases}$$

$A_{st,req} = 235.95mm^2$

$as = \frac{\pi d^2}{4} = \frac{\pi * 10^2}{4} = 78.54mm^2$

$S_{cal} = \frac{b * as}{A_{s,req}} = \frac{1000 * 78.54}{235.95mm^2} = 332.9mm$

$S_{provided} = 330$

$$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 \ * \ 78.54}{310} = 238.21$$

Provide ф10 C/C 330

Similarly in tabulated form 3F

Reinforcement for panel-7

Strip A-A

$M_{xs} = 4.14kNm/m$

$M_{xf} = \ 2.07kNm/m$ ↔ provide minimum reinforcement at the top

Strip E-E

$M_{xs} = 4.14kNm/m$

$M_{xf} = \ -3.84kNm/m$ ↔ provide minimum reinforcement at the bottom

Strip F-F

$M_{xs} = 0kNm/m$

$M_{xf} = \ 5.64KNm/m$ ↔ provide minimum reinforcement at the bottom

Strip H-H

Support moment $M_{xs} = 47kNm/m > M_{sd,min} = 8.88KNm/m$

Hence, design for this strip

$\mathbf{M_{xs} = 47kNm/m}$

$$\mu sd, s \ = \frac{M_{yf} * 10^6}{fcd * b * d^2} = \frac{47 * 10^6}{11.33 * 1000 * 200^2} = 0.135$$

$$z = \frac{d}{2} * \left(1 \ + \ \sqrt{1 - 2 * \mu sd, s}\,\right) = \frac{175}{2} * \left(1 \ + \ \sqrt{1 - 2 * 0.104}\,\right) \ = \ 162.22mm$$

$$A_{st,cal} = \frac{M_{xs}}{z * fyd} = \frac{47 * 10^6}{162.22 + \ 347.8} = \ 833.98mm^2$$

$$A_{s\,min} = \max\begin{cases} \dfrac{0.26 * fctm * b * d}{fyk} = \dfrac{0.26 * 2.2 * 1000 * 175}{400} = 250.25mm^2 \\ 0.0013 * b * d = 0.0013 * 1000 * 175 = 227.5mm^2 \end{cases}$$

$A_{st,min} = 235.95\ mm^2$

$$A_{st,req} = \max\begin{cases} A_{s,cal} = 833.98mm^2 \\ A_{s,min} = 235.95\ mm^2 \end{cases}$$

$A_{st,req} = 833.98mm^2$

$as = \frac{\pi d^2}{4} = \frac{\pi * 10^2}{4} = 78.54mm^2$

$S_{cal} = \frac{b * as}{A_{s,req}} = \frac{1000 * 78.54}{833.98} = 94.3mm$

$S_{provided} = 90mm$

$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 78.54}{90} = 872.66mm^2$

Provide ф10 C/C 90

Field moment

$M_{xf} = 22.10kNm/m > M_{sd,min} = 10.2KNm/m$

Hence, design for this strip

$M_{xf} = 22.10kNm/m$

$\mu sd,s = \frac{M_{yf} * 10^6}{fcd * b * d^2} = \frac{22.10 * 10^6}{11.33 * 1000 * 200^2} = 0.0064$

$z = \frac{d}{2} * \left(1 + \sqrt{1 - 2 * \mu sd,s}\right) = \frac{175}{2} * \left(1 + \sqrt{1 - 2 * 0.0064}\right) = 169.24mm$

$A_{st,cal} = \frac{M_{xs}}{z * fyd} = \frac{22.10 * 10^6}{169.24 + 347.8} = 375.43mm^2$

$$A_{s\ min} = \max\begin{cases} \frac{0.26 * fctm * b * d}{fyk} = \frac{0.26 * 2.2 * 1000 * 175}{400} = 250.25mm^2 \\ 0.0013 * b * d = 0.0013 * 1000 * 175 = 227.5mm^2 \end{cases}$$

$A_{st,min} = 250.25mm^2$

$$A_{st,req} = \max\begin{cases} A_{s,cal} = 375.43mm^2 \\ A_{s,min} = 250.25mm^2 \end{cases}$$

$A_{st,req} = 375.43mm^2$

$as = \frac{\pi d^2}{4} = \frac{\pi * 10^2}{4} = 78.54mm^2$

$$S_{cal} = \frac{b * as}{A_{s,req}} = \frac{1000 * 78.54}{375.43} = 209.2mm$$

$$S_{provided} = 200mm$$

$$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 78.54}{200} = 392.70mm^2$$

Provide ф10 C/C 200

Strip G-G

$M_{xs} = 5.02kNm/m$

$M_{xf} = 2.45kNm/m$ ↔ provide minimum reinforcement at the bottom

Strip C-C

Support moment

$M_{ys} = 29.45KNm/m > M_{sd,min} = 8.88KNm/m$

Hence, design for this strip

$$\mu sd, s = \frac{M_{ys} * 10^6}{fcd * b * d^2} = \frac{29.45 * 10^6}{11.33 * 1000 * 190^2} = 0.009$$

$$z = \frac{d}{2} * \left(1 + \sqrt{1 - 2 * \mu sd, s}\right) = \frac{165}{2} * \left(1 + \sqrt{1 - 2 * 0.072}\right) = 156.71mm$$

$$A_{st,cal} = \frac{M_{ys}}{z * fyd} = \frac{22.10 * 10^6}{156.71 * 347.8} = 540.59mm^2$$

$$A_{st,req} = \max\begin{cases} \frac{0.26 * fctm * b * d}{fyk} = \frac{0.26 * 2.2 * 1000 * 165}{400} = 235.95 \\ 0.0013 * b * d = 0.0013 * 1000 * 165 = 214.5 \end{cases}$$

$A_{st,min} = 235.95\ mm^2$

$$A_{st,req} = \max\begin{cases} A_{s,cal} = 540.59mm^2 \\ A_{s,min} = 235.95mm^2 \end{cases}$$

$A_{st,req} = 540.59mm^2$

$$as = \frac{\pi d^2}{4} = \frac{\pi * 10^2}{4} = 78.54mm^2$$

$S_{cal} = \frac{b * as}{A_{s,req}} = \frac{1000 * 78.54}{540.59} = 145.6mm$

$S_{provided} = 140mm$

$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 78.54}{140} = 490.561$

Provide ф10 C/C 140mm

Field moment

$M_{yf} = 22.10kNm/m > M\ sd, min = 10.2kNm/m$

Hence, design for this strip

$\mu sd, s = \frac{M_{yf} * 10^6}{fcd * b * d^2} = \frac{22.10 * 10^6}{11.33 * 1000 * 165^2} = 0.071$

$z = \frac{d}{2} * \left(1 + \sqrt{1 - 2 * \mu sd, s}\right) = \frac{190}{2} * \left(1 + \sqrt{1 - 2 * 0.071}\right) = 158.89mm$

$A_{st,cal} = \frac{M_{yf}}{z * fyd} = \frac{22.10 * 10^6}{158.89 + 347.8} = 398.07mm^2$

$$A_{st,req} = \max\begin{cases}\frac{0.26 * fctm * b * d}{fyk} = \frac{0.26 * 2.2 * 1000 * 165}{400} = 235.95\\ 0.0013 * b * d = 0.0013 * 1000 * 165 = 214.5\end{cases}$$

$A_{st,min} = 235.95mm^2$

$$A_{st,req} = \max\begin{cases}A_{s,cal} = 398.07mm^2\\ A_{s,min} = 235.95mm^2\end{cases}$$

$A_{st,req} = 398.07mm^2$

$as = \frac{\pi d^2}{4} = \frac{\pi * 10^2}{4} = 78.54mm^2$

$S_{cal} = \frac{b * as}{A_{s,req}} = \frac{1000 * 78.54}{398.07} = 197.3mm$

$S_{provided} = 190$

$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 78.54}{220} = 413.37mm^2$

Provide ф10 C/C 190

Strip D-D

$M_{ys} = 4.14kNm/m$

$M_{yf} = -0.152kNm/m$ ↔ provide minimum reinforcement at the top

Strip B-B

Support moment

$M_{xs} = 52.95kNm/m > M\ sd, min = 8.88kNm/m$

Hence, design for this strip

$$\mu sd, s = \frac{M_{xs} * 10^6}{fcd * b * d^2} = \frac{52.95 * 10^6}{11.33 * 1000 * 200^2} = 0.153$$

$$z = \frac{d}{2} * \left(1 + \sqrt{1 - 2 * \mu sd, s}\right) = \frac{175}{2} * \left(1 + \sqrt{1 - 2 * 0.153}\right) = 160.44mm$$

$$A_{st,cal} = \frac{M_{xs}}{z * fyd} = \frac{52.95 * 10^6}{160.44 * 347.8} = 948.84mm^2$$

$$A_{s\ min} = \max\begin{cases}\frac{0.26 * fctm * b * d}{fyk} = \frac{0.26 * 2.2 * 1000 * 175}{400} = 250.25mm^2\\ 0.0013 * b * d = 0.0013 * 1000 * 175 = 227.5mm^2\end{cases}$$

$A_{st,min} = 250.25mm^2$

$$A_{st,req} = \max\begin{cases}A_{s,cal} = 948.84mm^2\\ A_{s,min} = 235.95mm^2\end{cases}$$

$A_{st,req} = 948.84mm^2$

$$as = \frac{\pi d^2}{4} = \frac{\pi * 10^2}{4} = 78.54mm^2$$

$$S_{cal} = \frac{b * as}{A_{s,req}} = \frac{1000 * 78.54}{948.84} = 82.8mm$$

$S_{provided} = 80mm$

$$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 78.54}{80} = 981.75$$

Provide ф10 C/C 80

Shear force calculation

(1) A slab in which shear reinforcement is provided should have a depth of at least 200 mm.

Shear force for in two-way slab determine from:

Short direction $V_x = Pd * L_x * \beta_{vx}$

Longer direction $V_y = Pd * L_y * \beta_{vy}$

Table 3.9 Shear force coefficients

Table 3-10: Shear force coefficients for rectangular slabs

Table A-3 Shear Force Coefficients for Uniformly Loaded Rectangular Panels Supported on Four Sides with Provision fo Torsion at Corners

Type of panel and location	Edge	β_{vx} for values of L_y/L_x								β_{vy}
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
1	Continuous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
2	Continuous	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
	Discontinous	-	-	-	-	-	-	-	-	0.24
3	Continuous	0.36	0.40	0.44	0.47	0.49	0.51	0.55	0.59	0.36
	Discontinous	0.24	0.27	0.29	0.31	0.32	0.34	0.36	0.38	-
4	Continuous	0.40	0.44	0.47	0.50	0.52	0.54	0.57	0.60	0.40
	Discontinous	0.26	0.29	0.31	0.33	0.34	0.35	0.38	0.40	0.26
5	Continuous	0.40	0.43	0.45	0.47	0.48	0.49	0.52	0.54	-
	Discontinous	-	-	-	-	-	-	-	-	0.26
6	Continuous	-	-	-	-	-	-	-	-	0.40
	Discontinous	0.26	0.30	0.33	0.36	0.38	0.40	0.44	0.47	-
7	Continuous	0.45	0.48	0.51	0.53	0.55	0.57	0.60	0.63	-
	Discontinous	0.30	0.32	0.34	0.35	0.36	0.37	0.39	0.41	0.30
8	Continuous	-	-	-	-	-	-	-	-	0.45
	Discontinous	0.30	0.33	0.36	0.38	0.40	0.42	0.45	0.48	0.30
9	Discontinous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

Sample example on panel-1

$Pd = 13.29 KN/m^2$

$L_x = 3.78$

From the above table we can calculate the value β_{vx} and β_{vy} depending on the ly/lx ratio and the slab continuity.

$L_y = 4.72$ $L_x = 3.78$ $\frac{L_y}{L_x} = 1.25$ so we can interpolatethe value

$\beta_{vx,c} = 0.480$ $\beta_{vy,c} = 0.40$

$\beta_{vx,d} = 0.320$ $\beta_{vy,d} = 0.26$

Short span;

$V_{x,c} = 13.29 * 3.78 * 0.480 = 24.19$

$V_{x,d} = 13.29 * 3.78 * 0.320 = 16.12$

Long span;

$V_{y,c} = 13.29 * 3.78 * 0.40 = 20.17$

$V_{y,d} = 13.29 * 3.78 * 0.26 = 13.11$

Check Shear Capacity of Slab

Shear Capacity of the concrete EBCS EN 1992-1-1-2002 sec.6.2.2

Members not requiring design shear reinforcement

(1) The design value for the shear resistance $V_{Rd,c}$ is given by:

$V_{Rd,c} = [C_{Rd,c}\ K(100\rho 1 fck)^{1/3} + \sigma_{cp}]b_w * d.$ (6.2a)

With a minimum of $V_{Rd,c} = (V_{min} + K_1\sigma_{cp})b_w * d$ (6.2b)

Where:

fck is in MPa

$V_{Rd,c} =$ in N

$C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$

$K = \sqrt{\frac{200}{d}} \leq 2.0$ with d in mm

The value of K depends on the value effective depth. As the depth varies the value of K also varies. But, since the value of effective depth is below 200 so the calculated value of K will be greater than 2 so we have to provide 2.

K_1 = 0.15 (Note: recommended value from EBCS EN 1992)

$V_{min} = 0.035 * K^{3/2} * fck^{1/2} = 0.035 * 2^{3/2} * 20^{1/2}$ =0.44

$\sigma_{cp} = \frac{N_{Ed}}{A_c} =< 0.2fcd$

N_{Ed} Is the axial force in the cross − section due to loading or prestressing [in N]

(N_{Ed} > 0 for compression). The influence of imposed deformations on N_{Ed} may be ignore.

$\rho = \frac{A_s}{b_w * d}$ taking minimum reinforcement and its value depend on the depth

Taking 1meter strip b=1000mm

Design for shear force, $V_{ED} = 24.19$

Design for shear resistance

$V_{Rd,c} = \left[C_{Rd,c}\ K(100\rho 1 fck)^{\frac{1}{3}} + \sigma_{cp}\right] b_w * d$

$V_{Rd,c} = [0.12 * 2 * (100 * 0.00145 * 20)^{1/3} + 0]1000 * 175 = 77.48KN$

$V_{min} == [0.035 * 2^{3/2} * 20^{1/2}]1000 * 175 = 77KN$

So, $V_{Rd,c} = 77.48 > V_{ED} = 24.19$ Ok!!

Therefore shear reinforcement is not required

Design for shear force, $V_{ED} = 16.2$

Design for shear resistance

$V_{Rd,c} = \left[C_{Rd,c}\ K(100\rho 1 fck)^{\frac{1}{3}} + \sigma_{cp}\right] b_w * d$

$V_{Rd,c} = [0.12 * 2 * (100 ** 20)^{1/3} + 0]1000 * 175 = 88.82KN$

$V_{min} == [0.035 * 2^{3/2} * 20^{1/2}]1000 * 175 = 77.48KN$

So, $V_{Rd,c} = 77.48 > V_{ED} = 16.2$ Ok!!

Therefore shear reinforcement is not required

Design for shear force, $V_{ED} = 20.17$

Design for shear resistance

$V_{Rd,c} = \left[C_{Rd,c}\ K(100\rho 1 fck)^{\frac{1}{3}} + \sigma_{cp}\right] b_w * d$

$V_{Rd,c} = [0.12 * 2 * (100 * 0.00473 * 20)^{1/3} + 0]1000 * 165 = 173.2KN$

$V_{min} == [0.035 * 2^{3/2} * 20^{1/2}]1000 * 165 = 72.6KN$

So, $V_{Rd,c} = 88.82 > V_{ED} = 20.17$ Ok!!

Therefore shear reinforcement is not required

Design for shear force, $V_{ED} = 13.11$

Design for shear resistance

$$V_{Rd,c} = \left[C_{Rd,c}\ K(100\rho 1 fck)^{\frac{1}{3}} + \sigma_{cp}\right] b_w * d$$

$$V_{Rd,c} = [0.12 * 2 * (100 * 0.00473 * 20)^{1/3} + 0]1000 * 165 = 73.2KN$$

$$V_{min} == [0.035 * 2^{3/2} * 20^{1/2}]1000 * 165 = 72.6KN$$

So, $V_{Rd,c} = 77.48 > V_{ED} = 13.11$ Ok!!

Therefore shear reinforcement is not required

Check depth for serviceability

For $fck = 20$MPa

Let's do for P-1

$$\rho o = \sqrt{fck} * 10^{-3} = \sqrt{20} * 10^{-3} = 0.00447$$

$$\rho xf = \frac{A_{st,used}}{b * d} = \frac{253.35}{1000 * 175} = 0.00145$$

$$\rho xs = \frac{A_{st,used}}{b * d} = \frac{253.35}{1000 * 175} = 0.00145$$

$$\rho yf = \frac{A_{st,used}}{b * d} = \frac{238}{1000 * 165} = 0.00142$$

$$\rho ys = \frac{A_{st,used}}{b * d} = \frac{238}{1000 * 165} = 0.00142$$

All the above steel ratios are $< \rho o$ so we have to use the formula

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2} fck\right] \text{Fi}$$

But this formula is for $fyk = 500$MPa and we use $fyk = 400$MPa so we have to multiply by F1

$$F1 = \frac{500}{fyk/(A_{st,req}/A_{st,used})}$$

$$F1xf = \frac{500}{fyk/(A_{st,req}/A_{st,used})} = \frac{500}{400/(250.25/253.35)} = 1.26551$$

$$F1xs = \frac{500}{fyk/(A_{st,req}/A_{st,used})} = \frac{500}{400/(250.25/253.35)} = 1.26551$$

$$F1yf = \frac{500}{fyk/(A_{st,req}/A_{st,used})} = \frac{500}{400/(235.95/238)} = 1.26086$$

$$F1ys = \frac{500}{fyk/(A_{st,req}/A_{st,used})} = \frac{500}{400/(250.25/253.35)} = 1.26086$$

Length to depth ratio have been done as follow

$$\frac{l}{d}i = K\left[11 + 1.5\sqrt{20} + 3.2\sqrt{fck}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2} fck\right]\text{Fi}$$

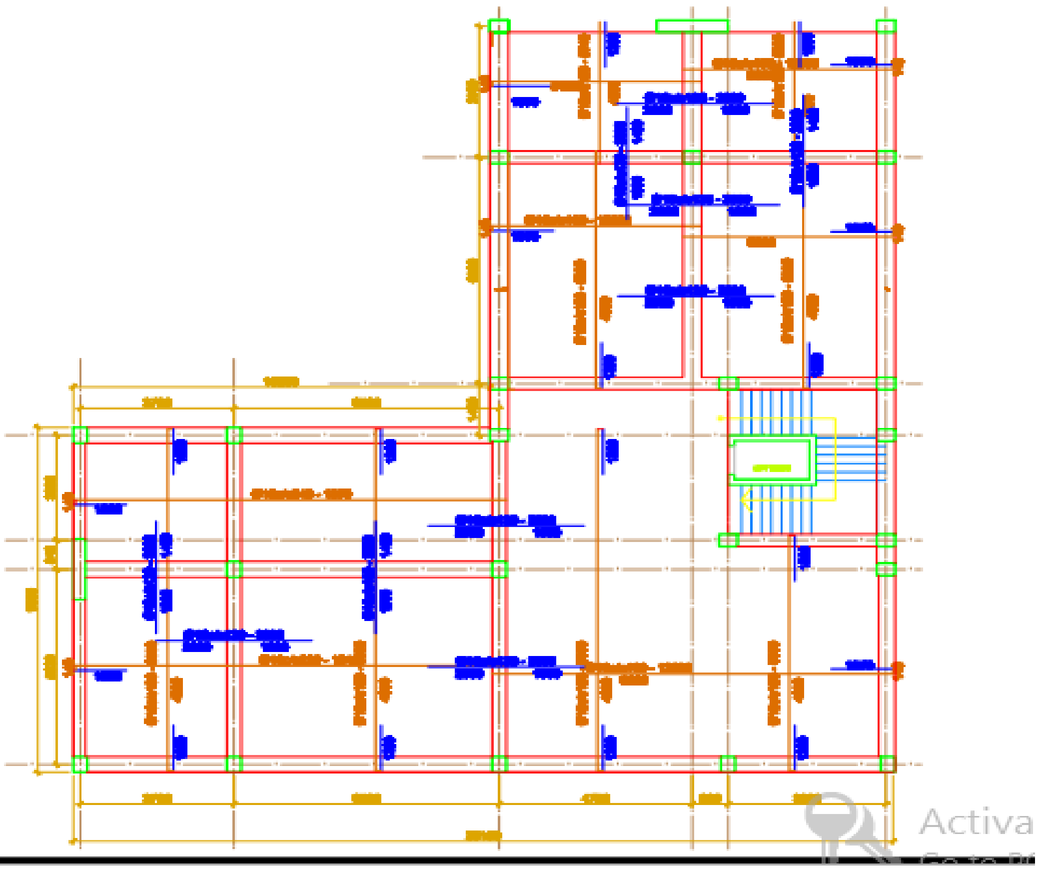
$$\frac{l}{d}xf = K\left[11 + 1.5\sqrt{f_{ck}} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2} fck\right]\text{Fi}$$

$$\frac{l}{d}xs = K\left[11 + 1.5\sqrt{f_{ck}} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2} fck\right]\text{Fi}$$

$$\frac{l}{d}yf = K\left[11 + 1.5\sqrt{f_{ck}} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2} fck\right]\text{Fi}$$

$$\frac{l}{d}ys = K\left[11 + 1.5\sqrt{f_{ck}} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2} fck\right]\text{Fi}$$

Slab reinforcement detailing



4 Stair case Design

4.1 Introduction

Functionally, the staircase is an important component of a building, and often the only means of access between the various floors in the building. It consists of a flight of steps, usually with one or more intermediate landings (horizontal slab platforms) provided between the floor levels. The horizontal top portion of a step (where the foot rests) is termed tread and the vertical projection of the step (i.e., the vertical distance between two neighboring steps) is called riser. Stair widths are encountered in entrance to public buildings. The horizontal Projection Plan of an inclined flight of steps, between the first and last risers, is termed going. Generally, risers in a flight should not exceed about 12 in number. The steps in the flight can be designed in a number of ways: with waist slab, with tread-riser arrangement (without waist slab) or with isolated tread slabs.

4.2 Design procedure

Stair case analysis and design is similar to one-way slab analysis and design. It involves the analysis steps followed for slabs. The inclined configuration is analyzed by projecting the loads on a horizontal plane. Determination of depth for deflection: 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: determines the total load in the stair and landing and determines moment and shear forces based on the analyzed moment.

Check depth for flexure: this step helps to cross check the design depth as it is safe for flexure or not, if not revise the depth determined in step 1 and also the loads.

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

Detailing: The arrangement of reinforcement bars and their length are determined and drawn.

- ✓ Stair continuity must be checked when the stair have cantilever landing, however cantilever length is less than main stair length divided by three. Therefore, our stair analysis is not continuous and we have three cases to be checked. One fully loaded, second only main stair is loaded and third only cantilever is loaded. For design we take maximum moment values for main stair and cantilever.

- ✓ Secondary reinforcements essential to distribute the load uniformly and arranged in opposite direction to the main reinforcement. Steel area of secondary reinforcement has to be a maximum of 20% of main reinforcement or minimum reinforcement.

✓ Diameter 8 center to center 200mm reinforcement is sufficient to fully support the raiser. Besides, it is enough to use Ø8 bar per steps.

✓ Main bottom and top negative reinforcement extend to the top part of the landing to avoid corner failure as a result of resultant force caused by tension forces at different axes.

✓ In stair detail reinforcement we do not take any risk we provide minimum reinforcement even if there is no negative moment at the end point of the stair. In addition to this, detail reinforcement is the best if we can form diamond shape at the point where we finish the stair and start the landing.

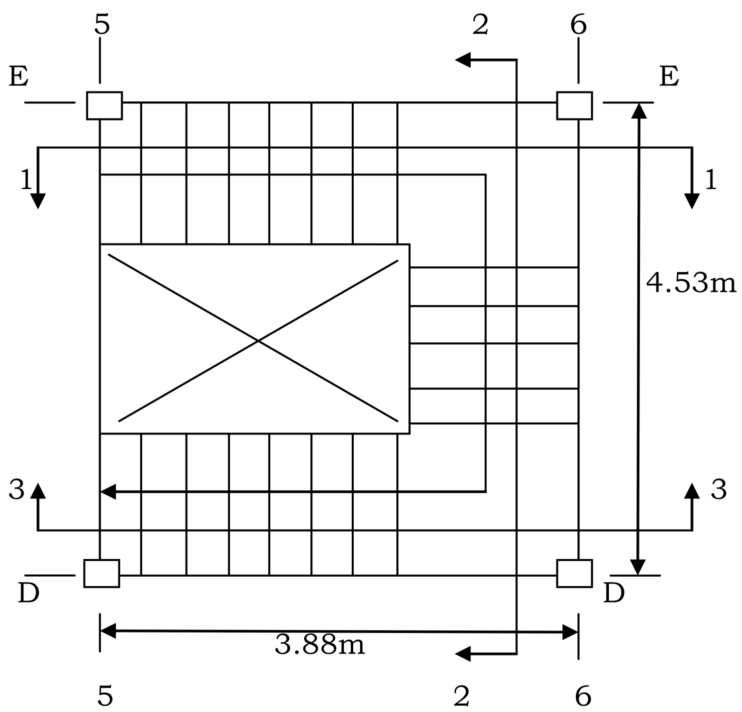


Figure 4.1 Typical Staircase from ground floor to 4th floor

4.3 Analysis and design

4.3.1 Material Properties

Concrete class C-20/25 Partial safety factor

$fck = 20Mpa$ $rc = 1.5$

$fcd = 11.33Mpa$ $rs = 1.15$

$Steel\ grade\ S - 400$ $\phi = 12\ mm$

$fyk = 400MPa$

Section 1-1

Step 1-Geometrical data

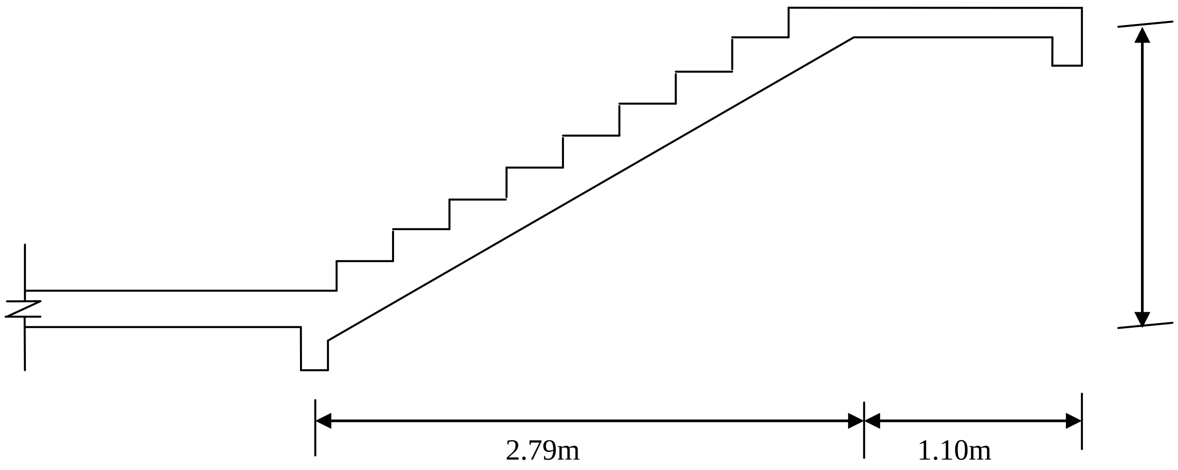


Figure 4.2 section 1-1

Table 4.1 Stair material Used

Material	Thickness (mm)	Unit weight (KN/m^3)
Rc slab	270	25
Cement screed	30	23
Plastering	25	23
Marble	20	27

- Floor finish

Marble = 20mm

Plastering = 25mm

Cement screed = 30mm

Number of thread (T) = 8

Number of riser(R) = 9

Thread = 310mm

Riser =1350/9 = 150mm Live load = 3KN/ m^2

Breadth of step along slope (B)

$$B = \sqrt{T^2 + R^2} = \sqrt{15^2 + 31^2} = 34.43\text{cm}$$

Take noising = 25mm

Width of going = T-N = 310-25 = 285mm

Step 2: Determination of depth for deflection

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if \;\; \rho \le \rho_o$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if \;\; \rho \ge \rho_o$$

Since the formula is done for S-500 in our case S-400

$$F1 = \frac{500}{fyk} = \frac{500}{400} = 1.25$$

$$p_o = 10^{-3}\sqrt{fck}$$

Where:

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

But the K value with different structural system:-

A. Simply supported beam, one-or two-way spanning simply supported slap. (k=1)

B. End span of continuous beam or one-way continuous slab or two-way spanning slab continuous slab continuous over one long side. (k=1.3)

C. Interior span of beam or one way or two-way spanning slab (k=1.5)

D. Slab supported on columns without beams (flat slab) (based on longer span) (k=1.2)

E. Cantilever (k=0.4)

Assume depth (D) = 200mm

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

For simply supported slab k=1.3 PREN 1992-1-1-table 7.4 N

$d = D - \frac{\phi}{2} - cover$

$c_{min} =$ diameter of bar which is $\phi = 12$mm

$c_{dur} = 10$mm With exposure class of Xo with $S - 4$ for 50 year life time

Allowance in design for variation

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

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

$c_{min,b} =$ diameter of bar which is Φ = 12mm

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

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

$\Delta c_{dur,st}$ = 0mm, EBCS EN: 1990: 2015 Recommended

$\Delta c_{dur,add}$ = 0mm, EBCS EN: 1990: 2015 Recommended

Δc_{dev} = 10mm, EBCS EN: 1990: 2014 Recommended

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

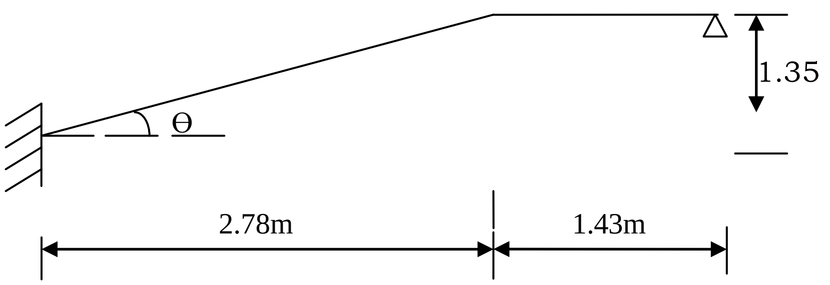
Therefore: c_{min} = max {12mm; 10mm+0 - 0 - 0; 10mm} take c_{min} = 12mm

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

d $= 200 - 22 - 122 = 172$ mm

4.4 Load computation

Taking 1m strip calculate the coming load on stair



$\Theta = \tan^{-1}(1.30/2.78) = 26^{o}$

Take-1m strip width as one- way slab

Dead load on flight

$\text{Own weight of flight} = 1\text{m} * 0.2\text{m} * 25 /(\cos 26^{o}) = 5.61\text{kN/m}$

$\text{Own weight of steps} = 1/2 * 1\text{m} * 0.15 * 25/(\cos 26^{o}) = 2.13\text{kN/m}$

$\text{Weight of soffit plaster} = 0.02 * 23/(\cos 26^{o}) = 0.52\text{kN/m}$

$\text{Weight of cement screed for tread} = 0.02 * 23 = 0.46\text{kN/m}$

$\text{Weightof cement screed for riser} = 0.03 * 9 * 0.15 * 23/2.48 = 0.38\text{kN/m}$

$\text{Weight of marble thread} = 0.03 * 27 = 0.81\text{kN/m}$

$\text{Weight of marble riser} = 0.03 * 9 * 0.15 * 27/2.48 = 0.29\text{kN/m}$

Total dead load = 10.28kN/m

Live load on stair

From ES EN 1991-1-1:2015, Table 6.2:

Stairs are under category A and a uniformly distributed area load between 2 to 4 is allowed.

$Q_k = 3\ \text{KN/m}^2$

Design load on flight=1.35*DL+1.5*LL =1.35*10.28+1.5*3

Design load on flight =18.35KN/m

Dead load on landing: -

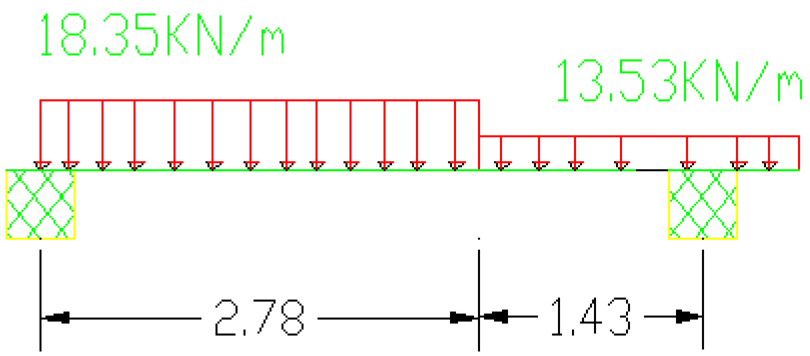
- $\text{Self} - \text{weight} = D_{used} * \gamma_{concrete} * 1m = 200mm * 25KN/m/1000 = 5KN/m$
- $\text{Ceiling} = t_{ceiling} * \gamma_{concrete} * 1m = 0.020 * 23 * 1m = 0.46KN/m$
- $\text{Mortar} = t_{mortar} * \gamma concrete * 1m = 0.03 * 23 * 1 = 0.69KN/m$
- $\text{Marble} = t_{marbler} * \gamma_{marbler} * 1m = 0.02 * 27 * 1 = 0.54KN/m$
- $\text{Dead load on landing} = (0.54 + 0.69 + 0.46 + 5)KN/m\ 6.69kN/m$

Live load in stair = 3KN/m

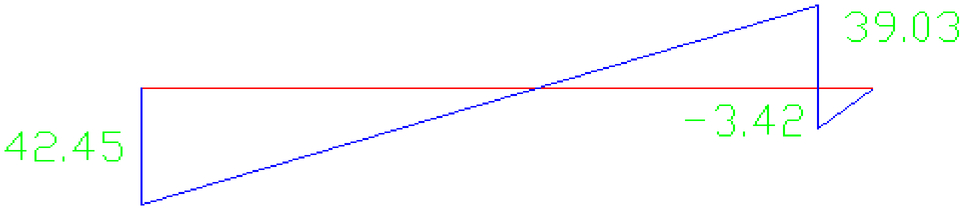
Load on stair

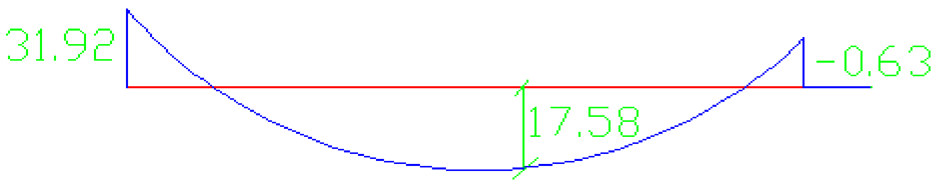
- ✓ Load on flight (Pd=18.35KN/m)
- ✓ Load on landing (Pd=13.53KN/m)

Modeling and analyzing



4.5 Moment and Shear force calculation





Support moment $M_{sd} = 31.92\text{kN/m}$

Span moment $M_{sd} = 17.58\ \text{kN/m}$

Design shear $V_{sd} = 42.45\ \text{kN/m}$

Check depth for flexure

$$d_{min} \geq \sqrt{\frac{M_{max}}{0.8 * fcd * b * K_{max}(1 - 0.4kmax)}}$$

$M_{max} = 31.29\text{KN/m}$ $\quad b = 1000\text{mm}$ $\quad K_{max} = 0.448$

$$d = D - \frac{\emptyset}{2} - \text{cover}, \quad d = 200 - \frac{12}{2} - 22 = 172\text{mm} \quad fcd = 11.33\text{Mpa}$$

$$d_{min} \geq \sqrt{\frac{31.29 * 10^6}{0.8 * 11.33 * 1000 * 0.448(1 - 0.4 * 0.448)}} = 96.44$$

=172mm ≥ 96.9mm OK!!

4.6 Design reinforcement

Main Reinforcement Design

For the given

- Material Data, C-20/25, S-400
- Effective depth, d = 172mm
- Width, b = 1000mm

$$A_{s,min} = 0.26 * fctm * b * \frac{d}{fyk} > 0.0013 * b * d$$

$$A_{s,min} = 0.26 * 2.21 * 1000 * \frac{172}{400} > 0.0013 * 1000 * 172$$

$= 247.1\text{mm}^2 > 223.6\text{mm}^2$ok!!

$\therefore A_{s,min} = 247\text{mm}^2$

$M_{max} = 31.29\text{KNm}$

$V_{max} = 42.45\text{KN}$

$$\mu sds = \frac{M_{sds}}{fcd * b * d^2} = \frac{31.29\text{KNm}}{11.33 * 1000 * 172^2} = 0.093$$

$$z = \frac{d}{2} * (1 + \sqrt{1 + 2 * \mu sds})$$

$$z = \frac{172}{2} * (1 + \sqrt{1 + 2 * 0.093}) = 179.6mm$$

$$A_{s,req} = \frac{Msds}{fyd * z} = \frac{31.29KNm}{347.83 * 179.6} = 500mm^2$$

$\therefore A_{s,min} < A_{s,req} = 247.1mm^2 < 500mm^2$, so take area of required ($A_{s,req}$).

$S = \frac{b * as}{A_{s,req}}$, assume the diameter of bar $\phi = 12$, $as = \frac{\pi * D^2}{4} = \frac{\pi * 12^2}{4} = 113.1mm^2$

$S = \frac{1000 * 113.1}{500mm^2} = 226.2mm$ take $S = 220mm$

According to EBCS-EN-1992-1-1-2015

$S_{max} \leq \{3D, 400mm, Scalcu\}, \{3 * 200, 400mm, 220mm\}$ EBCS – EN 1992 – 1 – 1 – 2014

$S_{max} \leq \{6000mm, 400\ mm, 220mm\}$,

$S_{max} = 220mm$

$$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 113.1}{220} = 514mm^2$$

Provide ф10 C/C 220mm Top bar

Since the span moment is half of the support moment, the spacing calculated will be twice that for support so the governing spacing is:-

$Smax \leq \{3D, 400mm, Scal\}, \{3 * 200, 400mm, 2 * 220mm\}$

$Smax \leq \{600, 400mm, 440mm\}$

Provide ф10 C/C 400mm bottom bar

Transversal (secondary) reinforcement design

According to Euro code 2 Part 1, 1 – EN 1992-1-1-2002 Section 9.3.1.1 the ratio of

Secondary to the main reinforcement shall be at least equal to 20% of the main reinforcement

A_s, transverse $\geq$ 20% $A_{st,cal}$

A_s, transverse $\geq 0.2 * 500mm^2 = 100mm^2$

d = 200 – 22 – 12 – 4 = 162mm, for Ø8 main transverse rebar

$$A_{s,min} = 0.26 * \frac{fctm}{fyk} * b * d$$

$$A_{s,min} = 0.26 * \frac{2.21}{400} * 1000 * 162 = 232.71mm^2$$

$\therefore\ A_{s,min} \geq A_s$, transverse, i.e $232.71mm^2 \geq 100mm^2$, so take $A_{s,min}$.

$S = \frac{b * as}{Asr}$, assume the diameter of bar$\phi = 8$, $as = \frac{\pi d^2}{4} = \frac{\pi * 8^2}{4} = 50.24mm^2$

$$S = \frac{1000 * 50.24}{232.71mm^2} = 205.89mm \quad \text{take } S = 200mm$$

$S_{max} \leq \{3.5D, 450mm, S_{cal}\}, \{3.5 * 160, 450mm, 200mm\},$

$S_{max} \leq \{700mm, 450mm, 200mm\},$

$S_{max} = 200mm$

Provide ϕ8 C/C 200mm

Stair design for shear

C-25 S-400

$fcd = 11.33Mpa \qquad fyd = 400Mpa$

Concrete shear capacity

$V_{Rd,c} = [C_{Rd,c}\ K(100\rho 1 fck)^{1/3} + \sigma_{cp}]b_w * d$[EBCS EN: 1992 − 1 − 1: 2015 Eqn. 6.2a]

$V_{Rd,c} = (V_{min} + K_1\sigma_{cp})b_w * d$[EBCS EN: 1992 − 1 − 1: 2015 Eqn. 6.2b]

Take which ever maximum shear resistance!

$$C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$$

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0 = 1 + \sqrt{\frac{200}{172}} = 2.078$$

K= 2.078> 2 take K= 2, K1= 0.15 recommended value based on [EBCS EN: 1992 -1-1: 2015]

$$V_{min} = [0.035 * 2^{3/2} * 20^{1/2}] = 0.442KN$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} =< 0.2fcd$$

N_{Ed} Is the axial force in the cross − section due to loading or prestressing [in N]

(N_{Ed} > 0 for compression). The influence of imposed deformations on N_{Ed} may be ignore.

$$\rho = \frac{A_s}{b_w * d} \leq 0.02$$

$$\frac{A_s}{b_w * d} = \frac{514}{1000 * 172} = 0.0029 \text{ Take } 0.0029 \text{ which is less than } 0.02$$

$$V_{Rd,s} = [0.12 * 2(100 * 0.002 * 20)1/3 + 0.15 * 0] * 1000 * 172 = 65.53KN$$

$$V_{Rd,s} = (V_{min} + K_1\sigma_{cp})b_w * d = (0.443 + 0) * 1000 * 172 = 76.2$$

65.53KN ≤ 76.2KN take 76.2KN and compare with maximum shear design

Design shear = 42.45KN

$V_{Rd,s} = 76.2KN > 42.45KN \ldots . ok!!$

Recheck depth with the new rebar

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

$$\rho = \frac{A_s}{b_w * d} = \frac{514}{000 * 172} = 0.0029 = 0.29\%$$

Therefore we use the first equations, because

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if\ \ \rho \leq \rho_o$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if\ \ \rho \geq \rho_o$$

$\rho = 0$ Compression reinforcement for slab

$$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{\rho o}{\rho} + 3.2\sqrt{fck} * \left(\frac{\rho o}{\rho} - 1\right)^{pkcf\frac{3}{2}})$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{20}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right]$$

$$\frac{l}{d} = 59.36 * \frac{500}{fyk\frac{A_{s,req}}{A_{s,prov}}} = 59.36 * \frac{500}{400\frac{500}{514}}$$

$$\frac{l}{d} = 76.27$$

$$d = \frac{5000}{76.27} = 65.55mm$$

Since the effective depth, deff=172mm, is greater than the depth of required, dre=65.55mm, so it satisfy the requirement and also it is safe and economical.

Section 2-2

Step-1 Support condition and span

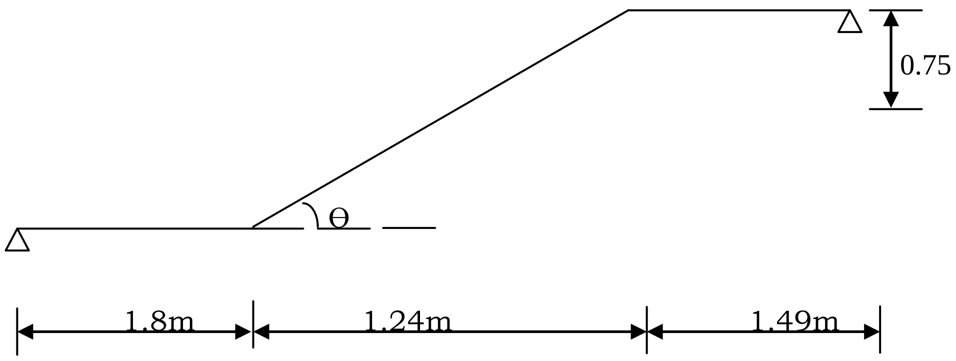


Figure 4.2 Section 2 -2

$$\Theta = \tan^{-1}(0.75/1.24) = 31.2^o$$

- Floor finish

Marble = 20mm Number of thread (T) = 4

Plastering = 25mm Number of riser(R) = 5

Cement screed = 30mm Thread = 310mm

Riser = 150mm Live load = 3KN/ m^2

Breadth of step along slope (B)

$$B = \sqrt{T^2 + R^2} = \sqrt{15^2 + 31^2} = 34.43cm$$

Take noising = 25mm

Width of going = T-N = 310-25 = 285mm

Assume depth (D) = 200mm

Dead load on flight

$$\text{Self} - \text{weight} = \frac{D_{used} * \gamma_{concrete}}{\cos\theta * 1000} = \frac{200mm * 25}{\cos 31.2^o * 1000} = 5.84 \text{ KN/m}$$

$$\text{Steps} = \frac{0.5 * \text{Volume} * \gamma_{concrete}}{\text{horizontal projection of inclined flight}} = \frac{0.5 * 0.172 * 1 * 0.31 * 4 * 25KN/M3}{1.24} = 2.15$$

$$\text{Ceiling} = \frac{t_{ceiling} * t_{mortar}}{\cos\theta} = \frac{0.025 * 23 * 1}{\cos 31.2} = 0.672KN/m$$

$$\text{Mortar} = \frac{t_{mortar} * (R + T) * \text{number of tread} * \gamma_{marbler}}{\text{horizontal projection of inclined flight}} = 1.072 \text{ KN/m}$$

$$\text{Mortar} = \frac{0.03 * (0.172 + 0.31) * 4 * 23}{1.24} = 1.072 \text{ KN/m}$$

$$\text{Marble} = \frac{t_{marbler} * (R + T) * \text{number of tread} * \gamma\text{marble}}{\text{horizontal projection of inclined flight}}$$

$$\text{Marble} = \frac{0.02 * (0.172 + 0.31) * 4 * 27}{1.24} = 0.839 \text{ KN/m}$$

Total Dead load on flight = 10.57KN/m

For simply supported slab k = 1.3 PREN 1992-1-1-table 7.4 N

Live load on stair

From ES EN 1991-1-1:2015, Table 6.2:

Stairs are under category A and a uniformly distributed area load between 2 to 4 is allowed.

Qk = 3 KN/m^2

Design load on flight = 1.35*DL+1.5*LL

= 1.35*10.57+1.5*3 = 18.76KN/m

Dead load on landing

- $\text{Self} - \text{weight} = D_{used} * \gamma_{concrete} * 1m = 200mm * 25KN/m/1000 = 5KN/m$
- $\text{Ceiling} = t_{ceiling} * \gamma_{concrete} * 1m = 0.020 * 23 * 1m = 0.46KN/m$
- $\text{Mortar} = t_{mortar} * \gamma\text{concrete} * 1m = 0.03 * 23 * 1 = 0.69KN/m$
- $\text{Marble} = t_{marbler} * \gamma_{marbler} * 1m = 0.02 * 27 * 1 = 0.54KN/m$
- $\text{Dead load on landing} = (0.54 + 0.69 + 0.46 + 5)KN/m\ 6.69kN/m$ and

Live load in stair = 3KN/m

Design load = 1.35*DL+1.5LL

= 1.35*6.69+1.5*3 = 13.53KN/m

Load on stair

- ✓ Load on flight (Pd = 18.35KN/m)
- ✓ Load on landing (Pd = 13.53KN/m)

Support moment $M_{sd} = 41.14 kN/m$

Span moment $M_{sd} = 17.58\ kN/m$

Design shear $V_{sd} = 33.94\ kN/m$

Check depth for flexure

$$d_{min} \geq \sqrt{\frac{M_{max}}{0.8 * fcd * b * K_{max}(1 - 0.4kmax)}}$$

$M_{max} = 41.14 KN/m \qquad b = 1000mm \qquad K_{max} = 0.448$

$d = D - \frac{\emptyset}{2} - cover, \quad d = 200 - \frac{12}{2} - 22 = 172mm \qquad fcd = 11.33Mpa$

$$d_{min} \geq \sqrt{\frac{41.14 * 10^6}{0.8 * 11.33 * 1000 * 0.448(1 - 0.4 * 0.448)}} = 111.1mm$$

= 172mm ≥ 111.1mm OK!!

Main Reinforcement Design

For the given

- Material Data, C-20/25, S-400
- Effective depth, d = 172mm
- Width, b = 1000mm

$$A_{s,min} = 0.26 * fctm * b * \frac{d}{fyk} > 0.0013 * b * d$$

$$A_{s,min} = 0.26 * 2.21 * 1000 * \frac{172}{400} > 0.0013 * 1000 * 172$$

$= 247.1mm^2 > 223.6mm^2 ok!!$

$\therefore A_{s,min} = 247.1mm^2$

$M_{max} = 41.14KNm$

$V_{max} = 33.94KN$

$$\mu sds = \frac{M_{sds}}{fcd * b * d^2} = \frac{31.29KNm}{11.33 * 1000 * 172^2} = 0.122$$

$$z = \frac{d}{2} * (1 + \sqrt{1 + 2 * \mu sds})$$

$$z = \frac{172}{2} * (1 + \sqrt{1 + 2 * 0.093}) = 181.91mm$$

$$A_{s,req} = \frac{Msds}{fyd * z} = \frac{31.29KNm}{347.83 * 181.91} = 650mm^2$$

$\therefore A_{s,min} < A_{s,req} = 247.1mm^2 < 650mm^2$, so take area of required ($A_{s,req}$).

$S = \frac{b * as}{A_{s,req}}$, assume the diameter of bar $\phi = 12$, $as = \frac{\pi * D^2}{4} = \frac{\pi * 12^2}{4} = 113.1mm^2$

$S = \frac{1000 * 113.1}{650mm^2} = 174mm$ take $S = 170mm$

According to EBCS-EN-1992-1-1-2015

$S_{max} \leq \{3D, 400mm, S_{cal}\}, \{3 * 200, 400mm, 220mm\}$ EBCS – EN 1992 – 1 – 1 – 2015

$S_{max} \leq \{6000mm, 400 mm, 220mm\}$,

$S_{max} = 170mm$

$$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 113.1}{170} = 665.2mm^2$$

Provid Ø12c/c 170mm top bar

Since the span moment is half of the support moment, the spacing calculated will be twice that for support so the governing spacing is:-

$Smax \leq \{3D, 400mm, Scal\}, \{3 * 200, 400mm, 2 * 170mm\}$

$Smax \leq \{600, 400mm, 340mm\}$

Provide ф10 C/C 340mm bottom bar

Transversal (secondary) reinforcement design

According to Euro code 2 Part 1, 1 – EN 1992-1-1-2002 Section 9.3.1.1 the ratio of

Secondary to the main reinforcement shall be at least equal to 20% of the main reinforcement

A_s, transverse $\geq$ 20% $A_{st,cal}$

A_s, transverse $\geq 0.2 * 500mm^2 = 100mm^2$

d = 200 – 22 – 12 – 4 = 162mm, for Ø8 main transverse rebar

$$A_{s,min} = 0.26 * \frac{fctm}{fyk} * b * d$$

$$A_{s,min} = 0.26 * \frac{2.21}{400} * 1000 * 162 = 232.71mm^2$$

$\therefore$ $A_{s,min} \geq A_s$, transverse, i.e $232.71mm^2 \geq 100mm^2$ so take $A_{s,min}$.

$$S = \frac{b * as}{Asr}, \text{assume the diameter of bar} \phi = 8, as = \frac{\pi d^2}{4} = \frac{\pi * 8^2}{4} = 50.24mm^2$$

$$S = \frac{1000 * 50.24}{232.71mm^2} = 205.89mm \quad \text{take } S = 200mm$$

$S_{max} \leq \{3.5D, 450mm, S_{cal}\}, \{3.5 * 160, 450mm, 200mm\}$,

$S_{max} \leq \{700mm, 450mm, 200mm\}$,

$S_{max} = 200mm$

Provide ф8 C/C 200mm

Stair design for shear

C-25 S-400

$fcd = 11.33Mpa \qquad fyd = 400Mpa$

Concrete shear capacity

$$V_{Rd,c} = [C_{Rd,c} K(100\rho 1 fck)^{1/3} + \sigma_{cp}]b_w * d[\text{EBCS EN: } 1992 - 1 - 1\text{: } 2015 \text{ Eqn. } 6.2a]$$

$$V_{Rd,c} = (V_{min} + K_1\sigma_{cp})b_w * d[\text{EBCS EN: } 1992 - 1 - 1\text{: } 2015 \text{ Eqn. } 6.2b]$$

Take which ever maximum shear resistance!

$$C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$$

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0 = 1 + \sqrt{\frac{200}{172}} = 2.078$$

K= 2.078> 2 take K= 2, K1= 0.15 recommended value based on [EBCS EN: 1992 -1-1: 2015]

$$V_{min} = \left[0.035 * 2^{3/2} * 20^{1/2}\right] = 0.442KN$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} =< 0.2fcd$$

N_{Ed} Is the axial force in the cross − section due to loading or prestressing [in N]

(N_{Ed} > 0 for compression). The influence of imposed deformations on N_{Ed} may be ignore.

$$\rho = \frac{A_s}{b_w * d} \leq 0.02$$

$$\frac{A_s}{b_w * d} = \frac{665}{1000 * 172} = 0.0038$$ Take 0.0029 which is less than 0.02

$$V_{Rd,s} = [0.12 * 2(100 * 0.0038 * 20)1/3 + 0.15 * 0] * 1000 * 172 = 81.16KN$$

$$V_{Rd,s} = \left(V_{min} + K_1\sigma_{cp}\right)b_w * d = (0.443 + 0) * 1000 * 172 = 76.2$$

81.16KN ≥ 76.2KN take 81.16KN and compare with maximum shear design

Design shear = 33.94KN

$V_{Rd,s} = 76.2KN > 33.94KN$ …. ok!!

Recheck depth with the new rebar

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

$$\rho = \frac{A_s}{b_w * d} = \frac{514}{000 * 172} = 0.0029 = 0.29\%$$

Therefore we use the first equations

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if \ \rho \leq \rho_o$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if \ \rho \geq \rho_o$$

$\rho = 0$ Compression reinforcement for slab

$$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{\rho o}{\rho} + 3.2\sqrt{fck} * \left(\frac{\rho o}{\rho} - 1\right)^{pkcf\frac{3}{2}})$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{20}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right]$$

$$\frac{\mathrm{l}}{\mathrm{d}} = 48.34$$

$$\frac{l}{d} = 48.34 * \frac{500}{fyk\frac{\mathrm{A_{s,req}}}{\mathrm{A_{s,prov}}}} = 59.36 * \frac{500}{400\frac{650}{665}}$$

$$\mathrm{d} = \frac{5000}{61.83} = 71.91\mathrm{mm}$$

Since the effective depth, deff=172mm, is greater than the depth of required, dre=65.55mm, so it satisfy the requirement and also it is safe and economical.

Section 3-3

Step-1 Support condition and span

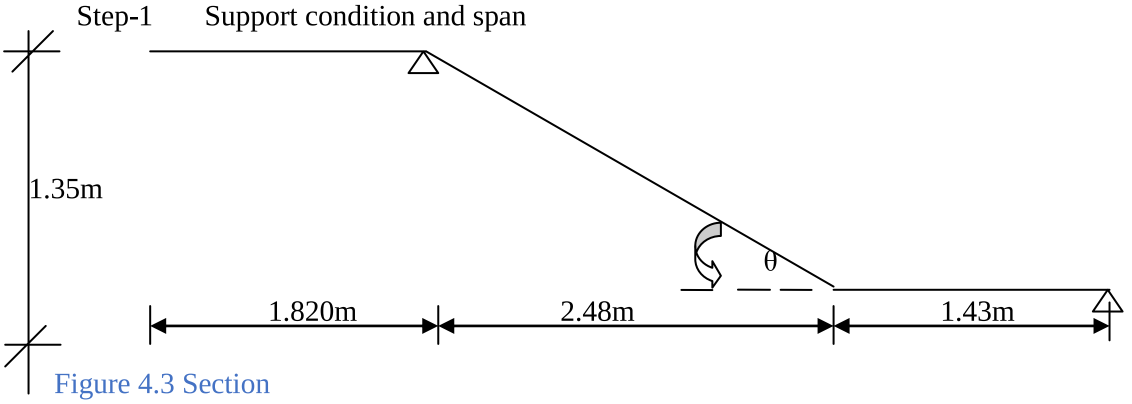


Figure 4.3 Section

$$\theta = \tan^{-1}(1.35/2.48) = (28.56)^{o}$$

Dead load on flight

Taking 1m strip calculate the coming load on stair

$$\mathrm{Self-weight} = \frac{\mathrm{Dused} * \gamma\mathrm{concrete}}{\cos\theta * 1000} = \frac{200mm * 25}{\cos 28.56^{o} * 1000} = 5.69\mathrm{KN/m}$$

$$\mathrm{Steps} = \frac{0.5 * \mathrm{Volume} * \gamma\mathrm{concrete}}{\text{horizontal projection of inclined flight}} = \frac{0.5 * 0.172 * 1 * 0.31 * 4 * 25\mathrm{KN/M3}}{2.48} = 2.15$$

$$\mathrm{Ceiling} = \frac{\mathrm{tceiling} * \gamma\mathrm{mortar}}{\cos\theta} = \frac{0.025 * 23 * 1}{\cos 28.56} = 0.672\mathrm{KN/m}$$

$$\mathrm{Mortar} = \frac{\mathrm{tmortar} * (\mathrm{R} + \mathrm{T}) * \mathrm{numberoftread} * \gamma\mathrm{mortar}}{\mathrm{horizontalprojectionofinclinedflight}} = 1.072\ \mathrm{KN/m}$$

$$\mathrm{Mortar} = \frac{0.03 * (0.172 + 0.31) * 4 * 23}{2.48} = 1.072\ \mathrm{KN/m}$$

$$\text{Marble} = \frac{\text{tmarble} * (R + T) * \text{number of tread} * \gamma\text{marble}}{\text{horizontalprojectionofinclinedflight}}$$

$$\text{Marble} = \frac{0.02 * (0.172 + 0.31) * 4 * 27}{2.48} = 0.839\ \text{KN/m}$$

Total Dead load on flight = 10.57KN/m

Dead load on landing

- $\text{Self} - \text{weight} = D_{used} * \gamma_{concrete} * 1\text{m} = 200\text{mm} * 25\text{KN/m}/1000 = 5\text{KN/m}$
- $\text{Ceiling} = t_{ceiling} * \gamma_{concrete} * 1\text{m} = 0.020 * 23 * 1\text{m} = 0.46\text{KN/m}$
- $\text{Mortar} = t_{mortar} * \gamma\text{concrete} * 1\text{m} = 0.03 * 23 * 1 = 0.69\text{KN/m}$
- $\text{Marble} = t_{marbler} * \gamma_{marbler} * 1\text{m} = 0.02 * 27 * 1 = 0.54\text{KN/m}$
- $\text{Dead load on landing} = (0.54 + 0.69 + 0.46 + 5)\text{KN/m}\ 6.69\text{kN/m and}$

Live load in stair = 3KN/m

Design load = 1.35*DL+1.5LL =1.35*6.69+1.5*3

Design load = 13.53KN/m

Load on stair

- ✓ Load on flight (Pd=18.35KN/m)
- ✓ Load on landing (Pd=13.53KN/m)

Support moment $M_{sd} = 22.36\text{kN/m}$

Span moment $M_{sd} = 25.1\ \text{kN/m}$

Design shear $V_{sd} = 37.97\ \text{kN/m/m}$

Check depth for flexure

$$d_{min} \geq \sqrt{\frac{M_{max}}{0.8 * \text{fcd} * b * K_{max}(1 - 0.4\text{kmax})}}$$

$M_{max} = 22.36\text{KN/m}$ $b = 1000\text{mm}$ $K_{max} = 0.448$

$d = D - \frac{\emptyset}{2} - \text{cover},\quad d = 200 - \frac{12}{2} - 22 = 172\text{mm}\qquad fcd = 11.33\text{Mpa}$

$$d_{min} \geq \sqrt{\frac{22.36 * 10^6}{0.8 * 11.33 * 1000 * 0.448(1 - 0.4 * 0.448)}} = 81.9$$

= 172mm ≥ 81.9mm OK!!

Design reinforcement

Main Reinforcement Design

For the given

- Material Data, C-20/25, S-400
- Effective depth, d = 172mm
- Width, b = 1000mm

$$A_{s,min} = 0.26 * fctm * b * \frac{d}{fyk} > 0.0013 * b * d$$

$$A_{s,min} = 0.26 * 2.21 * 1000 * \frac{172}{400} > 0.0013 * 1000 * 172$$

$$= 247.1mm^2 > 223.6mm^2 ok!!$$

$$\therefore A_{s,min} = 247mm^2$$

$$M_{max} = 22.36KNm$$

$$V_{max} = 37.97KN$$

$$\mu sds = \frac{M_{sds}}{fcd * b * d^2} = \frac{22.36}{11.33 * 1000 * 172^2} = 0.067$$

$$z = \frac{d}{2} * (1 + \sqrt{1 + 2 * \mu sds})$$

$$z = \frac{172}{2} * (1 + \sqrt{1 + 2 * 0.067}) = 1177.5mm$$

$$A_{s,req} = \frac{Msds}{fyd * z} = \frac{22.36}{347.83 * 177.5mm} = 362mm^2$$

$\therefore A_{s,min} < A_{s,req} = 247.1mm^2 < 362mm^2$, so take area of required ($A_{s,req}$).

$S = \frac{b * as}{A_{s,req}}$, assume the diameter of bar $\phi = 12$, $as = \frac{\pi * D^2}{4} = \frac{\pi * 12^2}{4} = 113.1mm^2$

$$S = \frac{1000 * 113.1}{362mm^2} = 226.2mm \quad \text{take } S = 310mm$$

According to EBCS-EN-1992-1-1-2015

$S_{max} \leq \{3D, 400mm, Scalcu\}, \{3 * 200, 400mm, 310mm\}$ EBCS − EN 1992 − 1 − 1 − 2014

$S_{max} \leq \{6000mm, 400\ mm, 310mm\},$

$S_{max} = 310mm$

$$A_{st,used} = \frac{b * as}{S_{provided}} = \frac{1000 * 113.1}{310} = 364.8mm^2$$

Provide ϕ10 C/C 310mm Top bar

Since the span moment is half of the support moment, the spacing calculated will be twice that for support so the governing spacing is:-

$Smax \leq \{3D, 400mm, Scal\}, \{3 * 200, 400mm, 2 * 220mm\}$

$Smax \leq \{600, 400mm, 440mm\}$

Provide ϕ10 C/C 400mm bottom bar

Transversal (secondary) reinforcement design

According to Euro code 2 Part 1, 1 – EN 1992-1-1-2002 Section 9.3.1.1 the ratio of

Secondary to the main reinforcement shall be at least equal to 20% of the main reinforcement

$A_s, \text{transverse} \geq 20\%\ A_{st,cal}$

$A_s, \text{transverse} \geq 0.2 * 500mm^2 = 100mm^2$

d = 200 – 22 – 12 – 4 = 162mm, for Ø8 main transverse rebar

$$A_{s,min} = 0.26 * \frac{fctm}{fyk} * b * d$$

$$A_{s,min} = 0.26 * \frac{2.21}{400} * 1000 * 162 = 232.71mm^2$$

$\therefore A_{s,min} \geq A_s$, transverse, i.e $232.71mm^2 \geq 100mm^2$, so take $A_{s,min}$.

$S = \frac{b * as}{Asr}$, assume the diameter of bar $\phi = 8$, $as = \frac{\pi d^2}{4} = \frac{\pi * 8^2}{4} = 50.24mm^2$

$$S = \frac{1000 * 50.24}{232.71mm^2} = 205.89mm \quad \text{take } S = 200mm$$

$S_{max} \leq \{3.5D, 450mm, S_{cal}\}, \{3.5 * 160, 450mm, 200mm\},$

$S_{max} \leq \{700mm, 450mm, 200mm\},$

$S_{max} = 200mm$

Provide ф8 C/C 200mm

Stair design for shear

C-25 S-400

$fcd = 11.33Mpa \qquad fyd = 400Mpa$

Concrete shear capacity

$V_{Rd,c} = [C_{Rd,c}\ K(100\rho 1 fck)^{1/3} + \sigma_{cp}]b_w * d$[EBCS EN: 1992 − 1 − 1: 2015 Eqn. 6.2a]

$V_{Rd,c} = (V_{min} + K_1\sigma_{cp})b_w * d$[EBCS EN: 1992 − 1 − 1: 2015 Eqn. 6.2b]

Take which ever maximum shear resistance!

$$C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$$

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0 = 1 + \sqrt{\frac{200}{172}} = 2.078$$

K= 2.078 > 2 take K = 2, K1= 0.15 recommended value based on [EBCS EN: 1992 -1-1: 2015]

$V_{min} = [0.035 * 2^{3/2} * 20^{1/2}] = 0.442KN$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} =< 0.2fcd$$

N_{Ed} Is the axial force in the cross − section due to loading or prestressing [in N]

(N_{Ed} > 0 for compression). The influence of imposed deformations on N_{Ed} may be ignore.

$$\rho = \frac{A_s}{b_w * d} \leq 0.02$$

$\frac{A_s}{b_w * d} = \frac{364.8}{1000 * 172} = 0.0021$ Take 0.0021 which is less than 0.02

$V_{Rd,s} = [0.12 * 2(100 * 0.0021 * 20)1/3 + 0.15 * 0] * 1000 * 172 = 65.55KN$

$V_{Rd,s} = (V_{min} + K_1\sigma_{cp})b_w * d = (0.443 + 0) * 1000 * 172 = 76.2$

65.55KN ≤ 76.2KN take 76.2KN and compare with maximum shear design

Design shear = 37.97KN

$V_{Rd,s} = 76.2KN > 37.97KN$ .... ok!!

Recheck depth with the new rebar

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

$$\rho = \frac{A_s}{b_w * d} = \frac{364.8}{000 * 172} = 0.0021 = 0.21\%$$

Therefore we use the first equations

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if \ \rho \le \rho_o$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] if \ \rho \ge \rho_o$$

$\rho = 0$ Compression reinforcement for slab

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{20}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right]$$

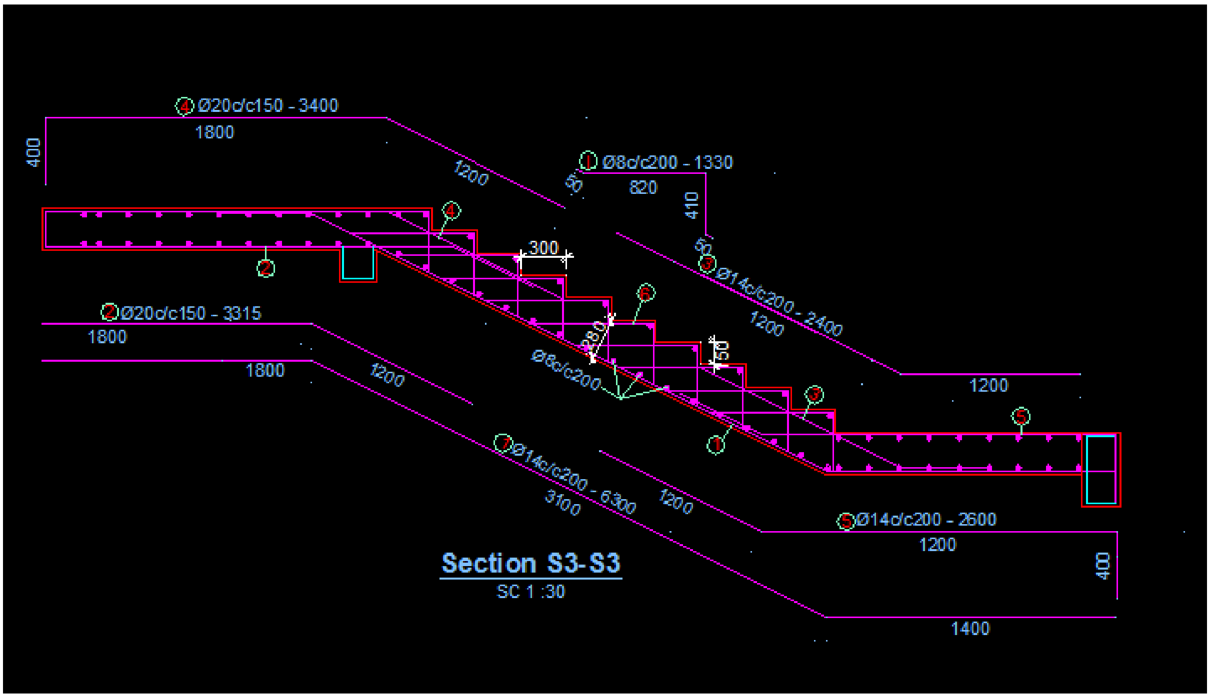
$$\frac{l}{d} = 59.36 * \frac{500}{fyk\frac{A_{s,req}}{A_{s,prov}}} = 59.36 * \frac{500}{400\frac{362}{364.8}}$$

$$\frac{l}{d} = 74.77$$

$$d = \frac{5000}{74.77} = 66.87mm$$

Since the effective depth, deff=172mm, is greater than the depth of required, dre=65.55mm, so it satisfy the requirement and also it is safe and economical.

Stair reinforcement detailing



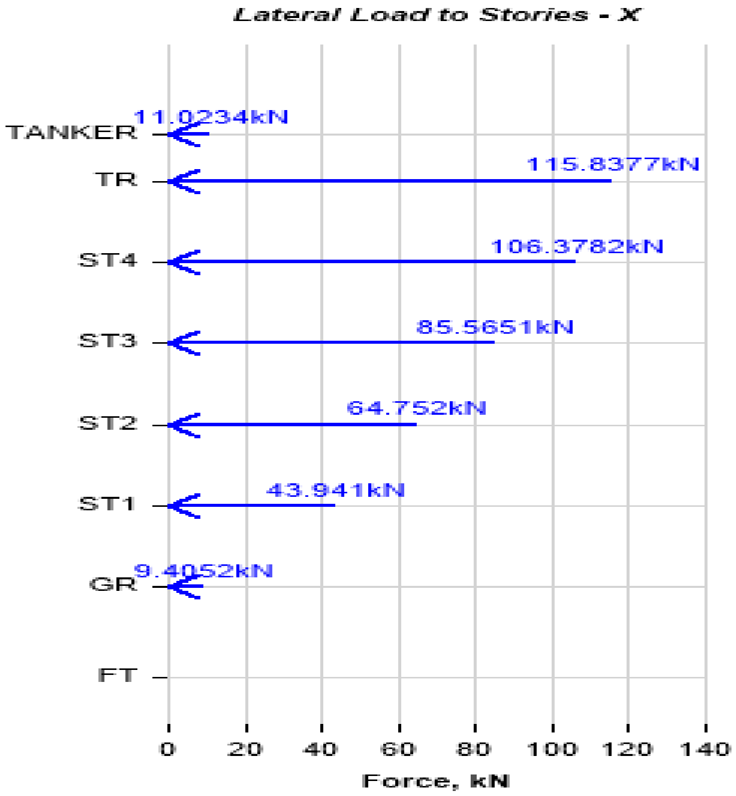
4 Ø20c/c150 - 3400
1800
400
1200
1 Ø8c/c200 - 1330
820
410
50
300
7 Ø14c/c200 - 2400
1200
2 Ø20c/c150 - 3315
1800
1800
1200
Ø8c/c200
3 Ø14c/c200 - 6300
3100
1200
5 Ø14c/c200 - 2600
1200
400
1400
Section S3-S3
SC 1 :30

5 Wind Load Analysis

Calculated Base Shear

$\mathbf{V_{coeff}} = \mathbf{S_d}(\mathbf{T_1})\boldsymbol{\lambda}$

Direction	Period Used (sec)	W (kN)	F_b (kN)
X - Ecc. Y	0.974	11060.2352	436.9025



Story	Elevation m	X-Dir kN	Y-Dir kN
TANKER	20.09	11.0234	0
TR	18	115.8377	0
ST4	14.4	106.3782	0
ST3	10.8	85.5651	0
ST2	7.2	64.752	0
ST1	3.6	43.941	0
GR	0	9.4052	0
FT	-4	0	0

EUROCODE8 2004 Auto Seismic Load Calculation

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

Direction and Eccentricity

Direction = X + Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

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

Structure Height above Base, H $H = 24.09m$

Factors and Coefficients

Country =

Design Ground Acceleration, $a_g = 0.1g$

Ground Type [EC Table 3.1] = B

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

Constant Acceleration Period Limit, T_B [EC Table 3.2] $T_B = 0.15$ sec

Constant Acceleration Period Limit, T_C [EC Table 3.2] $T_C = 0.5$ sec

Constant Displacement Period Limit, T_D [EC Table 3.2] $T_D = 2$ sec

Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$

Behavior Factor, q [EC 3.2.2.5(3)] $q = 3.9$

Seismic Response

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

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

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

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

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

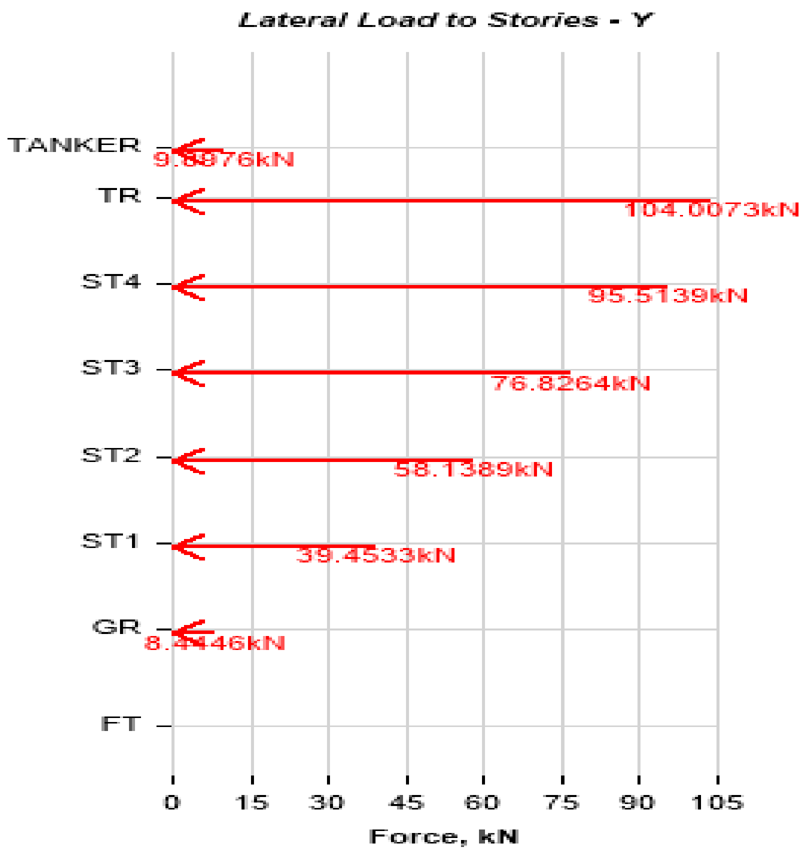
Equivalent Lateral Forces

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

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
Y - Ecc. X	1.084	11060.2352	392.2821

Applied Story Forces



Story	Elevation m	X-Dir kN	Y-Dir kN
TANKER	20.09	0	9.8976
TR	18	0	104.0073
ST4	14.4	0	95.5139
ST3	10.8	0	76.8264
ST2	7.2	0	58.1389
ST1	3.6	0	39.4533
GR	0	0	8.4446
FT	-4	0	0

EUROCODE8 2004 Auto Seismic Load Calculation

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

Direction and Eccentricity

Direction = Y + Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2] $C_t = 0.075\text{m}$

Structure Height above Base, H $H = 24.09$ m

Factors and Coefficients

Country =

Seismic Response

Design Ground Acceleration, a_g $a_g = 0.1g$

Ground Type [EC Table 3.1] = B

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

Constant Acceleration Period Limit, T_B [EC Table 3.2] $T_B = 0.15$ sec

Constant Acceleration Period Limit, T_C [EC Table 3.2] $T_C = 0.5$ sec

Constant Displacement Period Limit, T_D [EC Table 3.2] $T_D = 2$ sec

Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$

Behavior Factor, q [EC 3.2.2.5(3)] $q = 3.9$

Seismic Response

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

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

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

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

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

6 Earth quake Analysis

6.1 Introduction

Building structures are exposed to lateral loads of the earth quake and wind loads. The occurrence of these loads simultaneously on the structure is very rare and therefore, we will design the structure for the governing load among the two. Thus, the design process involves the determination of the two loads separately and designing for the maximum effect.

Using, Equivalent static (building code) analysis method

This type of analysis is applied to buildings whose response is not significantly affected by contribution from higher modes vibration. These requirements are claimed to be satisfied by buildings which;

a) Meets the criteria for regularity in plan and elevation.

b) Have fundamental periods of vibration T1≤ 4TC 2Sec

6.2 Determination of Center of Mass and Seismic Load

The horizontal forces at each floor level, Fi, are distributed to lateral load resistive structural elements in proportion to their rigidities assuming rigid floor diaphragms. Center of mass (Xm, Ym): it is a point on a floor level where the whole floor mass and its inertial effects can be replaced using a lumped equivalent mass.(Ministry of Ethiopian ecucation, 2015)

$$X_m = \frac{\sum w_i * x_i}{\sum w_i}$$

$$Y_m = \frac{\sum w_i * y_i}{\sum w_i}$$

Where: X_m and :Y_m are the coordinate of the point of application of Fi

When the seismic action is parallel to the Y-direction and X-direction respectively

Foundation

Table 6.1 Center of mass calculation for foundation

Column on	Width(m)	Depth(m)	Height(m)	Weight(KN)	Moment Arm		Moment	
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis -1 A	0.35	0.45	1.35	5.32	0	0	0	0
Axis -1 D	0.35	0.45	1.35	5.32	0	9.51	50.5932	0

Axis - 2A	0.35	0.45	1.35	5.32	3.77	0	0	20.0564
Axis - 2B	0.35	0.45	1.35	5.32	3.77	5.63	29.9516	20.0564
Axis - 2D	0.35	0.45	1.35	5.32	3.77	9.51	50.5932	20.0564
Axis - 3A	0.35	0.45	1.35	5.32	10.29	0	0	54.7428
Axis - 3B	0.35	0.45	1.35	5.32	10.29	5.63	29.9516	54.7428
Axis - 3D	0.35	0.45	1.35	5.32	10.29	9.51	50.5932	54.7428
Axis - 3E	0.35	0.45	1.35	5.32	10.29	11	58.52	54.7428
Axis - 3F	0.35	0.45	1.35	5.32	10.29	17.53	93.2596	54.7428
Axis - 4F	0.35	0.45	1.35	5.32	15.05	17.53	93.2596	80.066
Axis - 5A	0.35	0.45	1.35	5.32	15.91	0	0	84.6412
Axis - 5C	0.35	0.45	1.35	5.32	15.91	6.48	34.4736	84.6412
Axis - 5E	0.35	0.45	1.35	5.32	15.91	11	58.52	84.6412
Axis - 5F	0.35	0.45	1.35	5.32	15.91	17.53	93.2596	84.6412
Axis - 6A	0.35	0.45	1.35	5.32	19.8	0	0	105.336
Axis - 6C	0.35	0.45	1.35	5.32	19.8	6.48	34.4736	105.336
Axis - 6E	0.35	0.45	1.35	5.32	19.8	11	58.52	105.336
Axis - 6F	0.35	0.45	1.35	5.32	19.8	17.53	93.2596	105.336
Axis - 6G	0.35	0.45	1.35	5.32	19.8	21.3	113.316	105.336
			Total	106.4			849.2848	1089.217

$$X_m = \frac{\sum w_i * x_i}{\sum w_i} = \frac{410.144}{89.6} = 4.58\text{KNm}$$

$$Y_m = \frac{\sum w_i * y_i}{\sum w_i} = \frac{841.22}{89.6} = 9.39\text{KNm}$$

Table 6.2 Center of mass calculation for ground slab

Column on	Width	Depth(m)	Height(m)	Weight(KN)	Moment Arm		Moment	
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis -1 A	0.35	0.45	3.2	12.6	0	0	0	0
Axis -1 D	0.35	0.45	3.2	12.6	0	9.51	119.826	0
Axis - 2A	0.35	0.45	3.2	12.6	3.77	0	0	47.502
Axis - 2B	0.35	0.45	3.2	12.6	3.77	5.63	70.938	47.502
Axis - 2D	0.35	0.45	3.2	12.6	3.77	9.51	119.826	47.502
Axis - 3A	0.35	0.45	3.2	12.6	10.29	0	0	129.654
Axis - 3B	0.35	0.45	3.2	12.6	10.29	5.63	70.938	129.654
Axis - 3D	0.35	0.45	3.2	12.6	10.29	9.51	119.826	129.654

Axis - 3E	0.35	0.45	3.2	12.6	10.29	11	138.6	129.654
Axis - 3F	0.35	0.45	3.2	12.6	10.29	17.53	220.878	129.654
Axis - 4F	0.35	0.45	3.2	12.6	15.05	17.53	220.878	189.63
Axis - 5A	0.35	0.45	3.2	12.6	15.91	0	0	200.466
Axis - 5C	0.35	0.45	3.2	12.6	15.91	6.48	81.648	200.466
Axis - 5E	0.35	0.45	3.2	12.6	15.91	11	138.6	200.466
Axis - 5F	0.35	0.45	3.2	12.6	15.91	17.53	220.878	200.466
Axis - 6A	0.35	0.45	3.2	12.6	19.8	0	0	249.48
Axis - 6C	0.35	0.45	3.2	12.6	19.8	6.48	81.648	249.48
Axis - 6E	0.35	0.45	3.2	12.6	19.8	11	138.6	249.48
Axis - 6F	0.35	0.45	3.2	12.6	19.8	17.53	220.878	249.48
Axis - 6G	0.35	0.45	3.2	12.6	19.8	21.3	268.38	249.48
	TOTAL			252			2232.342	3029.67

Beam									
Beam	Between Axis	Width(m)	Depth(m)	Length	Weight(KN)	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis - 1	A&D	0.3	0.4	9.51	28.53	0	0	0	0
Axis - 2	A&D	0.3	0.4	9.51	28.53	1.89	4.76	135.8028	53.9217
Axis - 3	A&G	0.3	0.4	21.3	63.9	5.15	10.65	680.535	329.085
Axis - 4	F&G	0.3	0.4	21.3	63.9	7.53	10.65	680.535	481.167
Axis - 5	A&G	0.3	0.4	21.3	63.9	7.96	10.65	680.535	508.644
Axis - 6	A&G	0.3	0.4	21.3	63.9	9.9	10.65	680.535	632.61
Axis - 7	1&6	0.3	0.4	19.74	59.22	0	0	0	0
Axis - 8	1&6	0.3	0.4	19.74	59.22	9.9	2.82	167.0004	586.278
Axis - 9	1&6	0.3	0.4	19.74	59.22	9.9	3.24	191.8728	586.278
Axis - 10	1&6	0.3	0.4	19.74	59.22	9.9	4.75	281.295	586.278
Axis - 11	3&6	0.3	0.4	9.5	28.5	4.75	5.5	156.75	135.375
Axis - 12	3&6	0.3	0.4	9.5	28.5	4.75	8.76	249.66	135.375
Axis - 13	3&6	0.3	0.4	9.5	28.5	4.75	10.65	303.525	135.375
Total weight					635.04			4208.046	4170.3867

				Stair					
Stair Type	Name	Width	Length	DL(KN/m2)	Weight	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Stair 1	Flight 1	1.3	4.43	10.28	59.20252	1.35	2.78	164.583	164.583
Stair 1	Flight 2	1.3	3.88	10.6	53.4664	0.75	1.24	66.29834	66.29834
Stair 1	Flight 3	1.3	4.43	10.28	59.20252	1.35	2.78	164.583	164.583
Total weight					171.8714			395.4643	395.4643

				Partion wall					
Wall	Between Axis	Width/Diamter(m)	Depth(m)	Length	Weight(KN)	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
A&B	1	0.2	3.05	4.53	38.6862	0	2.49	96.32864	0
A&B	3	0.15	3.05	3.7	23.6985	11	2.075	49.17439	260.6835
A&B	5	0.15	3.05	2.68	17.1654	15.7	1.565	26.86385	269.4968
B&C	1	0.2	3.05	0.62	5.2948	0	5.75	30.4451	0
B&C	2&3	0.15	3.05	0.85	5.44425	4.845	5.75	31.30444	26.37739
B&C	2&3	0.15	3.05	0.85	5.44425	7.38	5.75	31.30444	40.17857
B&C	2&3	0.15	3.05	0.85	5.44425	9.53	5.75	31.30444	51.8837
C&D	1	0.2	3.05	0.62	5.2948	0	6.8	36.00464	0
C&D	2&3	0.15	3.05	2.93	18.76665	4.845	6.8	127.6132	90.92442
C&D	2&3	0.15	3.05	2.93	18.76665	7.38	6.8	127.6132	138.4979
C&D	2&3	0.15	3.05	2.93	18.76665	9.53	6.8	127.6132	178.8462
E&F		0.15	3.05	6.2	39.711	14.14	14.22	564.6904	561.7121

	3&4					5			
1&2	B	0.15	3.05	2.34	14.9877	3.825	5.48	82.1326	57.32795
2&3	B	0.15	3.05	6.18	39.5829	6.18	5.48	216.9143	244.6223
5&6	A&B	0.15	3.05	2.91	18.63855	17.36 5	6.21	115.7454	323.6584
2&3	D	0.15	3.05	4.09	26.19645	19.71	6.561	171.8749	516.332
3&4	E	0.15	3.05	4.3	27.5415	13.46 5	7.05	194.1676	370.8463
5&6	E	0.15	3.05	4.3	27.5415	17.36 5	12.06	332.1505	478.2581
			TOTAL		356.972			2393.245	3609.646

$$X_m = \frac{\sum w_i * x_i}{\sum w_i} = \frac{8536.033}{1142.74} = 7.48\text{KNm}$$

$$Y_m = \frac{\sum w_i * y_i}{\sum w_i} = \frac{10315.73}{1142.74} = 9.027\text{KNm}$$

Table 6.3 Center of mass calculation for first slab

Column on	Width	Depth(m)	Height(m)	Weight(KN)	Moment Arm		Moment	
					X(m)	Y(m)	Mx=Wi*Y i	My=Wi*X i
Axis -1 A	0.35	0.45	3.2	12.6	0	0	0	0
Axis -1 D	0.35	0.45	3.2	12.6	0	9.51	119.826	0
Axis - 2A	0.35	0.45	3.2	12.6	3.77	0	0	47.502
Axis - 2B	0.35	0.45	3.2	12.6	3.77	5.63	70.938	47.502
Axis - 2D	0.35	0.45	3.2	12.6	3.77	9.51	119.826	47.502
Axis - 3A	0.35	0.45	3.2	12.6	10.29	0	0	129.654
Axis - 3B	0.35	0.45	3.2	12.6	10.29	5.63	70.938	129.654
Axis - 3D	0.35	0.45	3.2	12.6	10.29	9.51	119.826	129.654
Axis - 3E	0.35	0.45	3.2	12.6	10.29	11	138.6	129.654
Axis - 3F	0.35	0.45	3.2	12.6	10.29	17.53	220.878	129.654
Axis - 4F	0.35	0.45	3.2	12.6	15.05	17.53	220.878	189.63
Axis - 5A	0.35	0.45	3.2	12.6	15.91	0	0	200.466
Axis - 5C	0.35	0.45	3.2	12.6	15.91	6.48	81.648	200.466
Axis - 5E	0.35	0.45	3.2	12.6	15.91	11	138.6	200.466

Axis - 5F	0.35	0.45	3.2	12.6	15.91	17.53	220.878	200.466
Axis - 6A	0.35	0.45	3.2	12.6	19.8	0	0	249.48
Axis - 6C	0.35	0.45	3.2	12.6	19.8	6.48	81.648	249.48
Axis - 6E	0.35	0.45	3.2	12.6	19.8	11	138.6	249.48
Axis - 6F	0.35	0.45	3.2	12.6	19.8	17.53	220.878	249.48
Axis - 6G	0.35	0.45	3.2	12.6	19.8	21.3	268.38	249.48
	TOTAL			252			2232.342	3029.67

Beam									
Beam	Between Axis	Width(m)	Depth(m)	Length	Weight(KN)	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis - 1	A&D	0.3	0.4	9.51	28.53	0	0	0	0
Axis - 2	A&D	0.3	0.4	9.51	28.53	1.89	4.76	135.8028	53.9217
Axis - 3	A&G	0.3	0.4	21.3	63.9	5.15	10.65	680.535	329.085
Axis - 4	F&G	0.3	0.4	21.3	63.9	7.53	10.65	680.535	481.167
Axis - 5	A&G	0.3	0.4	21.3	63.9	7.96	10.65	680.535	508.644
Axis - 6	A&G	0.3	0.4	21.3	63.9	9.9	10.65	680.535	632.61
Axis - 7	1&6	0.3	0.4	19.74	59.22	0	0	0	0
Axis - 8	1&6	0.3	0.4	19.74	59.22	9.9	2.82	167.0004	586.278
Axis - 9	1&6	0.3	0.4	19.74	59.22	9.9	3.24	191.8728	586.278
Axis - 10	1&6	0.3	0.4	19.74	59.22	9.9	4.75	281.295	586.278
Axis - 11	3&6	0.3	0.4	9.5	28.5	4.75	5.5	156.75	135.375
Axis - 12	3&6	0.3	0.4	9.5	28.5	4.75	8.76	249.66	135.375
Axis - 13	3&6	0.3	0.4	9.5	28.5	4.75	10.65	303.525	135.375
Total weight					635.04			4208.046	4170.3867

				Stair					
Stair Type	Name	Width	Length	DL(KN/m 2)	Weight	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi* Yi	My=Wi* Xi
Stair 1	Flight 1	1.3	4.43	10.28	59.20252	1.35	2.78	164.583	164.583
Stair 1	Flight 2	1.3	3.88	10.6	53.4664	0.75	1.24	66.29834	66.29834
Stair 1	Flight 3	1.3	4.43	10.28	59.20252	1.35	2.78	164.583	164.583
Total weight					171.8714			395.4643	395.4643

				Partion wall	1&4				
Wall	Between Axis	Width/Diamter(m)	Depth(m)	Length	Weight(KN)	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi* Yi	My=Wi* Xi
A&B	1	0.2	3.05	4.53	38.6862	0	2.49	96.328638	0
A&B	2	0.15	3.05	0.72	4.6116	3.62	2.85	13.14306	16.693992
A&B	2&3	0.15	3.05	4.08	26.1324	5.11	2.26	59.059224	133.536564
A&B	2&3	0.15	3.05	4.08	26.1324	9.57	2.26	59.059224	250.087068
A&B	3&4	0.2	3.05	3.02	25.7908	13	1.734	44.721247	335.2804
A&B	5	0.15	3.05	1.85	11.84925	15.57	1.15	13.626638	184.492823
A&B	5	0.15	3.05	2.38	15.2439	15.57	1.15	17.530485	237.347523
B&C	1	0.2	3.05	0.62	5.2948	0	5.75	30.4451	0

B&C	2	0.2	3.05	0.62	5.2948	3.6	5.75	30.4451	19.06128
B&C	3	0.15	3.05	0.62	3.9711	10.03	5.75	22.833825	39.830133
B&C	5	0.15	3.05	0.62	3.9711	15.57	5.75	22.833825	61.830027
B&C	6	0.15	3.05	0.62	3.9711	19.54	5.75	22.833825	77.595294
C&D	1	0.2	3.05	2.81	23.9974	0	7.655	183.7001	0
C&D	2	0.15	3.05	2.81	17.99805	3.62	7.655	137.77507	65.152941
C&D	2&3	0.15	3.05	2.81	17.99805	5.11	7.655	137.77507	91.9700355
C&D	3	0.15	3.05	2.81	17.99805	10.03	7.655	137.77507	180.520442
E&F	3	0.15	3.05	1.89	12.10545	10.03	11.89	143.9338	121.417664
E&F	3&4	0.15	3.05	1.89	12.10545	13	11.89	143.9338	157.37085
E&F	4	0.15	3.05	3.43	21.96915	13.465	11.89	261.21319	295.814605
E&F	5	0.15	3.05	1.89	12.10545	15.57	11.89	143.9338	188.481857
E&F	6	0.15	3.05	1.89	12.10545	19.54	11.89	143.9338	236.540493
F&G	3&4	0.15	3.05	3.68	23.5704	13	19.37	456.55865	306.4152
F&G	4	0.15	3.05	3.68	23.5704	13.465	19.37	456.55865	317.375436
F&G	5&6	0.15	3.05	3.68	23.5704	16	19.37	456.55865	377.1264
1&2	A&B	0.15	3.05	3.67	23.50635	2.06	3.11	73.104749	48.423081
1&2	A&B	0.15	3.05	3.67	23.50635	2.06	4.64	109.06946	48.423081
2&3	A&B	0.15	3.05	5.07	32.47335	5.85	3.11	100.99212	189.969098
2&3	A&B	0.15	3.05	5.07	32.47335	7.66	4.08	132.49127	248.745861

3&4	A&B	0.15	3.05	5.63	36.06015	12.99	2.91	104.93504	468.421349
4&5	A&B	0.15	3.05	0.87	5.57235	12.99	1.98	11.033253	72.3848265
5&6	A&B	0.15	3.05	3.88	24.8514	17.72	4.36	108.3521	440.366808
2&3	B	0.15	3.05	1.75	11.20875	8.99	5.63	63.105263	100.766663
5&6	C	0.15	3.05	3.43	21.96915	17.31	6.19	135.98904	380.285987
2&3	D	0.15	3.05	1.85	11.84925	9.005	9.34	110.672	106.702496
3&4	E	0.15	3.05	3.6	23.058	12.79	10.88	250.87104	294.91182
3&4	E&F	0.15	3.05	4.75	30.42375	12.79	12.76	388.20705	389.119763
5&6	E	0.15	3.05	3.43	21.96915	17.48	10.88	239.02435	384.020742
3&4	F	0.15	3.05	4.3	27.5415	19.5	17.53	482.8025	537.05925
3&4	F&G	0.15	3.05	4.19	26.83695	19.5	17.53	470.45173	523.320525
4&5	F&G	0.15	3.05	0.81	5.18805	19.5	10.88	56.445984	101.166975
5&6	F	0.15	3.05	3.65	23.37825	19.5	10.88	254.35536	455.875875
3&4	G	0.2	3.05	3.65	31.171	12.48	20.18	629.03078	389.01408
4&5	G	0.2	3.05	0.88	7.5152	17.48	20.18	151.65674	131.365696
5&6	G	0.2	3.05	3.65	31.171	19.54	20.18	629.03078	609.08134
			TOTAL		841.7665			7738.1304	9613.36634

				Slab				
Panel	Lx(m)	Ly(m)	DL(KN/m2	Weight(KN	Moment Arm		M	

			)	)			oment	
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
P1	3.78	4.72	7.66	136.6667	0	0	0	0
P2	3.78	4.79	10.05	181.9673	5.572	2	363.9346	1013.922
P3	4.72	6.53	7.85	241.9496	0	6.561	1587.431	0
P4	4.97	6.53	7.46	242.1076	5.572	9.509	2302.201	1349.023
P5	3.78	3.9	5.85	86.2407	9.509	12.409	1070.161	820.0628
P6	3.9	6.52	7.11	180.7931	12.409	6.561	1186.183	2243.461
P7	3.88	4.75	6.07	111.8701	5.572	6.561	733.9797	623.3402
P8	3.78	5.62	9.87	209.6743	0	9.509	1993.793	0
P9	5.62	6.52	9.37	343.3393	5.572	5.572	1913.087	1913.087
P10	5.62	5.63	10.47	331.2771	9.509	9.509	3150.114	3150.114
P11	3.88	6.48	7.15	179.7682	12.409	12.409	2230.743	2230.743
		TOTAL		2245.654			14618.54	13343.75

$$X_m = \frac{\sum w_i * x_i}{\sum w_i} = \frac{8536.033}{1142.74} = 7.48KNm$$

$$Y_m = \frac{\sum w_i * y_i}{\sum w_i} = \frac{10315.73}{1142.74} = 9.027KNm$$

Roof

Table 6.4 Center of mass calculation for roof slab

Beam									
Beam	Between Axis	Width(m)	Depth(m)	Length	Weight(KN)	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis - 1	A&D	0.3	0.4	9.51	28.53	0	0	0	0
Axis - 2	A&D	0.3	0.4	9.51	28.53	1.89	4.76	135.8028	53.9217
Axis - 3	A&G	0.3	0.4	21.3	63.9	5.15	10.65	680.535	329.085
Axis - 4	F&G	0.3	0.4	21.3	63.9	7.53	10.65	680.535	481.167
Axis - 5	A&G	0.3	0.4	21.3	63.9	7.96	10.65	680.535	508.644
Axis - 6	A&G	0.3	0.4	21.3	63.9	9.9	10.6	680.535	632.61

							5		
Axis - 7	1&6	0.3	0.4	19.74	59.22	0	0	0	0
Axis - 8	1&6	0.3	0.4	19.74	59.22	9.9	2.82	167.0004	586.278
Axis - 9	1&6	0.3	0.4	19.74	59.22	9.9	3.24	191.8728	586.278
Axis - 10	1&6	0.3	0.4	19.74	59.22	9.9	4.75	281.295	586.278
Axis - 11	3&6	0.3	0.4	9.5	28.5	4.75	5.5	156.75	135.375
Axis - 12	3&6	0.3	0.4	9.5	28.5	4.75	8.76	249.66	135.375
Axis - 13	3&6	0.3	0.4	9.5	28.5	4.75	10.65	303.525	135.375
Total weight					635.04			4208.046	4170.3867

				Stair					
Stair Type	Name	Width	Length	DL(KN/m2)	Weight	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Stair 1	Flight 1	1.3	4.43	10.28	59.20252	1.35	2.78	164.583	164.583
Stair 1	Flight 2	1.3	3.88	10.6	53.4664	0.75	1.24	66.29834	66.29834
Stair 1	Flight 3	1.3	4.43	10.28	59.20252	1.35	2.78	164.583	164.583
Total weight					171.8714			395.4643	395.4643

				Slab				
Panel	Lx(m)	Ly(m)	DL(KN/m2)	Weight(KN)	Moment Arm		Moment	
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
P1	3.78	4.72	14.09	251.3881	0	0	0	0
P2	3.78	4.79	14.09	255.1164	5.572	2	510.2327	1421.508
P3	4.72	6.53	14.09	434.2763	0	6.561	2849.287	0
P4	4.97	6.53	14.09	457.2783	5.572	9.509	4348.259	2547.955
P5	3.78	3.9	14.09	207.7148	9.509	12.409	2577.533	1975.16
P6	3.9	6.52	14.09	358.2805	12.409	6.561	2350.678	4445.903
P7	3.88	4.75	14.09	259.6787	5.572	6.561	1703.752	1446.93

P8	3.78	5.62	14.09	299.3223	0	9.509	2846.256	0
P9	5.62	6.52	14.09	516.2914	5.572	5.572	2876.776	2876.776
P10	5.62	5.63	14.09	445.8161	9.509	9.509	4239.265	4239.265
P11	3.88	6.48	14.09	354.2564	12.409	12.409	4395.968	4395.968
P12	3.93	4.53	14.09	250.8429	12.409	12.409	3112.709	3112.709
		TOTAL		3056.266			28933.94	26462.17

$$X_m = \frac{\sum w_i * x_i}{\sum w_i} = \frac{33141.98}{3056.266} = 10.84\text{KNm}$$

$$Y_m = \frac{\sum w_i * y_i}{\sum w_i} = \frac{30632.5567}{3056.266} = 10.02\text{KNm}$$

Function of our building Residential building. Hence, for ordinary buildings Importance class II with importance factor $(\gamma) = 1$ can be taken from "ES-EN 1998-1:2014, Section 4.2.5(4), Table 4.3."

Since our project site located in wolkita ; the seismic zone is zone 3.

Hence, From ES-EN1998-1:2014, Annex D (Informative) for zone 3.

$\alpha = agr/g = 0.1$

Where, agr Reference ground acceleration

g is gravitational force (9.81m/s^2)

$agr = 0.1 * 9.81 = 0.981\ m/s^2$.

Assuming a recorded surface wave magnitude of less than 5.5, we have decided to use type 2 spectrums. And the required information for Type 2 spectrum and Ground Type B, are

Table 6.5 spectrum and Ground Type B

Ground Type	S	TB(Sec)	Tc(sec)	TD(sec)
B	1.35	0.05	0.25	1.2

Fundamental period of vibration (T_1)

According to ESEN 1998, 2015 the fundamental period of vibration in two main directions should be smaller than the following values.

$$T_1 \leq \begin{cases} 4T_1 \\ 2 \text{ second} \end{cases}$$

And the fundamental period of vibration T_1 can be obtained from the expression below,

$$T_1 = C_t \times H^{0.75},$$

Where:

- H = 18.9 m, height of the building.
- C_t = 0.075, for reinforced concrete moment resisting frames
- $T_1 = C_t \times H^{0.75} = 0.075 \times (18.9)^{0.75}$ = 0.670 seconds

$$T_1 = 0.670 \leq \begin{cases} 4 * 0.25 = 1 \\ . \\ 2 \text{ second} \end{cases}$$ …………………. OK!!!

Base Shear determination

$F_b = S_d(T_1)m\lambda$

Where

$S_d(T_1)$ is the ordinate of the design specterum

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

M = is the total mass of the building above the foundation or above the top of a rigid basement,

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

Since, $T_1 > 2T_c$ = 0.670> 0.5, then ʎ = 1.0

Design spectrum $S_d(T_1)$ for the horizontal component of the seismic action the design spectrum $S_{d(T)}$ at period T shall be defined by the following expressions.

For $T_{C(T)} \leq T_{1(T)} \leq T_{D(T)}$ = 0.25 ≤0.670 ≤ 1.2, then

$$T_C \leq T \leq T_D:\ S_d(T_1) = \begin{cases} ag \times S * \frac{2.5}{q}\left[\frac{T_C}{T}\right] \\ , \\ \geq \beta \times ag \end{cases}$$

Where:

- ag = ground acceleration
- T_C, T_D and S are described on the above table for ground Type-B.
- q = behavioral factor

- β is the lower bound factor for horizontal Design spectrum and the recommended value is 0.2.

Ground acceleration (ag) It can be obtained from the country's hazard map for given site at seismic hazard zone which is found on zone 3

Table 6.6 the ground acceleration for different zones of the country's seismic hazard map.

Zone	5	4	3	2	1	0
$\alpha o = \frac{ag}{g}$	0.2	0.15	0.1	0.07	0.04	0

For zone 3,

$ag_R = \alpha o \times g = 0.1 \times 9.81 = 0.981$

$ag = ag_R \times \gamma,$

Where: γ- importance factor for category II, γ = 1

$ag_R = ag_R \times \gamma = 0.981 \times 1 = 0.981 m/s^2$

Behavioral factor (q): it will be obtained from the nature of the structure and its ductility nature (ductility medium or ductility high).

$$q = qo \times Kw \geq 1.5$$

Where:

- K_w = 1 for frame structures
- qo = basic values of behavioral factor, and determined as
- $qo = 3.0\alpha u/\alpha 1$ . . . . . for moderate ductility
- $\alpha u/\alpha 1 = 1.3$ .. . . for multi-story buildings

$q = qo \times Kw = 3 \times 1.3 \times 1 = 3.9$

$$Then, \quad Sd(T) = \begin{cases} ag \times S * \frac{2.5}{q}\left[\frac{TC}{T}\right] = 0.981 \times 1.35 \times \frac{2.5}{3.9} \times \frac{0.25}{0.670} = 0.317 \\ \geq \beta \times ag = 0.2 \times 0.981 = 0.1962 \end{cases}$$

$$S_d(T_1) = 0.317$$

Base Shear Force Calculation

$F_b = S_d(T_1) m\lambda$

Where

$S_d(T_1)$ is the ordinate of the design specterum

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

$M =$ is the total mass of the building above the foundation or above the top of a rigid basement,

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

Since, T = 0.670 sec ≥ 2 $*Tc = 0.5\ sec$ ………………… $take\ \lambda = 1$

Table 6.7 Story Mass

Story	Weight(KN)
Roof	3691.30
4th	4145.32
3rd	4145.32
2nd	4145.32
1st	4145.32
Ground	1415.88
Σ	21688.46

Total weight = 21688.46KN

Weight (Force) =mass*gravity

Mass = weight/gravity = 21688.46KN/10m/s^2 = 21688460kg m/s^2 /10m/s^2 = 2168846kg

$F_b = 0.317 m/s^2 * 2168846 * 1 = 687.524 KN$

6.3 Distribution of the horizontal Seismic force

According to "ES-EN1998-1:2014, Section 4.3.3.2.3(2)" The seismic action effects shall be determined by applying, to the two planar models, horizontal forces Fi to all story's,

$$F_i = F_b *$$

$$F_i = F_b * \frac{h_i m_i}{\sum h_i m_i}$$

Table 6.8 Story Shear

Story	mi(KN)	hi(m)	hi*mi(KNm)	Fi(KN)

Roof	3691.30	1.90	7013.47	76.92
4th floor	4145.32	3.05	12643.22	92.6249
3rd floor	4145.32	3.05	12643.22	92.6249
2nd floor	4145.32	3.05	12643.22	92.6249
1st floor	4145.32	3.05	12643.22	92.6249
Ground floor	1415.88	3.6	5097.168	55.90
Total sum	13867.27	18.9	62683.5	Fb=687.524

Base Shear determination

Fundamental period of vibration (T_1)

According to ESEN 1998, 2015 the fundamental period of vibration in two main directions should be smaller than the following values.

$$T_1 \leq \begin{cases} 4T_1 \\ 2 \text{ second} \end{cases}$$

And the fundamental period of vibration T_1 can be obtained from the expression below,

$$T_1 = C_t \times H^{0.75},$$

Where:

- H = 18.9 m, height of the building.
- C_t = 0.075, for reinforced concrete moment resisting frames
- $T_1 = C_t \times H^{0.75} = 0.075 \times (18.9)^{0.75} = 0.670$ seconds

$$T_1 = 0.670 \leq \begin{cases} 4 * 0.25 = 1 \\ . \\ 2 \text{ second} \end{cases} \text{…………………. OK!!!}$$

Base Shear determination

$F_b = S_d(T_1)m\lambda$

Where

$S_d(T_1)$ is the ordinate of the design specterum

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

M = is the total mass of the building above the foundation or above the top of a rigid basement,

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

Since, $T_1 > 2T_c$ = 0.670> 0.5, then λ = 1.0

Design spectrum $S_d(T_1)$ for the horizontal component of the seismic action the design spectrum $S_{d(T)}$ at period T shall be defined by the following expressions.

For $T_{C(T)} \leq T_{1(T)} \leq T_{D(T)}$ = 0.25 ≤0.670 ≤ 1.2, then

$$T_C \leq T \leq T_D:\ S_d(T_1) = \begin{cases} ag \times S * \frac{2.5}{q}\left[\frac{T_C}{T}\right], \\ \geq \beta \times ag \end{cases}$$

Where:

- ag = ground acceleration
- T_C, T_D and S are described on the above table for ground Type-B.
- q = behavioral factor
- β is the lower bound factor for horizontal Design spectrum and the recommended value is 0.2.

T1= $0.075*15.80^{0.75}$=0.59sec

4Tc(s) = 4*0.25=1sec

T1≤ min 2sec

T1=0.59sec < 1sec….. ok

Take T1=0.59sec

Table 6.9 Importance class for buildings from ES EN-8-1998:2015 Table 4.3

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

NOTE Importance classes I, II and III or IV correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EN 1990:2002, Annex B.

Table 6.10 Values of the parameters describing the recommended type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 6.11 Values of the parameters describing the recommended type 2 elastic response spectra from

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

Table 6.12 Recommended Values of the parameters

Describing the vertical elastic response spectra

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

Table 6.13 Ground type

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	–	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	–	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Weight of all floors

Story	Remark	Floor	Weignt(KN)
GR	W1	Foundation	89.6
1st	W2	Ground Floor Wt.	1142.74
2nd	W3	Wt. of first Floor	3618.47
3rd	W4	Wt. of second Floor	4538.43

Design spectrum (Sd(Td) =9389.24KN

For the horizontal components of the seismic action the design spectrum, Sd(T), shall be defined by the following expressions

(4)P For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \le T \le T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3}\right)\right] \quad (3.13)$$

$$T_B \le T \le T_C : \quad S_d(T) = a_g \cdot S \cdot \frac{2,5}{q} \quad (3.14)$$

$$T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T}\right] \\ \ge \beta \cdot a_g \end{cases} \quad (3.15)$$

$$T_D \le T : \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2}\right] \\ \ge \beta \cdot a_g \end{cases} \quad (3.16)$$

where

a_g, S, T_C and T_D	are as defined in **3.2.2.2**;
$S_d(T)$	is the design spectrum;
q	is the behaviour factor;
β	is the lower bound factor for the horizontal design spectrum.

NOTE The value to be ascribed to β for use in a country can be found in its National Annex. The recommended value for β is 0,2.

(5) For the vertical component of the seismic action the design spectrum is given by expressions (3.13) to (3.16), with the design ground acceleration in the vertical direction, a_{vg} replacing a_g, S taken as being equal to 1,0 and the other parameters as defined in **3.2.2.3**.

(6) For the vertical component of the seismic action a behaviour factor q up to to 1,5 should generally be adopted for all materials and structural systems.

(7) The adoption of values for q greater than 1,5 in the vertical direction should be justified through an appropriate analysis.

(8)P The design spectrum as defined above is not sufficient for the design of structures with base-isolation or energy-dissipation systems.

For frame system and DCM, $qo = 3\ \alpha u/\alpha 1$

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

$qo = 3\ \alpha u/\alpha 1 - 0.2 * 3\ \alpha u/\alpha 1 = 0.8 * 3\ \alpha u/\alpha 1$

- $\alpha 1$ is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant;

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

When the multiplication factor αu/α1 has not been evaluated through an explicit calculation, for buildings which are regular in plan the following approximate value of $\alpha u/\alpha 1$ may be used.

Frames or frame-equivalent dual systems

− One-storey buildings: $\alpha u/\alpha 1 = 1{,}1;$

− Multistory, one-bay frames: $\alpha u/\alpha 1 = 1{,}2;$

− Multistory, multi-bay frames or frame-equivalent dual structures: $\alpha u/\alpha 1 = 1{,}3.$

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

$Q = q_{ok_w} \geq 1.5$

Where $K_w = 1$

$q_{o=3} \ \frac{\alpha u}{\alpha 1}$

$q = 0.8 * 3 * 1.3 * 1 = 3.12$

$$T_C \leq T \leq T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2{,}5}{q} \cdot \left[\frac{T_C}{T}\right] \\ \geq \beta \cdot a_g \end{cases}$$

Where $Tc = 0.8, T = 0.59$

$\propto g = 3.5$

S =1.35

$S_d(T) = \text{Max } 5.133$

0.700

$F_b = S_d(T_1)W_{Tot} = 5.133 * 9389.24 = 48{,}199.10$

$F_t = 0.07 * 0.59 * 48{,}199.10 = 1{,}990.62$

Determination of D-Values of Frames

Portion of the base shear distributed over the height of the structure

$$F_i = \frac{(F_b - F_t)\, W_i\, h_i}{\Sigma\, W_j\, h_j}$$

Table 6.14 Computation of Story Shear Force

				BASE SHEAR				
Story	Remark	Wi	hi	Wihi	Fb	Ft	Fb-Ft	Fi
Ground Floor	W1	1142.74	3	3428.22	48,199.10	1,990.62	46,208.48	1002.25562
First Floor	W2	3618.47	6	21710.82	48,199.10	1,990.62	46,208.48	6347.25642
Second Floor	W3	4538.43	9	40845.87	48,199.10	1,990.62	46,208.48	11941.4748
Third Floor	W4	4538.43	12	54461.16	48,199.10	1,990.62	46,208.48	15921.9664
roof Floor	W5	2507.35	15	37610.25	48,199.10	1,990.62	46,208.48	10995.5267
			Total	**158056.32**				

7 Frame Analysis

7.1 Procedure for Modeling for 3D frame Analysis Using ETABS V9

Frame Analysis Procedure using ETABS

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

STEP 1: Select the base Unit and Design code

Unit: meter

Design code: Euro code

STEP 2: Plot Grid Coordinates

Plot grid coordinates that represent the given structural layout

STEP 3: 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
- The software itself calculates Shear modulus
- Unit weight: $25KN/m^3$

B. Rebar.

- Grade: S-400
- Unit weight: $7850KN/m^3$
- Modulus of elasticity: 200000MPa
- Poisson's ratio: 0.3
- Shear modulus (G): 76903069

- Minimum yield stress, fy: 435MPa
- Minimum tensile stress, fu: 500MPa
- Directional symmetry type:

STEP 4: Define Section properties

Beam

Cross section: 400x250 mm

Column

Cross section: 400x400 mm

Those materials are defined in which their composition is C-25 and S-400 defined previously.

STEP 5: Draw structural objects

STEP 6: Assign Joint restraint

STEP 7: Assign Joint diaphragm

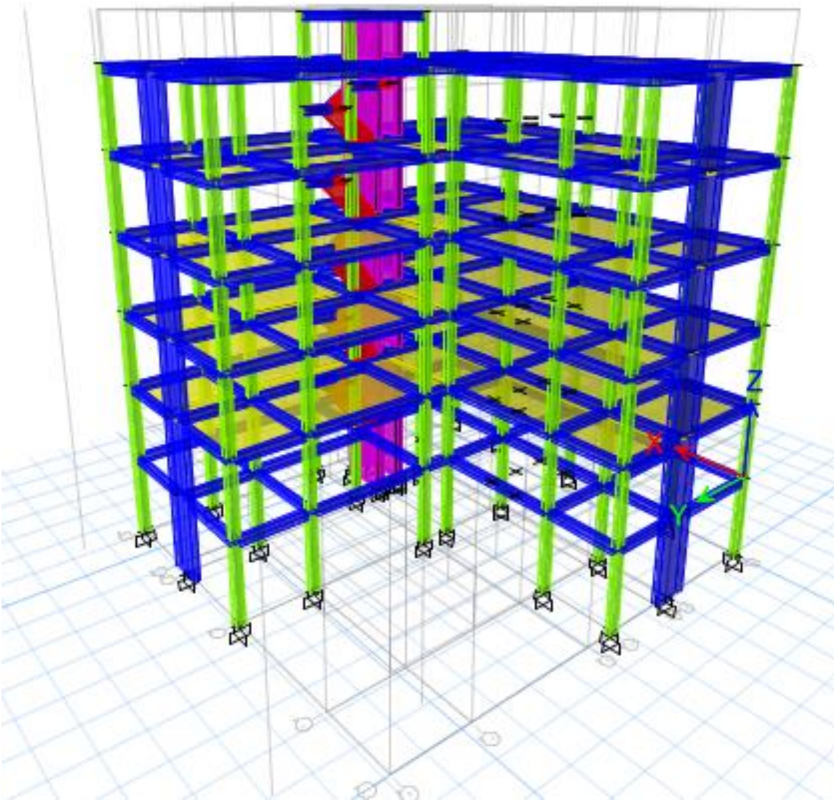
STEP 8: Assign loads

Define the following loads in the load pattern.

- Dead load
- Live load
- Wind load (X)
- Wind load (Y)
- EQ(X+)
- EQ (X-)
- EQ(Y+)
- EQ (Y-)

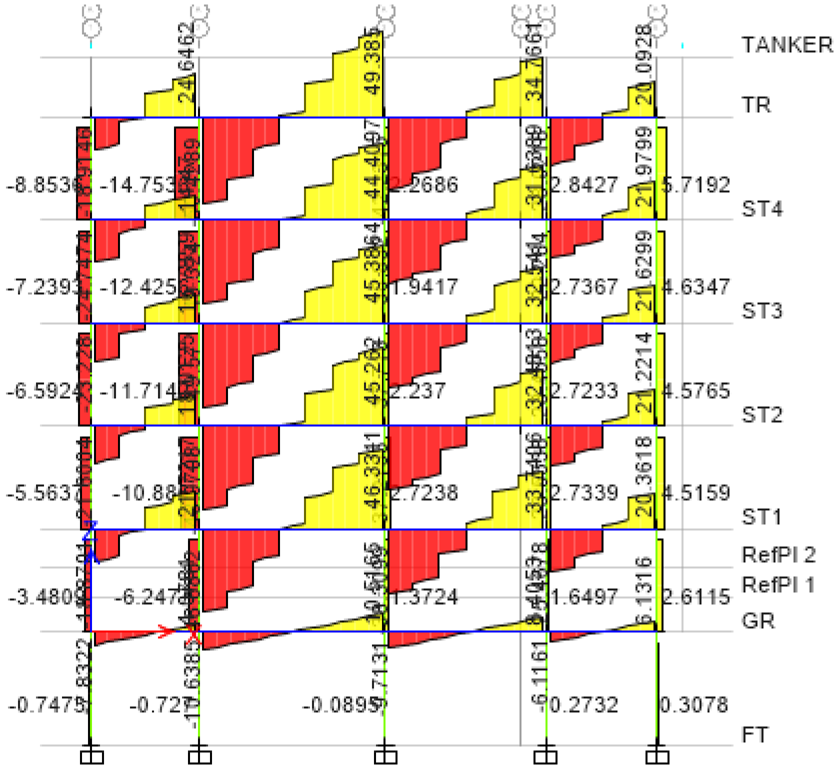
STEP 9: Analyze the model

STEP 10: Design results for checking

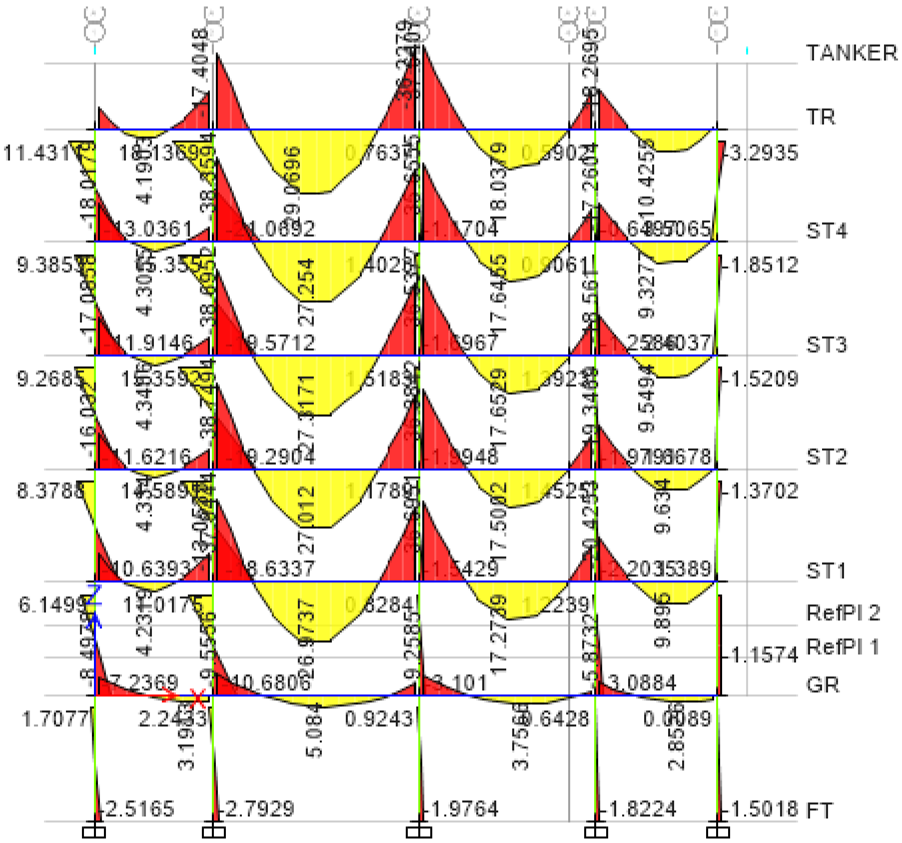


Z
X
Y

7.2 Analysis output of ETABS 2016 beam shear force diagram



7.3 Analysis output of ETABS/SAP Beam Bending Moment diagram



7.4 Analysis output of ETABS/SAP Column Axial force and Moments

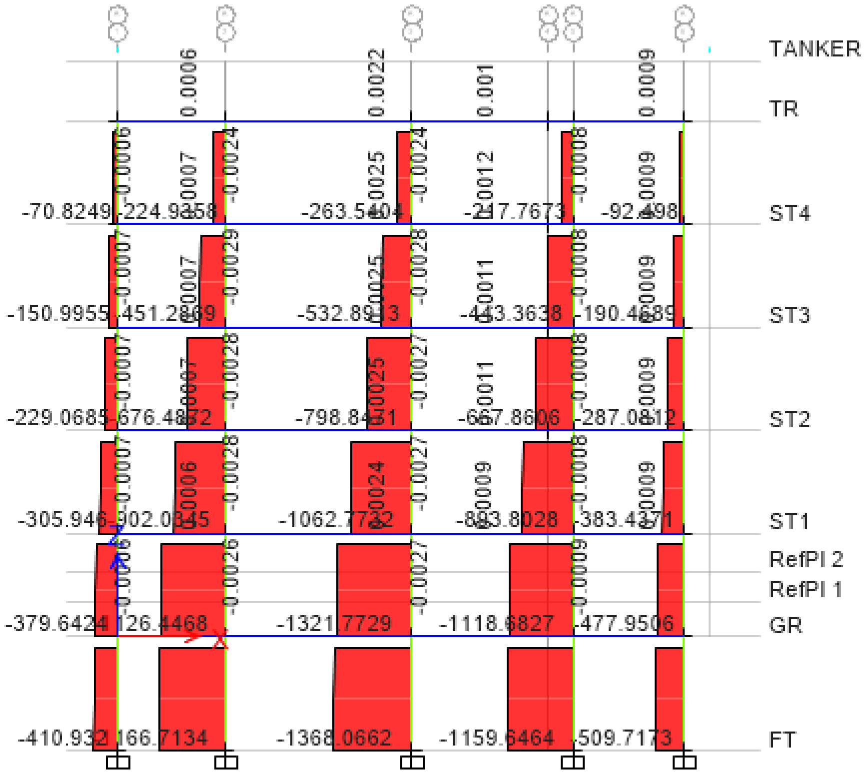
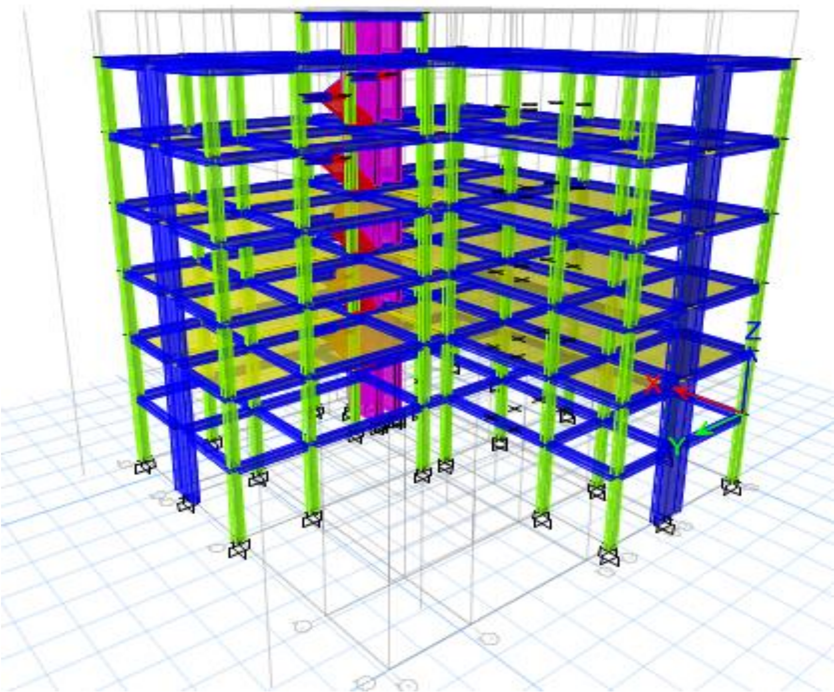


Figure ETABS output of column axial force

7.5 Analysis output of ETABS/SAP 3D Deflection Analysis



8 Design of Beams

8.1 Introduction

Beams are flexural members which are used to transfer the loads from slab to columns. Basically beams should be designed for flexure (moment). Furthermore, it is essential to check and design the beam sections for torsion and shear. Beams may be designed for flexural moment depending on the magnitude of the moment and the X- sectional dimensions. On the other hand, the beam can be singly reinforced, doubly reinforced section.

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.

Material data:

Dimension – 400x300mm

A. Concrete

- Grade of concrete C-25
- Concrete cube strength, $fcu = 25MPa$
- Concrete characteristic strength, ,$fck = 0.8 * fcu = 0.8 * 25 = 20MPa$
- Concrete design strength, $fcd = 0.85 * fck/\gamma c = 0.85 * 20/1.5 = 11.33MPa$

B. Steel

- S-400
- Steel tensile strength, $f$yk=400MPA
- Steel tensile design strength, $f$yd=$f$yk /ϒs=400/1.15=347.83MPA

8.2 Overall procedure/idea in design of beams

STEP 1: 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.(ministry of ethiopian education, 2015)

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

Where, $C_{min} = Mimum\ concrete\ cover$

$\Delta C_{dev} = Allowance\ in\ design\ for\ deviation$

C_{min}, **Minimum cover**

Minimum concrete cover shall be provided in order to ensure:

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

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

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

Where;

- Cmin,b -minimum cover due to bond requirement, see ES EN Art. 4.4.1.2 (3).
- Cmin,dur - minimum cover due to environmental conditions, see ES EN Art 4.4.1.2 (5)
- ΔCdur,γ- additive safety element, see ES EN Art 4.4.1.2 (6)
- ΔCdur,st -reduction of minimum cover for use of stainless steel, see ES EN Art 4.4.1.2 (7)
- ΔCdur,add -reduction of minimum cover for use of additional protection, see ES EN Art 4.4.1. 2 (8)

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

Cover Design for Bond

In order to transmit bond forces safely and to ensure adequate compaction of the

Concrete, the minimum cover should not be less than Cmin,b given in Table 4.2. (EBCS EN 2) Assum0e Φ16 longitudinal bar and Φ20 nominal maximum aggregate size; Therefore; Cmin ,b=20mm.

Cover Design for Corrosion/Durability

The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. But based on the table Exposure class according to Table 4.1 the exposure class isReduce by 1 and the structural class would be S3.

Therefore the value of minimum cover required for durability of reinforcement steel is determined using(ministry of ethiopian education, 2015)Table Value of minimum cover, Cmin,dur, requirements with regard to durability for reinforcement steel

Table 8.1 Enviromenal Requirement

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

$c_{min\,,dur} = 10m$

$c_{min} = 20mm$

$Therefore;\quad Cmin = 20mm$

ΔCdev (allowance in Design for Variation

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

$Th\,en;\quad Cnom = Cmin + \Delta Cdev = 20mm + 10mm = 30mm$

STEP 2: Effective Depth Determination: Serviceability requirement.

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

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] * f1 * f2 * f3 \dots\dots if\ \ \rho \leq \rho_o$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho}}\ \right] * f1 * f2 * f3 \dots\dots.if\ \ \rho \geq \rho_o$$

Since the formula is done for S-500 in our case S-400

$$F1 = \frac{500}{fyk} = \frac{500}{400} = 1.25$$

$$p_o = 10^{-3}\sqrt{fck}$$

Where;

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

$$F_1 = \frac{310}{\sigma_S} = \frac{500}{\frac{f_{yk} * A_{s,req}}{A_{S,prov}}}$$

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

F3=7/leff, for beams and slabs, other than flat slabs, with spans exceeding 7 m, which support

Partitions liable to be damaged by excessive deflections (leff in meters, see Art. 5.3.2.2 (1)). Or

F3=8.5/leff, for flat slabs where the greater span exceeds 8.5 m, and which support partition liable to be damaged by excessive deflections (leff in meters). Otherwise; F3=1 for both cases.

Assumption

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

As, req = As, provided.

$\frac{1}{d} = k * N$*F1*F2*F3

N=11+1.5$\sqrt{fck}$=11+1.5$\sqrt{20}$=17.17

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

$F_1 = \frac{310}{\sigma_S} = \frac{500}{\frac{f_{yk}*A_{s,req}}{A_{S,prov}}} = \frac{500}{400} = 1.25$

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

Now: $determineKusingtable$ 7.4Nof (ministry of ethiopian education, 2015)

So K = 1.3, F3 =1, and F2 = 1

l/d= $K*N*F1*F2*F3$ = 1.3* 17.71 * 1.75 * 1 * 1 = 40.29

L/d =40.29 Where; l = 5630 mm, this is the maximum span length.

d = 5630/40.29 = 139.74mm

So Effective depth d, = 180mm

Depth, D=d+d'

Where: d' = cover + Φstr + Φlon/2useΦstr = 8mmandΦlong = 20mm

d' = cover + Φstr + Φlong/2 = 30mm + 8mm + 20mm/2 = 48mm

d = D − d' = 400 − 48 = 352mm

Therefore, $D_{provided} = 400 \geq D_{required} = 352$ --- 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

<u>Example from terace on Axis-A</u>

Support moment=19.35KNm

b =0.3m

d=0.4m

$f$ck=20MPA

Where: $K = \frac{Msd}{b} * d2 \; * fck = \frac{19.35}{0.3} * 0.4 * 20 = 0.138$ <0.167.. single reinforced beam

STEP 4: Provide reinforcement

$Z = d/2(1 + \sqrt{1 - 3.53k)} = 352/2(1 + \sqrt{1 - 3.53 * 0.138)} = 302.04 \leq 0.95d \; = 334.4$

$As, calc = Msd/fyd * z = 19.35/347.83 * 302.04 = 184.18mm2$

STEP 5: Check for minimum & maximum reinforcement area

$$A_{s,min} = \frac{0.26 f_{ctm} b_t d}{f_{yk}} = 0.0013 b_t \mathrm{d} = 130\mathrm{mm2}$$

$A_{s,max} = 0.04\mathrm{bd} = 4000\mathrm{mm}$

Therefore $A_{s,min} \leq A_{s,calc} \leq A_{s,max}$

$A_{s,provided} = A_{s,calc} = 184.18\mathrm{mm}^2$

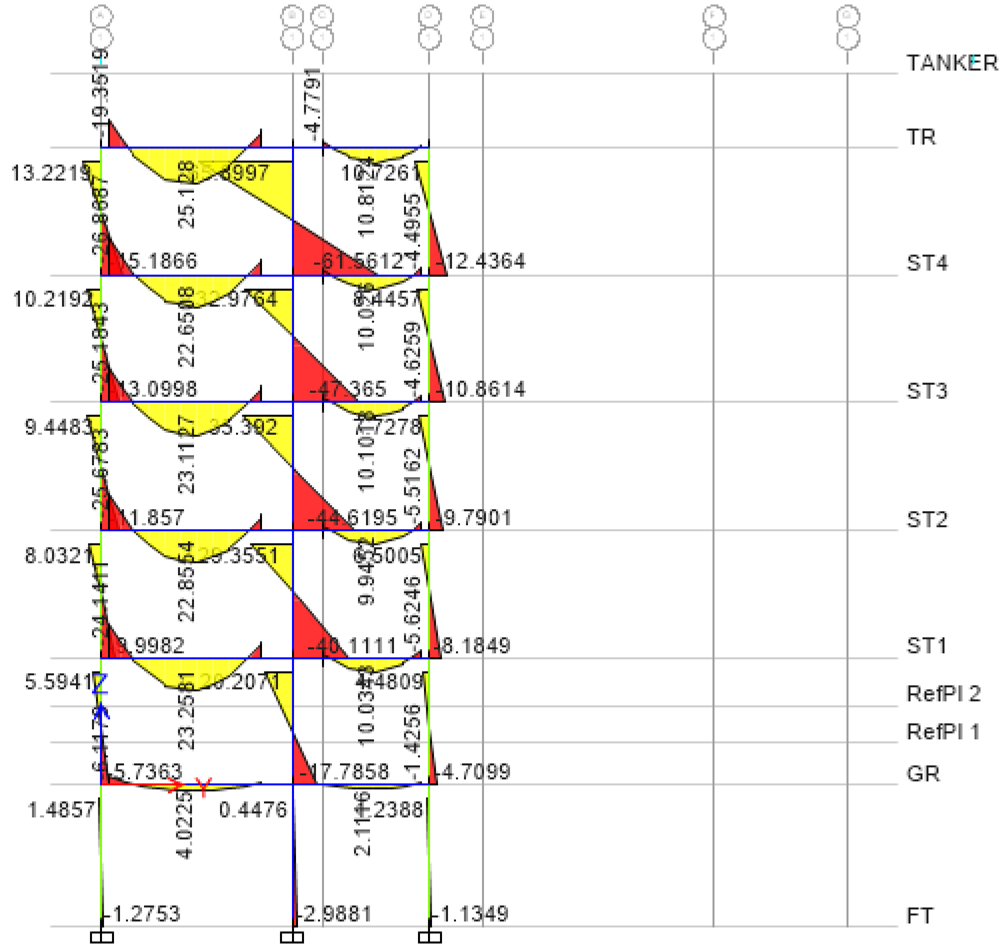
STEP 6: No of steel, n

$$a_s = \frac{\pi D^2}{4} = \frac{3.14 * 20^2}{4} = 314\mathrm{mm}^2$$

$$\mathrm{n} = \frac{A_{s,calc}}{a_{s,}} = \frac{184.18}{314} = 0.58 \approx 4\emptyset 20$$

So, use 4Ø20

Then, the other Storys in for each axis the same as the above step to determine by Excel.

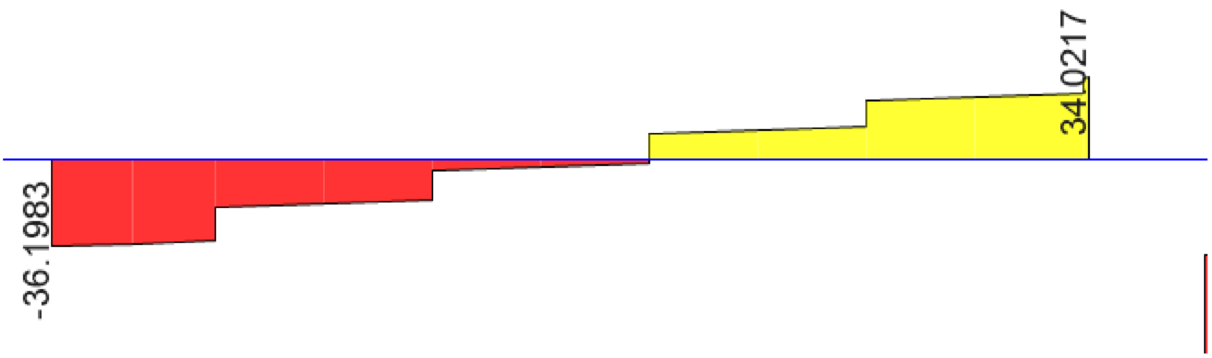


Bending moment diagram on Axis 1

Reinforcement of a beam on axis 1……………**Appendix 8H**

. Check the section for shear resistance

The design shear is located d distance from the face of the column. By similarity of triangle, V_{Ed} can obtain as follows:



Considering 200mm frame beam width, D + C/2= 400mm+200/2mm= 500mm

$\frac{34.021KN}{5.63-x} = \frac{36.198KN}{x}$, using the similarity of triangle x = 2.9m

$$\frac{V_{ED}}{0.85-0.45} = \frac{34.021}{2.82} = V_{ED} = 28.65$$

$$V_{Rd,max} =$$

Check for diagonal compression failure

$V_{rd,\,max}=V_1*f_{cd}*b_{w*}\frac{Z}{Cot\theta+tan\theta}$

Let us try,θ is less or equals to 21.8° , cotθ =2.5, tanθ =0.4

d= D-cover-$\frac{\phi}{2}$ -stirrup= 400-30-$\frac{20}{2}$-8=352mm

Z =0.9*d =0.9*352mm = 316.8mm

$V_1=0.6(1-\frac{fck}{250})=0.6(1-\frac{20}{250})=0.552$, f_{cd} =11.33, bw=300

$V_{rd,\,max}=0.552*11.33*300_*\frac{316.8}{2.5+0.4}=204.96KN$

Since, its resistance ($V_{rd,\,max}$ =204.96KN) more than acting shear (V_{ED} = 28.95KN), So the section is don't require shear reinforcement for diagonal.

Check concrete capacity of section for shear

$V_{Rd,\,c}$ = [C_{Rd}, c*k $(100\rho_1*f_{ck})^{1/3}$+ k_1*σcp]b_w*d ≤ (V_{min}+ k1*σcp) b_w*d

$K=1+\sqrt{\frac{200}{d}} \leq 2$

$K=1+\sqrt{\frac{200}{400}}$ = 1.71< 2, take, K=1.71 and d in mm

$\rho_1 = \frac{A_s}{b_w*d} = \frac{628}{300*400} = 0.01 \leq 0.02$

Where: -

A_s - is the area of the tensile reinforcement, which extends ≥ l_{bd} +d beyond the section considered.

b_w - is the smallest width of the cross-section in the tensile area in mm

$\sigma cp = \frac{N_{Ed}}{A_c} < 0.2f_{cd}$

$\sigma cp = \frac{0}{A_c} = 0 < 0.2*11.33 = 2.27$………………………………… it is OK!

N_{Ed} - is the axial force in the cross-section due to loading or prestressing [$N_{Ed} > 0$, for compression]. The influence of imposed deformations N_{Ed} on may be ignored.

A_c - is the area of concrete cross section

Note: For the use of C_{Rd}, c, V_{min} and k1, refer the National Annex. The recommended value for C_{Rd}, c is 0.18/ϒc, that for V_{min} is given by Expression (6.3N) and that for k1 is 0.15.

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

$V_{min} = 0.035*1.76^{3/2*}20^{1/2} = 0.365$

C_{Rd}, c = 0.18/ϒc = 0.18/1.5 = 0.12

$V_{Rd,c} = [\ C_{Rd}, c*k\ (100\rho_1*f_{ck}\ )^{1/3}+ k_1*\sigma cp]b_w*d \leq (V_{min}+ k1*\sigma cp)\ b_w*d$

$V_{Rd,c} = [\ 0.12*1.76\ (100*0.01*20\ )^{1/3}+ 0.15*0]300*400 \leq (0.36+ 0.15*0)\ 300*400$

$V_{Rd,c}$ = 68.96KN >V_{ED}= 43.2KN…………………………………………………....it is OK!

Reinforcement calculation

For member with vertical shear reinforcement the shear resistance, V_{RD} is the smaller value of

$$Vrd = Max \begin{cases} Vrds = \frac{Asw}{S} * Z * fyd * \cot\theta \\ Vrds = \alpha c * bw * Z * V * \frac{fcd}{\cot\theta + \tan\theta} \end{cases}$$

Where,

α= is the angle between shear reinforcement and the main tension chord (Recommended value of 1 for non pre stressed structures).

θ= is the angle between concrete compression struts and the main tension chord.

f_{td}= is the design value of the tensile force in the longitudinal reinforcement.

f_{cd} = is the design value of the concrete compression force in the direction of the longitudinal member axis.

bw =is the minimum width between tension and compression chords.

Z=is the inner lever arm, for a member with constant depth, corresponding to the maximum bending moment in the element under consideration. In the shear analysis, the approximate value Z = 0.9d may normally be used.

A_{sw} =is the cross-sectional area of the shear reinforcement and S=is the spacing of the stirrups

f_{yd} =is the design yield strength of the shear reinforcement.

V= 0.6, for $f_{ck} \leq 60$ MPa

Now let's determine the angle θ between the concrete compression strut and the main tension chord by considering Vrd, s = V_{ED}.

$$V_{ED} = \alpha c * b_w * z * v * \frac{fcd}{\cot\theta + \tan\theta} = \propto c * b_w * z * v * \frac{fcd}{\frac{\sin 2\theta}{2}}$$

$$V_{ED} = \alpha c * b_w * z * v * \frac{fcd}{\cot\theta + \tan\theta}$$

$$\theta = 0.5 * \sin^{-1}(2 * V_{ED} / (\alpha * bw * z * v * fcd))$$

$$\theta = 0.5 * \sin(2 * 68.79 / (1 * 300 * 316.8 * 0.6 * 11.33))$$

$$\theta = 0.614$$

$$\frac{VED}{Z * fcd * Cot\theta} = \frac{Asw}{S}, A_{sw} = 2 * \frac{3.14 * 8^2}{4} = 100.53 mm^2$$

$$\frac{68.79}{316.8 * 11.33 * 1} = \frac{100.53}{S}$$

S= 5245.5mm

Determination of maximum spacing

The ratio of shear reinforcement is given by the expression

$$\rho w = \frac{Asw}{S * bw * \sin\propto}$$

Where, ρw = is the shear reinforcement ratio

bw = is the breadth of the web of the member

$\rho w \leq \rho w, \min = \frac{0.08\sqrt{fck}}{fyk} = \frac{0.08\sqrt{20}}{400} = 0.000894$take ρw=0.000894

A_{sw}= is area of shear reinforcement with lengths

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

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

$S_{1, max}$ = 0.75d(1+cotα), considering, α=90°

$S_{1, max}$ = 0.75d=0.75*352 = 260mm

Thus, $S_{1, max}= \frac{Asw}{\rho w*bw*\sin\propto} = \frac{100.53}{0.000894*300*\sin 90°}$= 375mm

Check depth for flexure

M_{rd} = 0.8kx(1-0.4kx) *fcd*b*d^2

M_{rd}=0.8*0.448(1-0.4*0.448) *11.33*300*352^2 = 122.488KNm

Check depth for deflection: Serviceability requirement

$\frac{l}{d}$= N*K*F1*F2*F3, l = lx-shorter span

Assumption, initially we can't know ρ and ρ'. So, let's assume ρ = ρo and use eqn.7. 16a.area of steel required is equals to area of steel provided.

Where , N=$11+1.5\sqrt{fck}\frac{\rho}{\rho 0}+3.2\sqrt{fck}\,(\frac{\rho}{\rho 0}-1)^{3/2}$, if ρ≤ρo

$N=11+1.5\sqrt{fck}\frac{\rho}{\rho 0-\rho'}+\frac{1}{12}\sqrt{fck}\frac{\rho'}{\rho 0}^{1/2}$, if ρ>ρo

$N=11+1.5\sqrt{fck}=11+1.5\sqrt{20}=17.71$

$\rho o = \frac{\sqrt{fck}}{1000} = \frac{\sqrt{20}}{1000}$= 0.447%, $F_1 = \frac{500}{f_{ck}} = \frac{500}{400}$ = 1.25, F_2 =1, F_3 =1

$\rho = \frac{As,provided}{b*d} = \frac{628}{300*352}$= 0.6%

Since, 0.6% > ρ_o=0.447% using ES-EN 1992:2015 Art.7.4.2 (7.16b)

Where, $N=11+1.5\sqrt{fck}\frac{\rho 0}{\rho 0-\rho'}+\frac{1}{12}\sqrt{fck}\frac{\rho'}{\rho 0}^{1/2}$

$N=11+1.5\sqrt{fck}=11+1.5\sqrt{20}*\frac{0.477}{0.6-0}$ = 16.35

$F_1 = \frac{500}{fyk*\frac{As,req}{As,prov}} = \frac{500}{350*\frac{580.68.}{628}}$ = 1.54

F_2 = 1, since beff/bw <3

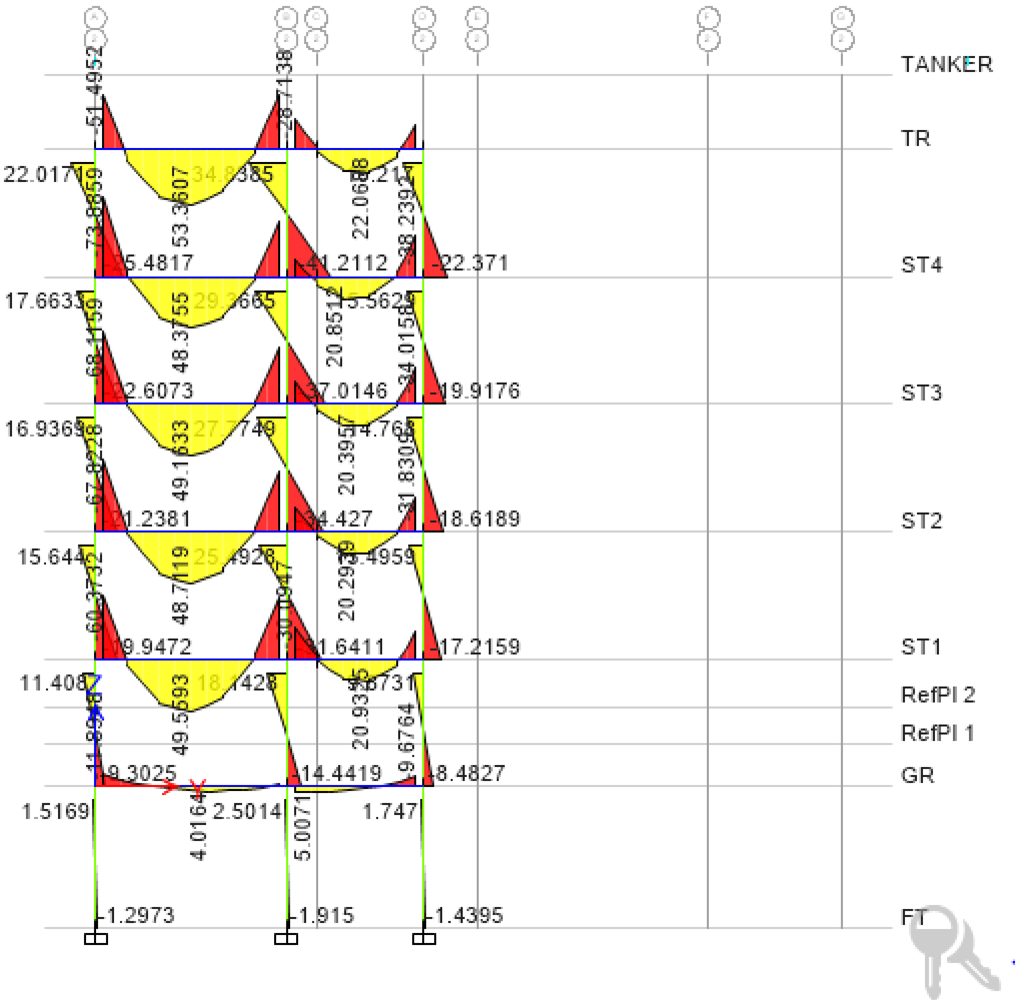
F_2 =1, since the span is less than 7

Now: determine K using table 7.4N ES EN 1992:2015. So, K = 1.3,F_1 = 1.79, F_2 =1 and F_3 =1

$$\frac{l}{d} = 1.3*16.35*1.54*1*1 = 32.73$$

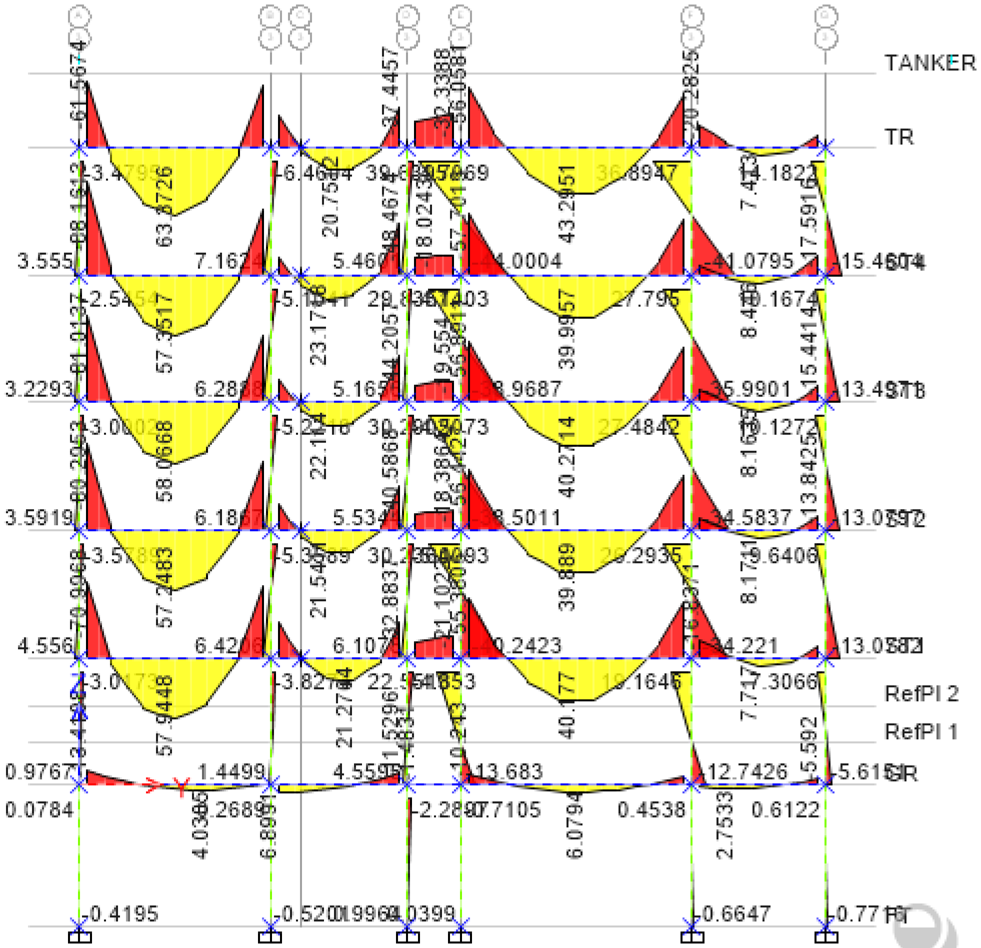
$$\frac{l}{d} = 32.73$$, Where; l = 5640 mm, this is the maximum span length

d = 5640/32.73 = 172.3mm<352mm



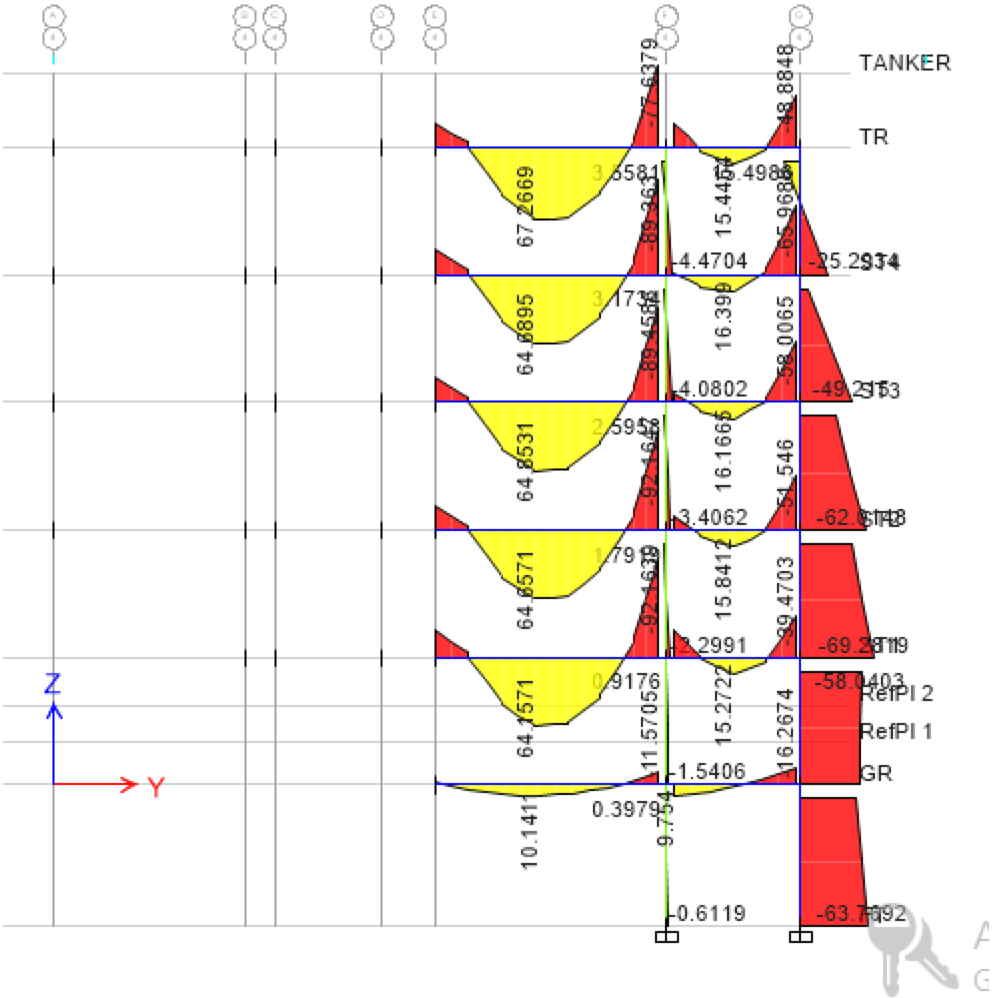
Bending moment diagram on Axis 2

Reinforcement of a beam on axis 2……………**Appendix 8I**

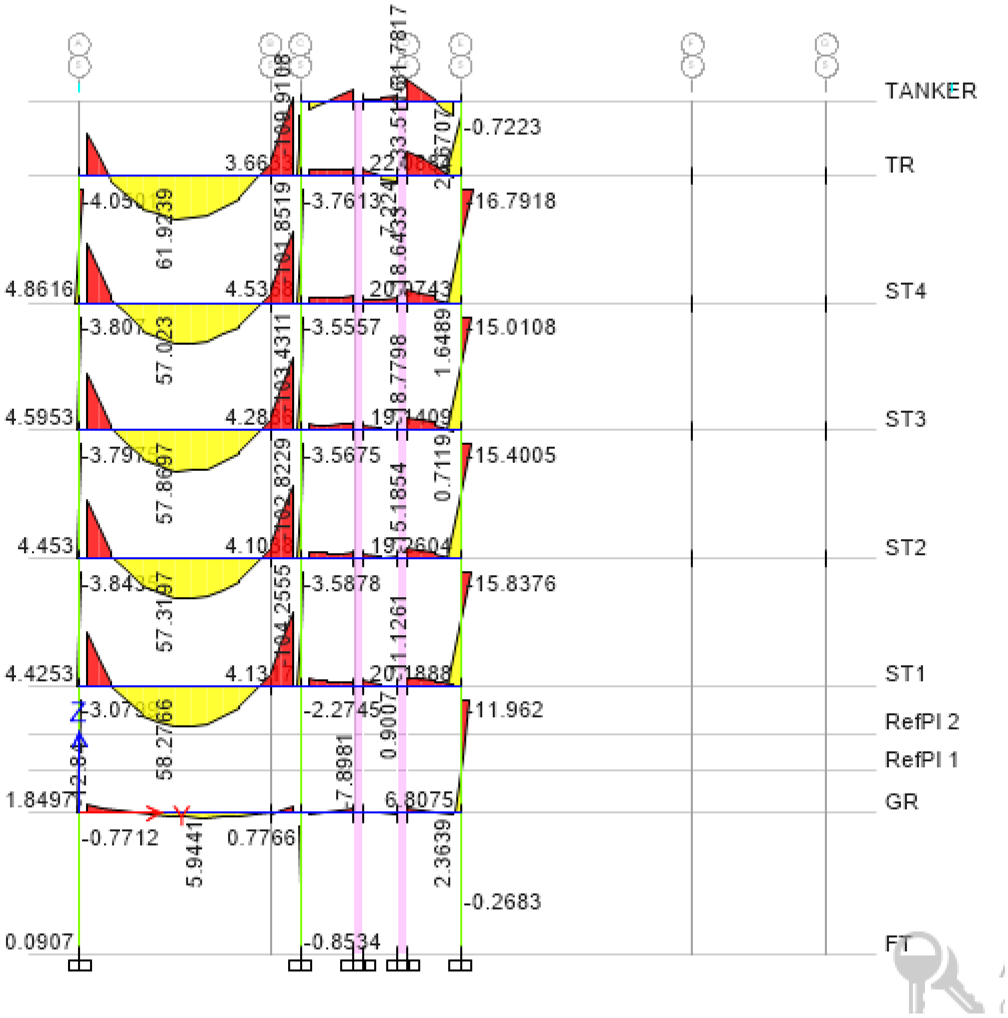


Bending moment diagram on Axis 3

Reinforcement of a beam on axis 3……………**Appendix 8J**

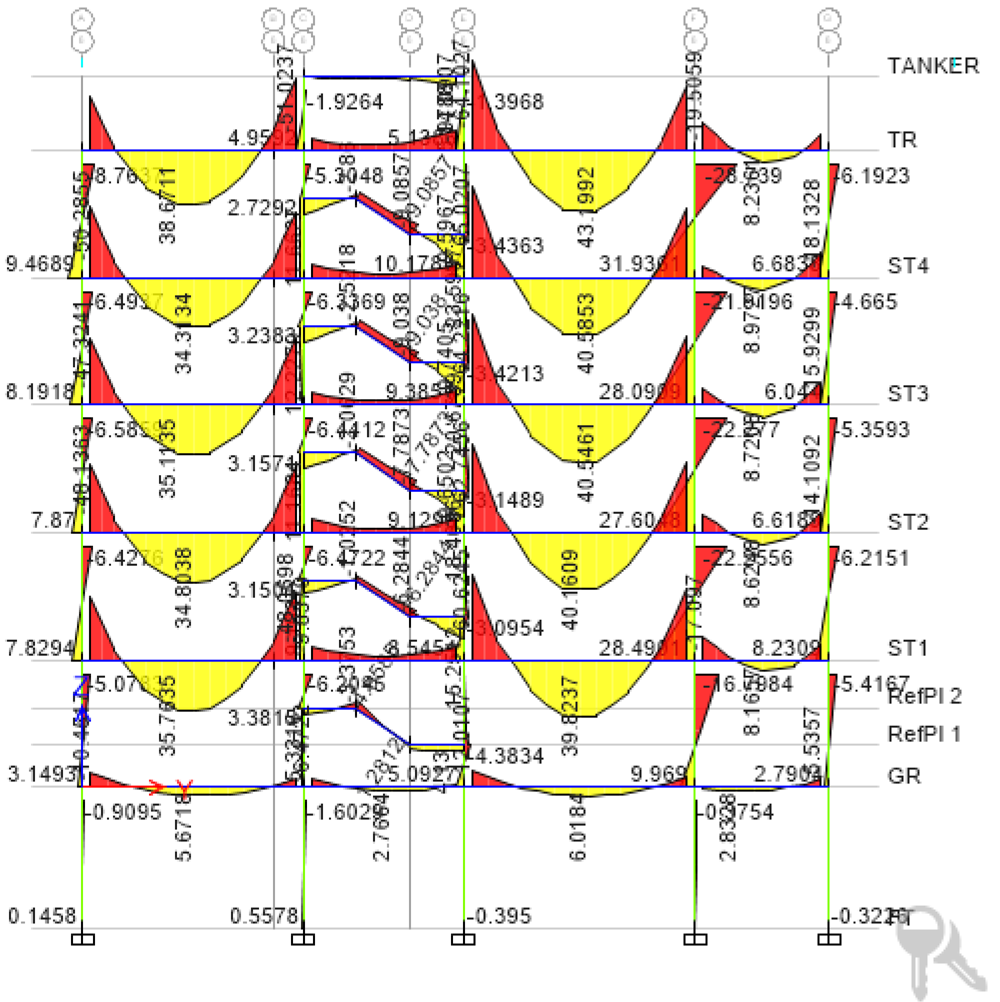


Bending moment diagram on Axis 4



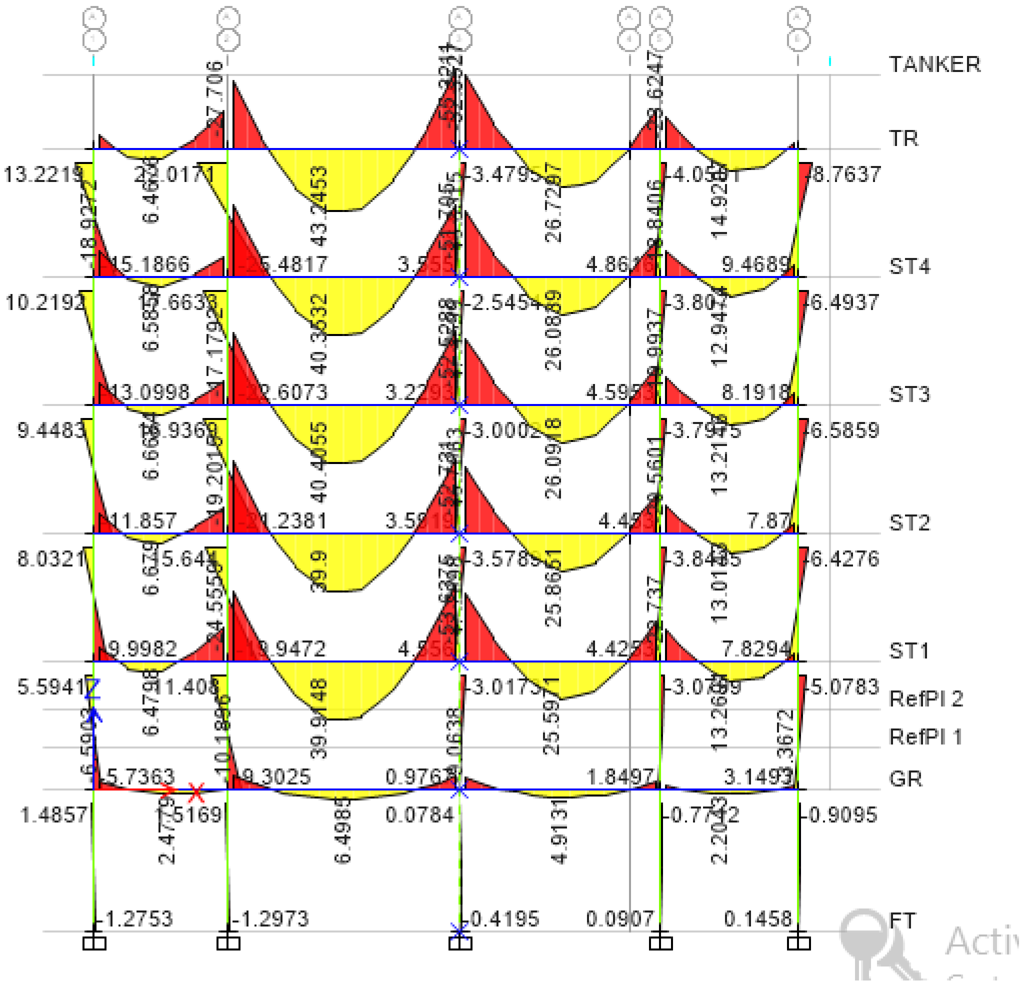
Bending moment diagram on Axis-5

Reinforcement of a beam on axis 5……………**Appendix 8L**



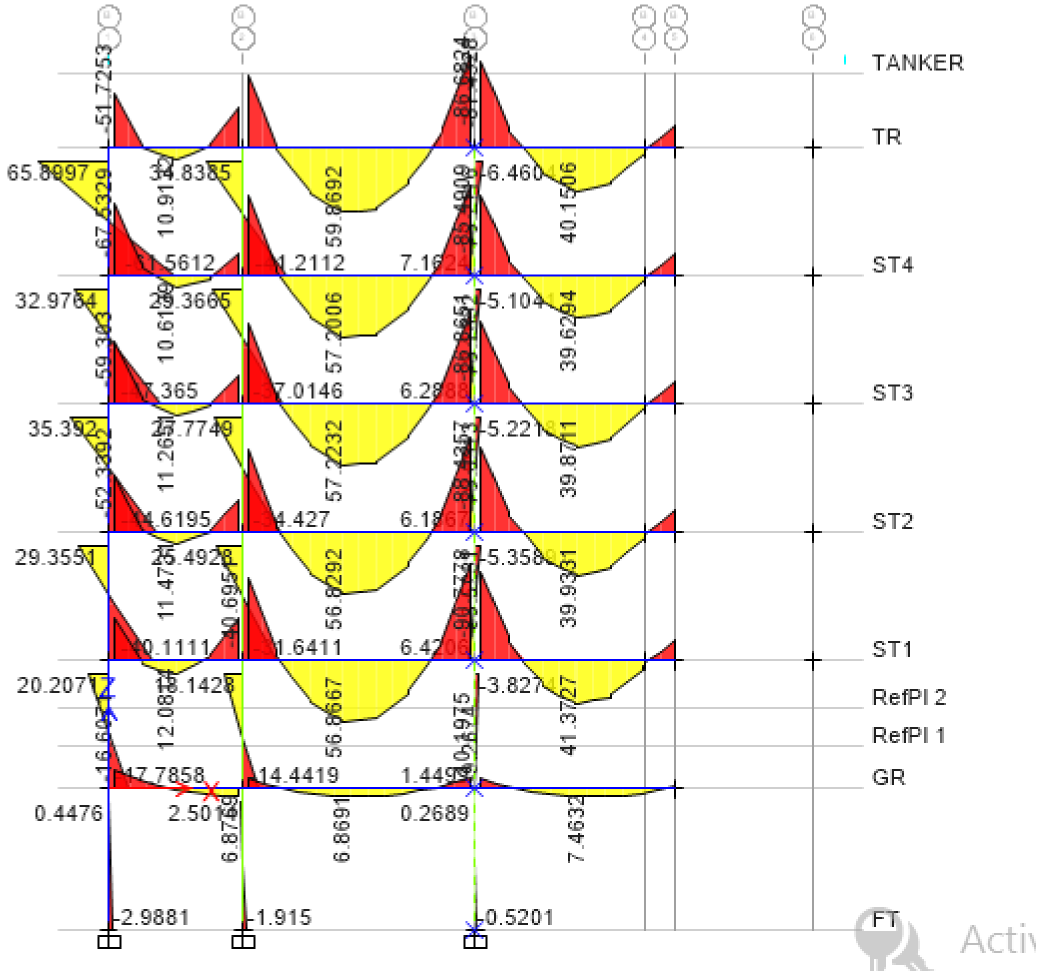
Bending moment diagram on Axis 6

Reinforcement of a beam on axis 6……………**Appendix 8M**



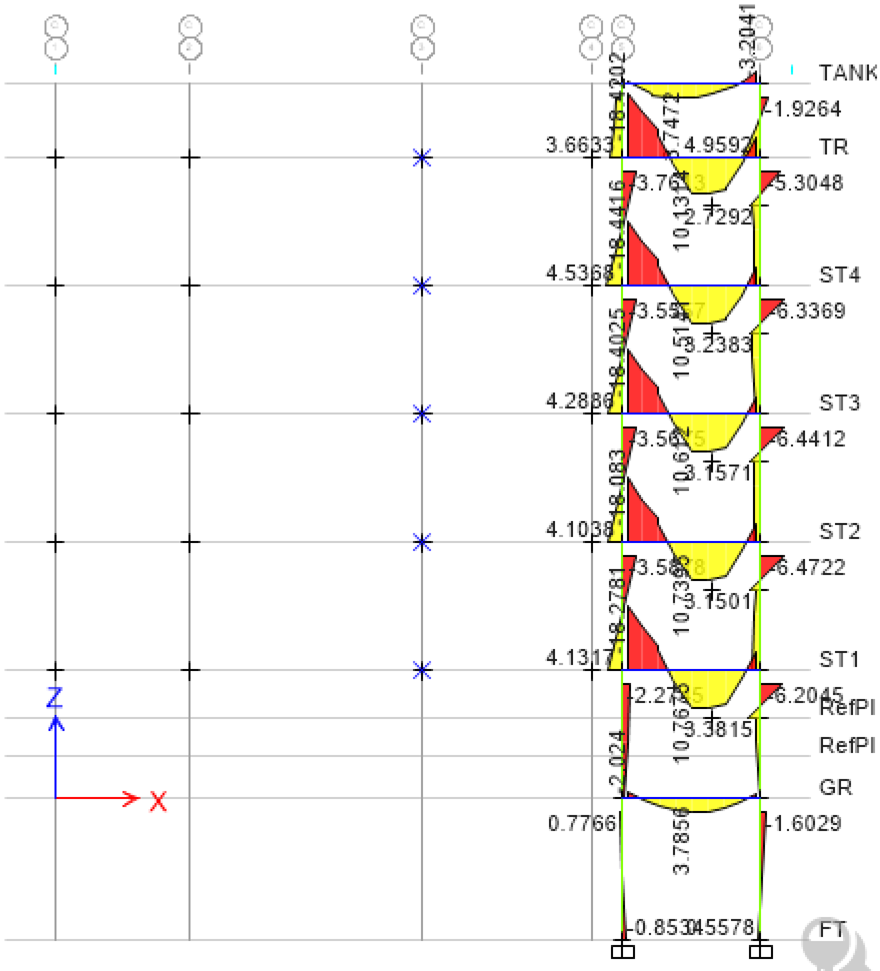
Bending moment diagram on Axis A

Reinforcement of a beam on axis A……………**Appendix 8A**



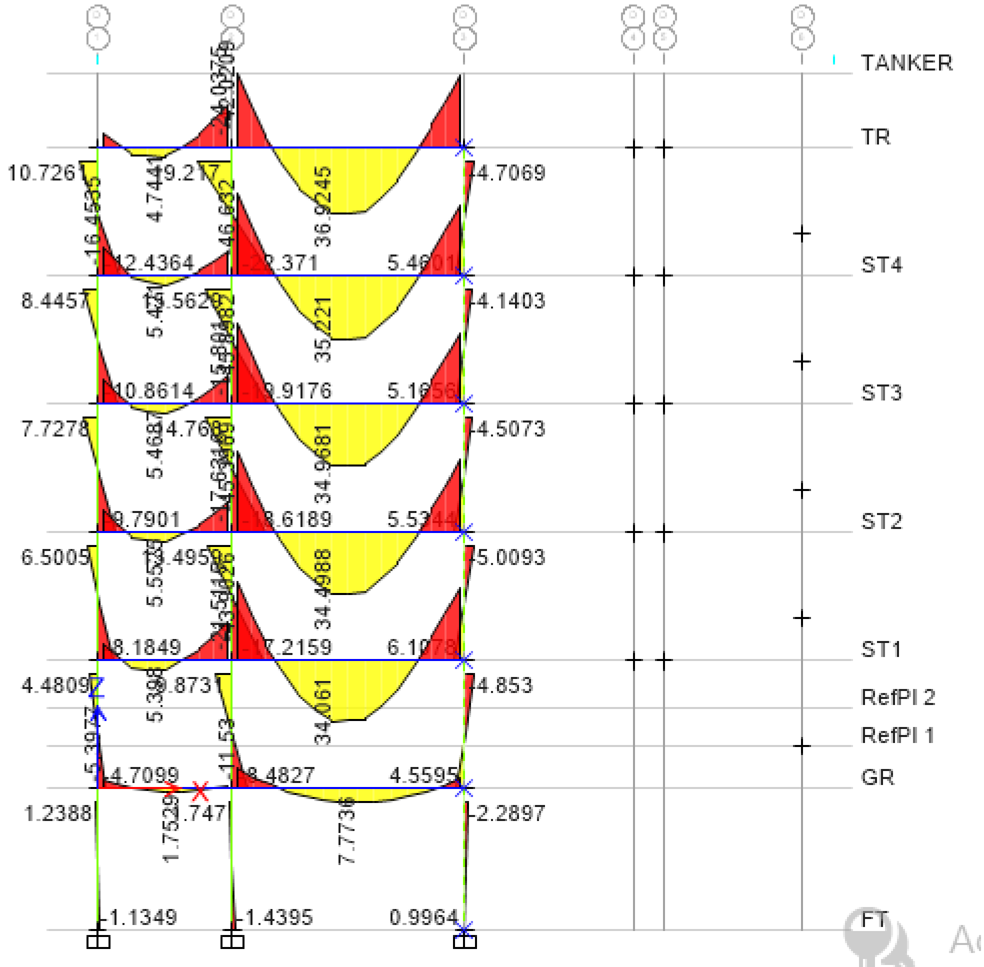
Bending moment diagram on Axis B

Reinforcement of a beam on axis B……………**Appendix 8B**



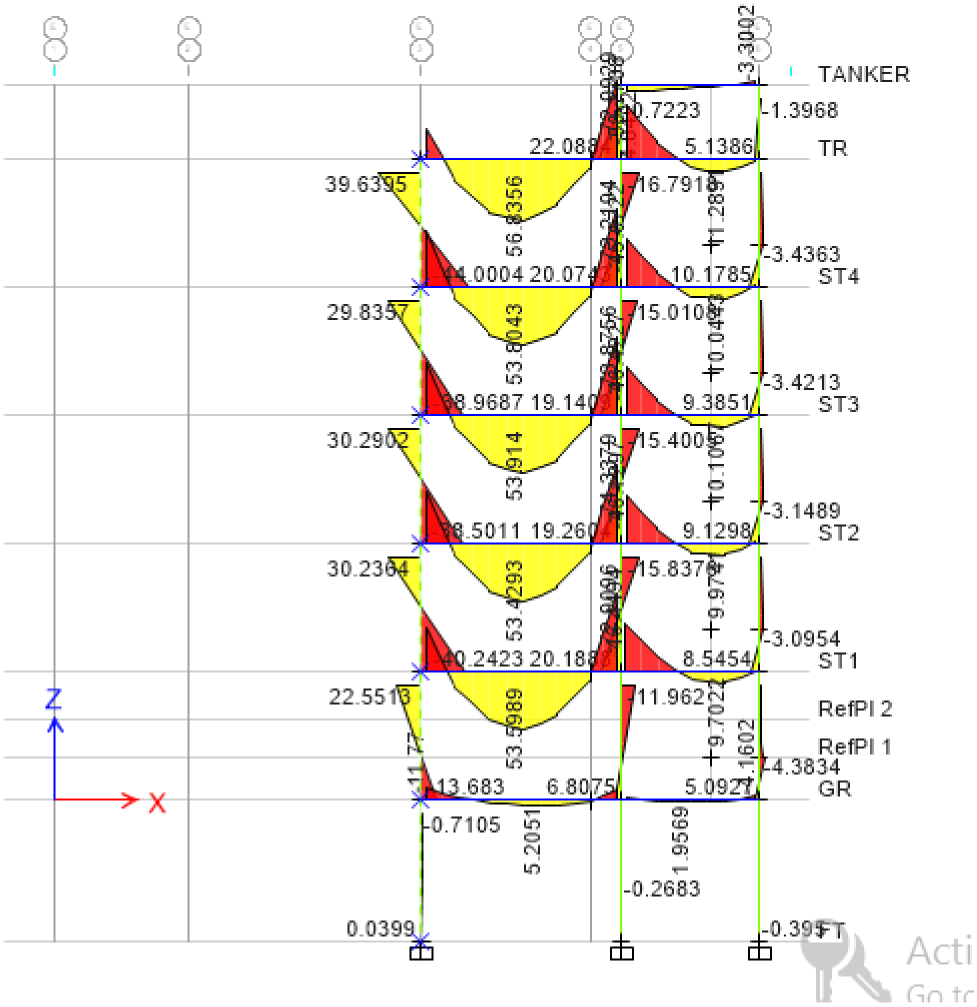
Bending moment diagram on Axis C

Reinforcement of a beam on axis C……………**Appendix 8C**



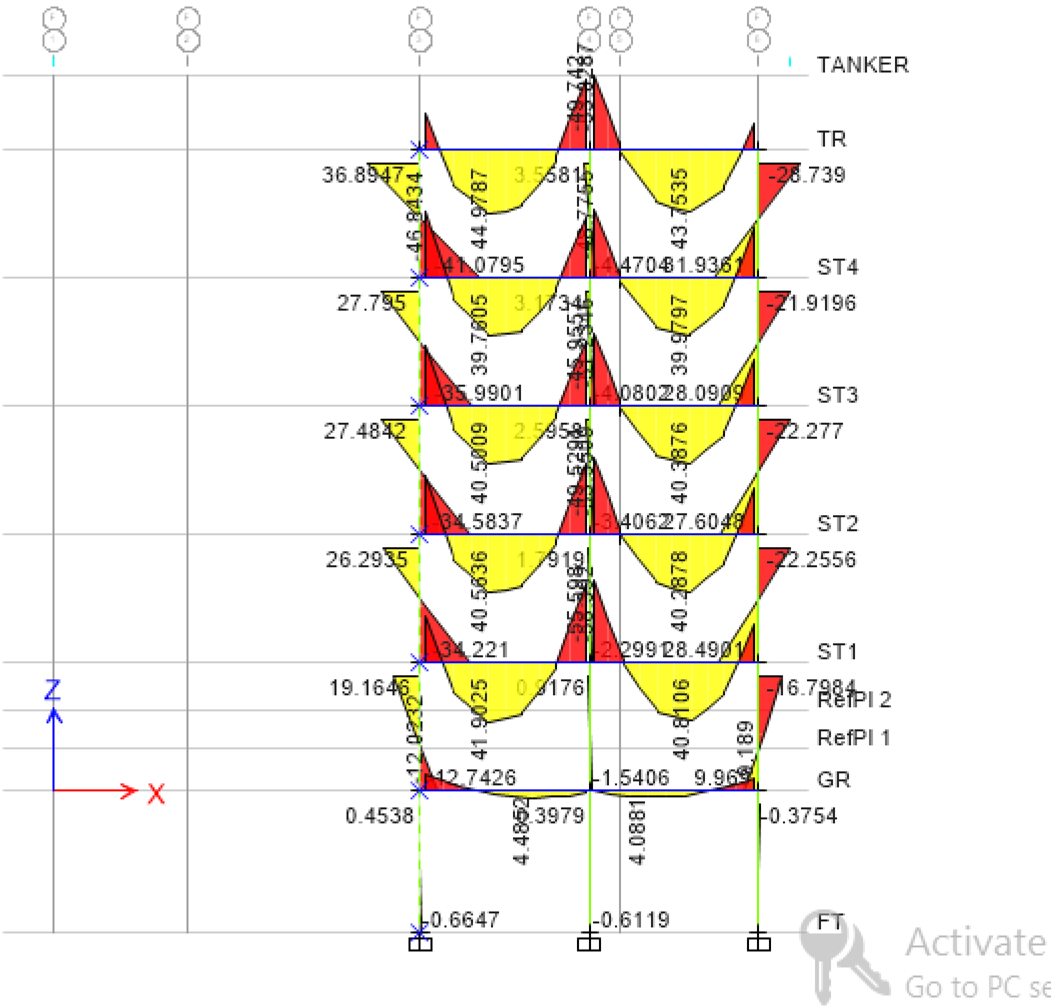
Bending moment diagram on Axis D

Reinforcement of a beam on axis D……………**Appendix 8D**



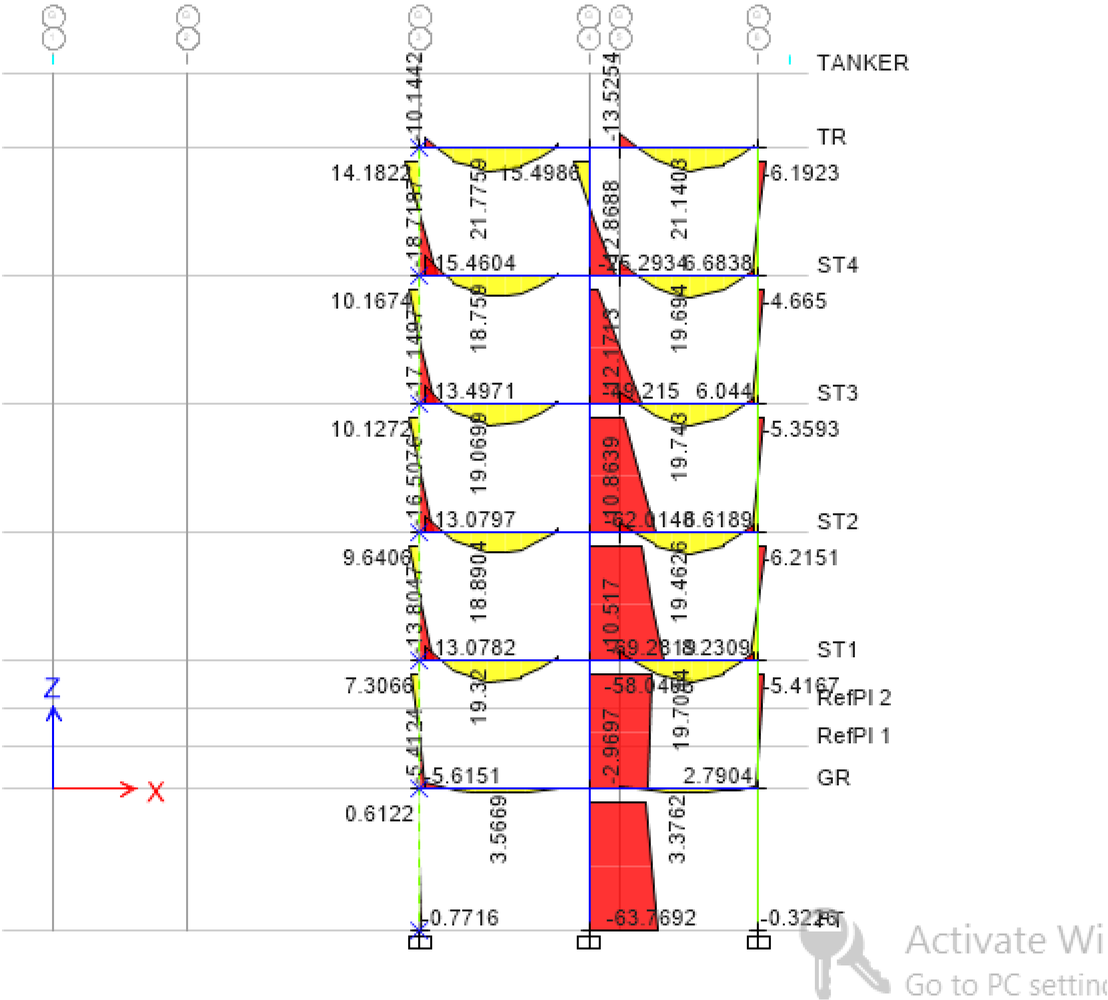
Bending moment diagram on Axis E

Reinforcement of a beam on axis E……………**Appendix 8E**



Bending moment diagram on Axis F

Reinforcement of a beam on axis F ……………**Appendix 8F**



Bending moment diagram on Axis G

Reinforcement of a beam on axis G……………**Appendix 8G**

9 Column Design

9.1 Introduction to column

A column is a vertical structural member transmitting axial compression loads with or without moments. The cross-sectional dimensions of a column are generally considerably less than its height. Columns support mainly vertical loads from the floors and roof and transmit these loads to the foundation. We have two types of column in our project which are rectangular and circular column which is design using 54 design charts by considering uniaxial and biaxial column.

The strength of a column depends on many factors including the following: -

- The strength of the material and shape and size of the cross section
- Length and degree of positional and directional restraints at its end

Classification of columns based on the following criteria: -

A. on the basis of geometry; rectangular, square, circular, L-shaped, T-shaped, etc, depending on the structural or architectural requirement

B. on the basis of degree of slenderness; short column, slender column

C. on the basis of loading: axially loaded column, columns under uni-axial bending or columns under biaxial bending.

D. on the basis of lateral reinforcement; tied columns, spiral columns.

e) On the basis of lateral stability is provided to the structure as a whole; braced or un-braced column.

G On the basis of composition; composite columns, in-filled columns, etc

Rectangular column sample calculation for C16 for 4^{th} floor column

Table 9.1 dimensions of top and bottom beams forC16- 4^{th} floor column

top beam	depth(m)	width(m)	length(m)
B1(x, x)	0.40	0.30	4.75
B1(y, y)	0.40	0.30	3.78
B2(x, x)	0.40	0.30	4.75
B2(y, y)	0.40	0.30	6.53

bottom beam	depth(m)	width(m)	length(m)
B3(x, x)	0.40	0.30	4.75
B3(y, y)	0.40	0.30	3.78
B4(x, x)	0.40	0.30	4.75
B4(y, y)	0.40	0.30	6.53

Column	depth(m)	width(m)	length(m)
	0.45	0.3	3.2

Mtopx-x = 69.42KNm Mtopy-y = 3.55KNm

Mbotx-x = 73.47KNm Mboty-y = 4.47KNm

N_{ED} = 407.44KN

Determination of concrete cover

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

$C_{nom} = C_{min+}\Delta C_{dev}$ …………………………. According to ES EN 1992: 2015 Art 4.4.12(1)

Minimum cover, C_{min}

Minimum concrete cover C_{min} shall be provided in order to ensure: -

- safe transmission bond forces
- protection of steel against corrosion or durability
- an adequate fire resistance

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

$$C_{min} = \text{Max} \begin{Bmatrix} C_{min,b} \\ C_{min,dur} \\ 10mm \end{Bmatrix}$$

where, $C_{min,b}$ -minimum cover due to bond requirement

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

In order to transmit bond forces safely and to ensure adequate compaction of concrete, the minimum cover should not be less then $C_{min,b}$ shown following

Table 9.2 Minimum cover, $C_{min,b}$ requirements with regard to bond

Bond requirement	
Arrangement of bars	Minimum cover, $C_{min,b}$
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕn)
If the nominal maximum aggregate size is greater than 32mm, $C_{min,b}$ should be increased by 5mm.	

Note: The values of $C_{min,b}$ for post- tensioned circular and rectangular ducts for bonded tendons, and pre-tensioned for use in country may be founded in National Annex. The recommended values for post-tensioned ducts are:

Circular ducts: diameter

Rectangular ducts: greater of the smaller dimension or half the greater dimension

There is no requirement for more than 80mm for either circular or rectangular ducts.

The recommended values for pre-tensioned tendon:

1.5*diameter of strand or plan wire

2.5*diameter of indented wire

For prestressing tendons the minimum cover of anchorage should be provided in accordance with the appropriate

Ethiopian Technical Approval

The minimum cover values for reinforcement and prestressing tendons in normal weight concrete taking account of the exposure classes and the structural classes is given by $C_{min,b}$

Note: The recommended structural class (design working life of 50 years) is S4 for the indicative concrete strengths and the recommended minimum structural class is S1. Assume Φ24 longitudinal bar and Φ20 nominal maximum aggregate size. Therefore; $C_{min,b}$=24mm.

Cover design for corrosion or durability

Design for exposure class related to environmental conditions class designation

Table 9.3 Design for exposure class related to environmental

Corrosion induced by carbonation		
XC1	Dry or permanent wet	Concrete inside buildings with low air humidity. Concrete permanently submerged in water
XC2	Wet, rarely day	Concrete surfaces subject to long-term water contact and many foundation
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity and external concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class CX2

Table 9.4 Structural class

Structural class							
	Exposure class						
	X0	XC1	XC2/XC3	XC4	XD1	XD2/XS1	XD3/XS2/ XS3
Design working life of 100 years	Increas e 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	≥C30/ 37 Reduce class by 1	≥C30/37 Reduce class by 1	≥C30/37 Reduce class by 1	≥C45/35R educe class by 1	≥C40/50 Reduce class by 1	≥C40/50 Reduce class by 1	≥C45/35 Reduce class by 1
Member with slab geometry (position of reinforcement is not affected the	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

construction process)							
Special quality control of 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 for corrosion

Table 9.5 indicative minimum strength class

Corrosion										
	Carination-induced corrosion				Chloride-induced corrosion			Chloride-induced corrosion from sea-water		
	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
Indicative strength class	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	

Damage of concrete							
	No risk	Freeze/thaw attack			Chemical attack		
	X0	XF1	XF2	XF3	XA1	XA2	XA3
Indicative strength class	C12/15	C30/C37	C25/C30	C30/C37	C30/37		C35/45

Note: The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. But based on the above table the exposure class is reducing by 1 and the structural class would be S3. Therefore, the value of minimum cover required for durability of reinforcement steel is determined using ES EN 1992:2015 table 4.4N.

Table 9.6 values of minimum cover

Enviromental requirement for $C_{min,dur}$(mm)							
Structural class	Exposure class						
	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

ΔC_{dev} (Allowance in Design for Variation)

Note. The value of allowance in design for variation use in a country may be found in its national annex. The recommended value is 10mm.

The nominal cover should be: -

$C_{nom} = C_{min+}\Delta C_{dev}$=10mm+24mm=34mm

Design for Fire

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

C_{fire}= 10mm, slab thickness (hs) = 80mm

Cover Design depends on the following factors according to ES EN 1991-1-2-2015: -

- ✓ Corrosion
- ✓ Bond or durability and fire

Therefore, the concrete cover should be 34mm.

Determination of effective length of the column

Effective length is used to account for the shape of the deflection curve. It can also be defined as

Buckling length i.e. the length of pin-ended column with constant normal force, having the same cross section and buckling load

- ❖ Effective length (lo) for braced member

From EBCS EN 1992-1-1:2015 (5:15)

l_o=0.5*l$\sqrt{(1+\frac{K1}{0.45+K1})*(1+\frac{K2}{0.45+K2})}$

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

l-length of column

K1-stiffness at end 1

K2-stiffness at end 2

Stiffness at each end (K)=$\frac{\text{column stiffness}}{\sum \text{beam stiffness}}$

K1x-x = $\frac{(\frac{EI}{l})\text{column}}{\sum(\frac{2EI}{l})\text{beam}}$

I_{column}= b*h^3/12 = 350*450^3/12=2,278,125,000mm^4

$I_{beam\ top}$= b*h^3/12 = 300*400^3/12=1,600,000,000mm^4

K1x-x=$\frac{2,278,125,000E/3200}{2*(\frac{1,600,000,000E}{4750})+2*(\frac{1,600,000,000}{4750})} = \frac{711,914.06}{1,347,368.4}$=0.528

Same for K2x-x

I_{column}= b*h^3/12 = 350*450^3/12=2,278,125,000mm^4

$I_{beam\ beam}$= b*h^3/12 = 300*400^3/12=1,600,000,000mm^4

K1y-y =$\frac{2,278,125,000E/3200}{2*(\frac{1,600,000,000E}{3780})+2*(\frac{1,600,000,000}{6530})}$=$\frac{711,914.06}{1,050,746.64}$=0.677

Same for K2y-y

l_ox-x=0.5*l$\sqrt{(1+\frac{K1x-x}{0.45+K1x-x})*(1+\frac{K2x-x}{0.45+K2x-x})}$

l_ox-x = 0.5*3200$\sqrt{(1+\frac{0.528}{0.45+0.528})*(1+\frac{0.528}{0.45+0.528})}$ = 2463.16mm

l_ox-x = 2463.16mm = 2.46m

l_oy-y=0.5*3200$\sqrt{(1+\frac{0.677}{0.45+0.677})*(1+\frac{0.677}{0.45+0.677})}$ =2560mm

l_oy-y = 2560mm = 2.56m

Maximum and Minimum reinforcement limit

$$As_{min} = \text{Max}\begin{cases} 0.1*\frac{N_{ED}}{fyd} = 0.1*\frac{407.44}{347.83} = 117.1mm^2 \\ 0.002Ac = 0.002*350*450 = 315mm^2 \end{cases}$$

$As_{max} = 0.08Ac$ = 0.08*350*450 =12600mm^2

Accidental eccentricity

The effect of imperfection taken from According to ES EN 1992: 2015, Art 5.2.7

$$ei_{x-x} = \text{Max}\begin{cases} \frac{l_o}{400} = \frac{2463.16}{400} = 6.15mm \\ \frac{h}{30} = \frac{320}{30} = 10.6mm \\ 20mm \end{cases}$$

ei_{x-x}= 20mm

$$ei_{y-y} = \text{Max}\begin{cases} \frac{l_o}{400} = \frac{2560}{400} = 6.4mm \\ \frac{h}{30} = \frac{320}{30} = 10.6mm \\ 20mm \end{cases}$$

ei_{y-y}= 20mm

Note:The first order effect includes the geometric imperfection but not consideration of slenderness.The second order effect is considered the slenderness of structural members or deformation of the structure.

First order iteration

M_{ED}= Max {M_{02}, M_{01}+M_2, M_{01}+0.5M_2}

M_{01}= Min {|M_{top}|, |M_{bottom}|+ei*N_{ED}}

M_{02}= Max {|M_{top}|, |M_{bottom}|+ei*N_{ED}}

M_{0e}= 0.4M_{01}+0.6$M_{02} \geq 0.4M_{02}$

For first order moment, M_2= 0.

In the X-direction

$M_{01,x-x}$= Min {|$M_{top,x-x}$|, |$M_{bottom,x-x}$|+ei*N_{ED}}

$M_{01,x-x}$ =Min {|69.42|, |73.47|+0.02*407.4 =81.6}

$M_{01,x-x}$ = 69.42KNm

$M_{02,x-x}$= Max {|$M_{top,x-x}$|, |$M_{bottom,x-x}$|+ei*N_{ED}}

$M_{02,x-x}$ = Max {|69.42|, |73.47|+0.02*407.4 =81.6}

$M_{02,x-x}$ = 81.6KNm

$M_{0e,x-x}$= 0.4$M_{01,x-x}$+0.6$M_{02,x-x}$ ≥ 0.4$M_{02,x-x}$

$M_{0e,x-x}$= 0.4*69.42+0.6*81.6≥ 0.4*81.6

$M_{0e,x-x}$ = 76.72KNm, $M_{ED,x-x}$ = 81.6KNm

In the Y-direction

$M_{01,y-y}$= Min {|$M_{top,y-y}$|, |$M_{bottom,y-y}$|+ei*N_{ED}}

$M_{01,y-y}$ = Min {|3.55|, |4.47|+0.02*407.4 = 12.6}

$M_{01,y-y}$ =3.55KNm

$M_{02,y-y}$= Max {|$M_{top,y-y}$|, |$M_{bottom,y-y}$|+ei*N_{ED}}

$M_{02,y-y}$ = Max {|3.55|, |4.47|+0.02*407.4 = 12.6}

$M_{02,y-y}$ = 12.6KNm

$M_{0e,y-y}$= 0.4$M_{01,y-y}$+0.6$M_{02,y-y}$ ≥ 0.4$M_{02,y-y}$

$M_{0e,y-y}$= 0.4*3.55+0.6*12.6≥ 0.4*12.6

$M_{0e,y-y}$ = 8.98KNm, $M_{ED,y-y}$ =12.6KNm

Equivalent first order eccentricity

In the X-direction

$$e_e = \text{Max}\begin{cases} 0.6e_{02} + 0.4e_{01} \\ 0.4e_{02} \end{cases}$$

$e_{02} = \frac{M_{o2}}{N_{sd}} = \frac{81.6}{407.4}*10^3$ = 200.29mm

$e_{01} = \frac{M_{o1}}{N_{sd}} = \frac{69.42}{407.4} * 10^3 = 170.39mm$

$e_e = Max \begin{cases} 0.6e_{02} + 0.4e_{01} = 0.6 * 200.29 + 0.4 * 170.39 = 188.33mm \\ 0.4e_{02} = 0.4 * 200.29 = 80.11mm \end{cases}$

$e_e = 188.33mm$

In the Y-direction

$e_e = Max \begin{cases} 0.6e_{02} + 0.4e_{01} \\ 0.4e_{02} \end{cases}$

$e_{02} = \frac{M_{o2}}{N_{sd}} = \frac{12.6}{407.4} * 10^3 = 30.92mm$

$e_{01} = \frac{M_{o1}}{N_{sd}} = \frac{3.55}{407.4} * 10^3 = 18.7mm$

$e_e = Max \begin{cases} 0.6e_{02} + 0.4e_{01} = 0.6 * 30.92 + 0.4 * 18.7 = 26.03mm \\ 0.4e_{02} = 0.4 * 30.92 = 12.36mm \end{cases}$

$e_e = 26.03mm$

Check slenderness Limit

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

i - is radius of gyration of the uncracked concrete

$\lambda_{x-x} = \frac{l_o, x-x}{\mathbf{i}}$, $i = \sqrt{\frac{I}{A}}$

$I = b*h^3/12 = 350*450^3/12 = 2,278,125,000mm^4$

$A = 350*450 = 157,500mm^2$

$i = \sqrt{\frac{I}{A}} = \sqrt{\frac{2,278,125,000}{157,500}} = 120.26mm$

$\lambda_{x-x} = \frac{l_o, x-x}{\mathbf{i}} = \frac{2463.16}{120.26} = 20.48mm$

$\lambda_{y-y} = \frac{l_o, y-y}{\mathbf{i}} = \frac{2560}{120.26} = 21.28mm$

The minimum limiting value of slenderness is

$\lambda_{lim} = \frac{20ABC}{\sqrt{n}}$

Where, $A = \frac{1}{1 + 0.2\varphi ef}$, φef- effective creep ratio

$B = \sqrt{1 + 2\omega}$, $\omega = \frac{As*fyd}{Ac*fcd}$, ω - mechanical reinforcement ratio

$C = 1.7 - r_m$, $n = \frac{N_{ed}}{A_c * f_{cd}}$, n - relative normal force

$r_m = \frac{M_{01}}{M_{02}}$, r_m - moment ratio

If φefis not known, A=0.7

If $\omega$is not known, B=1.1

$\lambda_{lim,x-x} = \frac{20ABC}{\sqrt{n}}$,Where, A = 0.7, B = 1.1

$C = 1.7 - r_m$, $r_m = \frac{M_{01,x-x}}{M_{02},x-x} = \frac{69.42}{81.6} = 0.85$

$n = \frac{N_{ed}}{A_c * f_{cd}} = \frac{407.4*1000}{11.33*350*450} = 0.228$,C = 1.7- 0.85 =0.85

Therefore, slenderness limit ($\lambda_{lim,x-x}$) = $\frac{20ABC}{\sqrt{n}} = \frac{20*0.7*1.1*0.85}{\sqrt{0.228}} = 27.85$mm

$\lambda_{lim,y-y} = \frac{20ABC}{\sqrt{n}}$Where, A = 0.7, B = 1.1

$C = 1.7 - r_m$, $r_m = \frac{M_{01,y-y}}{M_{02},y-y} = \frac{3.55}{12.6} = 0.28$

$n = \frac{N_{ed}}{A_c * f_{cd}} = \frac{407.4*1000}{11.33*350*450} = 0.228$,C = 1.7- 0.28 = 1.42

Therefore, slenderness limit (λ_{y-y}) = $\frac{20ABC}{\sqrt{n}} = \frac{20*0.7*1.1*1.42}{\sqrt{0.228}} = 46.52$mm

Check slenderness

$\lambda_{x-x} > \lambda_{lim,x-x}$, slender column otherwise short column

$\lambda_{x-x} = 20.48$mm$< \lambda_{lim,x-x} = 27.85$mm, it isshort column

$\lambda_{y-y} > \lambda_{lim,y-y}$, slender column otherwise short column

$\lambda_{y-y} = 21.28$mm$< \lambda_{lim,y-y} = 46.52$mm, it isshort column

$V_{sd} = \frac{N_{sd}}{A_c * f_{cd}} = \frac{407.4*1000}{11.33*350*450} = 0.22$

$\mu_{sd,x-x} = \frac{M_{ED,x-x}}{A_c * f_{cd} * h} = \frac{81.6*10^6}{11.33*450*350*450} = 0.101$

$\mu_{sd,y-y} = \frac{M_{ED,y-y}}{A_c * f_{cd} * h} = \frac{12.6*10^6}{11.33*450*350*450} = 0.015$

Take $\frac{d'}{h}$ = 0.2 andusing biaxial rectangular chart number read the mechanical steel ratio from biaxial interaction chart number 37, = 0.34

Reinforcement calculation

V_{sd}=0.22,$\mu_{sd,y-y}$= 0.101, $\mu_{sd,z-z}$= 0.015,ω = 0.34

- ✓ Longitudinal reinforcement

$As_{tot} = \frac{\omega * Ac * fcd}{fyd} = \frac{0.34*350*450*11.33}{347.8} = 1744.4mm^2$

As_{min}= 320mm^2,As_{max}= 12600mm^2

Take Φ20mm, as =3.14*20^2/4 = 314mm^2

No of bar = $\frac{As}{as} = \frac{1744.4}{314}$ = 5.5≈6

s = $\frac{b*as}{As} = \frac{350*314}{1744.4}$ = 70mm

Provide Φ20C/C70mm

- ✓ For shear reinforcement

$$\text{Diameter of bar} = \text{Max}\begin{cases} \frac{\text{Longitudinal bar diameter}}{4} = \frac{20}{6} = 4mm \\ \text{Diameter of stirrup} = 8mm \end{cases}$$

From above take Φ8mm diameter of bar for shear reinforcement

$$\text{Spacing of shear reinforcement} = \text{Min}\begin{cases} 20 * \Phi long, = 20 * 20mm = 400mm \\ \text{lesser dimension of column} = 320mm \\ 400mm \end{cases}$$

Finally, provide ΦC/C 320mm

10 Shear wall Design

A reinforced concrete wall is a vertical load bearing member whose greatest lateral dimension is more than four times its least lateral dimension, and in which the reinforcement is taken in to account when considering its strength A reinforced wall shall be considered as either short or slender and as either braced or un braced as follows; Short or Slender Walls: A wall may be considered short when the ratio of its effective height to its thickness does not exceed 7. It shall otherwise be considered slender. Braced or un-braced Walls: A wall may be considered as braced if, at right Angles to the plane of the wall, lateral stability to the structure as a whole is provided by walls or Other suitable bracing designed to resist all lateral forces in that direction.

It shall otherwise be considered as un-braced .The shear wall is subjected to both flexure and axial load, so we designed as column load bearing member resist lateral and axial load. From ETABS 2013 modeling analysis we get the maximum axial load and moments from Envelope.

Wall A

Axial load, P=1094.62KN

Shear force=60.15KN

Given parameters

C25

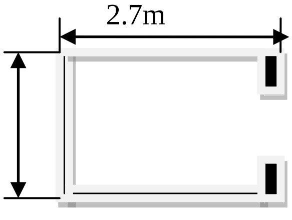
S-400 class B

Step 1) Determine material design Constant

$$\text{fcd} = \frac{\propto \text{cc} * \text{fck}}{\gamma\text{c}} = \frac{0.85 * 20}{1.5} = 11.33\text{Mpa}$$

$$\text{fyd} = \frac{400}{1.15} = 347.8\text{Mpa}$$

Step 2) Determination of effective depth and length



The total depth of the shear wall is given 200mm from the architectural drawing using Ø10mm bar and the clear cover 20mm

Effective depth of the wall become d=t-cover-Ø/2=200-20-10/2 =175mm

The effective height L of reinforced concrete wall in the non-sway mode shall be determined as from

Le=βL, where L-the store height of the wall

β-the coefficient defined below

β =1.0 Wall with two edge restrained

So Le=1*3.2=3.2m

Step 3 Design the Shear wall

Hence the wall loads are Vary from floor to floor we consider the wall result which found on the ground to base and provided this for the rest of the floor. The effect of imperfection may be taken as according to ES EN 1992:Art5.2.7

$$e_i = \max\begin{cases} \frac{Lo}{400} \\ \frac{h}{30} \\ 20mm \end{cases}$$

$$e_i = \max\begin{cases} \frac{3200mm}{400} \\ \frac{200}{30} \\ 20mm \end{cases} = \max\begin{cases} 8mm \\ 6.6mm \\ 20mm \end{cases} \text{take } e_i=20mm$$

Hence the shear wall is subjected to the pure axial load and so let's reduces the capacity by 20%

$$NED = 0.8 * (fcd * Ac + As * fyd)$$

$$As = (NED/0.8) - fcd * Ac$$

For wall A

$$As = 1.25NED - fcd * Ac$$

$$= 1.25 * 1094.62 * 10^6) - 11.33 * 1800 * 200$$

=1364.19 mm2

Asr in the rows become AS/2=1364.19/2 mm2 =682.098mm2

Using Ø10 bars

Spacing becomes

$$S = asxb/Asr$$

=78.5x1800/682.098=207.2mm

Use Ø10 bar c/c 200mm on the two face and the reinforcement for the transverse direction becomes

Spacing =2xspacing of the main bar

=2x200=400

Use Ø10 bar c/c 400mm on the two face of the wall

Step 4) Check for shear resistance of the wall

VRD,c = (CRD,c*K(100*ϱ*fck)1/3+K1*δcp)bw*d (Vmin+K1*δcp)bw*d

where

CRD,c=0.18/ϒc = 0.18/1.5=0.12

K = (1+√200/d) 2.0 d in mm

= (1+√200/2175) ≤ 2.0 = 2.06 ≤2.0

K1=0.15

ϱ=Asl/bwd≤ 2.0

Asl=is the area of the tensile reinforcement ,which extends (lbd+d) beyond the section considered

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

ϱ=682.098/(175x1800)≤ 0.02

=0.00216 ≤ 0.02

ϱ=0.00216

Vmin = 0.035*K3/2*fck1/2

= 0.035*2.03/2*201/2

= 0.443

δcp = NED/AC 0.2fcd

NED = the axial force in the cross section in the tensile area (mm)

AC=is the area of the concrete cross section (mm2)

NED=1094.62

δcp=1094.62/(1800*200*0.2*11.33)=0.0013

VRD,c = (CRD,c*K(100*ϱ*fck)1/3+K1*δcp)bw*d (Vmin+K1*δcp)bw*d

= (0.12*2.0*(100*0.00216*20)1/3+0.15*0.0013)2700*175 (0.433+0.15*0.0013)2700*175

= 376.738KN

VRD,c=376.738KN>122.763KN..OK!

Therefore no need of shear Reinforcement

11 Foundation Analysis and Design

11.1 Introduction

Foundation is that part of a structure which transmits the weight of the structure to the ground.

The main objectives of a foundation are the following

- To distribute the weight of the structure over larger area so as to avoid over loading of the soil beneath.
- To load the sub structure evenly so as to avoid unequal settlement.
- To provide a level surface for building operations.
- To take the sub structure deep into the ground and thus increase its stability and avoid overturning.

The foundation should be design such that

- The soil below doesn't fail in shear.
- The settlement is within the safe limit

From the ETABS output of axial loads with serviceable combination of loads, we group the foundations based on their respective axial loads and tabulated below

.

Table 11.1 ETABS output of axial loads

Story	Joint Label	Load Case/Co	FZ kN	MX kN-m	MY kN-m	MZ kN-m
FT	1	Comb1	410.932	-0.2238	1.2753	-0.0002
FT	2	Comb1	1166.713	1.8289	1.2973	-0.0002
FT	3	Comb1	1368.066	2.5263	0.4195	-0.0002
FT	4	Comb1	1159.646	2.8295	-0.0907	-0.0002
FT	5	Comb1	509.7173	0.3061	-0.1458	-0.0002
FT	6	Comb1	1130.43	-18.951	2.9881	-0.0015
FT	7	Comb1	1862.495	-0.6416	1.915	-0.0002
FT	8	Comb1	2279.114	-0.8395	0.5201	-0.0002
FT	9	Comb1	996.2479	1.5735	0.8534	-0.0002
FT	10	Comb1	567.5317	1.0781	-0.5578	-0.0002
FT	11	Comb1	294.2205	-0.2928	1.1349	-0.0002
FT	12	Comb1	871.5318	0.1688	1.4395	-0.0002
FT	13	Comb1	1023.613	0.5119	-0.9964	-0.0002
FT	14	Comb1	967.4587	-0.4358	-0.0399	-0.0002
FT	15	Comb1	927.6568	-2.0537	0.2509	-0.0002
FT	16	Comb1	532.4998	0.2819	0.395	-0.0002
FT	17	Comb1	1009.243	0.0973	0.6647	-0.0002
FT	18	Comb1	2041.734	0.7972	0.6119	-0.0002
FT	19	Comb1	975.4796	0.0079	0.1654	-0.0002
FT	20	Comb1	378.6508	0.7271	0.7716	-0.0002
FT	21	Comb1	1192.051	1.1874	63.7692	-0.0015
FT	22	Comb1	350.0074	0.8238	0.3226	-0.0002
FT	41	Comb1	129.5969	-0.8571	0.0411	-0.1237
FT	42	Comb1	845.991	-1.078	2.4001	0.1573
FT	47	Comb1	1058.05	-1.5249	1.7084	0.0906
FT	49	Comb1	1094.623	-1.4893	7.933	-0.1566
FT	50	Comb1	915.8848	-0.2928	4.0766	-0.0899
FT	51	Comb1	99.3662	-0.1505	0.0614	0.1235

Design of footing

Design constants

A. Material data

Design of F1

Design constants

A. Material data(concrete)

Structural class	S-4
Exposure class	-XC2/XC3
Cmin, durability	25mm
Cmin, bond ϕ16mm	16mm
Cmin, used	25mm
Cover	25+10+25=60mm

B. (concrete) Concrete data

$fcu(\text{MPa}) = C25/30$ $\quad$ $\text{fctm(MPa)} = \frac{0.3*25^2}{3}$

$\text{fck} = 25$ $\quad$ $\gamma_{mc} = 1.5$

$$f\text{cd} = \frac{0.85 * 25}{1.5} = 14.17$$

$E_{cm}(\text{GPa}) = 31$ $\quad$ Cover = 60

C. Reinforcement data

$$f\text{yk(MPa)} = 400$$

$$fyd(\text{MPa}) = \frac{400}{1.15} = 347.826$$

$$\gamma_{ms} = 1.15$$

$$E_s(\text{GPa}) = 200$$

Main bar (mm)= $\emptyset 16$

Since our site is located in Wolkite, it is difficult to undergo with soil investigation so we assume the site to have soil type cohesive clay soil with allowable bearing capacity of 350Kpa!

For F1 Bearing capacity of soil 350KN/m^2

Dimension of column is 500*500mm

$\gamma_{conc.} = 25\text{KN/m}^3$, $\quad$ $\gamma_{soil} = 150\text{KN/m}^3$

Area proportioning

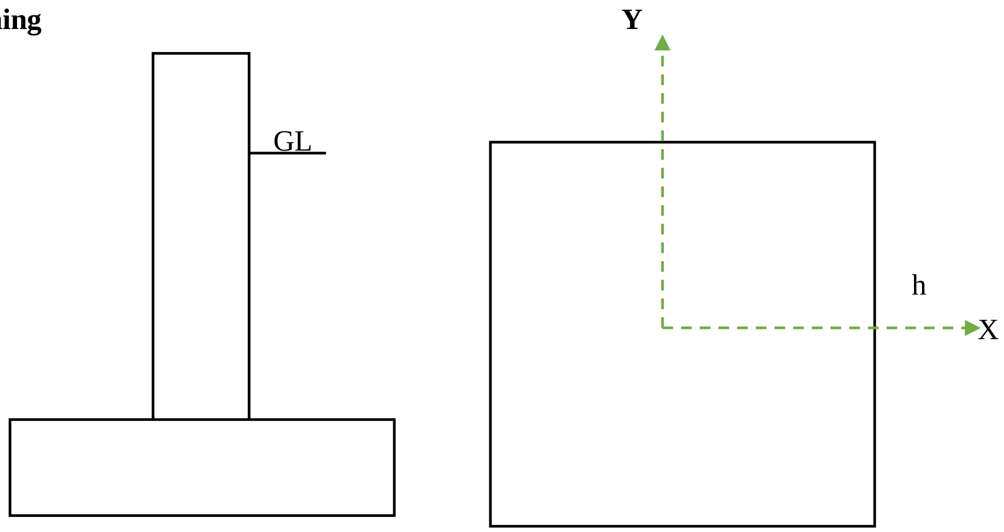


Figure 10.3 isolated footing and footing pad

First let us assume that the footing pad is square

Area = B*L=B*B as L/B=1

From bearing capacity of the soil using the theoretical method $q_{max} \leq q_{all}$

From F2 group we take the maximum values as follows ETABS 16.2 out put

$F_z = 1368.066$

$M_x = 2.5263$

$M_y = 0.4195$

Area of footing required

$$A = \frac{(N + W)}{\gamma_{soil}} = \frac{1368.066 + 136.8066}{150} = 10.032$$

Where W=10%N

$B * H * h = 3.25 * 2.6 * 0.7 = 5.915m^2$

$Area = 2.6 * 3.25 = 8.45m^2$

$\text{Weight of footing} = 8.45 * 0.7 * 25 = 147.875KN$

$$\delta = \frac{P}{A}\left(1 \pm \frac{6e_y}{B} \pm \frac{6e_x}{L}\right)$$

$$e_x = \frac{M_y}{Fz} = \frac{0.4195}{1368.066} = 0.00031$$

$$e_y = \frac{M_x}{Fz} = \frac{2.5263}{1368.066} = 0.00185$$

$$\delta_{max} = \frac{P}{A}\left(1 + \frac{6e_y}{B} + \frac{6e_x}{L}\right) = \frac{1368.066}{8.45}\left(1 + \frac{6 * 0.00185}{2.6} + \frac{6 * 0.00031}{3.25}\right) = 162.6852$$

$$\delta_{min} = \frac{P}{A}\left(1 - \frac{6e_y}{B} - \frac{6e_x}{L}\right) = \frac{1368.066}{8.45}\left(1 - \frac{6 * 0.00185}{2.6} - \frac{6 * 0.00031}{3.25}\right) = 161.1175$$

So, determine self-weight of the footing and surcharge load on the footing

Weight of the soil= (2.2*2.2-0.6*0.5) *1.7*19 = 146.64KN

Weight of footing = 2.2*2.2*0.7*25 =84.7 KN

Total design load = 1245.54+146.64+84.7= 1477KN (serviceability)

Total design load=2660.27+146.64+84.7=2891.61KN(ultimate

Q_{max} or min$\delta = \frac{P}{A}\left(1 \pm \frac{6e_y}{B} \pm \frac{6e_x}{L}\right)$

Assuming square footing of area

$A = B * B = B^2 = \frac{P}{A}\left(1 \pm \frac{6e_y}{B} \pm \frac{6e_x}{L}\right)$

$= \frac{1368.066}{B * B}\left(1 \pm \frac{6 * 0.00185}{B} \pm \frac{6 * 0.00031}{B}\right)$

$q_{max} \leq q_{all} == \frac{1368.066}{B * B}\left(1 \pm \frac{6 * 0.00185}{B} \pm \frac{6 * 0.00031}{B}\right)$

Column size h = 0.5m, b = 0.5m

For initial assumption factor safety

$$F.S = \frac{(F_z + M_y + M_z)\text{Ultimate}}{(F_z + M_y + M_z)\text{Servicebility}} = \frac{2279.1141 + 0.5201 + (-0.0002)}{1654.454 + 0.3767 + (-0.0001)} = 1.377$$

$$Q_{ult} = F.S \text{ x Qall} = 1.377 * 400\text{kpa} = 550.8\text{Kpa}$$

Let us assume depth of footing = 0.7m

Depth of soil = 1.7m

So, determine self-weight of the footing and surcharge load on the footing

Weight of the soil= $(2.2 * 2.2 - 0.5 * 0.5) * 1.7 * 19 = 148.25\text{KN}$

Weight of footing $= 2.2 * 2.2 * 0.7 * 25 = 84.7\text{KN}$

Total design load $= 1654.454 + 148.25 + 84.7 = 1887.4\text{KN}$(serviceability)

Total design load= $2279.1141 + 148.25 + 84.7 = 2512.06\text{KN}$(ultimate)

Eccentricity at serviceability limit

$ex = My/Fz = 0.3767KNm/1887.4KN = 0.0000199m$

$$ey = Mx/Fz = 2.6KNm/1245.54KN = 0.0017$$

Footing 1

Dimension of column is 350*450mm

From bearing capacity of the soil using the theoretical method

FZ=P=567.53

M_Y=1.07

M_X=0.55

Area of footing req

$$A = \frac{(N+W)}{\gamma_{soil}} = \frac{567.53+56.7}{150} = 4.16$$

where W=10%N

B*H*h =3.25*2.6*0.7

Area =3.25*2.6**= 8.45m²**

Weight =8.45*0.7*25 =147.87KN

δ=$\frac{P}{A}*(1 \pm \frac{6ey}{B} \pm \frac{6eX}{B})$ where ex=$\frac{My}{P} = \frac{1.07}{567.53}$ =0.002

ey=$\frac{Mx}{P} = \frac{0.55}{567.53}$ =0.0009

$\delta_{max} = \frac{567.53}{8.45}*(1+\frac{6(0.0009)}{3.25}+\frac{6(0.002)}{2.6})$ = 67.50

$\delta_{min} = \frac{567.53}{8.45}*(1-\frac{6(0.0009)}{2.6}-\frac{6(0.002)}{2.6})$ = 66.81

ơmax=67.50<66.81 ơ(allowable)=350

Punching shear resistance

$M_{xx}=79.07*\frac{1.1^2}{2}+(67.50-66.81)(\frac{1.1}{2})*(1.1*\frac{2}{3}$=47.84+5.336= **48.11**

$M_{yy}=(\frac{67.50+66.81}{2})*\frac{1.1^2}{2}$**= 40.62**

- Effective depth

  dx = h-c-o.5øbar=700-35-(0.5*20) = **655**

  dy = h-c-15øbar =700-35-(1.5*20) = **635**

- Main reinforcement longitudinal bar

K= $\frac{Mxx}{fck*bd^2} = \frac{48.11}{20*3.25*70^2}$ = 0.00015< 0.167Wherefck=0.8*25= 20

Z= d(0.25-($\frac{K}{1.134}$))≤0.95d , 0.95*0.7≤0.665

$Z=0.7(0.25-(\frac{0.0021}{1.134}))$

Z= **<u>0.174</u>**

$Z=\frac{d}{2}(1+\sqrt{1-3.53k} \leq 0.665$

Z= 0.7/2(1+√(1-3.53(0.0021))≤ 0.665

Z= 0.699≤ 0.665

Z=0.665

AS= M/fydz = 48.11/(347.82*0.665) =208 mm2

Min & max area of reinforcement

Asmin = (0.26*fctm*b*d)/fyk where fctm=0.3*〖fck〗^(2⁄3) ,fck=400/1.15 = 347.82

=0.3*〖20〗^(2⁄3) =2.21

= (0.26*2.21*3.25*0.7)/400

Asmin= 3268mm2

Asmax=0.04*Ac=0.04*3.25*0.7

Asmax =91000mm2

Use Asmin=3268 mm2

UsProvide 16Ø16(As=2614mm2)

Main reinforcement-transvers bar

k=M_yy/(fck*b*d^2)=40.62/(20*3.25*〖629〗^2)=0.0023<Kbal=0.167

There for compression reinforcement is not required.

z=d(0.25-k/1.134)=0.23d<0.95d

Asreq=Myy/(0.87fyk*z)=(47.22*〖10〗^6)/(0.87*400*0.239*629)=902.6〖mm〗^2

Minimum and maximum area of reinforcement

Asmin=0.26*(fctm/fyk)*bd=0.26*(2.5/400)*3.25*629=3321.9〖mm〗^2

Asmax=0.04*Ac=0.04*3.25*0.629=81770〖mm〗^2

Since As<Amin use Asmin= 3321.9〖mm〗^2

Provide 16Ø16(As=3321.9mm2)

1.Vertical shear

Critical shear at load from face of column

Average pressure critical section

=66.81+(2.15/3.25)*13.23 =75.56

Design shear force

V€d= 75.56*0.35*3.25=85.95KN

K=1+√(200/d)= 1+√(200/700) = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm2

ჯ1=Asl/bd =0

VRdC=0

Vmin=[0.035K^(3⁄2) √fck]bd

Vmin=[0.035*〖1.532〗^(3⁄2) √20]3250*700

Vmin= 657.23 KN

V€d<Vmin OK

Punching shear

Critical shear at 2.0d from face of column

Average d=(700+635)/2 = 667.5

2d=1335

Control perimeter

U=2(350+450)+(2π*1335)=9984mm

Area within perimeter

A=[0.35*0.45]+[2*0.35*1.335]+[2*0.45*1.335]+[π*1.3352]

=7.89m2

Average punching shear strees

V€d=((67.5+66.81)/2)[(3.25*2.6)-7.89]= 559.56 KN

Punching shear stress

V€d=BV€d/Ud , B=1+K(M€d/V€d)(U1/W1)

W1=0.5c12+c1c2+4c2d+16d2+2πdc1

=0.5(400) 2+400*400+4*400*667.5+16*667.52 +2*π*667.5*400

W1= 10113660mm2

B=1+0.659((6.05*〖10〗^6)/(559.56*〖10〗^3))(9984/10113660)=1.007

V€d=(1.007*559.56*〖10〗^3)/(9984*700) =0.08N/mm2

Punching shear resistance

K=1+√(200/d) =1+√(200/700) =1.535<2

VRdc=Vmin=0.035K^(3⁄2) 〖fck〗^(1⁄2)

=0.035〖(1.535)〗^(3⁄2) 〖(20)〗^(1⁄2) =0.298

0.298>V€d(0.08) OK

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=527.7085

U=2(350+450)=1600

V€d=(BV€d/Uod) B=1+K(M€d/V€d)(Uo/w1)

W1=0.5*4002+400*400

W1=240000mm2

B=1+0.65((6.05*〖10〗^6)/(527.7085*〖10〗^3))(1600/240000)

B=1.05

V€d=((1.05*527.7085*〖10〗^3)/(1600*886))=0.4N/mm2

Maximum shear resistance

Ved.max=0.5[0.6(1- fck/250)]fck/1.5

=0.5[0.6(1- 20/250)](20/1.5)

=3.68>V€dOK

Footing 2

Dimension of column is 350*450mm

From bearing capacity of the soil using the theoretical method

FZ=P=2279.114

MY=0.5201

MX=0.8395

Area of footing req

A=((N+W))/(ϒsoil) =(2279.114+227.9)/150 = 10.03 where W=10%N

B*H*h =3.25*2.6*0.7

Area =3.25*2.6=8.45m2

Weight =8.45*0.7*25 =147.87KN

δ =P/A *(1±6ey/B±6eX/B) where ex=My/P =0.5201/1368.06 =0.0003

=Mx/P =0.8395/2279.114 =0.0018

δmax=2279.114/8.45 *(1+6(0.0003)/3.25+(6(0.0018))/2.6) = 170.69

δmin=2279.114/8.45 *(1-6(0.0003)/3.25-(6(0.0018))/2.6) = 161.14

σmax= 170.69<161.14σ(allowable)=350

Punching shear resistance

Mxx=165.9*〖1.1〗^2/2+(170.69-161.14)(1.1/2)*(1.1*2/3 = 104.22

Myy=((170.69+161.14)/2)*〖1.1〗^2/2= 100.34

Effective depth

dx = h-c-o.5øbar=700-35-(0.5*20) = 655

dy = h-c-15øbar =700-35-(1.5*20) = 635

Main reinforcement longitudinal bar

K= Mxx/〖fck*bd〗^2 =48.11/〖20*3.25*70〗^2 = 0.00015< 0.167Wherefck=0.8*25= 20

Z= d(0.25-(K/1.134))≤0.95d , 0.95*0.7≤0.665

Z=0.7(0.25-(0.0021/1.134))

Z= 0.174

Z= d/2(1+√(1-3.53k)≤ 0.665

Z= 0.7/2(1+√(1-3.53(0.0021))≤ 0.665

Z= 0.699≤ 0.665

Z=0.665

AS= M/fydz = 104.22/(347.82*0.665) =450 mm2

Min & max area of reinforcement

Asmin = (0.26*fctm*b*d)/fyk where fctm=0.3*〖fck〗^(2⁄3) ,fck=400/1.15 = 347.82

=0.3*〖20〗^(2⁄3) =2.21

= (0.26*2.21*3.25*0.7)/400

Asmin= 3268mm2

Asmax=0.04*Ac=0.04*3.25*0.7

Asmax =91000mm2

Use Asmin=3268 mm2

UsProvide 16Ø16(As=2614mm2)

Main reinforcement-transvers bar

$k=M_{yy}/(f_{ck}*b*d^2)=40.62/(20*3.25*〖629〗^2)=0.0023<K_{bal}=0.167$

There for compression reinforcement is not required.

$z=d(0.25-k/1.134)=0.23d<0.95d$

Asreq=Myy/(0.87fyk*z)=(47.22*〖10〗^6)/(0.87*400*0.239*629)=902.6〖mm〗^2

Minimum and maximum area of reinforcement

Asmin=0.26*(fctm/fyk)*bd=0.26*(2.5/400)*3.25*629=3321.9〖mm〗^2

Asmax=0.04*Ac=0.04*3.25*0.629=81770〖mm〗^2

Since As<Amin use Asmin= 3321.9〖mm〗^2

Provide 16Ø16(As=3321.9mm2)

1.Vertical shear

Critical shear at load from face of column

Average pressure critical section

=100.34+(2.15/3.25)*13.23 =109.15

Design shear force

V€d= 109.15*0.35*3.25=124.15KN

K=1+√(200/d)= 1+√(200/700) = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm2

ʒ1=Asl/bd =0

VRdC=0

Vmin=[0.035K^(3⁄2) √fck]bd

Vmin=[0.035*〖1.532〗^(3⁄2) √20]3250*700

Vmin= 657.23 KN

V€d<Vmin OK

Punching shear

Critical shear at 2.0d from face of column

Average d=(700+635)/2 = 667.5

2d=1335

Control perimeter

U=2(350+450)+(2π*1335)=9984mm

Area within perimeter

A=[0.35*0.45]+[2*0.35*1.335]+[2*0.45*1.335]+[π*1.3352]

=7.89m2

Average punching shear strees

V€d=((170.69+161.141)/2)[(3.25*2.6)-7.89]= 559.56 KN

Punching shear stress

V€d=BV€d/Ud , B=1+K(M€d/V€d)(U1/W1)

W1=0.5c12+c1c2+4c2d+16d2+2πdc1

=0.5(400) 2+400*400+4*400*667.5+16*667.52 +2*π*667.5*400

W1= 10113660mm2

B=1+0.659((6.05*〖10〗^6)/(559.56*〖10〗^3))(9984/10113660)=1.007

V€d=(1.007*559.56*〖10〗^3)/(9984*700) =0.08N/mm2

Punching shear resistance

K=1+√(200/d) =1+√(200/700) =1.535<2

VRdc=Vmin=0.035K^(3⁄2) 〖fck〗^(1⁄2)

=0.035〖(1.535)〗^(3⁄2) 〖(20)〗^(1⁄2) =0.298

0.298>V€d(0.08) OK

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=527.7085

U=2(350+450)=1600

V€d=(BV€d/Uod) B=1+K(M€d/V€d)(Uo/w1)

W1=0.5*4002+400*400

W1=240000mm2

B=1+0.65((6.05*〖10〗^6)/(527.7085*〖10〗^3))(1600/240000)

B=1.05

V€d=((1.05*527.7085*〖10〗^3)/(1600*886))=0.65N/mm2

Maximum shear resistance

Ved.max=0.5[0.6(1- fck/250)]fck/1.5

=0.5[0.6(1- 20/250)](20/1.5)

=3.68>V€dOK

Footing 3

Dimension of column is 350*450mm

From bearing capacity of the soil using the theoretical method

FZ=P=2279.114

MY=0.5201

MX=0.8395

Area of footing req

A=((N+W))/(ϒsoil) =(2279.114+227.9)/150 = 16.7 where W=10%N

B*H*h =3.25*2.6*0.7

Area =3.25*2.6=8.45m2

Weight =8.45*0.7*25 =147.87KN

δ=P/A *(1±6ey/B±6eX/B) where ex=My/P =0.419/2279.114 =0.00018

ey=Mx/P =0.8395/2279.114 =0.0003

δmax=2279.114/8.45 *(1+6(0.0003)/3.25+(6(0.00018))/2.6) = 270.96

δmin=2279.114/8.45 *(1-6(0.0003)/3.25-(6(0.0018))/2.6) = 268.43

σmax=67.50<66.81 σ(allowable)=350

Punching shear resistance

Mxx=269.96*〖1.1〗^2/2+(270.96-268.43)(1.1/2)*(1.1*2/3 = 164.34

Myy=((270.96+268.43)/2)*〖1.1〗^2/2=63.13

Effective depth

dx = h-c-o.5øbar=700-35-(0.5*20) = 655

dy = h-c-15øbar =700-35-(1.5*20) = 635

Main reinforcement longitudinal bar

K= Mxx/〖fck*bd〗^2 =48.11/〖20*3.25*70〗^2 = 0.00015< 0.167Wherefck=0.8*25= 20

Z= d(0.25-(K/1.134))≤0.95d , 0.95*0.7≤0.665

Z=0.7(0.25-(0.0021/1.134))

Z= 0.174

Z= d/2(1+√(1-3.53k)≤ 0.665

Z= 0.7/2(1+√(1-3.53(0.0021))≤ 0.665

Z= 0.699≤ 0.665

Z=0.665

AS= M/fydz = 164.34/(347.82*0.665) =450 mm2

Min & max area of reinforcement

Asmin = (0.26*fctm*b*d)/fyk where fctm=0.3*〖fck〗^(2⁄3) ,fck=400/1.15 = 347.82

=0.3*〖20〗^(2⁄3) =2.21

= (0.26*2.21*3.25*0.7)/400

Asmin= 3268mm2

Asmax=0.04*Ac=0.04*3.25*0.7

Asmax =91000mm2

Use Asmin=3268 mm2

UsProvide 16Ø16(As=2614mm2)

Main reinforcement-transvers bar

k=M_yy/(fck*b*d^2)=40.62/(20*3.25*〖629〗^2)=0.0023<Kbal=0.167

There for compression reinforcement is not required.

z=d(0.25-k/1.134)=0.23d<0.95d

Asreq=Myy/(0.87fyk*z)=(47.22*〖10〗^6)/(0.87*400*0.239*629)=902.6〖mm〗^2

Minimum and maximum area of reinforcement

Asmin=0.26*(fctm/fyk)*bd=0.26*(2.5/400)*3.25*629=3321.9〖mm〗^2

Asmax=0.04*Ac=0.04*3.25*0.629=81770〖mm〗^2

Since As<Amin use Asmin= 3321.9〖mm〗^2

Provide 16Ø16(As=3321.9mm2)

1.Vertical shear

Critical shear at load from face of column

Average pressure critical section

=66.81+(2.15/3.25)*13.23 =75.56

Design shear force

V€d= 75.56*0.35*3.25=85.95KN

K=1+√(200/d)= 1+√(200/700) = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm2

ʒ1=Asl/bd =0

VRdC=0

Vmin=[0.035K^(3⁄2) √fck]bd

Vmin=[0.035*〖1.532〗^(3⁄2) √20]3250*700

Vmin= 657.23 KN

V€d<Vmin OK

Punching shear

Critical shear at 2.0d from face of column

Average d=(700+635)/2 = 667.5

2d=1335

Control perimeter

U=2(350+450)+(2π*1335)=9984mm

Area within perimeter

A=[0.35*0.45]+[2*0.35*1.335]+[2*0.45*1.335]+[π*1.3352]

=7.89m2

Average punching shear strees

V€d=((67.5+66.81)/2)[(3.25*2.6)-7.89]= 559.56 KN

Punching shear stress

V€d=BV€d/Ud , B=1+K(M€d/V€d)(U1/W1)

W1=0.5c12+c1c2+4c2d+16d2+2πdc1

=0.5(400) 2+400*400+4*400*667.5+16*667.52 +2*π*667.5*400

W1= 10113660mm2

B=1+0.659((6.05*〖10〗^6)/(559.56*〖10〗^3))(9984/10113660)=1.007

V€d=(1.007*559.56*〖10〗^3)/(9984*700) =0.08N/mm2

Punching shear resistance

K=1+√(200/d) =1+√(200/700) =1.535<2

VRdc=Vmin=0.035K^(3⁄2) 〖fck〗^(1⁄2)

=0.035〖(1.535)〗^(3⁄2) 〖(20)〗^(1⁄2) =0.298

0.298>V€d(0.08) OK

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=527.7085

U=2(350+450)=1600

V€d=(BV€d/Uod)   B=1+K(M€d/V€d)(Uo/w1)

W1=0.5*4002+400*400

W1=240000mm2

B=1+0.65((6.05*〖10〗^6)/(527.7085*〖10〗^3))(1600/240000)

B=1.05

V€d=((1.05*527.7085*〖10〗^3)/(1600*886))=0.92N/mm2

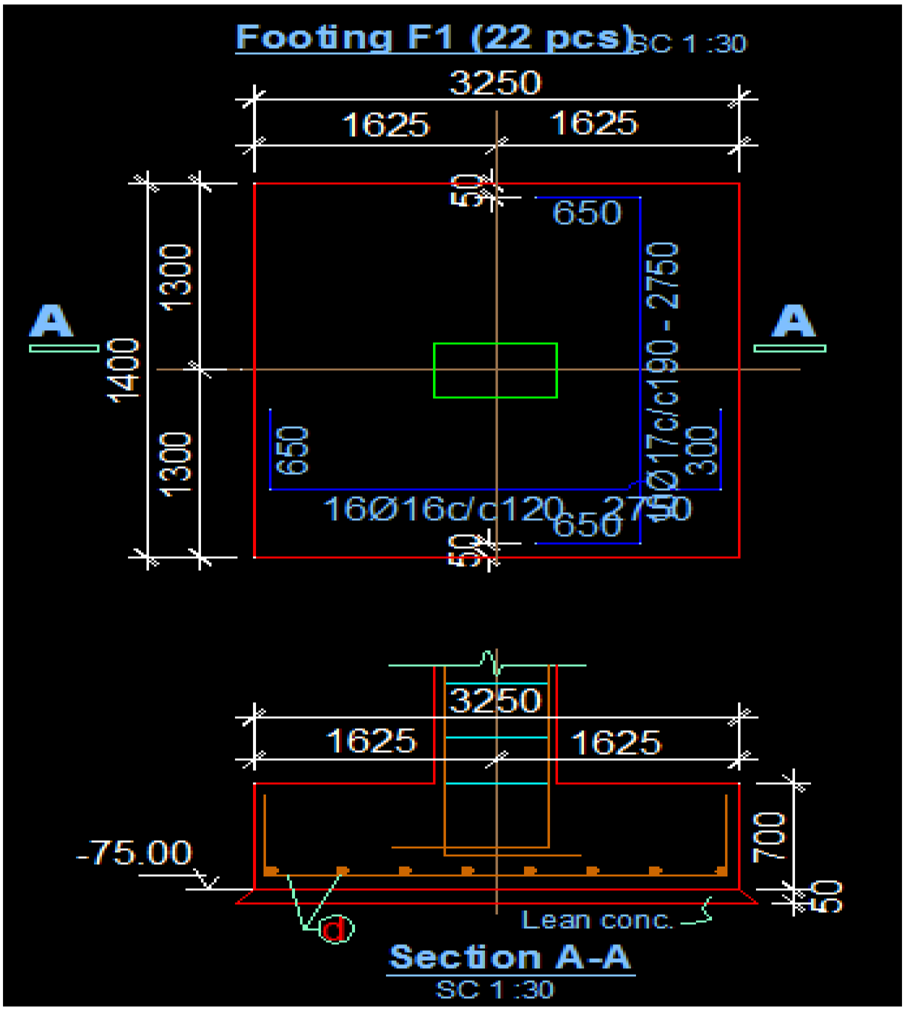
Maximum shear resistance

Ved.max=0.5[0.6(1- fck/250)]fck/1.5

=0.5[0.6(1- 20/250)](20/1.5)

=3.68>V€dOK

Footing reinforcement detailing



12 Conclusion and Recommendation

Conclusion

In this final year project which required a lot struggle to fruit out the final design of this G + 4Resendetial building and each important part of the 1 building was carefully analyzed and designed for the appropriate loading conditions as much as possible to get the final design result. In design of this building, we have tried to satisfy the most basic requirements of design of a structure in accordance with revised Ethiopian building code standard derived from the European building code standard. The design of reinforced concrete members were designed economical, safe and serviceable as much as possible.

Recommendation

Nowadays it is well known that analysis and design of any Engineering Structure is supported with Design Software, hence we recommend that it would have been better if Design Software Courses is given for Civil Engineers as a subject in order to make the students familiar with different software.

Finally, we recommend that, since design & analysis of a structure with the Euro Code is new in our context, it demands patience and hard work and hence need to be thoroughly referred to get much out of it.

To make the students rich in information, there should be internet access in the computer room.

It is better to teach the software that is used to design civil engineering structures just like as a one course.

13 REFERENCES

1. Dr. Suresh Borra, P.M.B. RajKiran Nanduri and Sk. Naga Raju,Design Method of Reinforced Concrete Shear Wall Using EBCS, American Journal of Engineering Research (AJER),Volume-4, Issue-3,2015, pp.31-43

2. Abdeta D and Yohanes , Design of Column and Beam Handbook, Wolkite university, Wolkite, Ethiopia,February11, 2021.

3. Bowels T. Foundation Analysis and Design,University of Kansas, Kansa, America,1974,pp.1-14.

4. Arthur H. Nilson, Design of Concrete Structures, 12thEdition, McGraw Hill, 1997, 780pp.

5. European Standard Based on Ethiopia Norm,1992-1-1:2015, Basis of design and actions on structures,Addis Ababa university, Addis Ababa, pp.1-87, December 01, 2015.

6. H, N. Design of Concrete Structures 14thEdition, Cornell University, Cornell, America, pp.293, 1974.

7. Mustafa T. Designof Foundation Hand Book, WolkiteUniversity, Wolkite,Ethiopia February 2, 2021.

8. Ali Abel-Rahman, Fundamentals of Reinforced Concrete, New Offset,1991.pp.246.

9. European Standard Based on Ethiopia Norm,1998.1-1.2015, Design of Structres for Earth Quake Resistances,Addis Ababa university, Addis Ababa, pp.1-238, December 01, 2015.

10. CSA Committee A23.3, design of Concrete Structures for Buildings, CAN3-a23.3-M84 Canadian standards Association, Rexdale, Canada, 1994,199pp.

Appendaix

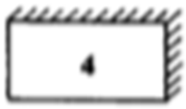
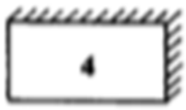
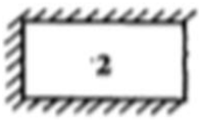
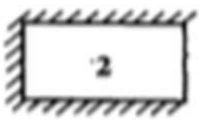
Appendix 2A Effective depth will be calculated for roof floor slab

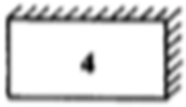
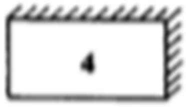
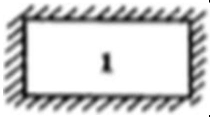
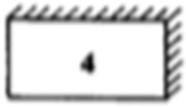
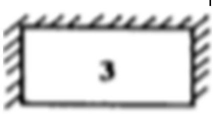
Panel	Lx	Ly	N	K	F1	F2&F3	d (mm)	D (mm)
P1	3780	4720	17.1	1.3	1.25	1	131.3412	156.3412
P2	3780	4790	17.1	1.3	1.25	1	131.3412	156.3412
P3	4720	6530	17.1	1.3	1.25	1	164.0028	189.0028
P4	4790	6530	17.1	1.3	1.25	1	166.435	191.435
P5	3780	3900	17.1	1.3	1.25	1	131.3412	156.3412
P6	3900	6520	17.1	1.3	1.25	1	135.5108	160.5108
P7	5390	5630	17.1	1.3	1.25	1	168.1513	193.1556
P8	3780	5620	17.1	1.3	1.25	1	131.3412	156.3412
P9	5620	6520	17.1	1.3	1.25	1	170.2745	195.2745
P10	5620	5630	17.1	1.3	1.25	1	170.2745	194.2745
P11	3880	6480	17.1	1.3	1.25	1	134.8158	159.8158
P12	3880	4530	17.1	1.3	1.25	1	131.3412	156.3412

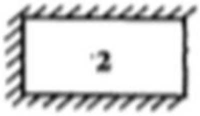
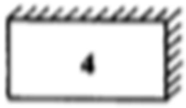
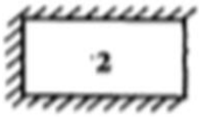
Appendix 2B design loads for first floor slab

Panel	Wind load	Water proofing bitumen	Cement screed	RC slab weight	Ceiling plaster load	Total dead load	Live load	design load
P1	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P2	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P3	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P4	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P5	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P6	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P7	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P8	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P9	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P10	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P11	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77
P12	8.41	0.28	0.69	4.25	0.46	14.09	0.5	19.77

Appendix 2C Moment analysis for two way roof slab

Panel		L_x(m)	L_y(m)	L_y/L_x	α_i	$\alpha_i$value	Pd (KN/m^2)	M_i (KNm/m)	moment value
P1		3.78	4.72	1.25	a_{xs}	0.066	19.77	M_{xs}	18.56
					a_{xf}	0.049	19.77	M_{xf}	13.92
					a_{ys}	0.047	19.77	M_{ys}	12.81
					a_{yf}	0.036	19.77	M_{yf}	9.60
P2		3.78	4.79	1.27	a_{xs}	0.066	19.77	M_{xs}	18.86
					a_{xf}	0.049	19.77	M_{xf}	14.15
					a_{ys}	0.047	19.77	M_{ys}	12.81
					a_{yf}	0.036	19.77	M_{yf}	9.60
P3		4.72	6.53	1.38	a_{xs}	0.055	19.77	M_{xs}	23.97
					a_{xf}	0.041	19.77	M_{xf}	17.98
					a_{ys}	0.039	19.77	M_{ys}	16.15
					a_{yf}	0.029	19.77	M_{yf}	12.11
P4		4.79	6.53	1.36	a_{xs}	0.072	19.77	M_{xs}	24.42
					a_{xf}	0.053	19.77	M_{xf}	18.31
					a_{ys}	0.047	19.77	M_{ys}	12.47
					a_{yf}	0.036	19.77	M_{yf}	16.63

P5		3.78	3.9	1.03	a_{xs}	0.052	19.77	M_{xs}	14.20
					a_{xf}	0.039	19.77	M_{xf}	10.65
					a_{ys}	0.047	19.77	M_{ys}	12.81
					a_{yf}	0.036	19.77	M_{yf}	9.60
P6		3.9	6.52	1.67	a_{xs}	0.083	19.77	M_{xs}	23.91
					a_{xf}	0.063	19.77	M_{xf}	17.93
					a_{ys}	0.047	19.77	M_{ys}	11.03
					a_{yf}	0.036	19.77	M_{yf}	8.27
P7					a_{xs}	0.083	19.77	M_{xs}	19.65
					a_{xf}	0.063	19.77	M_{xf}	14.74
					a_{ys}	0.047	19.77	M_{ys}	18.38
					a_{yf}	0.036	19.77	M_{yf}	13.78
P8		3.78	5.62	1.49	a_{xs}	0.078	19.77	M_{xs}	21.88
					a_{xf}	0.059	19.77	M_{xf}	16.41
					a_{ys}	0.047	19.77	M_{ys}	12.81
					a_{yf}	0.036	19.77	M_{yf}	9.60
P9		5.62	6.52	1.16	a_{xs}	0.053	19.77	M_{xs}	33.23
					a_{xf}	0.04	19.77	M_{xf}	24.92

					a_{ys}	0.039	19.77	M_{ys}	22.90
					a_{yf}	0.03	19.77	M_{yf}	17.17
P10		5.63	5.63	1.00	a_{xs}	0	19.77	M_{xs}	24.74
					a_{xf}	0.034	19.77	M_{xf}	18.56
					a_{ys}	0.045	19.77	M_{ys}	22.90
					a_{yf}	0.034	19.77	M_{yf}	17.17
P11		3.88	6.48	1.67	a_{xs}	0.089	19.77	M_{xs}	25.07
					a_{xf}	0.067	19.77	M_{xf}	18.80
					a_{ys}	0	19.77	M_{ys}	13.49
					a_{yf}	0.044	19.77	M_{yf}	10.12
P12					a_{xs}	0.089	19.77	M_{xs}	13.95
					a_{xf}	0.067	19.77	M_{xf}	10.47
					a_{ys}	0	19.77	M_{ys}	10.91
					a_{yf}	0.044	19.77	M_{yf}	8.18

Appendix 3A Effective depth will be calculated for solid slab

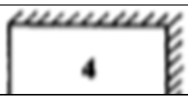
Panel	Lx	Ly	N	K	F1	F2&F3	d (mm)	D (mm)
P1	3780	4720	17.1	1.3	1.25	1	131.3412	156.3412
P2	3780	4790	17.1	1.3	1.25	1	131.3412	156.3412
P3	4720	6530	17.1	1.3	1.25	1	164.0028	189.0028
P4	4790	6530	17.1	1.3	1.25	1	166.435	191.435
P5	3780	3900	17.1	1.3	1.25	1	131.3412	156.3412
P6	3900	6520	17.1	1.3	1.25	1	135.5108	160.5108

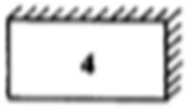
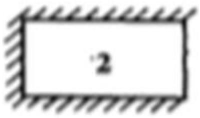
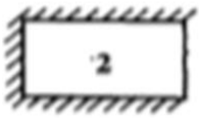
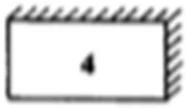
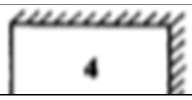
P7	5390	5630	17.1	1.3	1.25	1	168.1513	193.1556
P8	3780	5620	17.1	1.3	1.25	1	131.3412	156.3412
P9	5620	6520	17.1	1.3	1.25	1	170.2745	195.2745
P10	5620	5630	17.1	1.3	1.25	1	170.2745	194.2745
P11	3880	6480	17.1	1.3	1.25	1	134.8158	159.8158
P12	3880	4530	17.1	1.3	1.25	1	131.3412	156.3412

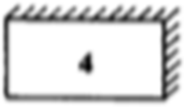
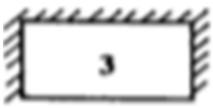
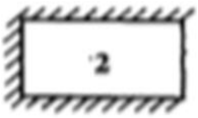
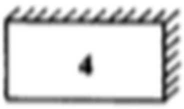
Appendix 3B Design load for solid slab

Panel	Partition load	Wall plastering load	F.F load	Cement screed	RC slab weight	Ceiling plaster load	Total dead load	Live load	(Pd)design load
P1	1.02	0.65	0.46	0.69	4.38	0.46	7.66	2	13.34
P2	3.60	0.46	0.46	0.69	4.38	0.46	10.05	2	16.57
P3	1.30	0.56	0.46	0.69	4.38	0.46	7.85	2	13.60
P4	1.23	0.24	0.46	0.69	4.38	0.46	7.46	2	13.07
P5	0	0	0.32	0.69	4.38	0.46	5.85	2	10.90
P6	0.93	0.19	0.46	0.69	4.38	0.46	7.11	2	12.60
P7	0	0	0.54	0.69	4.38	0.46	6.07	2	11.19
P8	2.76	1.12	0.46	0.69	4.38	0.46	9.87	2	16.32
P9	2.93	0.59	0.32	0.69	4.38	0.46	9.37	5	20.15
P10	3.66	0.74	0.54	0.69	4.38	0.46	10.47	5	21.63
P11	0.97	0.19	0.46	0.69	4.38	0.46	7.15	2	12.65

Appendix 3C Moment analysis for two way $\mathbf{1^{st} - 4^{th}}$ floor slab

Panel		L_x(m)	L_y(m)	L_y/L_x	α_i	α_i value	Pd (KN/m^2)	M_i (KNm/m)	Moment value
P1		3.78	4.72	1.25	a_{xs}	0.066	13.34	M_{xs}	12.58

					a_{xf}	0.049	13.34	M_{xf}	9.34
					a_{ys}	0.047	13.34	M_{ys}	8.96
					a_{yf}	0.036	13.34	M_{yf}	6.86
P2		3.78	4.79	1.27	a_{xs}	0.066	16.57	M_{xs}	15.63
					a_{xf}	0.049	16.57	M_{xf}	11.60
					a_{ys}	0.047	16.57	M_{ys}	11.13
					a_{yf}	0.036	16.57	M_{yf}	8.52
P3		4.72	6.53	1.38	a_{xs}	0.055	13.60	M_{xs}	16.66
					a_{xf}	0.041	13.60	M_{xf}	12.42
					a_{ys}	0.039	13.60	M_{ys}	11.82
					a_{yf}	0.029	13.60	M_{yf}	8.79
P4		4.79	6.53	1.36	a_{xs}	0.072	13.07	M_{xs}	21.59
					a_{xf}	0.053	13.07	M_{xf}	15.90
					a_{ys}	0.047	13.07	M_{ys}	14.09
					a_{yf}	0.036	13.07	M_{yf}	10.79
P5		3.78	3.9	1.03	a_{xs}	0.052	10.90	M_{xs}	8.10
					a_{xf}	0.039	10.90	M_{xf}	6.07
					a_{ys}	0.047	10.90	M_{ys}	7.32
					a_{yf}	0.036	10.90	M_{yf}	5.61
P6		3.9	6.52	1.67	a_{xs}	0.083	12.60	M_{xs}	15.91

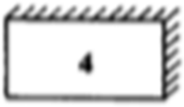
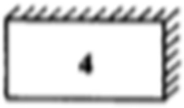
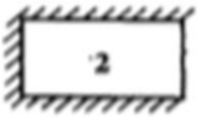
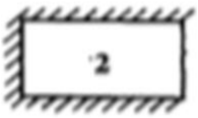
					a_{xf}	0.063	12.60	M_{xf}	12.07
					a_{ys}	0.047	12.60	M_{ys}	9.01
					a_{yf}	0.036	12.60	M_{yf}	6.90
P7									
P8		3.78	5.62	1.49	a_{xs}	0.078	16.32	M_{xs}	18.19
					a_{xf}	0.059	16.32	M_{xf}	13.76
					a_{ys}	0.047	16.32	M_{ys}	10.96
					a_{yf}	0.036	16.32	M_{yf}	8.40
P9		5.62	6.52	1.16	a_{xs}	0.053	20.15	M_{xs}	33.73
					a_{xf}	0.04	20.15	M_{xf}	25.45
					a_{ys}	0.039	20.15	M_{ys}	24.82
					a_{yf}	0.03	20.15	M_{yf}	19.09
P10		5.63	5.63	1.00	a_{xs}	0	21.63	M_{xs}	12.73
					a_{xf}	0.034	21.63	M_{xf}	23.22
					a_{ys}	0.045	21.63	M_{ys}	30.74
					a_{yf}	0.034	21.63	M_{yf}	23.22
P11		3.88	6.48	1.67	a_{xs}	0.089	12.65	M_{xs}	16.94
					a_{xf}	0.067	12.65	M_{xf}	12.75
					a_{ys}	0	12.65	M_{ys}	0
					a_{yf}	0.044	12.65	M_{yf}	8.38

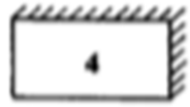
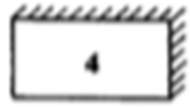
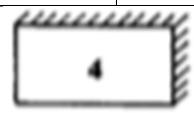
Appendix 3DSpan moment adjustment$1^{st} - 4^{th}$ solid slab

panel to be adjusted	Support moment		ΔM_{sup}	$\frac{\Delta M_{sup.}}{M_{large}}$	Adjusted Support moment		L_y/L_x	C_x	C_y	ΔM	ΔM_{xf}	ΔM_{yf}
	M_{large}	M_{small}	.		M_{averag}	$M_{balad.}$						
P1 & P2	11.13	8.96	2.17	0.19	10.045							
P1 & P3	12.58	11.82	0.76	0.06	12.2							
P2 & P4	15.63	14.09	1.54	0.10	14.86							
P3 & P4	21.59	16.66	4.93	0.23		19.13	1.36	0.388	0.339	2.47	0.956	0.835
P3 & P7	1	1	1	1.00	1							
P5 & P6	9.01	8.1	0.91	0.10	8.56							
P5 & P8	10.96	7.32	3.64	0.33		9.5	1.49	0.306	0.096	1.82	0.557	0.175
P6 & P9	33.73	15.91	17.82	0.53		26.45	1.16	0.346	0.381	8.91	3.083	3.395
P7 & P10	1	1	1	1.00	1							
P8 & P9	24.82	18.19	6.63	0.27	21.51							
P9 & P10	30.74	24.82	5.92	0.19	27.78							
P10 & P11	30.74	16.94	13.8	0.45		23.55	1	0.380	0.280	7.20	2.733	2.014

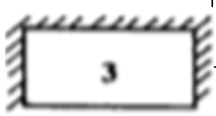
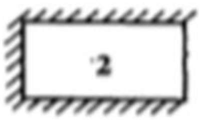
Panel to be adjusted		P1 & P2	P1 & P3	P2 & P4	P3 & P4	P3 & P7	P5 & P6	P5 & P8	P6 & P9	P7 & P10	P8 & P9	P9 & P10	P10 & P11
Field moment	M_{xf}				15.9			13.76	25.45				23.22
	M_{yf}				10.79			8.40	19.09				23.22
Adjusted field moment	M_{xf}^*				16.86			14.32	28.53				26
	M_{yf}^*				11.63			8.54	22.48				25.23

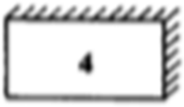
Appendix 3D Adjusted Span moment1st – 4^{th} solid slab

Panel		α_i	$\alpha_i$value	Pd (KN/m^2)	M_i (KNm/m)	Moment value	Adjusted moment
P1		a_{xs}	0.066	13.34	M_{xs}	12.58	12.2
		a_{xf}	0.049	13.34	M_{xf}	9.34	9.34
		a_{ys}	0.047	13.34	M_{ys}	8.96	10.045
		a_{yf}	0.036	13.34	M_{yf}	6.86	6.86
P2		a_{xs}	0.066	16.57	M_{xs}	15.63	14.86
		a_{xf}	0.049	16.57	M_{xf}	11.60	11.60
		a_{ys}	0.047	16.57	M_{ys}	11.13	10.045
		a_{yf}	0.036	16.57	M_{yf}	8.52	8.52
P3		a_{xs}	0.055	13.60	M_{xs}	16.66	19.14
		a_{xf}	0.041	13.60	M_{xf}	12.42	12.42
		a_{ys}	0.039	13.60	M_{ys}	11.82	12.2
		a_{yf}	0.029	13.60	M_{yf}	8.79	8.79
P4		a_{xs}	0.072	13.07	M_{xs}	21.59	19.14
		a_{xf}	0.053	13.07	M_{xf}	15.90	16.86
		a_{ys}	0.047	13.07	M_{ys}	14.09	14.86
		a_{yf}	0.036	13.07	M_{yf}	10.79	11.63

P5		a_{xs}	0.052	10.90	M_{xs}	8.10	8.58
		a_{xf}	0.039	10.90	M_{xf}	6.07	6.07
		a_{ys}	0.047	10.90	M_{ys}	7.32	9.5
		a_{yf}	0.036	10.90	M_{yf}	5.61	5.61
P6		a_{xs}	0.083	12.60	M_{xs}	15.91	26.46
		a_{xf}	0.063	12.60	M_{xf}	12.07	12.07
		a_{ys}	0.047	12.60	M_{ys}	9.01	8.58
		a_{yf}	0.036	12.60	M_{yf}	6.90	6.90
P7							
P8		a_{xs}	0.078	16.32	M_{xs}	18.19	21.51

Appendix 3D adjusted Span moment1^{st} – 4^{th} solid sla

		a_{xf}	0.059	16.32	M_{xf}	13.76	14.32
		a_{ys}	0.047	16.32	M_{ys}	10.96	9.5
		a_{yf}	0.036	16.32	M_{yf}	8.40	8.54
P9		a_{xs}	0.053	20.15	M_{xs}	33.73	26.46
		a_{xf}	0.04	20.15	M_{xf}	25.45	28.53
		a_{ys}	0.039	20.15	M_{ys}	24.82	21.51
		a_{yf}	0.03	20.15	M_{yf}	19.09	25.45
P10		a_{xs}	0	21.63	M_{xs}	12.73	12.73
		a_{xf}	0.034	21.63	M_{xf}	23.22	25.15

		a_{ys}	0.045	21.63	M_{ys}	30.74	24.11
		a_{yf}	0.034	21.63	M_{yf}	23.22	25.15
P11		a_{xs}	0.089	12.65	M_{xs}	16.94	24.11
		a_{xf}	0.067	12.65	M_{xf}	12.75	12.75
		a_{ys}	0	12.65	M_{ys}	0	0
		a_{yf}	0.044	12.65	M_{yf}	8.38	8.38

Appendix 3RReinforcement design for $1^{st} - 4^{th}$ floor slab

PANEL			**Loc**	**M_{adj}**	**d**	μsd,s	**z**	$A_{st,cal}$	$A_{st,min}$	$A_{st,req}$	S_{cal}	S_{prov}	$A_{st,used}$	ρ_{cal}
	Type	4	M_{xf}	9.34	175	0.027	172.61	155.56	250.25	250.5	313.8	310	253.35	0.00145
	0		M_{xs}	12.2	175	0.035	171.87	204.08	250.25	250.5	313.8	310	253.35	0.00145
0	S-1	1	M_{yf}	6.86	165	0.022	163.14	120.89	235.95	235.5	332.9	330	238.00	0.00145
	1		M_{ys}	10.05	165	0.033	162.27	177.97	235.95	235.5	332.9	330	238.00	0.00145
	Type	4	M_{xf}	11.60	175	0.033	172.03	193.87	250.25	250.5	313.8	310	253.35	0.00145
	0		M_{xs}	14.86	175	0.043	171.17	249.59	250.25	250.5	313.8	310	253.35	0.00145
1	S-1	0	M_{yf}	8.52	165	0.028	162.69	150.56	235.95	235.5	332.9	330	238.00	0.00145
	1		M_{ys}	10.05	165	0.033	162.27	177.97	235.95	235.5	332.9	330	238.00	0.00145
	Type	2	M_{xf}	12.42	175	0.036	171.81	207.83	250.25	250.5	313.8	310	253.35	0.00145
	1		M_{xs}	19.13	175	0.055	170.04	323.45	250.25	323.5	242.8	240	327.25	0.00145
0	S-1	1	M_{yf}	8.79	165	0.028	162.62	155.41	235.95	235.5	332.9	330	238.00	0.00145

	1		M_{ys}	12.20	165	0.040	161.67	216.95	235.95	235.5	332.9	330	238.00	0.00145
	Type	2	M_{xf}	11.63	175	0.034	172.02	194.38	250.25	250.5	313.8	310	253.35	0.00145
	1		M_{xs}	19.13	175	0.055	170.04	323.45	250.25	323.5	242.8	240	327.25	0.00145
1	S-1	0	M_{yf}	16.86	165	0.055	160.36	302.27	235.95	302.7	259.8	250	314.16	0.00145
	1		M_{ys}	14.86	165	0.048	160.93	265.48	235.95	265.8	295.8	290	270.83	0.00145
	Type	4	M_{xf}	6.07	175	0.017	173.46	100.61	250.25	250.5	313.8	310	253.35	0.00145
	0		M_{xs}	8.58	175	0.025	172.81	142.74	250.25	250.5	313.8	310	253.35	0.00145
0	S-1	1	M_{yf}	5.61	165	0.018	163.49	98.66	235.95	235.5	332.9	330	238.00	0.00145
	1		M_{ys}	9.50	165	0.031	162.42	168.16	235.95	235.5	332.9	330	238.00	0.00145
	Type	4	M_{xf}	12.07	175	0.035	171.90	201.87	250.25	250.5	313.8	310	253.35	0.00145
	0		M_{xs}	26.46	175	0.076	168.05	452.67	250.25	452.7	173.5	170	462.00	0.197
1	S-1	0	M_{yf}	6.90	165	0.022	163.13	121.60	235.95	235.5	332.9	330	238.00	0.00145
	1		M_{ys}	8.58	165	0.028	162.67	151.64	235.95	235.5	332.9	330	238.00	0.00145
	Type	4	M_{xf}	14.32	175	0.041	171.31	240.32	250.25	250.5	313.8	310	253.35	0.00138

	1		M_{xs}	21.51	175	0.062	169.40	365.06	250.25	365.6	215.1	210	374.00	0.00164
0	S-1	1	M_{yf}	8.54	165	0.028	162.68	150.92	235.95	235.5	332.9	310	253.35	0.00145
	0		M_{ys}	9.50	165	0.031	162.42	168.16	235.95	235.5	332.9	330	238.00	0.00145
	Type	3	M_{xf}	25.45	175	0.073	168.33	434.67	250.25	434.7	180.7	180	436.33	0.00197
	1		M_{xs}	26.46	175	0.076	168.05	452.67	250.25	452.7	173.5	200	392.70	0.00197
1	S-1	1	M_{yf}	19.05	165	0.062	159.74	342.86	235.95	342.6	229.1	220	357.00	0.00159
	0		M_{ys}	21.51	165	0.070	159.03	388.86	235.95	388.8	202.0	200	392.70	0.00180
	Type	2	M_{xf}	26.00	175	0.075	168.18	444.47	250.25	444.5	176.7	170	462.00	0.00197

	1		M_{xs}	12.73	175	0.037	171.73	213.12	250.25	250.3	313.8	310	253.35	0.00146
0	S-1	1	M_{yf}	25.15	165	0.082	157.98	457.70	235.95	457.7	171.6	170	462.00	0.00218
	1		M_{ys}	24.11	165	0.078	158.28	437.93	235.95	438.0	179.3	170	462.00	0.00207
	Type	4	M_{xf}	12.75	175	0.037	171.72	213.46	250.25	250.3	313.8	310	253.35	0.00143
	0		M_{xs}	24.11	175	0.069	168.69	410.90	250.25	411.0	191.1	210	374.00	0.00187
1	S-1	0	M_{yf}	8.38	165	0.027	162.73	148.05	235.95	236.0	332.9	330	238.00	0.00145
	1		M_{ys}	0.00	165	0.000	165.00	0.00	235.95	236.0	332.9	330	238.00	0.00145

Appendix 8A Reinforcement of a beam on axis A

Axis A	TERACE											
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	13.22	300	352	0.018	346.39	Single	137.28	4224	109.72	137.28	0.4372	4Φ 20
Span	b/n1&2	6.4	300	352	0.009	349.31	Single	137.28	4224	52.675	137.28	0.4372	4Φ 20
Support	2	27.7	300	352	0.037	340.02	Single	137.28	4224	234.21	234.21	0.7459	4Φ 20
Span	b/n2&3	43.2	300	352	0.058	332.91	Single	137.28	4224	373.07	373.07	1.1881	4Φ 20
Support	3	55.2	300	352	0.074	327.19	double	137.28	4224	485.04	485.04	1.5447	4Φ 20
Span	b/n3&4	26.7	300	352	0.036	340.47	Single	137.28	4224	225.46	225.46	0.718	4Φ 20
Support	4	29.6	300	352	0.040	339.16	double	137.28	4224	250.91	250.91	0.7991	4Φ 20
Span	b/n4&5	14.92	300	352	0.020	345.65	double	137.28	4224	124.1	137.28	0.4372	4Φ 21

Axis A	STORY4											
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	18.92	300	352	0.025	343.91	Single	137.28	4224	158.17	158.17	0.5037	4Φ 20
Span	b/n1&2	6.58	300	352	0.009	349.23	Single	137.28	4224	54.169	137.28	0.4372	4Φ 20
Support	2	24.35	300	352	0.033	341.51	Single	137.28	4224	204.99	204.99	0.6528	4Φ 20
Span	b/n2&3	40.35	300	352	0.054	334.24	Single	137.28	4224	347.07	347.07	1.1053	4Φ 20
Support	3	51.7	300	352	0.070	328.88	double	137.28	4224	451.95	451.95	1.4393	4Φ 20
Span	b/n3&4	26.08	300	352	0.035	340.74	Single	137.28	4224	220.05	220.05	0.7008	4Φ 20
Support	4	18.84	300	352	0.025	343.94	double	137.28	4224	157.48	157.48	0.5015	4Φ 20
Span	b/n4&5	12.94	300	352	0.017	346.51	double	137.28	4224	107.36	137.28	0.4372	4Φ 21

Axis A	STORY3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	12.13	300	352	0.016	346.86	Single	137.28	4224	100.54	137.28	0.4372	4Φ 20
Span	b/n1&2	6.66	300	352	0.009	349.19	Single	137.28	4224	54.833	137.28	0.4372	4Φ 20
Support	2	17.17	300	352	0.023	344.67	Single	137.28	4224	143.22	143.22	0.4561	4Φ 20
Span	b/n2&3	40.4	300	352	0.054	334.22	Single	137.28	4224	347.52	347.52	1.1068	4Φ 20
Support	3	52.52	300	352	0.071	328.48	double	137.28	4224	459.67	459.67	1.4639	4Φ 20
Span	b/n3&4	26.09	300	352	0.035	340.74	Single	137.28	4224	220.13	220.13	0.7011	4Φ 20
Support	4	19.99	300	352	0.027	343.44	double	137.28	4224	167.34	167.34	0.5329	4Φ 20
Span	b/n4&5	13.21	300	352	0.018	346.39	double	137.28	4224	109.64	137.28	0.4372	4Φ 21

Appendix 8B Reinforcement of a beam on axis B

Axis B	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	51.72	300	352	0.070	328.87	Single	137.28	4224	452.14	452.14	1.4399	4Φ 20
Span	b/n1&2	10.91	300	352	0.015	347.38	Single	137.28	4224	90.293	137.28	0.4372	4Φ 20
Support	2	0	300	352	0.000	352	Single	137.28	4224	0	137.28	0.4372	4Φ 20
Span	b/n2&3	59.86	300	352	0.081	324.9	Single	137.28	4224	529.69	529.69	1.6869	4Φ 20
Support	3	86.68	300	352	0.117	311.01	double	137.28	4224	801.28	801.28	2.5518	4Φ 20
Span	b/n1&2	40.15	300	352	0.054	334.34	Single	137.28	4224	345.25	345.25	1.0995	4Φ 20
Support	3	0	300	352	0.000	352	double	137.28	4224	0	137.28	0.4372	4Φ 20
Axis B	STORY 4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	67.53	300	352	0.091	321.06	Single	137.28	4224	604.7	604.7	1.9258	4Φ 20
Span	b/n1&2	10.61	300	352	0.014	347.51	Single	137.28	4224	87.777	137.28	0.4372	4Φ 20
Support	2	0	300	352	0.000	352	Single	137.28	4224	0	137.28	0.4372	4Φ 20
Span	b/n2&3	57.2	300	352	0.077	326.21	Single	137.28	4224	504.12	504.12	1.6055	4Φ 20
Support	3	85.4	300	352	0.115	311.7	double	137.28	4224	787.68	787.68	2.5085	4Φ 20
Span	b/n1&2	39.6	300	352	0.053	334.59	Single	137.28	4224	340.26	340.26	1.0836	4Φ 20
Support	3	0	300	352	0.000	352	double	137.28	4224	0	137.28	0.4372	4Φ 20
Axis B	STORY 3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	59.3	300	352	0.080	325.18	Single	137.28	4224	524.28	524.28	1.6697	4Φ 20
Span	b/n1&2	11.26	300	352	0.015	347.23	Single	137.28	4224	93.23	137.28	0.4372	4Φ 20
Support	2	0	300	352	0.000	352	Single	137.28	4224	0	137.28	0.4372	4Φ 20
Span	b/n2&3	57.22	300	352	0.077	326.2	Single	137.28	4224	504.31	504.31	1.6061	4Φ 20
Support	3	86.86	300	352	0.117	310.91	double	137.28	4224	803.19	803.19	2.5579	4Φ 20
Span	b/n1&2	39.87	300	352	0.054	334.47	Single	137.28	4224	342.71	342.71	1.0914	4Φ 20
Support	3	0	300	352	0.000	352	double	137.28	4224	0	137.28	0.4372	4Φ 20

Axis B	STORY 2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	52.33	300	352	0.070	328.57	Single	137.28	4224	457.88	457.88	1.4582	4Φ 20
Span	b/n1&2	11.47	300	352	0.015	347.14	Single	137.28	4224	94.993	137.28	0.4372	4Φ 20
Support	2	0	300	352	0.000	352	Single	137.28	4224	0	137.28	0.4372	4Φ 20
Span	b/n2&3	56.8	300	352	0.076	326.41	Single	137.28	4224	500.29	500.29	1.5933	4Φ 20
Support	3	88.4	300	352	0.119	310.07	double	137.28	4224	819.65	819.65	2.6104	4Φ 20
Span	b/n1&2	39.9	300	352	0.054	334.45	Single	137.28	4224	342.98	342.98	1.0923	4Φ 20
Support	3	0	300	352	0.000	352	double	137.28	4224	0	137.28	0.4372	4Φ 20

Axis B	STORY 1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	0	300	352	0.000	352	Single	137.28	4224	0	137.28	0.4372	4Φ 20
Span	b/n1&2	12.08	300	352	0.016	346.88	Single	137.28	4224	100.12	137.28	0.4372	4Φ 20
Support	2	40.6	300	352	0.055	334.13	Single	137.28	4224	349.34	349.34	1.1125	4Φ 20
Span	b/n2&3	56.86	300	352	0.076	326.38	Single	137.28	4224	500.87	500.87	1.5951	4Φ 20
Support	3	90.7	300	352	0.122	308.8	double	137.28	4224	844.43	844.43	2.6893	4Φ 20
Span	b/n1&2	41.37	300	352	0.056	333.77	Single	137.28	4224	356.35	356.35	1.1349	4Φ 20
Support	3	0	300	352	0.000	352	double	137.28	4224	0	137.28	0.4372	4Φ 20

Axis B	GROUND FLOOR												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	16.6	300	352	0.022	344.92	Single	137.28	4224	138.36	138.36	0.4406	4Φ 20
Span	b/n1&2	2.5	300	352	0.003	350.95	Single	137.28	4224	20.48	137.28	0.4372	4Φ 20
Support	2	6.87	300	352	0.009	349.11	Single	137.28	4224	56.576	137.28	0.4372	4Φ 20
Span	b/n2&3	6.86	300	352	0.009	349.11	Single	137.28	4224	56.493	137.28	0.4372	4Φ 20
Support	3	0.26	300	352	0.000	351.89	double	137.28	4224	2.1242	137.28	0.4372	4Φ 20
Span	b/n1&2	7.4	300	352	0.010	348.88	Single	137.28	4224	60.98	137.28	0.4372	4Φ 20
Support	3	0	300	352	0.000	352	double	137.28	4224	0	137.28	0.4372	4Φ 20

Appendix 8C Reinforcement of a beam on axis C

Axis C	TAKER											
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	5	0.786	300	352	0.001	351.67	Single	137.28	4224	6.4257	137.28	0.4372	4Φ 20
Span	b/n5&6	3.74	300	352	0.005	350.43	Single	137.28	4224	30.683	137.28	0.4372	4Φ 20
Support	6	3.2	300	352	0.004	350.66	Single	137.28	4224	26.236	137.28	0.4372	4Φ 20

Axis C	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	5	18.42	300	352	0.025	344.13	Single	137.28	4224	153.89	153.89	0.4901	4Φ 20
Span	b/n5&6	10.13	300	352	0.014	347.72	Single	137.28	4224	83.757	137.28	0.4372	4Φ 20
Support	6	1.79	300	352	0.002	351.25	Single	137.28	4224	14.651	137.28	0.4372	4Φ 20

Axis C	STORY4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	5	18.44	300	352	0.025	344.12	Single	137.28	4224	154.06	154.06	0.4906	4Φ 20
Span	b/n5&6	10.51	300	352	0.014	347.55	Single	137.28	4224	86.939	137.28	0.4372	4Φ 20
Support	6	1.77	300	352	0.002	351.26	Single	137.28	4224	14.487	137.28	0.4372	4Φ 20

Axis C	STORY3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	5	18.402	300	352	0.025	344.13	Single	137.28	4224	153.73	153.73	0.4896	4Φ 20
Span	b/n5&6	10.6	300	352	0.014	347.51	Single	137.28	4224	87.693	137.28	0.4372	4Φ 20
Support	6	1.7	300	352	0.002	351.29	Single	137.28	4224	13.913	137.28	0.4372	4Φ 20

Axis C	STORY2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	5	18.08	300	352	0.024	344.28	Single	137.28	4224	150.98	150.98	0.4808	4Φ 20
Span	b/n5&6	10.73	300	352	0.014	347.46	Single	137.28	4224	88.783	137.28	0.4372	4Φ 20
Support	6	1.8	300	352	0.002	351.25	Single	137.28	4224	14.733	137.28	0.4372	4Φ 20

Axis C	STORY1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	5	18.27	300	352	0.025	344.19	Single	137.28	4224	152.61	152.61	0.486	4Φ 20
Span	b/n5&6	10.76	300	352	0.014	347.44	Single	137.28	4224	89.035	137.28	0.4372	4Φ 20
Support	6	2.03	300	352	0.003	351.15	Single	137.28	4224	16.62	137.28	0.4372	4Φ 20

Axis C	GR												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	5	2.04	300	352	0.003	351.15	Single	137.28	4224	16.702	137.28	0.4372	4Φ 20
Span	b/n5&6	3.78	300	352	0.005	350.41	Single	137.28	4224	31.013	137.28	0.4372	4Φ 20
Support	6	0.44	300	352	0.001	351.82	Single	137.28	4224	3.5956	137.28	0.4372	4Φ 20

Appendix 8D Reinforcement of a beam on axis D

Axis D	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	15.236	300	352	0.020	345.51	Single	137.28	4224	126.78	137.28	0.4372	4Φ 20
Span	b/n1&2	4.744	300	352	0.006	350.01	Single	137.28	4224	38.967	137.28	0.4372	4Φ 20
Support	2	24.03	300	352	0.032	341.66	Single	137.28	4224	202.21	202.21	0.644	4Φ 20
Span	b/n2&3	36.92	300	352	0.050	335.83	Single	137.28	4224	316.06	316.06	1.0066	4Φ 20
Support	3	25.32	300	352	0.034	341.08	double	137.28	4224	213.42	213.42	0.6797	4Φ 20

Axis D	STOREY 4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	21.21	300	352	0.029	342.9	Single	137.28	4224	177.83	177.83	0.5663	4Φ 20
Span	b/n1&2	54.71	300	352	0.074	327.42	Single	137.28	4224	480.39	480.39	1.5299	4Φ 20
Support	2	46.63	300	352	0.063	331.3	Single	137.28	4224	404.65	404.65	1.2887	4Φ 20
Span	b/n2&3	35.221	300	352	0.047	336.61	Single	137.28	4224	300.82	300.82	0.958	4Φ 20
Support	3	12.25	300	352	0.016	346.8	double	137.28	4224	101.55	137.28	0.4372	4Φ 20

Axis D	STOREY 3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	12.236	300	352	0.016	346.81	Single	137.28	4224	101.43	137.28	0.4372	4Φ 20
Span	b/n1&2	5.468	300	352	0.007	349.7	Single	137.28	4224	44.954	137.28	0.4372	4Φ 20
Support	2	45.89	300	352	0.062	331.65	Single	137.28	4224	397.81	397.81	1.2669	4Φ 20
Span	b/n2&3	34.96	300	352	0.047	336.73	Single	137.28	4224	298.49	298.49	0.9506	4Φ 20
Support	3	0	300	352	0.000	352	double	137.28	4224	0	137.28	0.4372	4Φ 20

Axis D	STOREY 2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	7.25	300	352	0.010	348.94	Single	137.28	4224	59.733	137.28	0.4372	4Φ 20
Span	b/n1&2	5.556	300	352	0.007	349.66	Single	137.28	4224	45.682	137.28	0.4372	4Φ 20
Support	2	45.99	300	352	0.062	331.6	Single	137.28	4224	398.73	398.73	1.2698	4Φ 20
Span	b/n2&3	34.499	300	352	0.046	336.94	Single	137.28	4224	294.36	294.36	0.9375	4Φ 20
Support	3	15.26	300	352	0.021	345.5	double	137.28	4224	126.98	137.28	0.4372	4Φ 20

Axis D	STOREY 1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	1	4.256	300	352	0.006	350.21	Single	137.28	4224	34.938	137.28	0.4372	4Φ 20
Span	b/n1&2	5.398	300	352	0.007	349.73	Single	137.28	4224	44.374	137.28	0.4372	4Φ 20
Support	2	3.98	300	352	0.005	350.33	Single	137.28	4224	32.662	137.28	0.4372	4Φ 20
Span	b/n2&3	34.061	300	352	0.046	337.14	Single	137.28	4224	290.46	290.46	0.925	4Φ 20
Support	3	5.268	300	352	0.007	349.78	double	137.28	4224	43.299	137.28	0.4372	4Φ 20

Appendix 8E Reinforcement of a beam on axis E

Axis E	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	36.25	300	352	0.049	336.14	Single	137.28	4224	310.04	310.04	0.9874	4Φ 20
Span	b/n3&4	56.836	300	352	0.076	326.39	Single	137.28	4224	500.63	500.63	1.5944	4Φ 20
Support	4	0.928	300	352	0.001	351.61	Single	137.28	4224	7.5878	137.28	0.4372	4Φ 20
Span	b/n4&5	28.9	300	352	0.039	339.48	Single	137.28	4224	244.75	244.75	0.7795	4Φ 20
Support	5	78.25	300	352	0.105	315.52	double	137.28	4224	712.99	712.99	2.2707	4Φ 20

Axis E	STOREY 4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	25.26	300	352	0.034	341.11	Single	137.28	4224	212.9	212.9	0.678	4Φ 20
Span	b/n3&4	53.804	300	352	0.072	327.86	Single	137.28	4224	471.8	471.8	1.5025	4Φ 20
Support	4	53.914	300	352	0.073	327.81	Single	137.28	4224	472.84	472.84	1.5059	4Φ 20
Span	b/n4&5	78.25	300	352	0.105	315.52	Single	137.28	4224	712.99	712.99	2.2707	4Φ 20
Support	5	23.165	300	352	0.031	342.04	double	137.28	4224	194.71	194.71	0.6201	4Φ 20

Axis E	STOREY 3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	12.26	300	352	0.016	346.8	Single	137.28	4224	101.64	137.28	0.4372	4Φ 20
Span	b/n3&4	53.804	300	352	0.072	327.86	Single	137.28	4224	471.8	471.8	1.5025	4Φ 20
Support	4	8.8756	300	352	0.012	348.25	Single	137.28	4224	73.272	137.28	0.4372	4Φ 20
Span	b/n4&5	10.106	300	352	0.014	347.73	Single	137.28	4224	83.556	137.28	0.4372	4Φ 20
Support	5	46.26	300	352	0.062	331.47	double	137.28	4224	401.23	401.23	1.2778	4Φ 20

Axis E	STOREY 2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	26.263	300	352	0.035	340.66	Single	137.28	4224	221.64	221.64	0.7059	4Φ 20
Span	b/n3&4	53.329	300	352	0.072	328.09	Single	137.28	4224	467.3	467.3	1.4882	4Φ 20
Support	4	1.357	300	352	0.002	351.43	Single	137.28	4224	11.101	137.28	0.4372	4Φ 20
Span	b/n4&5	9.928	300	352	0.013	347.8	Single	137.28	4224	82.066	137.28	0.4372	4Φ 20
Support	5	14.236	300	352	0.019	345.95	double	137.28	4224	118.31	137.28	0.4372	4Φ 20

Axis E	STOREY 1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	39.56	300	352	0.053	334.61	Single	137.28	4224	339.9	339.9	1.0825	4Φ 20
Span	b/n3&4	53.598	300	352	0.072	327.96	Single	137.28	4224	469.85	469.85	1.4963	4Φ 20
Support	4	16.981	300	352	0.023	344.76	Single	137.28	4224	141.61	141.61	0.451	4Φ 20
Span	b/n4&5	46.419	300	352	0.062	331.4	Single	137.28	4224	402.7	402.7	1.2825	4Φ 20
Support	5	49.23	300	352	0.066	330.06	double	137.28	4224	428.81	428.81	1.3656	4Φ 20

Axis E	GROUND												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	5.895	300	352	0.008	349.52	Single	137.28	4224	48.489	137.28	0.4372	4Φ 20
Span	b/n3&4	5.2051	300	352	0.007	349.81	Single	137.28	4224	42.779	137.28	0.4372	4Φ 20
Support	4	5.895	300	352	0.008	349.52	Single	137.28	4224	48.489	137.28	0.4372	4Φ 20
Span	b/n4&5	5.0927	300	352	0.007	349.86	Single	137.28	4224	41.849	137.28	0.4372	4Φ 20
Support	5	6.89	300	352	0.009	349.1	double	137.28	4224	56.742	137.28	0.4372	4Φ 20

Appendix 8F Reinforcement of a beam on axis F

Axis F	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	44.126	300	352	0.059	332.48	Single	137.28	4224	381.56	381.56	1.2151	4Φ 20
Span	b/n3&4	44.979	300	352	0.061	332.08	Single	137.28	4224	389.4	389.4	1.2401	4Φ 20
Support	4	49.742	300	352	0.067	329.82	Single	137.28	4224	433.59	433.59	1.3809	4Φ 20
Span	b/n4&5	43.754	300	352	0.059	332.65	Single	137.28	4224	378.14	378.14	1.2043	4Φ 20
Support	5	48.235	300	352	0.065	330.54	double	137.28	4224	419.54	419.54	1.3361	4Φ 20

Axis F	STOREY 4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	39.756	300	352	0.053	334.52	Single	137.28	4224	341.68	341.68	1.0882	4Φ 20
Span	b/n3&4	39.761	300	352	0.053	334.52	Single	137.28	4224	341.72	341.72	1.0883	4Φ 20
Support	4	77.55	300	352	0.104	315.89	Single	137.28	4224	705.79	705.79	2.2477	4Φ 20
Span	b/n4&5	39.98	300	352	0.054	334.42	Single	137.28	4224	343.7	343.7	1.0946	4Φ 20
Support	5	40.256	300	352	0.054	334.29	double	137.28	4224	346.21	346.21	1.1026	4Φ 20

Axis F	STOREY 3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	39.265	300	352	0.053	334.75	Single	137.28	4224	337.23	337.23	1.074	4Φ 20
Span	b/n3&4	40.501	300	352	0.054	334.17	Single	137.28	4224	348.44	348.44	1.1097	4Φ 20
Support	4	45.955	300	352	0.062	331.62	Single	137.28	4224	398.41	398.41	1.2688	4Φ 20
Span	b/n4&5	40.388	300	352	0.054	334.23	Single	137.28	4224	347.41	347.41	1.1064	4Φ 20
Support	5	46.235	300	352	0.062	331.49	double	137.28	4224	401	401	1.2771	4Φ 20

Axis F	STOREY 2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	39.235	300	352	0.053	334.76	Single	137.28	4224	336.95	336.95	1.0731	4Φ 20
Span	b/n3&4	40.564	300	352	0.055	334.14	Single	137.28	4224	349.01	349.01	1.1115	4Φ 20
Support	4	49.53	300	352	0.067	329.92	Single	137.28	4224	431.61	431.61	1.3746	4Φ 20
Span	b/n4&5	40.388	300	352	0.054	334.23	Single	137.28	4224	347.41	347.41	1.1064	4Φ 20
Support	5	45.236	300	352	0.061	331.96	double	137.28	4224	391.77	391.77	1.2477	4Φ 20

Axis F	STOREY 1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	36.256	300	352	0.049	336.14	Single	137.28	4224	310.1	310.1	0.9876	4Φ 20
Span	b/n3&4	41.903	300	352	0.056	333.52	Single	137.28	4224	361.2	361.2	1.1503	4Φ 20
Support	4	55.598	300	352	0.075	326.99	Single	137.28	4224	488.83	488.83	1.5568	4Φ 20
Span	b/n4&5	40.811	300	352	0.055	334.03	Single	137.28	4224	351.25	351.25	1.1186	4Φ 20
Support	5	45.237	300	352	0.061	331.96	double	137.28	4224	391.78	391.78	1.2477	4Φ 20

Axis F	GROUND												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	12.023	300	352	0.016	346.9	Single	137.28	4224	99.643	137.28	0.4372	4Φ 20
Span	b/n3&4	4.4852	300	352	0.006	350.12	Single	137.28	4224	36.83	137.28	0.4372	4Φ 20
Support	4	8.2356	300	352	0.011	348.52	Single	137.28	4224	67.935	137.28	0.4372	4Φ 20
Span	b/n4&5	4.0881	300	352	0.005	350.28	Single	137.28	4224	33.553	137.28	0.4372	4Φ 20
Support	5	9.2356	300	352	0.012	348.1	double	137.28	4224	76.278	137.28	0.4372	4Φ 20

Appendix 8G Reinforcement of a beam on axis G

Axis G	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	10.144	300	352	0.014	347.71	Single	137.28	4224	83.875	137.28	0.4372	4Φ 20
Span	b/n3&4	21.776	300	352	0.029	342.65	Single	137.28	4224	182.71	182.71	0.5819	4Φ 20
Support	4	13.525	300	352	0.018	346.25	Single	137.28	4224	112.3	137.28	0.4372	4Φ 20
Span	b/n4&5	21.14	300	352	0.028	342.93	Single	137.28	4224	177.23	177.23	0.5644	4Φ 20
Support	5	25.369	300	352	0.034	341.06	double	137.28	4224	213.85	213.85	0.681	4Φ 20

Axis G	STOREY 4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	18.72	300	352	0.025	344	Single	137.28	4224	156.45	156.45	0.4983	4Φ 20
Span	b/n3&4	18.759	300	352	0.025	343.98	Single	137.28	4224	156.79	156.79	0.4993	4Φ 20
Support	4	26.293	300	352	0.035	340.65	Single	137.28	4224	221.91	221.91	0.7067	4Φ 20
Span	b/n4&5	19.694	300	352	0.026	343.57	Single	137.28	4224	164.8	164.8	0.5248	4Φ 20
Support	5	21.236	300	352	0.029	342.89	double	137.28	4224	178.05	178.05	0.567	4Φ 20

Axis G	STOREY 3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	17.149	300	352	0.023	344.68	Single	137.28	4224	143.04	143.04	0.4555	4Φ 20
Span	b/n3&4	19.07	300	352	0.026	343.84	Single	137.28	4224	159.45	159.45	0.5078	4Φ 20
Support	4	49.215	300	352	0.066	330.07	Single	137.28	4224	428.67	428.67	1.3652	4Φ 20
Span	b/n4&5	19.743	300	352	0.027	343.55	Single	137.28	4224	165.22	165.22	0.5262	4Φ 20
Support	5	20.125	300	352	0.027	343.38	double	137.28	4224	168.5	168.5	0.5366	4Φ 20

Axis G	STOREY 2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	16.508	300	352	0.022	344.96	Single	137.28	4224	137.58	137.58	0.4381	4Φ 20
Span	b/n3&4	18.89	300	352	0.025	343.92	Single	137.28	4224	157.91	157.91	0.5029	4Φ 20
Support	4	62.014	300	352	0.083	323.83	Single	137.28	4224	550.56	550.56	1.7534	4Φ 20
Span	b/n4&5	19.463	300	352	0.026	343.67	Single	137.28	4224	162.81	162.81	0.5185	4Φ 20
Support	5	22.315	300	352	0.030	342.41	double	137.28	4224	187.36	187.36	0.5967	4Φ 20

Axis G	STOREY 1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	13.804	300	352	0.019	346.13	Single	137.28	4224	114.66	137.28	0.4372	4Φ 20
Span	b/n3&4	19.3	300	352	0.026	343.74	Single	137.28	4224	161.42	161.42	0.5141	4Φ 20
Support	4	9.2818	300	352	0.012	348.08	Single	137.28	4224	76.664	137.28	0.4372	4Φ 20
Span	b/n4&5	17.08	300	352	0.023	344.71	Single	137.28	4224	142.45	142.45	0.4537	4Φ 20
Support	5	19.235	300	352	0.026	343.77	double	137.28	4224	160.86	160.86	0.5123	4Φ 20

Axis G	GROUND												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	3	5.4124	300	352	0.007	349.72	Single	137.28	4224	44.494	137.28	0.4372	4Φ 20
Span	b/n3&4	3.5669	300	352	0.005	350.5	Single	137.28	4224	29.257	137.28	0.4372	4Φ 20
Support	4	4.265	300	352	0.006	350.21	Single	137.28	4224	35.013	137.28	0.4372	4Φ 20
Span	b/n4&5	3.3762	300	352	0.005	350.58	Single	137.28	4224	27.687	137.28	0.4372	4Φ 20
Support	5	2.465	300	352	0.003	350.97	double	137.28	4224	20.192	137.28	0.4372	4Φ 20

Appendix 8H Reinforcement of a beam on axis 1

Axis 1	TERACE											
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	19.35	300	352	0.026	343.72	Single	137.28	4224	161.85	161.85	0.5154	4Φ 20
Span	b/nA&B	25.12	300	352	0.034	341.17	Single	137.28	4224	211.68	211.68	0.6741	4Φ 20
Support	B	4.77	300	352	0.006	350	Single	137.28	4224	39.182	137.28	0.4372	4Φ 20
Span	b/nB&C	10.81	300	352	0.015	347.42	Single	137.28	4224	89.454	137.28	0.4372	4Φ 20
Support	C	1.98	300	352	0.003	351.17	double	137.28	4224	16.21	137.28	0.4372	4Φ 20

Axis 1	STORY4											
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	26.86	300	352	0.036	340.39	Single	137.28	4224	226.86	226.86	0.7225	4Φ 20
Span	b/nA&B	22.65	300	352	0.030	342.27	Single	137.28	4224	190.26	190.26	0.6059	4Φ 20
Support	B	10.17	300	352	0.014	347.7	Single	137.28	4224	84.091	137.28	0.4372	4Φ 20
Span	b/nB&C	10.02	300	352	0.013	347.76	Single	137.28	4224	82.836	137.28	0.4372	4Φ 20
Support	C	4.49	300	352	0.006	350.11	double	137.28	4224	36.87	137.28	0.4372	4Φ 20

Axis 1	STORY3											
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	25.18	300	352	0.034	341.14	Single	137.28	4224	212.2	212.2	0.6758	4Φ 20
Span	b/nA&B	23.11	300	352	0.031	342.06	Single	137.28	4224	194.23	194.23	0.6186	4Φ 20
Support	B	10.46	300	352	0.014	347.57	Single	137.28	4224	86.52	137.28	0.4372	4Φ 20
Span	b/nB&C	10.1	300	352	0.014	347.73	Single	137.28	4224	83.505	137.28	0.4372	4Φ 20
Support	C	4.62	300	352	0.006	350.06	double	137.28	4224	37.943	137.28	0.4372	4Φ 20

Axis 1	STORY2												
Type	Loc.	Momen t(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,pro v(mm^2)	No.of bar	Remark
Support	A	25.67	300	352	0.035	340.93	Single	137.28	4224	216.47	216.47	0.6894	4Φ 20
Span	b/nA&B	22.85	300	352	0.031	342.18	Single	137.28	4224	191.98	191.98	0.6114	4Φ 20
Support	B	10.87	300	352	0.015	347.4	Single	137.28	4224	89.957	137.28	0.4372	4Φ 20
Span	b/nB&C	9.94	300	352	0.013	347.8	Single	137.28	4224	82.166	137.28	0.4372	4Φ 20
Support	C	5.51	300	352	0.007	349.68	double	137.28	4224	45.301	137.28	0.4372	4Φ 20

Axis 1	STORY1												
Type	Loc.	Momen t(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,pro v(mm^2)	No.of bar	Remark
Support	A	24.14	300	352	0.032	341.61	Single	137.28	4224	203.16	203.16	0.647	4Φ 20
Span	b/nA&B	23.25	300	352	0.031	342	Single	137.28	4224	195.45	195.45	0.6224	4Φ 20
Support	B	11.03	300	352	0.015	347.33	Single	137.28	4224	91.299	137.28	0.4372	4Φ 20
Span	b/nB&C	10.03	300	352	0.013	347.76	Single	137.28	4224	82.92	137.28	0.4372	4Φ 20
Support	C	5.62	300	352	0.008	349.64	double	137.28	4224	46.212	137.28	0.4372	4Φ 20

Axis 1	GR												
Type	Loc.	Momen t(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,pro v(mm^2)	No.of bar	Remark
Support	A	6.11	300	352	0.008	349.43	Single	137.28	4224	50.271	137.28	0.4372	4Φ 20
Span	b/nA&B	4.02	300	352	0.005	350.31	Single	137.28	4224	32.992	137.28	0.4372	4Φ 20
Support	B	1.37	300	352	0.002	351.43	Single	137.28	4224	11.208	137.28	0.4372	4Φ 20
Span	b/nB&C	2.11	300	352	0.003	351.12	Single	137.28	4224	17.277	137.28	0.4372	4Φ 20
Support	C	1.42	300	352	0.002	351.41	double	137.28	4224	11.617	137.28	0.4372	4Φ 20

Appendix 8I Reinforcement of a beam on axis 2

Axis 2	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm²)	As,max (mm²)	As,cal(mm²)	As,prov(mm²)	No.of bar	Remark
Support	A	51.4	300	352	0.069	329.02	Single	137.28	4224	449.13	449.13	1.4303	4Φ 20
Span	b/nA&B	53.36	300	352	0.072	328.08	Single	137.28	4224	467.6	467.6	1.4892	4Φ 20
Support	B	28.71	300	352	0.039	339.56	Single	137.28	4224	243.08	243.08	0.7741	4Φ 20
Span	b/nB&C	22.06	300	352	0.030	342.53	Single	137.28	4224	185.16	185.16	0.5897	4Φ 20
Support	C	23.49	300	352	0.032	341.89	double	137.28	4224	197.53	197.53	0.6291	4Φ 20

Axis 2	STORY4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm²)	As,max (mm²)	As,cal(mm²)	As,prov(mm²)	No.of bar	Remark
Support	A	73.88	300	352	0.099	317.81	Single	137.28	4224	668.34	668.34	2.1285	4Φ 20
Span	b/nA&B	48.37	300	352	0.065	330.47	Single	137.28	4224	420.8	420.8	1.3401	4Φ 20
Support	B	51.2	300	352	0.069	329.12	Single	137.28	4224	447.25	447.25	1.4244	4Φ 20
Span	b/nB&C	20.85	300	352	0.028	343.06	Single	137.28	4224	174.73	174.73	0.5565	4Φ 20
Support	C	38.23	300	352	0.051	335.23	double	137.28	4224	327.87	327.87	1.0442	4Φ 20

Axis 2	STORY3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm²)	As,max (mm²)	As,cal(mm²)	As,prov(mm²)	No.of bar	Remark
Support	A	68.11	300	352	0.092	320.77	Single	137.28	4224	610.45	610.45	1.9441	4Φ 20
Span	b/nA&B	49.16	300	352	0.066	330.1	Single	137.28	4224	428.16	428.16	1.3636	4Φ 20
Support	B	5366	300	352	7.218	#####	Single	137.28	4224	#####	#####	#####	4Φ 20
Span	b/nB&C	20.39	300	352	0.027	343.26	Single	137.28	4224	170.77	170.77	0.5439	4Φ 20
Support	C	34.01	300	352	0.046	337.16	double	137.28	4224	290	290	0.9236	4Φ 20

Axis 2	STORY2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	67.82	300	352	0.091	320.92	Single	137.28	4224	607.57	607.57	1.9349	4Φ 20
Span	b/nA&E	48.71	300	352	0.066	330.31	Single	137.28	4224	423.96	423.96	1.3502	4Φ 20
Support	B	55.84	300	352	0.075	326.87	Single	137.28	4224	491.13	491.13	1.5641	4Φ 20
Span	b/nB&C	20.29	300	352	0.027	343.31	Single	137.28	4224	169.92	169.92	0.5411	4Φ 20
Support	C	31.83	300	352	0.043	338.16	double	137.28	4224	270.62	270.62	0.8618	4Φ 20

Axis 2	STORY1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	60.37	300	352	0.081	324.65	Single	137.28	4224	534.61	534.61	1.7026	4Φ 20
Span	b/nA&E	49.55	300	352	0.067	329.91	Single	137.28	4224	431.8	431.8	1.3752	4Φ 20
Support	B	30.09	300	352	0.040	338.94	Single	137.28	4224	255.23	255.23	0.8128	4Φ 20
Span	b/nB&C	20.93	300	352	0.028	343.03	Single	137.28	4224	175.42	175.42	0.5587	4Φ 20
Support	C	24.4	300	352	0.033	341.49	double	137.28	4224	205.42	205.42	0.6542	4Φ 20

Axis 2	GR												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	11.89	300	352	0.016	346.96	Single	137.28	4224	98.523	137.28	0.4372	4Φ 20
Span	b/nA&E	4.01	300	352	0.005	350.32	Single	137.28	4224	32.909	137.28	0.4372	4Φ 20
Support	B	1.97	300	352	0.003	351.17	Single	137.28	4224	16.128	137.28	0.4372	4Φ 20
Span	b/nB&C	5.007	300	352	0.007	349.9	Single	137.28	4224	41.141	137.28	0.4372	4Φ 20
Support	C	9.67	300	352	0.013	347.91	double	137.28	4224	79.908	137.28	0.4372	4Φ 20

Appendix 8J Reinforcement of a beam on axis 3

Axis 3	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	61.56	300	352	0.083	324.06	Single	137.28	4224	546.14	546.14	1.7393	4Φ 20
Span	b/nA&B	63.37	300	352	0.085	323.16	Single	137.28	4224	563.77	563.77	1.7954	4Φ 20
Support	B	58.58	300	352	0.079	325.53	Single	137.28	4224	517.35	517.35	1.6476	4Φ 20
Span	b/nB&C	20.75	300	352	0.028	343.1	Single	137.28	4224	173.87	173.87	0.5537	4Φ 20
Support	C	56.05	300	352	0.075	326.77	double	137.28	4224	493.13	493.13	1.5705	4Φ 20
Span	b/nC&D	43.29	300	352	0.058	332.87	Single	137.28	4224	373.89	373.89	1.1907	4Φ 20
Support	D	20.28	300	352	0.027	343.31	double	137.28	4224	169.83	169.83	0.5409	4Φ 20
Span	b/nD&E	7.41	300	352	0.010	348.88	double	137.28	4224	61.063	137.28	0.4372	4Φ 20
Support	E	9.06	300	352	0.012	348.17	double	137.28	4224	74.811	137.28	0.4372	4Φ 20
Axis 3	STORY4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	88.16	300	352	0.119	310.2	Single	137.28	4224	817.08	817.08	2.6022	4Φ 20
Span	b/nA&B	57.35	300	352	0.077	326.14	Single	137.28	4224	505.55	505.55	1.61	4Φ 20
Support	B	61.17	300	352	0.082	324.25	Single	137.28	4224	542.36	542.36	1.7273	4Φ 20
Span	b/nB&C	23.17	300	352	0.031	342.04	Single	137.28	4224	194.75	194.75	0.6202	4Φ 20
Support	C	48.46	300	352	0.065	330.43	double	137.28	4224	421.64	421.64	1.3428	4Φ 20
Span	b/nC&D	39.99	300	352	0.054	334.41	Single	137.28	4224	343.8	343.8	1.0949	4Φ 20
Support	D	51.66	300	352	0.069	328.9	double	137.28	4224	451.57	451.57	1.4381	4Φ 20
Span	b/nD&E	8.44	300	352	0.011	348.44	double	137.28	4224	69.639	137.28	0.4372	4Φ 20
Span	E	17.59	300	352	0.024	344.49	Span	137.28	4224	146.8	146.8	0.4675	4Φ 20

Axis 3	STORY3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	81.01	300	352	0.109	314.06	Single	137.28	4224	741.58	741.58	2.3617	4Φ 20
Span	b/nA&B	58.06	300	352	0.078	325.79	Single	137.28	4224	512.36	512.36	1.6317	4Φ 20
Support	B	62.7	300	352	0.084	323.49	Single	137.28	4224	557.23	557.23	1.7746	4Φ 20
Span	b/nB&C	22.11	300	352	0.030	342.51	Single	137.28	4224	185.59	185.59	0.5911	4Φ 20
Support	C	44.2	300	352	0.059	332.44	double	137.28	4224	382.24	382.24	1.2173	4Φ 20
Span	b/nC&D	40.27	300	352	0.054	334.28	Single	137.28	4224	346.34	346.34	1.103	4Φ 20
Support	D	51.37	300	352	0.069	329.04	double	137.28	4224	448.85	448.85	1.4294	4Φ 20
Span	b/nD&E	8.16	300	352	0.011	348.56	double	137.28	4224	67.305	137.28	0.4372	4Φ 20
Span	E	15.44	300	352	0.021	345.43	Span	137.28	4224	128.51	137.28	0.4372	4Φ 20

Axis 3	STORY2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	80.2	300	352	0.108	314.49	Single	137.28	4224	733.16	733.16	2.3349	4Φ 20
Span	b/nA&E	57.24	300	352	0.077	326.19	Single	137.28	4224	504.5	504.5	1.6067	4Φ 20
Support	B	65.01	300	352	0.087	322.34	Single	137.28	4224	579.84	579.84	1.8466	4Φ 20
Span	b/nB&C	21.54	300	352	0.029	342.76	Single	137.28	4224	180.67	180.67	0.5754	4Φ 20
Support	C	40.58	300	352	0.055	334.14	double	137.28	4224	349.16	349.16	1.112	4Φ 20
Span	b/nC&D	39.88	300	352	0.054	334.46	Single	137.28	4224	342.8	342.8	1.0917	4Φ 20
Support	D	51.86	300	352	0.070	328.8	double	137.28	4224	453.45	453.45	1.4441	4Φ 20
Span	b/nD&E	8.17	300	352	0.011	348.55	double	137.28	4224	67.389	137.28	0.4372	4Φ 20
Span	E	13.84	300	352	0.019	346.12	Span	137.28	4224	114.96	137.28	0.4372	4Φ 20

Axis 3	STORY1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	70.99	300	352	0.095	319.3	Single	137.28	4224	639.19	639.19	2.0357	4Φ 20
Span	b/nA&E	57.94	300	352	0.078	325.85	Single	137.28	4224	511.21	511.21	1.6281	4Φ 20
Support	B	62.78	300	352	0.084	323.45	Single	137.28	4224	558.01	558.01	1.7771	4Φ 20
Span	b/nB&C	21.27	300	352	0.029	342.88	Single	137.28	4224	178.35	178.35	0.568	4Φ 20
Support	C	32.88	300	352	0.044	337.68	double	137.28	4224	279.94	279.94	0.8915	4Φ 20
Span	b/nC&D	40.77	300	352	0.055	334.05	Single	137.28	4224	350.88	350.88	1.1175	4Φ 20
Support	D	16.83	300	352	0.023	344.82	double	137.28	4224	140.32	140.32	0.4469	4Φ 20
Span	b/nD&E	7.71	300	352	0.010	348.75	double	137.28	4224	63.559	137.28	0.4372	4Φ 20
Span	E	10.39	300	352	0.014	347.6	Span	137.28	4224	85.934	137.28	0.4372	4Φ 20

Axis 3	GR												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	13.4	300	352	0.018	346.31	Single	137.28	4224	111.24	137.28	0.4372	4Φ 20
Span	b/nA&E	4.03	300	352	0.005	350.31	Single	137.28	4224	33.074	137.28	0.4372	4Φ 20
Support	B	0.94	300	352	0.001	351.61	Single	137.28	4224	7.6861	137.28	0.4372	4Φ 20
Span	b/nB&C	6.89	300	352	0.009	349.1	Single	137.28	4224	56.742	137.28	0.4372	4Φ 20
Support	C	10.24	300	352	0.014	347.67	double	137.28	4224	84.678	137.28	0.4372	4Φ 20
Span	b/nC&D	6.07	300	352	0.008	349.45	Single	137.28	4224	49.939	137.28	0.4372	4Φ 20
Support	D	12.5	300	352	0.017	346.7	double	137.28	4224	103.66	137.28	0.4372	4Φ 20
Span	b/nD&E	2.75	300	352	0.004	350.85	double	137.28	4224	22.534	137.28	0.4372	4Φ 20
Support	E	5.59	300	352	0.008	349.65	Span	137.28	4224	45.964	137.28	0.4372	4Φ 20

Appendix 8K Reinforcement of a beam on axis 4

Axis 4	TERACE											
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	D	23.034	300	352	0.031	342.1	Single	137.28	4224	193.58	193.58	0.6165	4Φ 20
Span	b/nD&E	67.69	300	352	0.091	320.98	Single	137.28	4224	606.28	606.28	1.9308	4Φ 20
Support	E	17.638	300	352	0.024	344.47	Single	137.28	4224	147.21	147.21	0.4688	4Φ 20
Span	b/nE&F	15.446	300	352	0.021	345.42	Single	137.28	4224	128.56	137.28	0.4372	4Φ 20
Support	F	18.884	300	352	0.025	343.92	double	137.28	4224	157.86	157.86	0.5027	4Φ 20

Axis 4	SROREY 4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	D	23.237	300	352	0.031	342.01	Single	137.28	4224	195.34	195.34	0.6221	4Φ 20
Span	b/nD&E	64.686	300	352	0.087	322.5	Single	137.28	4224	576.65	576.65	1.8365	4Φ 20
Support	E	89.363	300	352	0.120	309.54	Single	137.28	4224	830	830	2.6433	4Φ 20
Span	b/nE&F	16.399	300	352	0.022	345.01	Single	137.28	4224	136.65	137.28	0.4372	4Φ 20
Support	F	65.968	300	352	0.089	321.85	double	137.28	4224	589.26	589.26	1.8766	4Φ 20

Axis 4	SROREY 3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	D	23.15	300	352	0.031	342.05	Single	137.28	4224	194.58	194.58	0.6197	4Φ 20
Span	b/nD&E	64.853	300	352	0.087	322.41	Single	137.28	4224	578.29	578.29	1.8417	4Φ 20
Support	E	89.363	300	352	0.120	309.54	Single	137.28	4224	830	830	2.6433	4Φ 20
Span	b/nE&F	16.167	300	352	0.022	345.11	Single	137.28	4224	134.68	137.28	0.4372	4Φ 20
Support	F	49.252	300	352	0.066	330.05	double	137.28	4224	429.01	429.01	1.3663	4Φ 20

Axis 4	SROREY 2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	D	23	300	352	0.031	342.11	Single	137.28	4224	193.28	193.28	0.6155	4Φ 20
Span	b/nD&E	64.854	300	352	0.087	322.41	Single	137.28	4224	578.3	578.3	1.8417	4Φ 20
Support	E	89.459	300	352	0.120	309.48	Single	137.28	4224	831.03	831.03	2.6466	4Φ 20
Span	b/nE&F	16.399	300	352	0.022	345.01	Single	137.28	4224	136.65	137.28	0.4372	4Φ 20
Support	F	65.968	300	352	0.089	321.85	double	137.28	4224	589.26	589.26	1.8766	4Φ 20

Axis 4	SROREY 1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	D	24.6	300	352	0.033	341.4	Single	137.28	4224	207.16	207.16	0.6597	4Φ 20
Span	b/nD&E	64.157	300	352	0.086	322.76	Single	137.28	4224	571.47	571.47	1.82	4Φ 20
Support	E	92.165	300	352	0.124	307.99	Single	137.28	4224	860.33	860.33	2.7399	4Φ 20
Span	b/nE&F	15.272	300	352	0.021	345.5	Single	137.28	4224	127.08	137.28	0.4372	4Φ 20
Support	F	39.47	300	352	0.053	334.65	double	137.28	4224	339.09	339.09	1.0799	4Φ 20

Axis 4	GROUND												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	D	2.2013	300	352	0.003	351.08	Single	137.28	4224	18.026	137.28	0.4372	4Φ 20
Span	b/nD&E	10.142	300	352	0.014	347.71	Single	137.28	4224	83.855	137.28	0.4372	4Φ 20
Support	E	11.571	300	352	0.016	347.1	Single	137.28	4224	95.837	137.28	0.4372	4Φ 20
Span	b/nE&F	9.245	300	352	0.012	348.09	Single	137.28	4224	76.356	137.28	0.4372	4Φ 20
Support	F	16.267	300	352	0.022	345.07	double	137.28	4224	135.53	137.28	0.4372	4Φ 20

Appendix 8L Reinforcement of a beam on axis 5

Axis 5	TANKER												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	10.186	300	352	0.014	347.69	Single	137.28	4224	84.223	137.28	0.4372	4Φ 20
Span	b/nC&D	0.5453	300	352	0.001	351.77	Single	137.28	4224	4.4566	137.28	0.4372	4Φ 20
Support	D	1.7817	300	352	0.002	351.25	Single	137.28	4224	14.583	137.28	0.4372	4Φ 20
Span	b/nD&E	14.238	300	352	0.019	345.95	Single	137.28	4224	118.32	137.28	0.4372	4Φ 20
Support	E	0.7223	300	352	0.001	351.7	double	137.28	4224	5.9045	137.28	0.4372	4Φ 20

Axis 5	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	59.036	300	352	0.079	325.31	Single	137.28	4224	521.74	521.74	1.6616	4Φ 20
Span	b/nC&D	61.924	300	352	0.083	323.88	Single	137.28	4224	549.68	549.68	1.7506	4Φ 20
Support	D	100.91	300	352	0.136	303.02	Single	137.28	4224	957.41	957.41	3.0491	4Φ 20
Span	b/nD&E	22	300	352	0.030	342.55	Single	137.28	4224	184.64	184.64	0.588	4Φ 20
Support	E	0.6707	300	352	0.001	351.72	double	137.28	4224	5.4823	137.28	0.4372	4Φ 20

Axis 5	STOREY 4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	79.113	300	352	0.106	315.07	Single	137.28	4224	721.9	721.9	2.299	4Φ 20
Span	b/nC&D	57.023	300	352	0.077	326.3	Single	137.28	4224	502.43	502.43	1.6001	4Φ 20
Support	D	101.85	300	352	0.137	302.47	Single	137.28	4224	968.09	968.09	3.0831	4Φ 20
Span	b/nD&E	20.074	300	352	0.027	343.4	Single	137.28	4224	168.06	168.06	0.5352	4Φ 20
Support	E	15.67	300	352	0.021	345.33	double	137.28	4224	130.46	137.28	0.4372	4Φ 20

Axis 5	STOREY 3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	79.624	300	352	0.107	314.8	Single	137.28	4224	727.18	727.18	2.3159	4Φ 20
Span	b/nC&D	57.87	300	352	0.078	325.88	Single	137.28	4224	510.53	510.53	1.6259	4Φ 20
Support	D	103.43	300	352	0.139	301.55	Single	137.28	4224	986.11	986.11	3.1405	4Φ 20
Span	b/nD&E	19.01	300	352	0.026	343.87	Single	137.28	4224	158.94	158.94	0.5062	4Φ 20
Support	E	4.09	300	352	0.006	350.28	double	137.28	4224	33.569	137.28	0.4372	4Φ 20

Axis 5	STOREY 2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	81.407	300	352	0.110	313.85	Single	137.28	4224	745.72	745.72	2.3749	4Φ 20
Span	b/nC&D	57.87	300	352	0.078	325.88	Single	137.28	4224	510.53	510.53	1.6259	4Φ 20
Support	D	102.82	300	352	0.138	301.91	Single	137.28	4224	979.15	979.15	3.1183	4Φ 20
Span	b/nD&E	3.58	300	352	0.005	350.5	Single	137.28	4224	29.365	137.28	0.4372	4Φ 20
Support	E	19.26	300	352	0.026	343.76	double	137.28	4224	161.08	161.08	0.513	4Φ 20

Axis 5	STOREY 1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	74.693	300	352	0.100	317.39	Single	137.28	4224	676.59	676.59	2.1548	4Φ 20
Span	b/nC&D	58.277	300	352	0.078	325.68	Single	137.28	4224	514.44	514.44	1.6383	4Φ 20
Support	D	104.26	300	352	0.140	301.07	Single	137.28	4224	995.56	995.56	3.1706	4Φ 20
Span	b/nD&E	1.1261	300	352	0.002	351.53	Single	137.28	4224	9.2098	137.28	0.4372	4Φ 20
Support	E	1.888	300	352	0.003	351.21	double	137.28	4224	15.455	137.28	0.4372	4Φ 20

Axis 5	GROUND												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	12.84	300	352	0.017	346.55	Single	137.28	4224	106.52	137.28	0.4372	4Φ 20
Span	b/nC&D	5.9441	300	352	0.008	349.5	Single	137.28	4224	48.896	137.28	0.4372	4Φ 20
Support	D	104.26	300	352	0.140	301.07	Single	137.28	4224	995.56	995.56	3.1706	4Φ 20
Span	b/nD&E	1.1261	300	352	0.002	351.53	Single	137.28	4224	9.2098	137.28	0.4372	4Φ 20
Support	E	1.888	300	352	0.003	351.21	double	137.28	4224	15.455	137.28	0.4372	4Φ 20

Axis 5	STOREY 2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	81.407	300	352	0.110	313.85	Single	137.28	4224	745.72	745.72	2.3749	4Φ 20
Span	b/nC&D	57.87	300	352	0.078	325.88	Single	137.28	4224	510.53	510.53	1.6259	4Φ 20
Support	D	102.82	300	352	0.138	301.91	Single	137.28	4224	979.15	979.15	3.1183	4Φ 20
Span	b/nD&E	3.58	300	352	0.005	350.5	Single	137.28	4224	29.365	137.28	0.4372	4Φ 20
Support	E	19.26	300	352	0.026	343.76	double	137.28	4224	161.08	161.08	0.513	4Φ 20

Axis 5	STOREY 1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	74.693	300	352	0.100	317.39	Single	137.28	4224	676.59	676.59	2.1548	4Φ 20
Span	b/nC&D	58.277	300	352	0.078	325.68	Single	137.28	4224	514.44	514.44	1.6383	4Φ 20
Support	D	104.26	300	352	0.140	301.07	Single	137.28	4224	995.56	995.56	3.1706	4Φ 20
Span	b/nD&E	1.1261	300	352	0.002	351.53	Single	137.28	4224	9.2098	137.28	0.4372	4Φ 20
Support	E	1.888	300	352	0.003	351.21	double	137.28	4224	15.455	137.28	0.4372	4Φ 20

Axis 5	GROUND												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	C	12.84	300	352	0.017	346.55	Single	137.28	4224	106.52	137.28	0.4372	4Φ 20
Span	b/nC&D	5.9441	300	352	0.008	349.5	Single	137.28	4224	48.896	137.28	0.4372	4Φ 20
Support	D	104.26	300	352	0.140	301.07	Single	137.28	4224	995.56	995.56	3.1706	4Φ 20
Span	b/nD&E	1.1261	300	352	0.002	351.53	Single	137.28	4224	9.2098	137.28	0.4372	4Φ 20
Support	E	1.888	300	352	0.003	351.21	double	137.28	4224	15.455	137.28	0.4372	4Φ 20

Appendix 8M Reinforcement of a beam on axis 6

Axis 6	TERACE												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	38.09	300	352	0.051	335.29	Single	137.28	4224	326.6	326.6	1.0401	4Φ 20
Span	b/nA&B	38.67	300	352	0.052	335.02	Single	137.28	4224	331.84	331.84	1.0568	4Φ 20
Support	B	51.023	300	352	0.069	329.2	Single	137.28	4224	445.59	445.59	1.4191	4Φ 20
Span	b/nB&C	STAIR	300	352	#####	#####	Single	137.28	4224	#####	#####	#####	4Φ 20
Support	C	64.1	300	352	0.086	322.79	double	137.28	4224	570.91	570.91	1.8182	4Φ 20
Span	b/nC&D	43.19	300	352	0.058	332.92	Single	137.28	4224	372.97	372.97	1.1878	4Φ 20
Support	D	19.6	300	352	0.026	343.61	double	137.28	4224	163.99	163.99	0.5223	4Φ 20
Span	b/nD&E	8.23	300	352	0.011	348.53	double	137.28	4224	67.889	137.28	0.4372	4Φ 20
Support	E	10.5	300	352	0.014	347.56	Span	137.28	4224	86.855	137.28	0.4372	4Φ 20

Axis 6	STROY4												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	38.28	300	352	0.051	335.2	Single	137.28	4224	328.32	328.32	1.0456	4Φ 20
Span	b/nA&B	34.31	300	352	0.046	337.03	Single	137.28	4224	292.68	292.68	0.9321	4Φ 20
Support	B	11.66	300	352	0.016	347.06	Single	137.28	4224	96.589	137.28	0.4372	4Φ 20
Span	b/nB&C	STAIR	300	352	#####	#####	Single	137.28	4224	#####	#####	#####	4Φ 20
Support	C	65.02	300	352	0.087	322.33	double	137.28	4224	579.93	579.93	1.8469	4Φ 20
Span	b/nC&D	40.58	300	352	0.055	334.14	Single	137.28	4224	349.16	349.16	1.112	4Φ 20
Support	D	49.11	300	352	0.066	330.12	double	137.28	4224	427.69	427.69	1.3621	4Φ 20
Span	b/nD&E	8.97	300	352	0.012	348.21	double	137.28	4224	74.06	137.28	0.4372	4Φ 20
Support	E	18.13	300	352	0.024	344.25	Span	137.28	4224	151.41	151.41	0.4822	4Φ 20

Axis 6	STROY3												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	47.32	300	352	0.064	330.97	Single	137.28	4224	411.04	411.04	1.3091	4Φ 20
Span	b/nA&B	35.11	300	352	0.047	336.66	Single	137.28	4224	299.83	299.83	0.9549	4Φ 20
Support	B	12.22	300	352	0.016	346.82	Single	137.28	4224	101.3	137.28	0.4372	4Φ 20
Span	b/nB&C	STAIR	300	352	#####	#####	Single	137.28	4224	#####	#####	#####	4Φ 20
Support	C	64.2	300	352	0.086	322.74	double	137.28	4224	571.89	571.89	1.8213	4Φ 20
Span	b/nC&D	40.45	300	352	0.054	334.2	Single	137.28	4224	347.98	347.98	1.1082	4Φ 20
Support	D	48.9	300	352	0.066	330.22	double	137.28	4224	425.73	425.73	1.3558	4Φ 20
Span	b/nD&E	8.72	300	352	0.012	348.32	double	137.28	4224	71.974	137.28	0.4372	4Φ 20
Support	E	15.92	300	352	0.021	345.22	Span	137.28	4224	132.58	137.28	0.4372	4Φ 20

Axis 6	STROY2												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	48.13	300	352	0.065	330.59	Single	137.28	4224	418.57	418.57	1.333	4Φ 20
Span	b/nA&E	34.8	300	352	0.047	336.8	Single	137.28	4224	297.05	297.05	0.946	4Φ 20
Support	B	11.16	300	352	0.015	347.27	Single	137.28	4224	92.39	137.28	0.4372	4Φ 20
Span	b/nB&C	STAIR	300	352	#####	#####	Single	137.28	4224	#####	#####	#####	4Φ 20
Support	C	62.7	300	352	0.084	323.49	double	137.28	4224	557.23	557.23	1.7746	4Φ 20
Span	b/nC&D	40.16	300	352	0.054	334.33	Single	137.28	4224	345.34	345.34	1.0998	4Φ 20
Support	D	48.5	300	352	0.065	330.41	double	137.28	4224	422.01	422.01	1.344	4Φ 20
Span	b/nD&E	8.62	300	352	0.012	348.36	double	137.28	4224	71.14	137.28	0.4372	4Φ 20
Support	E	14.1	300	352	0.019	346.01	Span	137.28	4224	117.16	137.28	0.4372	4Φ 20

Axis 6	STROY1												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	0	300	352	0.000	352	Single	137.28	4224	0	137.28	0.4372	4Φ 20
Span	b/nA&E	35.76	300	352	0.048	336.36	Single	137.28	4224	305.65	305.65	0.9734	4Φ 20
Support	B	48.06	300	352	0.065	330.62	Single	137.28	4224	417.92	417.92	1.3309	4Φ 20
Span	b/nB&C	STAIR	300	352	#####	#####	Single	137.28	4224	#####	#####	#####	4Φ 20
Support	C	60.62	300	352	0.082	324.53	double	137.28	4224	537.03	537.03	1.7103	4Φ 20
Span	b/nC&D	39.82	300	352	0.054	334.49	Single	137.28	4224	342.26	342.26	1.09	4Φ 20
Support	D	17.09	300	352	0.023	344.71	double	137.28	4224	142.54	142.54	0.4539	4Φ 20
Span	b/nD&E	8.16	300	352	0.011	348.56	double	137.28	4224	67.305	137.28	0.4372	4Φ 20
Support	E	10.5	300	352	0.014	347.56	Span	137.28	4224	86.855	137.28	0.4372	4Φ 20

Axis 6	GR												
Type	Loc.	Moment(KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal(mm^2)	As,prov(mm^2)	No.of bar	Remark
Support	A	10.46	300	352	0.014	347.57	Single	137.28	4224	86.52	137.28	0.4372	4Φ 20
Span	b/nA&E	5.67	300	352	0.008	349.61	Single	137.28	4224	46.626	137.28	0.4372	4Φ 20
Support	B	5.33	300	352	0.007	349.76	Single	137.28	4224	43.812	137.28	0.4372	4Φ 20
Span	b/nB&C	STAIR	300	352	#####	#####	Single	137.28	4224	#####	#####	#####	4Φ 20
Support	C	2.01	300	352	0.003	351.16	double	137.28	4224	16.456	137.28	0.4372	4Φ 20
Span	b/nC&D	6.01	300	352	0.008	349.47	Single	137.28	4224	49.442	137.28	0.4372	4Φ 20
Support	D	7.3	300	352	0.010	348.92	double	137.28	4224	60.149	137.28	0.4372	4Φ 20
Span	b/nD&E	2.83	300	352	0.004	350.81	double	137.28	4224	23.192	137.28	0.4372	4Φ 20
Support	E	5.53	300	352	0.007	349.67	Span	137.28	4224	45.467	137.28	0.4372	4Φ 20

Appendix 8N Shear Reinforcement of a beam on axis A

	Axis-A	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	18.91	1.641327	13.12638	2103	370	264	264	Ø8 C/C 264
	NEAR-2	24.64	2.138673	18.85638	1464	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	50.42	3.293972	42.73601	646	370	264	264	Ø8 C/C 264
	NEAR-3	49.38	3.226028	41.69601	662	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	42.23	3.088127	35.36517	780	370	264	264	Ø8 C/C 264
	NEAR-4	34.76	2.541873	27.89517	989	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	28.02	2.259771	21.79546	1266	370	264	264	Ø8 C/C 264
	NEAR-6	20.09	1.620229	13.86546	1991	370	264	264	Ø8 C/C 264
	Axis-A	STORY-4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	24.74	2.385034	19.53275	1413	370	264	264	Ø8 C/C 264
	NEAR-2	14.47	1.394966	9.262746	2980	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	19.32	1.976874	14.41395	1915	370	264	264	Ø8 C/C 264
	NEAR-3	44.4	4.543126	39.49395	699	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	36.83	3.028818	30.72575	898	370	264	264	Ø8 C/C 264
	NEAR-4	31.63	2.601182	25.52575	1081	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	22.28	1.953591	16.55487	1667	370	264	264	Ø8 C/C 264
	NEAR-6	21.97	1.926409	16.24487	1699	370	264	264	Ø8 C/C 264

	Axis-A	STORY-1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	18.87	1.758161	13.48213	2047	370	264	264	Ø8 C/C 264
	NEAR-2	21.7	2.021839	16.31213	1692	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	48.88	3.347312	41.54941	664	370	264	264	Ø8 C/C 264
	NEAR-3	46.33	3.172688	38.99941	708	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	38.3	2.993184	31.87654	866	370	264	264	Ø8 C/C 264
	NEAR-4	33.74	2.636816	27.31654	1010	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	25.46	2.155932	19.53174	1413	370	264	264	Ø8 C/C 264
	NEAR-6	20.36	1.724068	14.43174	1913	370	264	264	Ø8 C/C 264
	Axis-A	GROUND							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	7.83	2.487176	6.24963	4417	370	264	264	Ø8 C/C 264
	NEAR-2	4.07	1.292824	2.48963	11087	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	10.6	3.273899	8.97466	3075	370	264	264	Ø8 C/C 264
	NEAR-3	10.51	3.246101	8.88466	3107	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	9.71	3.018625	8.095218	3410	370	264	264	Ø8 C/C 264
	NEAR-4	8.4	2.611375	6.785218	4068	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	6.11	1.93683	4.526371	6098	370	264	264	Ø8 C/C 264
	NEAR-6	6.13	1.94317	4.546371	6071	370	264	264	Ø8 C/C 264

	Axis-A	STORY-3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	23.22	2.198687	17.91846	1540	370	264	264	Ø8 C/C 26
	NEAR-2	16.7	1.581313	11.39846	2422	370	264	264	Ø8 C/C 26
			6.52						
	NEAR-2	49.57	3.403859	42.25943	653	370	264	264	Ø8 C/C 26
	NEAR-3	45.38	3.116141	38.06943	725	370	264	264	Ø8 C/C 26
			5.63						
	NEAR-3	37.17	3.001967	30.95429	892	370	264	264	Ø8 C/C 26
	NEAR-4	32.54	2.628033	26.32429	1049	370	264	264	Ø8 C/C 26
			3.88						
	NEAR-5	23.45	2.018771	17.61878	1567	370	264	264	Ø8 C/C 26
	NEAR-6	21.62	1.861229	15.78878	1748	370	264	264	Ø8 C/C 26
	Axis-A	STORY-2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	21.6	2.059738	16.33564	1690	370	264	264	Ø8 C/C 26
	NEAR-2	18.04	1.720262	12.77564	2160	370	264	264	Ø8 C/C 26
			6.52						
	NEAR-2	48.97	3.388352	41.71487	662	370	264	264	Ø8 C/C 26
	NEAR-3	45.26	3.131648	38.00487	726	370	264	264	Ø8 C/C 26
			5.63						
	NEAR-3	31.21	2.758868	25.53107	1081	370	264	264	Ø8 C/C 26
	NEAR-4	32.48	2.871132	26.80107	1030	370	264	264	Ø8 C/C 26
			3.88						
	NEAR-5	23.55	2.040965	17.75759	1554	370	264	264	Ø8 C/C 26
	NEAR-6	21.22	1.839035	15.42759	1789	370	264	264	Ø8 C/C 26

Appendix 8O Shear Reinforcement of a beam on axis B

	Axis-B	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	62.45	2.263723	48.60117	568	370	264	264	Ø8 C/C 264
	NEAR-2	41.83	1.516277	27.98117	986	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	81.7	3.201034	68.88745	401	370	264	264	Ø8 C/C 264
	NEAR-3	84.71	3.318966	71.89745	384	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	75.62	3.988202	66.10162	418	370	264	264	Ø8 C/C 264
	NEAR-4	31.13	1.641798	21.61162	1277	370	264	264	Ø8 C/C 264
	Axis-B	STORY-4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	74.32	2.741579	60.71155	455	370	264	264	Ø8 C/C 264
	NEAR-2	28.15	1.038421	14.54155	1898	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	86.34	3.324888	73.30417	377	370	264	264	Ø8 C/C 264
	NEAR-3	82.97	3.195112	69.93417	395	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	71.46	3.933514	62.34018	443	370	264	264	Ø8 C/C 264
	NEAR-4	30.82	1.696486	21.70018	1272	370	264	264	Ø8 C/C 264

	Axis-B	STORY-3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	68.28	2.566866	54.92653	503	370	264	264	Ø8 C/C 264
	NEAR-2	32.27	1.213134	18.91653	1459	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	86.42	3.306875	73.30102	377	370	264	264	Ø8 C/C 264
	NEAR-3	83.97	3.213125	70.85102	390	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	72.96	3.957653	63.70555	433	370	264	264	Ø8 C/C 264
	NEAR-4	30.83	1.672347	21.57555	1279	370	264	264	Ø8 C/C 264

	Axis-B	STORY-2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	62.92	2.411168	49.82019	554	370	264	264	Ø8 C/C 264
	NEAR-2	35.72	1.368832	22.62019	1220	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	86.32	3.287613	73.13942	377	370	264	264	Ø8 C/C 264
	NEAR-3	84.87	3.232387	71.68942	385	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	73.97	3.979846	64.63975	427	370	264	264	Ø8 C/C 264
	NEAR-4	30.67	1.650154	21.33975	1293	370	264	264	Ø8 C/C 264
	Axis-B	STORY-1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	53.65	2.097828	40.81181	676	370	264	264	Ø8 C/C 264
	NEAR-2	43.02	1.682172	30.18181	915	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	85.76	3.246938	72.50089	381	370	264	264	Ø8 C/C 264
	NEAR-3	86.45	3.273062	73.19089	377	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	76.34	4.12075	67.04007	412	370	264	264	Ø8 C/C 264
	NEAR-4	27.96	1.50925	18.66007	1479	370	264	264	Ø8 C/C 264
	Axis-B	GROUND							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	12.61	3.78	10.93534	2524	370	264	264	Ø8 C/C 264
	NEAR-2	0	0	#DIV/0!	#DIV/0!	370	264	#DIV/0!	#DIV/0!
			6.52						
	NEAR-2	10.4	3.206052	8.77158	3147	370	264	264	Ø8 C/C 264
	NEAR-3	10.75	3.313948	9.12158	3026	370	264	264	Ø8 C/C 264
			5.63						
	NEAR-3	10.63	3.22973	8.977769	3074	370	264	264	Ø8 C/C 264
	NEAR-4	7.9	2.40027	6.247769	4418	370	264	264	Ø8 C/C 264
	Axis-C	TANKER							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.88						
	NEAR-5	6.59	1.827677	4.779954	5774	370	264	264	Ø8 C/C 264
	NEAR-6	7.4	2.052323	5.589954	4938	370	264	264	Ø8 C/C 264

Appendix 8P Shear Reinforcement of a beam on axis C

	Axis-C	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.88						
	NEAR-5	18.43	1.935275	13.64936	2022	370	264	264	Ø8 C/C 264
	NEAR-6	18.52	1.944725	13.73936	2009	370	264	264	Ø8 C/C 264
	Axis-C	STORY-4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.88						
	NEAR-5	18.38	1.956499	13.66405	2020	370	264	264	Ø8 C/C 264
	NEAR-6	18.07	1.923501	13.35405	2067	370	264	264	Ø8 C/C 264
	Axis-C	STORY-3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.88						
	NEAR-5	18.41	1.959155	13.69275	2016	370	264	264	Ø8 C/C 264
	NEAR-6	18.05	1.920845	13.33275	2070	370	264	264	Ø8 C/C 264
	Axis-C	STORY-2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.88						
	NEAR-5	18.21	1.935218	13.48628	2047	370	264	264	Ø8 C/C 264
	NEAR-6	18.3	1.944782	13.57628	2033	370	264	264	Ø8 C/C 264
	Axis-C	STORY-1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.88						
	NEAR-5	18.25	1.936287	13.51852	2042	370	264	264	Ø8 C/C 264
	NEAR-6	18.32	1.943713	13.58852	2031	370	264	264	Ø8 C/C 264
	Axis-C	GROUND							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.88						
	NEAR-5	6.26	1.984379	4.676371	5902	370	264	264	Ø8 C/C 264
	NEAR-6	5.98	1.895621	4.396371	6278	370	264	264	Ø8 C/C 264
	Axis-D	TERACE							

Appendix 8Q Shear Reinforcement of a beam on axis D

	Axis-D	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	16.38	1.654634	11.41047	2419	370	264	264	Ø8 C/C 264
	NEAR-2	21.04	2.125366	16.07047	1718	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	44.17	3.646346	38.08902	725	370	264	264	Ø8 C/C 264
	NEAR-3	34.81	2.873654	28.72902	961	370	264	264	Ø8 C/C 264
	Axis-D	STORY-4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	21.85	2.305779	17.09295	1615	370	264	264	Ø8 C/C 264
	NEAR-2	13.97	1.474221	9.212952	2996	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	45.31	3.71599	39.18899	704	370	264	264	Ø8 C/C 264
	NEAR-3	34.19	2.80401	28.06899	983	370	264	264	Ø8 C/C 264
	Axis-D	STORY-3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	20.55	2.15416	15.76108	1751	370	264	264	Ø8 C/C 264
	NEAR-2	15.51	1.62584	10.72108	2575	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	44.88	3.694667	38.78209	712	370	264	264	Ø8 C/C 264
	NEAR-3	34.32	2.825333	28.22209	978	370	264	264	Ø8 C/C 264
	Axis-D	STORY-2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	19.17	2.010058	14.38241	1919	370	264	264	Ø8 C/C 264
	NEAR-2	16.88	1.769942	12.09241	2283	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	44.33	3.667448	38.26211	721	370	264	264	Ø8 C/C 264
	NEAR-3	34.48	2.852552	28.41211	971	370	264	264	Ø8 C/C 264

	Axis-D	STORY-1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			3.78						
	NEAR-1	16.83	1.754479	12.01452	2297	370	264	264	Ø8 C/C 264
	NEAR-2	19.43	2.025521	14.61452	1889	370	264	264	Ø8 C/C 264
			6.52						
	NEAR-2	43.38	3.619163	37.36293	739	370	264	264	Ø8 C/C 264
	NEAR-3	34.77	2.900837	28.75293	960	370	264	264	Ø8 C/C 264

Appendix 8R Shear Reinforcement of a beam on axis E

	Axis-E	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	59.2	2.056748	44.75078	617	370	264	264	Ø8 C/C 264
	NEAR-2	102.85	3.573252	88.40078	312	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	40.5	2.944351	33.59491	822	370	264	264	Ø8 C/C 264
	NEAR-3	12.87	0.935649	5.964912	4627	370	264	264	Ø8 C/C 264
	Axis-E	STORY-4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	74.49	2.344338	58.53924	472	370	264	264	Ø8 C/C 264
	NEAR-2	104.4	3.285662	88.44924	312	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	37.74	2.765984	30.89055	894	370	264	264	Ø8 C/C 264
	NEAR-3	15.2	1.114016	8.350546	3305	370	264	264	Ø8 C/C 264
	Axis-E	STORY-3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	72.25	2.300071	56.48114	489	370	264	264	Ø8 C/C 264
	NEAR-2	104.6	3.329929	88.83114	311	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	38.03	2.789346	31.18572	885	370	264	264	Ø8 C/C 264
	NEAR-3	14.87	1.090654	8.025722	3439	370	264	264	Ø8 C/C 264
	Axis-E	STORY-2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	71.95	2.294282	56.20699	491	370	264	264	Ø8 C/C 264
	NEAR-2	104.61	3.335718	88.86699	311	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	37.82	2.777093	30.98348	891	370	264	264	Ø8 C/C 264
	NEAR-3	15.02	1.102907	8.183485	3373	370	264	264	Ø8 C/C 264

	Axis-E	STORY-1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	66.87	2.277682	52.13189	529	370	264	264	Ø8 C/C 264
	NEAR-2	98.42	3.352318	83.68189	330	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	35.35	2.760274	28.92104	954	370	264	264	Ø8 C/C 264
	NEAR-3	14.34	1.119726	7.911036	3489	370	264	264	Ø8 C/C 264
	Axis-E	GROUND							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	10.7	2.209868	8.269357	3338	370	264	264	Ø8 C/C 264
	NEAR-2	16.56	3.420132	14.12936	1953	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	5.81	1.841732	4.226371	6531	370	264	264	Ø8 C/C 264
	NEAR-3	6.43	2.038268	4.846371	5695	370	264	264	Ø8 C/C 264
	Axis-F	TERACE							

Appendix 8S Shear Reinforcement of a beam on axis F

	Axis-F	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	59.84	2.659661	48.54545	569	370	264	264	Ø8 C/C 264
	NEAR-2	66.83	2.970339	55.53545	497	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	68.4	2.183039	52.6711	524	370	264	264	Ø8 C/C 264
	NEAR-3	53.17	1.696961	37.4411	737	370	264	264	Ø8 C/C 264
	Axis-F	STORY-4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	73.07	3.07462	61.1397	451	370	264	264	Ø8 C/C 264
	NEAR-2	60.73	2.55538	48.7997	566	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	64.79	1.869591	47.39337	582	370	264	264	Ø8 C/C 264
	NEAR-3	69.67	2.010409	52.27337	528	370	264	264	Ø8 C/C 264
	Axis-F	STORY-3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	70.46	2.974579	58.56893	471	370	264	264	Ø8 C/C 264
	NEAR-2	62.9	2.655421	51.00893	541	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	66.41	2.006001	49.79095	554	370	264	264	Ø8 C/C 264
	NEAR-3	62.04	1.873999	45.42095	608	370	264	264	Ø8 C/C 264

	Axis-F	STORY-2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	69.02	2.89598	57.05582	484	370	264	264	Ø8 C/C 264
	NEAR-2	65.16	2.73402	53.19582	519	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	67.76	2.029087	50.99605	541	370	264	264	Ø8 C/C 264
	NEAR-3	61.81	1.850913	45.04605	613	370	264	264	Ø8 C/C 264

	Axis-F	STORY-1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	63.83	2.70218	51.97192	531	370	264	264	Ø8 C/C 264
	NEAR-2	69.16	2.92782	57.30192	482	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	69.83	2.252393	54.26671	509	370	264	264	Ø8 C/C 264
	NEAR-3	50.46	1.627607	34.89671	791	370	264	264	Ø8 C/C 264
	Axis-F	GROUND							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-1	10.56	3.919103	9.207364	2998	370	264	264	Ø8 C/C 264
	NEAR-2	4.61	1.710897	3.257364	8474	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	5.69	1.45532	3.727284	7405	370	264	264	Ø8 C/C 264
	NEAR-3	9.48	2.42468	7.517284	3672	370	264	264	Ø8 C/C 264

Appendix 8T Shear Reinforcement of a beam on axis G

	Axis-G	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-3	27	2.937391	22.3857	1233	370	264	264	Ø8 C/C 264
	NEAR-4	24.75	2.692609	20.1357	1371	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	26.78	2.155734	20.54381	1344	370	264	264	Ø8 C/C 264
	NEAR-6	21.42	1.724266	15.18381	1818	370	264	264	Ø8 C/C 264
	Axis-G	STORY-4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-3	30.73	3.324556	26.08984	1058	370	264	264	Ø8 C/C 264
	NEAR-4	21.31	2.305444	16.66984	1656	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	25.78	2.05393	19.47912	1417	370	264	264	Ø8 C/C 264
	NEAR-6	22.92	1.82607	16.61912	1661	370	264	264	Ø8 C/C 264
	Axis-G	STORY-3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-3	29.91	3.222222	25.25023	1093	370	264	264	Ø8 C/C 264
	NEAR-4	22.35	2.407778	17.69023	1560	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	25.36	2.0288	19.085	1446	370	264	264	Ø8 C/C 264
	NEAR-6	23.14	1.8512	16.865	1637	370	264	264	Ø8 C/C 264
	Axis-G	STORY-2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-3	29.4	3.1498	24.71437	1117	370	264	264	Ø8 C/C 264
	NEAR-4	23.15	2.4802	18.46437	1495	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	24.44	1.95843	18.17535	1519	370	264	264	Ø8 C/C 264
	NEAR-6	23.98	1.92157	17.71535	1558	370	264	264	Ø8 C/C 264

	Axis-G	STORY-1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-3	27.93	2.979835	23.22475	1188	370	264	264	Ø8 C/C 264
	NEAR-4	24.84	2.650165	20.13475	1371	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	29.73	2.132601	22.73176	1214	370	264	264	Ø8 C/C 264
	NEAR-6	24.36	1.747399	17.36176	1590	370	264	264	Ø8 C/C 264
	Axis-G	GROUND							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.63						
	NEAR-3	7.78	3.465301	6.652952	4149	370	264	264	Ø8 C/C 264
	NEAR-4	4.86	2.164699	3.732952	7394	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-5	6.54	2.007532	4.904619	5628	370	264	264	Ø8 C/C 264
	NEAR-6	6.1	1.872468	4.464619	6182	370	264	264	Ø8 C/C 264

Appendix 8U Shear Reinforcement of a beam on axis 1

	Axis-1	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.62						
	NEAR-A	361983	5.619472	329646.2	0.083731	370	264	264	Ø8 C/C 264
	NEAR-	34.0271	0.000528	-32302.7	-0.85446	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	15.9879	1.930437	11.83033	2333.115	370	264	264	Ø8 C/C 264
	NEAR-3	16.1463	1.949563	11.98873	2302.288	370	264	264	Ø8 C/C 264
	Axis-1	STOREY4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.62						
	NEAR-1	38.8332	3.16027	32.66466	844.9964	370	264	264	Ø8 C/C 264
	NEAR-2	30.225	2.45973	24.05646	1147.364	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	15.2798	1.803557	11.02684	2503.122	370	264	264	Ø8 C/C 264
	NEAR-3	17.5917	2.076443	13.33874	2069.275	370	264	264	Ø8 C/C 264
	Axis-1	STOREY3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.62						
	NEAR-1	38.1928	3.073766	31.95524	863.7555	370	264	264	Ø8 C/C 264
	NEAR-2	31.638	2.546234	25.40044	1086.655	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	15.07	1.779844	10.81955	2551.079	370	264	264	Ø8 C/C 264
	NEAR-3	17.7821	2.100156	13.53165	2039.775	370	264	264	Ø8 C/C 264

	Axis-1	STOREY2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.62						
	NEAR-1	38.3734	3.081848	32.12279	859.2504	370	264	264	Ø8 C/C 264
	NEAR-2	31.6036	2.538152	25.35299	1088.689	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	14.5721	1.712335	10.30004	2679.748	370	264	264	Ø8 C/C 264
	NEAR-3	18.447	2.167665	14.17494	1947.205	370	264	264	Ø8 C/C 264
	Axis-1	STOREY1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.62						
	NEAR-1	3.4809	0.542079	0.257363	107247.3	370	264	264	Ø8 C/C 264
	NEAR-2	32.6073	5.077921	29.38376	939.3459	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	14.2132	1.678871	9.963304	2770.318	370	264	264	Ø8 C/C 264
	NEAR-3	18.6346	2.201129	14.3847	1918.81	370	264	264	Ø8 C/C 264
	Axis-1	GROUD							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			5.62						
	NEAR-1	0.2744	0.235731	-0.30995	-89052.2	370	264	-89052.2	Ø8 C/C 264
	NEAR-2	6.2675	5.384269	5.683152	4856.727	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	4.4556	1.847967	3.245237	8505.24	370	264	264	Ø8 C/C 264
	NEAR-3	4.8994	2.032033	3.689037	7482.039	370	264	264	Ø8 C/C 264

Appendix 8V Shear Reinforcement of a beam on axis 2

	Axis-2	TERACE							REMARK
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Ø8 C/C 264
			5.62						
	NEAR-A	88.371	3.018475	73.67409	374.6435	370	264	264	
	NEAR-	76.1641	2.601525	61.46719	449.0447	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	45.9897	2.871766	37.95045	727.304	370	264	264	Ø8 C/C 264
	NEAR-3	16.1463	1.008234	8.107055	3404.629	370	264	264	Ø8 C/C 264
	Axis-2	STOREY4							REMARK
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	102.7069	3.23731	86.78045	318.0615	370	264	264	Ø8 C/C 264
	NEAR-2	75.5932	2.38269	59.66675	462.5946	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	35.97	1.393142	23.0087	1199.612	370	264	264	Ø8 C/C 264
	NEAR-3	64.209	2.486858	51.2477	538.5904	370	264	264	Ø8 C/C 264
	Axis-2	STOREY3							Ø8 C/C 264
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	98.8222	3.162273	83.13451	332.0103	370	264	264	Ø8 C/C 264
	NEAR-2	76.8049	2.457727	61.11721	451.6161	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	39.008	1.499475	25.94875	1063.693	370	264	264	Ø8 C/C 264
	NEAR-3	61.928	2.380525	48.86875	564.8091	370	264	264	Ø8 C/C 264
	Axis-2	STOREY2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	33.654	1.66954	23.53486	1172.793	370	264	264	Ø8 C/C 264
	NEAR-2	79.632	3.95046	69.51286	397.0707	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	45.6798	1.786468	32.84371	840.3897	370	264	264	Ø8 C/C 264
	NEAR-3	53.5314	2.093532	40.69531	678.2481	370	264	264	Ø8 C/C 264

	Axis-2	STOREY1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	REMARK
			5.62						
	NEAR-1	3.4809	0.542079	0.257363	107247.3	370	264	264	Ø8 C/C 264
	NEAR-2	32.6073	5.077921	29.38376	939.3459	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	14.2132	1.678871	9.963304	2770.318	370	264	264	Ø8 C/C 264
	NEAR-3	18.6346	2.201129	14.3847	1918.81	370	264	264	Ø8 C/C 264
	Axis-2	GROUD							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	33.897	2.393952	26.78897	1030.332	370	264	264	Ø8 C/C 264
	NEAR-2	45.679	3.226048	38.57097	715.6035	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	45.215	3.178265	38.07339	724.9556	370	264	264	Ø8 C/C 264
	NEAR-3	9.9831	0.701735	2.84149	9713.747	370	264	264	Ø8 C/C 264

Appendix 8W Shear Reinforcement of a beam on axis 3

	Axis-3	TERACE							
	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	104.143	2.992844	86.67474	318.4494	370	264	264	Ø8 C/C 264
	NEAR-2	91.418	2.627156	73.94974	373.247	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	48.9	1.636608	33.90081	814.1846	370	264	264	Ø8 C/C 264
	NEAR-3	67.03	2.243392	52.03081	530.4841	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-3	59.64	3.409044	50.85769	542.7206	370	264	264	Ø8 C/C 264
	NEAR-4	54.6	3.120956	45.81769	602.4205	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-4	22.69	1.977593	16.93028	1630.305	370	264	264	Ø8 C/C 264
	NEAR-5	20.68	1.802407	14.92028	1849.933	370	264	264	Ø8 C/C 264
	Axis-3	STOREY4							
	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	121.28	3.189488	102.1915	270.096	370	264	264	Ø8 C/C 264
	NEAR-2	92.42	2.430512	73.33149	376.3938	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	39.09	1.305917	24.06364	1147.022	370	264	264	Ø8 C/C 264
	NEAR-3	77.05	2.574083	62.02364	445.0161	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-3	57.67	3.385165	49.11788	561.9444	370	264	264	Ø8 C/C 264
	NEAR-4	53.5757	3.144835	45.02358	613.0458	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-4	13.8	1.305732	8.494471	3249.351	370	264	264	Ø8 C/C 264
	NEAR-5	26.15	2.474268	20.84447	1324.165	370	264	264	Ø8 C/C 264

	Axis-3	STOREY3							
	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	116.27	3.122163	97.57541	282.8737	370	264	264	Ø8 C/C 264
	NEAR-2	93.02	2.497837	74.32541	371.3604	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	42.22	1.418914	27.28291	1011.678	370	264	264	Ø8 C/C 264
	NEAR-3	73.23	2.461086	58.29291	473.497	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-3	57.64	3.386927	49.09677	562.1859	370	264	264	Ø8 C/C 264
	NEAR-4	53.49	3.143073	44.94677	614.0934	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-4	15.73	1.478722	10.38994	2656.561	370	264	264	Ø8 C/C 264
	NEAR-5	24.48	2.301278	19.13994	1442.09	370	264	264	Ø8 C/C 264
	Axis-3	STOREY2							
	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	115.36	3.100838	96.68417	285.4812	370	264	264	Ø8 C/C 264
	NEAR-2	93.72	2.519162	75.04417	367.8036	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	44.29	1.501225	29.47971	936.2887	370	264	264	Ø8 C/C 264
	NEAR-3	70.18	2.378775	55.36971	498.4949	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-3	37.42	2.692887	30.44428	906.6242	370	264	264	Ø8 C/C 264
	NEAR-4	53.32	3.837113	46.34428	595.5755	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-4	16.48	1.562438	11.1851	2467.705	370	264	264	Ø8 C/C 264
	NEAR-5	23.39	2.217562	18.0951	1525.359	370	264	264	Ø8 C/C 264

	Axis-3	STOREY1							
	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	108.34	3.0169	90.31266	305.6218	370	264	264	Ø8 C/C 264
	NEAR-2	93.48	2.6031	75.45266	365.8124	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	50.93	1.728101	36.13523	763.8395	370	264	264	Ø8 C/C 264
	NEAR-3	63.42	2.151899	48.62523	567.6377	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-3	57.42	3.257342	48.57081	568.2738	370	264	264	Ø8 C/C 264
	NEAR-4	57.69	3.272658	48.84081	565.1323	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-4	53.38	2.72634	43.55116	633.7723	370	264	264	Ø8 C/C 264
	NEAR-5	20.63	1.05366	10.80116	2555.422	370	264	264	Ø8 C/C 264
	Axis-3	GROUD							
	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	
			5.62						
	NEAR-1	10.88	3.654848	9.385612	2940.833	370	264	264	Ø8 C/C 264
	NEAR-2	5.85	1.965152	4.355612	6337.001	370	264	264	Ø8 C/C 264
			3.88						
	NEAR-2	0.38	0.127875	-1.11177	-24826.7	370	264	-24826.7	Ø8 C/C 264
	NEAR-3	11.15	3.752125	9.658232	2857.823	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-3	2.52	1.263871	1.519075	18169.95	370	264	264	Ø8 C/C 264
	NEAR-4	10.5	5.266129	9.499075	2905.706	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-4	10	2.157534	7.67327	3597.1	370	264	264	Ø8 C/C 264
	NEAR-5	7.52	1.622466	5.19327	5314.863	370	264	264	Ø8 C/C 264

Appendix 8X Shear Reinforcement of a beam on axis 4

	Axis-4	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.53						
	NEAR-E	48.6	2.170711	37.36074	739	370	264	264	Ø8 C/C 264
	NEAR-F	97.6	4.359289	86.36074	320	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	35	1.353591	22.01971	1253	370	264	264	Ø8 C/C 264
	NEAR-G	62.74	2.426409	49.75971	555	370	264	264	Ø8 C/C 264
	Axis-4	STORY-4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.53						
	NEAR-E	47.47	2.02773	35.71797	773	370	264	264	Ø8 C/C 264
	NEAR-F	105.4	4.50227	93.64797	295	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	17.72	0.713633	5.254995	5252	370	264	264	Ø8 C/C 264
	NEAR-G	76.14	3.066367	63.67499	433	370	264	264	Ø8 C/C 264
	Axis-4	STORY-3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.53						
	NEAR-E	47.46	2.03449	35.74949	772	370	264	264	Ø8 C/C 264
	NEAR-F	104.87	4.49551	93.15949	296	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	22.88	0.932568	10.56374	2613	370	264	264	Ø8 C/C 264
	NEAR-G	69.86	2.847432	57.54374	480	370	264	264	Ø8 C/C 264

	Axis-4	STORY-2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.53						
	NEAR-E	47.06	2.011664	35.31643	782	370	264	264	Ø8 C/C 264
	NEAR-F	105.7	4.518336	93.95643	294	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	26.34	1.094243	14.25614	1936	370	264	264	Ø8 C/C 264
	NEAR-G	64.65	2.685757	52.56614	525	370	264	264	Ø8 C/C 264
	Axis-4	STORY-1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.53						
	NEAR-E	46.76	2.024014	35.16249	785	370	264	264	Ø8 C/C 264
	NEAR-F	104.1	4.505986	92.50249	298	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	35.35	1.483711	23.38965	1180	370	264	264	Ø8 C/C 264
	NEAR-G	54.71	2.296289	42.74965	646	370	264	264	Ø8 C/C 264
	Axis-4	GROUND							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.53						
	NEAR-E	9.15	2.809097	7.514848	3673	370	264	264	Ø8 C/C 264
	NEAR-F	12.12	3.720903	10.48485	2633	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	0	0	#DIV/0!	#DIV/0!	370	264	#DIV/0!	#DIV/0!
	NEAR-G	13.38	3.78	11.60308	2379	370	264	264	Ø8 C/C 264

Appendix 8Y Shear Reinforcement of a beam on axis 5

AXIS-5	TANKER							
Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
		4.53						
NEAR-C	24.19	1.672477	16.92928	1630	370	264	264	Ø8 C/C 264
NEAR-E	41.33	2.857523	34.06928	810	370	264	264	Ø8 C/C 264
AXIS-5	TERACE							
Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
		6.48						
NEAR-A	94.93	2.584649	76.49235	361	370	264	264	Ø8 C/C 264
NEAR-C	143.07	3.895351	124.6323	221	370	264	221	Ø8 C/C 221
AXIS-5	STORY-4							
Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
		6.48						
NEAR-A	111.79	2.919787	92.56991	298	370	264	264	Ø8 C/C 264
NEAR-C	136.31	3.560213	117.0899	236	370	264	236	Ø8 C/C 235
AXIS-5	STORY-3							
Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
		6.48						
NEAR-A	108.45	2.857661	89.39879	309	370	264	264	Ø8 C/C 264
NEAR-C	137.47	3.622339	118.4188	233	370	264	233	Ø8 C/C 233
AXIS-5	STORY-2							
Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
		6.48						
NEAR-A	109.6	2.883976	90.52245	305	370	264	264	Ø8 C/C 264
NEAR-C	136.66	3.596024	117.5825	235	370	264	235	Ø8 C/C 234
AXIS-5	STORY-1							
Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
		6.48						
NEAR-A	104.59	2.789411	85.76732	322	370	264	264	Ø8 C/C 264
NEAR-C	138.38	3.690589	119.5573	231	370	264	231	Ø8 C/C 230
AXIS-5	GROUND							
Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
		6.48						
NEAR-A	11.25	2.654771	9.122698	3026	370	264	264	Ø8 C/C 264
NEAR-C	16.21	3.825229	14.0827	1960	370	264	264	Ø8 C/C 264
Axis-6	TERACE							

Appendix 8Z Shear Reinforcement of a beam on axis 6

	Axis-6	TERACE							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.48						
	NEAR-A	52.85	3.070911	44.21064	624	370	264	264	Ø8 C/C 264
	NEAR-B	58.67	3.409089	50.03064	552	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-E	68.03	3.640383	58.64883	471	370	264	264	Ø8 C/C 264
	NEAR-F	54	2.889617	44.61883	619	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	23.46	1.967579	17.47451	1580	370	264	264	Ø8 C/C 264
	NEAR-G	21.61	1.812421	15.62451	1767	370	264	264	Ø8 C/C 264
	Axis-6	STORY-4							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.48						
	NEAR-A	57.28	3.312283	48.59881	568	370	264	264	Ø8 C/C 264
	NEAR-B	54.78	3.167717	46.09881	599	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-E	67.71	3.635474	58.36035	473	370	264	264	Ø8 C/C 264
	NEAR-F	53.91	2.894526	44.56035	619	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	15.42	1.372119	9.778476	2823	370	264	264	Ø8 C/C 264
	NEAR-G	27.06	2.407881	21.41848	1289	370	264	264	Ø8 C/C 264

	Axis-6	STORY-3							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.48						
	NEAR-A	56.12	3.250716	47.45353	582	370	264	264	Ø8 C/C 264
	NEAR-B	55.75	3.229284	47.08353	586	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-E	67.28	3.632097	57.98108	476	370	264	264	Ø8 C/C 264
	NEAR-F	53.68	2.897903	44.38108	622	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	17.1	1.523044	11.46379	2408	370	264	264	Ø8 C/C 264
	NEAR-G	25.34	2.256956	19.70379	1401	370	264	264	Ø8 C/C 264

	Axis-6	STORY-2							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.48						
	NEAR-A	56.41	3.266638	47.7412	578	370	264	264	Ø8 C/C 264
	NEAR-B	55.49	3.213362	46.8212	590	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-E	66.39	3.613024	57.16565	483	370	264	264	Ø8 C/C 264
	NEAR-F	53.6	2.916976	44.37565	622	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	17.81	1.612498	12.26542	2250	370	264	264	Ø8 C/C 264
	NEAR-G	23.94	2.167502	18.39542	1500	370	264	264	Ø8 C/C 264
	Axis-6	STORY-1							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.48						
	NEAR-A	55.39	3.21044	46.72895	591	370	264	264	Ø8 C/C 264
	NEAR-B	56.41	3.26956	47.74895	578	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-E	64.92	3.588349	55.83787	494	370	264	264	Ø8 C/C 264
	NEAR-F	53.22	2.941651	44.13787	625	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	20.64	1.848796	15.03566	1836	370	264	264	Ø8 C/C 264
	NEAR-G	21.56	1.931204	15.95566	1730	370	264	264	Ø8 C/C 264
	Axis-6	GROUND							
Span	Loac	Vmax	Leng	Ved	Scalc	Smin	Smax	Sprov	Remark
			6.48						
	NEAR-A	10.45	3.451376	8.930056	3091	370	264	264	Ø8 C/C 264
	NEAR-B	9.17	3.028624	7.650056	3608	370	264	264	Ø8 C/C 264
			6.53						
	NEAR-E	11.03	3.511745	9.453274	2920	370	264	264	Ø8 C/C 264
	NEAR-F	9.48	3.018255	7.903274	3492	370	264	264	Ø8 C/C 264
			3.78						
	NEAR-F	3.7	1.24875	2.212593	12475	370	264	264	Ø8 C/C 264
	NEAR-G	7.5	2.53125	6.012593	4591	370	264	264	Ø8 C/C 264

Appendix 9A determination of effective length of column

location	column	bc	hc	bb	hb	Lc	Lbx left	lbx right	lby left	lby right	k1x	k2x	k1y	k2y	Lox	Loy
Wt	C14	350	450	300	400	1800	0	3880	6480	4530	3.58	3.58	2.46	2.46	1699.52	1660.85
Wt	C15	350	450	300	400	1800	5630	3880	4530	0	2.12	2.12	4.18	4.18	1642.40	1712.54
Wt	C19	350	450	300	400	1800	3880	0	6480	4530	3.58	3.58	2.46	2.46	1699.52	1660.85
Wt	C20	350	450	300	400	1800	3880	0	4530	6530	3.58	3.58	2.47	2.47	1699.52	1661.22
fourth	C1	350	450	300	400	3200	0	3780	0	5620	1.96	1.96	2.92	2.92	2901.52	2986.18
fourth	C2	350	450	300	400	3200	0	3780	5620	3880	1.96	1.96	1.19	1.19	2901.52	2761.38
fourth	C3	350	450	300	400	3200	0	3780	3880	0	1.96	1.96	2.01	2.01	2901.52	2907.81
fourth	C4	350	450	300	400	3200	3780	6520	0	5620	1.24	1.24	2.92	2.92	2774.49	2986.18
fourth	C5	350	450	300	400	3200	3780	6520	5620	3880	1.24	1.24	1.19	1.19	2774.49	2761.38
fourth	C6	350	450	300	400	3200	3780	6520	3880	0	1.24	1.24	2.01	2.01	2774.49	2907.81
fourth	C7	350	450	300	400	3200	6520	5630	0	5620	1.57	1.57	2.92	2.92	2843.27	2986.18
fourth	C8	350	450	300	400	3200	6520	5630	5440	3490	1.57	1.57	1.10	1.10	2843.27	2736.57
fourth	C9	350	450	300	400	3200	6520	5630	3490	1540	1.57	1.57	0.55	0.55	2843.27	2483.34
fourth	C10	350	450	300	400	3200	0	5630	1540	6530	2.92	2.92	0.65	0.65	2986.51	2543.58
fourth	C11	350	450	300	400	3200	0	4750	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
fourth	C12	350	450	300	400	3200	0	4750	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52
fourth	C13	350	450	300	400	3200	5630	3880	0	6480	1.19	1.19	3.36	3.36	2761.61	3011.21
fourth	C14	350	450	300	400	3200	0	3880	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
fourth	C15	350	450	300	400	3200	5630	3880	4530	0	1.19	1.19	2.35	2.35	2761.61	2943.00
fourth	C16	350	450	300	400	3200	4750	4750	0	3780	1.23	1.23	1.96	1.96	2772.16	2901.52
fourth	C17	350	450	300	400	3200	4750	4750	3780	0	1.23	1.23	1.96	1.96	2772.16	2901.52
fourth	C18	350	450	300	400	3200	3880	0	0	648	2.01	2.01	0.34	0.34	2907.81	2284.41
fourth	C19	350	450	300	400	3200	3880	0	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
fourth	C20	350	450	300	400	3200	3880	0	4530	6530	2.01	2.01	1.39	1.39	2907.81	2808.35
fourth	C21	350	450	300	400	3200	4750	0	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
fourth	C22	350	450	300	400	3200	4750	0	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52
Third	C1	350	450	300	400	3200	0	3780	0	5620	1.96	1.96	2.92	2.92	2901.52	2986.18
Third	C2	350	450	300	400	3200	0	3780	5620	3880	1.96	1.96	1.19	1.19	2901.52	2761.38
Third	C3	350	450	300	400	3200	0	3780	3880	0	1.96	1.96	2.01	2.01	2901.52	2907.81
Third	C4	350	450	300	400	3200	3780	6520	0	5620	1.24	1.24	2.92	2.92	2774.49	2986.18
Third	C5	350	450	300	400	3200	3780	6520	5620	3880	1.24	1.24	1.19	1.19	2774.49	2761.38
Third	C6	350	450	300	400	3200	3780	6520	3880	0	1.24	1.24	2.01	2.01	2774.49	2907.81
Third	C7	350	450	300	400	3200	6520	5630	0	5620	1.57	1.57	2.92	2.92	2843.27	2986.18
Third	C8	350	450	300	400	3200	6520	5630	5440	3490	1.57	1.57	1.10	1.10	2843.27	2736.57
Third	C9	350	450	300	400	3200	6520	5630	3490	1540	1.57	1.57	0.55	0.55	2843.27	2483.34
Third	C10	350	450	300	400	3200	0	5630	1540	6530	2.92	2.92	0.65	0.65	2986.51	2543.58
Third	C11	350	450	300	400	3200	0	4750	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
Third	C12	350	450	300	400	3200	0	4750	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52
Third	C13	350	450	300	400	3200	5630	3880	0	6480	1.19	1.19	3.36	3.36	2761.61	3011.21
Third	C14	350	450	300	400	3200	0	3880	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
Third	C15	350	450	300	400	3200	5630	3880	4530	0	1.19	1.19	2.35	2.35	2761.61	2943.00
Third	C16	350	450	300	400	3200	4750	4750	0	3780	1.23	1.23	1.96	1.96	2772.16	2901.52
Third	C17	350	450	300	400	3200	4750	4750	3780	0	1.23	1.23	1.96	1.96	2772.16	2901.52
Third	C18	350	450	300	400	3200	3880	0	0	648	2.01	2.01	0.34	0.34	2907.81	2284.41
Third	C19	350	450	300	400	3200	3880	0	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
Third	C20	350	450	300	400	3200	3880	0	4530	6530	2.01	2.01	1.39	1.39	2907.81	2808.35
Third	C21	350	450	300	400	3200	4750	0	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
Third	C22	350	450	300	400	3200	4750	0	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52

Secod	C1	350	450	300	400	3200	0	3780	0	5620	1.96	1.96	2.92	2.92	2901.52	2986.18
Secod	C2	350	450	300	400	3200	0	3780	5620	3880	1.96	1.96	1.19	1.19	2901.52	2761.38
Secod	C3	350	450	300	400	3200	0	3780	3880	0	1.96	1.96	2.01	2.01	2901.52	2907.81
Secod	C4	350	450	300	400	3200	3780	6520	0	5620	1.24	1.24	2.92	2.92	2774.49	2986.18
Secod	C5	350	450	300	400	3200	3780	6520	5620	3880	1.24	1.24	1.19	1.19	2774.49	2761.38
Secod	C6	350	450	300	400	3200	3780	6520	3880	0	1.24	1.24	2.01	2.01	2774.49	2907.81
Secod	C7	350	450	300	400	3200	6520	5630	0	5620	1.57	1.57	2.92	2.92	2843.27	2986.18
Secod	C8	350	450	300	400	3200	6520	5630	5440	3490	1.57	1.57	1.10	1.10	2843.27	2736.57
Secod	C9	350	450	300	400	3200	6520	5630	3490	1540	1.57	1.57	0.55	0.55	2843.27	2483.34
Secod	C10	350	450	300	400	3200	0	5630	1540	6530	2.92	2.92	0.65	0.65	2986.51	2543.58
Secod	C11	350	450	300	400	3200	0	4750	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
Secod	C12	350	450	300	400	3200	0	4750	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52
Secod	C13	350	450	300	400	3200	5630	3880	0	6480	1.19	1.19	3.36	3.36	2761.61	3011.21
Secod	C14	350	450	300	400	3200	0	3880	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
Secod	C15	350	450	300	400	3200	5630	3880	4530	0	1.19	1.19	2.35	2.35	2761.61	2943.00
Secod	C16	350	450	300	400	3200	4750	4750	0	3780	1.23	1.23	1.96	1.96	2772.16	2901.52
Secod	C17	350	450	300	400	3200	4750	4750	3780	0	1.23	1.23	1.96	1.96	2772.16	2901.52
Secod	C18	350	450	300	400	3200	3880	0	0	648	2.01	2.01	0.34	0.34	2907.81	2284.41
Secod	C19	350	450	300	400	3200	3880	0	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
Secod	C20	350	450	300	400	3200	3880	0	4530	6530	2.01	2.01	1.39	1.39	2907.81	2808.35
Secod	C21	350	450	300	400	3200	4750	0	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
Secod	C22	350	450	300	400	3200	4750	0	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52
First	C1	350	450	300	400	3200	0	3780	0	5620	1.96	1.96	2.92	2.92	2901.52	2986.18
First	C2	350	450	300	400	3200	0	3780	5620	3880	1.96	1.96	1.19	1.19	2901.52	2761.38
First	C3	350	450	300	400	3200	0	3780	3880	0	1.96	1.96	2.01	2.01	2901.52	2907.81
First	C4	350	450	300	400	3200	3780	6520	0	5620	1.24	1.24	2.92	2.92	2774.49	2986.18
First	C5	350	450	300	400	3200	3780	6520	5620	3880	1.24	1.24	1.19	1.19	2774.49	2761.38
First	C6	350	450	300	400	3200	3780	6520	3880	0	1.24	1.24	2.01	2.01	2774.49	2907.81
First	C7	350	450	300	400	3200	6520	5630	0	5620	1.57	1.57	2.92	2.92	2843.27	2986.18
First	C8	350	450	300	400	3200	6520	5630	5440	3490	1.57	1.57	1.10	1.10	2843.27	2736.57
First	C9	350	450	300	400	3200	6520	5630	3490	1540	1.57	1.57	0.55	0.55	2843.27	2483.34
First	C10	350	450	300	400	3200	0	5630	1540	6530	2.92	2.92	0.65	0.65	2986.51	2543.58
First	C11	350	450	300	400	3200	0	4750	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
First	C12	350	450	300	400	3200	0	4750	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52
First	C13	350	450	300	400	3200	5630	3880	0	6480	1.19	1.19	3.36	3.36	2761.61	3011.21
First	C14	350	450	300	400	3200	0	3880	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
First	C15	350	450	300	400	3200	5630	3880	4530	0	1.19	1.19	2.35	2.35	2761.61	2943.00
First	C16	350	450	300	400	3200	4750	4750	0	3780	1.23	1.23	1.96	1.96	2772.16	2901.52
First	C17	350	450	300	400	3200	4750	4750	3780	0	1.23	1.23	1.96	1.96	2772.16	2901.52
First	C18	350	450	300	400	3200	3880	0	0	648	2.01	2.01	0.34	0.34	2907.81	2284.41
First	C19	350	450	300	400	3200	3880	0	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
First	C20	350	450	300	400	3200	3880	0	4530	6530	2.01	2.01	1.39	1.39	2907.81	2808.35
First	C21	350	450	300	400	3200	4750	0	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
First	C22	350	450	300	400	3200	4750	0	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52
GR	C1	350	450	300	400	3200	0	3780	0	5620	1.96	1.96	2.92	2.92	2901.52	2986.18
GR	C2	350	450	300	400	3200	0	3780	5620	3880	1.96	1.96	1.19	1.19	2901.52	2761.38
GR	C3	350	450	300	400	3200	0	3780	3880	0	1.96	1.96	2.01	2.01	2901.52	2907.81
GR	C4	350	450	300	400	3200	3780	6520	0	5620	1.24	1.24	2.92	2.92	2774.49	2986.18
GR	C5	350	450	300	400	3200	3780	6520	5620	3880	1.24	1.24	1.19	1.19	2774.49	2761.38
GR	C6	350	450	300	400	3200	3780	6520	3880	0	1.24	1.24	2.01	2.01	2774.49	2907.81
GR	C7	350	450	300	400	3200	6520	5630	0	5620	1.57	1.57	2.92	2.92	2843.27	2986.18
GR	C8	350	450	300	400	3200	6520	5630	5440	3490	1.57	1.57	1.10	1.10	2843.27	2736.57
GR	C9	350	450	300	400	3200	6520	5630	3490	1540	1.57	1.57	0.55	0.55	2843.27	2483.34
GR	C10	350	450	300	400	3200	0	5630	1540	6530	2.92	2.92	0.65	0.65	2986.51	2543.58
GR	C11	350	450	300	400	3200	0	4750	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
GR	C12	350	450	300	400	3200	0	4750	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52
GR	C13	350	450	300	400	3200	5630	3880	0	6480	1.19	1.19	3.36	3.36	2761.61	3011.21
GR	C14	350	450	300	400	3200	0	3880	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
GR	C15	350	450	300	400	3200	5630	3880	4530	0	1.19	1.19	2.35	2.35	2761.61	2943.00
GR	C16	350	450	300	400	3200	4750	4750	0	3780	1.23	1.23	1.96	1.96	2772.16	2901.52
GR	C17	350	450	300	400	3200	4750	4750	3780	0	1.23	1.23	1.96	1.96	2772.16	2901.52
GR	C18	350	450	300	400	3200	3880	0	0	648	2.01	2.01	0.34	0.34	2907.81	2284.41
GR	C19	350	450	300	400	3200	3880	0	6480	4530	2.01	2.01	1.38	1.38	2907.81	2807.42
GR	C20	350	450	300	400	3200	3880	0	4530	6530	2.01	2.01	1.39	1.39	2907.81	2808.35
GR	C21	350	450	300	400	3200	4750	0	6530	3780	2.47	2.47	1.24	1.24	2953.06	2774.67
GR	C22	350	450	300	400	3200	4750	0	3780	0	2.47	2.47	1.96	1.96	2953.06	2901.52

Appendix 9B slenderness check for column

location	column	Mx top	Mx botm	My top	My bot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slendernes(x)	slendernes(y)
Wt	C14	1.65	3.66	8.81	35.23	10.25	1.86	9.02	3.87	35.44	13.08	12.79	247.91	293.74	short	short
Wt	C15	0.72	22.08	18.01	53.06	38.26	1.49	18.78	22.85	53.83	12.64	13.18	171.96	142.11	short	short
Wt	C19	1.92	4.59	27.18	15.85	29.26	2.51	16.44	5.18	27.77	13.08	12.79	146.23	133.26	short	short
Wt	C20	1.3	5.13	28.56	62.47	22.83	1.76	29.02	5.59	62.93	13.08	12.79	188.65	168.68	short	short
fourth	C1	13.22	15.18	27.57	25.40	70.28	14.63	26.81	16.59	28.98	22.34	22.99	63.49	60.13	short	short
fourth	C2	65.89	61.56	23.09	3.18	208.99	65.74	7.36	70.07	27.27	22.34	21.26	34.28	64.36	short	short
fourth	C3	10.72	12.43	6.53	6.28	154.23	13.80	9.36	15.51	9.61	22.34	22.38	42.44	38.03	short	short
fourth	C4	22.01	25.48	78.18	74.46	224.93	26.51	78.96	29.98	82.68	21.36	22.99	35.38	32.32	short	short
fourth	C5	34.83	41.21	30.03	30.68	73.88	36.31	31.51	42.69	32.16	21.36	21.26	64.29	54.51	short	short
fourth	C6	9.21	22.37	39.63	40.01	163.88	12.49	42.91	25.65	43.29	21.36	22.38	61.65	36.02	short	short
fourth	C7	3.47	3.55	93.67	89.13	263.54	8.74	94.40	8.82	98.94	21.89	22.99	28.41	29.89	short	short
fourth	C8	6.48	7.16	40.04	40.32	453.04	15.54	49.10	16.22	49.38	21.89	21.07	22.68	21.57	short	short
fourth	C9	4.70	5.46	33.17	36.64	196.26	8.63	37.10	9.39	40.57	21.89	19.12	36.27	36.48	short	short
fourth	C10	39.06	44.00	33.84	34.75	182.88	42.72	37.50	47.66	38.41	22.99	19.58	38.66	34.81	short	short
fourth	C11	36.89	41.07	33.47	33.46	193.43	40.76	37.33	44.94	37.34	22.73	21.36	37.09	32.76	short	short
fourth	C12	14.18	15.40	3.20	15.50	64.35	15.47	4.49	16.69	16.79	22.73	22.34	62.70	116.19	short	short
fourth	C13	4.05	4.86	87.00	83.25	217.76	8.41	87.61	9.22	91.36	21.26	23.18	34.73	32.67	short	short
fourth	C14	3.76	4.53	23.96	30.33	199.90	7.76	27.96	8.53	34.33	22.38	21.61	36.36	40.75	short	short
fourth	C15	16.79	20.07	30.84	35.91	271.62	22.22	36.27	25.50	41.34	21.26	22.66	32.71	32.47	short	short
fourth	C16	3.55	4.47	69.42	73.47	407.44	11.70	77.57	12.62	81.62	21.34	22.34	24.91	24.16	short	short
fourth	C17	15.49	25.25	63.43	59.26	220.42	19.90	63.67	29.66	67.84	21.34	22.34	45.09	33.37	short	short
fourth	C18	8.76	9.46	48.42	44.09	92.49	10.61	45.94	11.31	50.27	22.38	17.59	51.54	53.18	short	short
fourth	C19	5.30	1.72	27.18	15.85	127.97	4.28	18.41	7.86	29.74	22.38	21.61	66.45	62.16	short	short
fourth	C20	1.36	10.17	28.56	62.47	118.65	3.73	30.93	12.54	64.84	22.38	21.62	83.75	73.04	short	short
fourth	C21	28.73	37.93	30.85	32.20	186.64	32.46	34.58	41.66	35.93	22.73	21.36	43.85	35.12	short	short
fourth	C22	6.19	6.68	9.21	15.96	59.18	7.37	10.39	7.86	17.14	22.73	22.34	64.46	92.49	short	short
Third	C1	10.21	13.09	14.06	20.43	150.99	13.23	17.08	16.11	23.45	22.34	22.99	46.52	51.44	short	short
Third	C2	32.97	47.36	17.13	0.55	425.00	41.47	9.05	55.86	25.63	22.34	21.26	30.22	42.50	short	short
Third	C3	8.44	10.86	4.19	5.53	104.31	10.53	6.28	12.95	7.62	22.34	22.38	56.49	55.79	short	short
Third	C4	17.66	22.60	43.28	60.27	451.28	26.69	52.31	31.63	69.30	21.36	22.99	26.22	28.94	short	short
Third	C5	29.36	37.01	19.88	26.20	718.20	43.72	34.24	51.37	40.56	21.36	21.26	20.61	20.77	slender	slender
Third	C6	15.56	19.91	25.16	32.92	334.67	22.25	31.85	26.60	39.61	21.36	22.38	30.71	31.86	short	short
Third	C7	2.54	3.22	51.70	71.83	532.89	13.20	62.36	13.88	82.49	21.89	22.99	21.11	26.60	slender	short
Third	C8	5.10	6.28	25.70	34.09	884.87	22.80	43.40	23.98	51.79	21.89	21.07	16.38	18.85	slender	slender
Third	C9	4.35	5.18	21.45	31.61	403.72	12.42	29.52	13.25	39.68	21.89	19.12	24.69	30.95	short	short
Third	C10	29.93	38.96	21.53	29.11	409.90	38.13	29.73	47.16	37.31	22.99	19.58	28.65	29.02	short	short
Third	C11	27.79	35.99	21.19	28.09	389.28	35.58	28.98	43.78	35.88	22.73	21.36	29.26	29.42	short	short
Third	C12	10.16	13.49	10.53	13.31	138.63	12.93	13.30	16.26	16.08	22.73	22.34	49.99	48.23	short	short
Third	C13	3.80	4.59	48.80	68.25	413.36	12.07	57.07	12.86	76.52	21.26	23.18	24.36	30.53	short	short
Third	C14	3.55	4.28	23.94	30.29	370.82	10.97	31.36	11.70	37.71	22.38	21.61	25.76	29.34	short	short
Third	C15	15.01	17.99	24.44	30.79	3875.12	92.51	101.94	95.49	108.29	21.26	22.66	7.64	7.93	slender	slender
Third	C16	3.17	4.08	48.49	62.97	786.61	18.90	64.22	19.81	78.70	21.34	22.34	17.30	20.50	slender	slender
Third	C17	7.58	49.20	34.30	46.09	450.66	16.59	43.31	58.21	55.10	21.34	22.34	43.36	28.01	short	short
Third	C18	6.49	8.19	24.02	34.93	190.46	10.30	27.83	12.00	38.74	22.38	17.59	39.68	46.27	short	short
Third	C19	6.33	3.23	26.02	17.03	223.85	7.71	21.51	10.81	30.50	22.38	21.61	42.91	43.25	short	short
Third	C20	3.42	9.12	19.78	60.82	205.22	7.52	23.88	13.22	64.92	22.38	21.62	51.36	60.49	short	short
Third	C21	21.90	28.09	20.19	26.62	374.60	29.39	27.68	35.58	34.11	22.73	21.36	29.38	29.86	short	short
Third	C22	4.66	6.04	10.67	13.57	126.17	7.18	13.19	8.56	16.09	22.73	22.34	49.87	50.98	short	short

Secod	C1	9.44	11.85	16.47	20.57	229.06	14.02	21.05	16.43	25.15	22.34	22.99	36.39	37.10	short	short
Secod	C2	35.39	44.61	21.44	1.89	633.88	48.07	14.57	57.29	34.12	22.34	21.26	22.25	32.89	slender	short
Secod	C3	7.72	9.70	4.89	5.86	159.31	10.91	8.08	12.89	9.05	22.34	22.38	44.00	41.61	short	short
Secod	C4	16.93	21.23	48.31	59.97	676.48	30.46	61.84	34.76	73.50	21.36	22.99	20.60	21.48	slender	slender
Secod	C5	27.77	34.42	20.96	25.73	1076.48	49.30	42.49	55.95	47.26	21.36	21.26	16.24	15.88	slender	slender
Secod	C6	14.76	18.61	28.80	31.45	502.45	24.81	38.85	28.66	41.50	21.36	22.38	24.21	22.17	short	slender
Secod	C7	3.00	3.59	57.63	71.20	798.84	18.98	73.61	19.57	87.18	21.89	22.99	16.81	19.69	slender	slender
Secod	C8	5.22	6.18	27.11	33.24	1325.00	31.72	53.61	32.68	59.74	21.89	21.07	13.04	14.34	slender	slender
Secod	C9	4.42	5.53	23.04	29.62	604.20	16.50	35.12	17.61	41.70	21.89	19.12	20.19	22.70	slender	short
Secod	C10	29.27	28.50	23.01	29.31	556.66	39.63	34.14	40.40	40.44	22.99	19.58	19.83	23.60	slender	short
Secod	C11	27.48	34.58	22.34	27.48	584.33	39.17	34.03	46.27	39.17	22.73	21.36	22.97	22.37	short	short
Secod	C12	10.12	13.06	10.28	12.36	210.57	14.33	14.49	17.27	16.57	22.73	22.34	39.01	37.01	short	slendernes(y)
Secod	C13	3.84	4.42	55.38	69.05	667.86	17.20	68.74	17.78	82.41	21.26	23.18	18.44	21.80	slender	slender
Secod	C14	3.58	4.13	23.22	28.89	544.57	14.47	34.11	15.02	39.78	22.38	21.61	20.53	23.49	slender	short
Secod	C15	15.83	20.18	22.75	28.29	521.72	26.26	33.18	30.61	38.72	21.26	22.66	23.98	24.01	short	short
Secod	C16	2.59	3.40	49.11	60.99	1178.20	26.15	72.67	26.96	84.55	21.34	22.34	13.84	15.93	slender	slender
Secod	C17	32.61	62.05	36.08	42.74	674.15	46.09	49.56	75.53	56.22	21.34	22.34	27.30	20.51	short	slender
Secod	C18	6.58	7.87	28.76	35.59	287.08	12.32	34.50	13.61	41.33	22.38	17.59	30.52	33.22	short	short
Secod	C19	6.44	3.15	26.65	17.31	322.48	9.60	23.76	12.89	33.10	22.38	21.61	34.61	35.58	short	short
Secod	C20	3.14	9.12	18.83	59.10	292.91	9.00	24.69	14.98	64.96	22.38	21.62	41.78	50.17	short	short
Secod	C21	22.27	27.60	201.02	26.08	562.50	33.52	37.33	38.85	212.27	22.73	21.36	22.96	41.81	short	short
Secod	C22	5.30	6.61	10.38	12.54	191.89	9.14	14.22	10.45	16.38	22.73	22.34	38.76	39.07	short	short
First	C1	8.03	9.99	16.31	23.37	305.94	14.15	22.43	16.11	29.49	22.34	22.99	30.56	34.94	short	short
First	C2	35.39	44.61	24.68	3.19	841.42	52.22	20.02	61.44	41.51	22.34	21.26	19.06	27.31	slender	short
First	C3	7.72	9.70	5.38	70.22	214.04	12.00	9.66	13.98	74.50	22.34	22.38	37.42	69.83	short	short
First	C4	15.64	19.94	47.53	65.96	902.03	33.68	65.57	37.98	84.00	21.36	22.99	17.61	19.91	slender	slender
First	C5	25.49	31.64	20.72	27.36	1443.84	54.37	49.60	60.52	56.24	21.36	21.26	13.72	14.01	slender	slender
First	C6	13.49	17.21	24.18	30.82	668.70	26.86	37.55	30.58	44.19	21.36	22.38	20.67	21.39	slender	slender
First	C7	3.57	4.55	56.40	77.86	1062.70	24.82	77.65	25.80	99.11	21.89	22.99	14.73	18.29	slender	slender
First	C8	5.35	6.42	26.45	34.99	1772.90	40.81	61.91	41.88	70.45	21.89	21.07	11.21	12.69	slender	slender
First	C9	5.00	6.10	23.27	28.11	800.12	21.00	39.27	22.10	44.11	21.89	19.12	17.24	18.62	slender	slender
First	C10	30.23	40.24	23.17	31.66	778.22	45.79	38.73	55.80	47.22	22.99	19.58	20.51	20.52	slender	short
First	C11	6.29	34.22	21.80	28.85	281.25	11.92	27.43	39.85	34.48	22.73	21.36	54.34	35.09	short	short
First	C12	9.64	13.05	9.26	11.04	210.57	13.85	13.47	17.26	15.25	22.73	22.34	40.24	36.61	short	short
First	C13	3.84	4.42	56.01	78.77	893.80	21.72	73.89	22.30	96.65	21.26	23.18	15.80	20.36	slender	slender
First	C14	3.58	4.13	21.90	27.03	732.90	18.24	36.56	18.79	41.69	22.38	21.61	17.52	19.78	slender	slender
First	C15	15.83	20.18	20.71	26.12	698.90	29.81	34.69	34.16	40.10	21.26	22.66	20.36	20.55	slender	slender
First	C16	1.79	2.29	47.53	61.54	1581.47	33.42	79.16	33.92	93.17	21.34	22.34	11.69	13.91	slender	slender
First	C17	48.68	69.28	31.13	38.37	890.73	66.49	48.94	87.09	56.18	21.34	22.34	20.41	18.07	slender	slender
First	C18	6.42	7.82	28.54	40.54	383.43	14.09	36.21	15.49	48.21	22.38	17.59	26.26	31.53	short	short
First	C19	6.47	3.15	26.70	18.41	424.54	11.64	26.90	14.96	35.19	22.38	21.61	29.11	29.54	short	short
First	C20	3.09	8.54	18.77	57.57	383.51	10.76	26.44	16.21	65.24	22.38	21.62	34.42	43.01	short	short
First	C21	22.25	28.49	20.65	27.31	750.70	37.26	35.66	43.50	42.32	22.73	21.36	20.03	20.36	slender	slender
First	C22	6.21	8.23	9.25	11.13	257.53	11.36	14.40	13.38	16.28	22.73	22.34	34.50	33.06	short	short
GR	C1	5.59	5.73	13.81	8.20	379.64	13.18	15.79	13.32	21.40	22.34	22.99	23.72	32.12	short	short
GR	C2	20.20	17.78	22.09	13.83	1036.23	38.50	34.55	40.92	42.81	22.34	21.26	15.34	18.05	slender	slender
GR	C3	4.48	4.70	4.65	2.38	267.18	9.82	7.72	10.04	9.99	22.34	22.38	28.73	36.90	short	short
GR	C4	11.40	9.30	37.69	18.66	1126.44	31.83	41.19	33.93	60.22	21.36	22.99	14.77	19.69	slender	slender
GR	C5	18.14	14.44	15.95	8.28	1829.74	51.03	44.87	54.73	52.54	21.36	21.26	11.67	12.87	slender	slender
GR	C6	9.87	8.48	17.97	10.84	830.03	25.08	27.44	26.47	34.57	21.36	22.38	16.99	20.46	slender	slender
GR	C7	3.01	0.97	44.31	21.60	1321.77	27.41	48.04	29.45	70.75	21.89	22.99	13.77	18.27	slender	slender
GR	C8	3.82	1.44	20.17	10.20	2236.30	46.17	54.93	48.55	64.90	21.89	21.07	10.30	11.74	slender	slender
GR	C9	4.82	4.55	18.05	10.25	985.30	24.26	29.96	24.53	37.76	21.89	19.12	14.74	18.79	slender	slender
GR	C10	22.55	13.68	18.65	11.87	930.00	32.28	30.47	41.15	37.25	22.99	19.58	19.53	18.82	slender	slender
GR	C11	19.16	12.74	17.18	10.00	969.77	32.14	29.40	38.56	36.58	22.73	21.36	18.10	18.72	slender	slender
GR	C12	7.30	5.61	6.79	5.08	348.16	12.57	12.04	14.26	13.75	22.73	22.34	28.54	28.74	short	short
GR	C13	3.07	1.84	45.33	21.81	1118.68	24.21	44.18	25.44	67.70	21.26	23.18	14.56	20.37	slender	slender
GR	C14	2.27	0.77	16.21	11.88	968.14	20.13	31.24	21.63	35.57	22.38	21.61	16.09	17.18	slender	slender
GR	C15	11.96	6.80	15.65	13.28	936.62	25.53	32.01	30.69	34.38	21.26	22.66	18.45	16.34	slender	slender
GR	C16	0.91	1.54	36.03	22.21	2006.01	41.03	62.33	41.66	76.15	21.34	22.34	10.39	12.80	slender	slender
GR	C17	58.04	55.62	22.29	14.49	1096.39	77.55	36.42	79.97	44.22	21.34	22.34	14.35	17.22	slender	slender
GR	C18	5.07	3.14	24.08	14.02	477.95	12.70	23.58	14.63	33.64	22.38	17.59	24.76	29.73	short	short
GR	C19	6.21	3.38	26.39	12.51	529.34	13.97	23.10	16.80	36.98	22.38	21.61	24.56	30.41	short	short
GR	C20	4.38	5.09	17.63	12.16	493.98	14.26	22.04	14.97	27.51	22.38	21.62	21.88	26.31	slender	short
GR	C21	16.79	9.96	16.05	8.79	937.62	28.71	27.54	35.54	34.80	22.73	21.36	18.95	19.30	slender	slender
GR	C22	5.41	2.79	6.78	4.86	321.18	9.21	11.28	11.83	13.20	22.73	22.34	33.45	30.69	short	short

Appendix 9C reinforcement of column

colmn	location	MEDx	MEDy	Vsd	µsdy	µsdx	ω	As	As, min	As, max	As, prov	NO	Prov
Wt	C14	3.87	35.44	0.0	0.04	0.00	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
Wt	C15	22.85	53.83	0.0	0.07	0.03	0.19	1146.838	315	6300	1146.838	2.537253	4ø24
Wt	C19	5.18	27.77	0.0	0.03	0.01	0.2	1207.198	315	6300	1207.198	2.670792	4ø24
Wt	C20	5.59	62.93	0.0	0.08	0.01	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
fourth	C1	16.59	28.98	0.0	0.04	0.02	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
fourth	C2	70.07	27.27	0.1	0.03	0.09	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
fourth	C3	15.51	9.61	0.1	0.01	0.02	0.7	4225.193	315	6300	4225.193	9.347773	10ø24
fourth	C4	29.98	82.68	0.1	0.10	0.04	0.5	3017.995	315	6300	3017.995	6.676981	7ø24
fourth	C5	42.69	32.16	0.0	0.04	0.05	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
fourth	C6	25.65	43.29	0.1	0.05	0.03	0.2	1207.198	315	6300	1207.198	2.670792	4ø24
fourth	C7	8.82	98.94	0.1	0.12	0.01	0.19	1146.838	315	6300	1146.838	2.537253	4ø24
fourth	C8	16.22	49.38	0.3	0.06	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
fourth	C9	9.39	40.57	0.1	0.05	0.01	1.2	7243.189	315	6300	7243.189	16.02475	17ø24
fourth	C10	47.66	38.41	0.1	0.05	0.06	0.7	4225.193	315	6300	4225.193	9.347773	10ø24
fourth	C11	44.94	37.34	0.1	0.05	0.06	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
fourth	C12	16.69	16.79	0.0	0.02	0.02	0.2	1207.198	315	6300	1207.198	2.670792	4ø24
fourth	C13	9.22	91.36	0.1	0.11	0.01	0.21	1267.558	315	6300	1267.558	2.804332	4ø24
fourth	C14	8.53	34.33	0.1	0.04	0.01	1.2	7243.189	315	6300	7243.189	16.02475	17ø24
fourth	C15	25.50	41.34	0.2	0.05	0.03	1.3	7846.788	315	6300	7846.788	17.36015	18ø24
fourth	C16	12.62	81.62	0.2	0.10	0.02	1.1	6639.59	315	6300	6639.59	14.68936	15ø24
fourth	C17	29.66	67.84	0.1	0.08	0.04	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
fourth	C18	11.31	50.27	0.1	0.06	0.01	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
fourth	C19	7.86	29.74	0.1	0.04	0.01	0.32	1931.517	315	6300	1931.517	4.273268	5ø24
fourth	C20	12.54	64.84	0.1	0.08	0.02	1.1	6639.59	315	6300	6639.59	14.68936	15ø24
fourth	C21	41.66	35.93	0.1	0.04	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
fourth	C22	7.86	17.14	0.0	0.02	0.01	0.23	1388.278	315	6300	1388.278	3.071411	4ø24
Third	C1	16.11	23.45	0.1	0.03	0.02	0.2	1207.198	315	6300	1207.198	2.670792	4ø24
Third	C2	55.86	25.63	0.2	0.03	0.07	0.12	724.3189	315	6300	724.3189	1.602475	4ø24
Third	C3	12.95	7.62	0.1	0.01	0.02	0.11	663.959	315	6300	663.959	1.468936	4ø24
Third	C4	31.63	69.30	0.3	0.09	0.04	0.61	3681.954	315	6300	3681.954	8.145917	9ø24
Third	C5	51.37	40.56	0.4	0.05	0.06	0.45	2716.196	315	6300	2716.196	6.009283	7ø24
Third	C6	26.60	39.61	0.2	0.05	0.03	0.26	1569.358	315	6300	1569.358	3.47203	4ø24
Third	C7	13.88	82.49	0.3	0.10	0.02	0.5	3017.995	315	6300	3017.995	6.676981	7ø24
Third	C8	23.98	51.79	0.5	0.06	0.03	0.7	4225.193	315	6300	4225.193	9.347773	10ø24
Third	C9	13.25	39.68	0.2	0.05	0.02	0.79	4768.433	315	6300	4768.433	10.54963	11ø24
Third	C10	47.16	37.31	0.2	0.05	0.06	0.6	3621.594	315	6300	3621.594	8.012377	9ø24
Third	C11	43.78	35.88	0.2	0.04	0.05	0.5	3017.995	315	6300	3017.995	6.676981	7ø24
Third	C12	16.26	16.08	0.1	0.02	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
Third	C13	12.86	76.52	0.2	0.10	0.02	0.31	1871.157	315	6300	1871.157	4.139728	5ø24
Third	C14	11.70	37.71	0.2	0.05	0.01	0.15	905.3986	315	6300	905.3986	2.003094	4ø24
Third	C15	95.49	108.29	2.2	0.13	0.12	0.2	1207.198	315	6300	1207.198	2.670792	4ø24
Third	C16	19.81	78.70	0.4	0.10	0.02	0.32	1931.517	315	6300	1931.517	4.273268	5ø24
Third	C17	58.21	55.10	0.3	0.07	0.07	0.32	1931.517	315	6300	1931.517	4.273268	5ø24
Third	C18	12.00	38.74	0.1	0.05	0.01	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
Third	C19	10.81	30.50	0.1	0.04	0.01	0.9	5432.392	315	6300	5432.392	12.01857	13ø24
Third	C20	13.22	64.92	0.1	0.08	0.02	0.9	5432.392	315	6300	5432.392	12.01857	413ø24
Third	C21	35.58	34.11	0.2	0.04	0.04	0.79	4768.433	315	6300	4768.433	10.54963	11ø24
Third	C22	8.56	16.09	0.1	0.02	0.01	0.6	3621.594	315	6300	3621.594	8.012377	9ø24

Secod	C1	16.43	25.15	0.1	0.03	0.02	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
Secod	C2	57.29	34.12	0.4	0.04	0.07	0.57	3440.515	315	6300	3440.515	7.611758	8ø24
Secod	C3	12.89	9.05	0.1	0.01	0.02	0.6	3621.594	315	6300	3621.594	8.012377	9ø24
Secod	C4	34.76	73.50	0.4	0.09	0.04	0.25	1508.998	315	6300	1508.998	3.33849	4ø24
Secod	C5	55.95	47.26	0.6	0.06	0.07	0.6	3621.594	315	6300	3621.594	8.012377	9ø24
Secod	C6	28.66	41.50	0.3	0.05	0.04	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
Secod	C7	19.57	87.18	0.4	0.11	0.02	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
Secod	C8	32.68	59.74	0.7	0.07	0.04	0.21	1267.558	315	6300	1267.558	2.804332	4ø24
Secod	C9	17.61	41.70	0.3	0.05	0.02	0.33	1991.877	315	6300	1991.877	4.406807	5ø24
Secod	C10	40.40	40.44	0.3	0.05	0.05	0.36	2172.957	315	6300	2172.957	4.807426	5ø24
Secod	C11	46.27	39.17	0.3	0.05	0.06	0.44	2655.836	315	6300	2655.836	5.875743	6ø24
Secod	C12	17.27	16.57	0.1	0.02	0.02	0.44	2655.836	315	6300	2655.836	5.875743	6ø25
Secod	C13	17.78	82.41	0.4	0.10	0.02	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
Secod	C14	15.02	39.78	0.3	0.05	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
Secod	C15	30.61	38.72	0.3	0.05	0.04	0.7	4225.193	315	6300	4225.193	9.347773	10ø24
Secod	C16	26.96	84.55	0.7	0.11	0.03	0.5	3017.995	315	6300	3017.995	6.676981	7ø24
Secod	C17	75.53	56.22	0.4	0.07	0.09	0.3	1810.797	315	6300	1810.797	4.006188	5ø24
Secod	C18	13.61	41.33	0.2	0.05	0.02	0.2	1207.198	315	6300	1207.198	2.670792	4ø24
Secod	C19	12.89	33.10	0.2	0.04	0.02	0.19	1146.838	315	6300	1146.838	2.537253	4ø24
Secod	C20	14.98	64.96	0.2	0.08	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
Secod	C21	38.85	212.27	0.3	0.26	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
Secod	C22	10.45	16.38	0.1	0.02	0.01	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C1	16.11	29.49	0.2	0.04	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C2	61.44	41.51	0.5	0.05	0.08	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C3	13.98	74.50	0.1	0.09	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C4	37.98	84.00	0.5	0.10	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C5	60.52	56.24	0.8	0.07	0.08	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C6	30.58	44.19	0.4	0.06	0.04	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C7	25.80	99.11	0.6	0.12	0.03	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C8	41.88	70.45	1.0	0.09	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C9	22.10	44.11	0.4	0.05	0.03	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C10	55.80	47.22	0.4	0.06	0.07	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C11	39.85	34.48	0.2	0.04	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C12	17.26	15.25	0.1	0.02	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C13	22.30	96.65	0.5	0.12	0.03	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C14	18.79	41.69	0.4	0.05	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C15	34.16	40.10	0.4	0.05	0.04	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C16	33.92	93.17	0.9	0.12	0.04	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C17	87.09	56.18	0.5	0.07	0.11	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C18	15.49	48.21	0.2	0.06	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C19	14.96	35.19	0.2	0.04	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C20	16.21	65.24	0.2	0.08	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C21	43.50	42.32	0.4	0.05	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
First	C22	13.38	16.28	0.1	0.02	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C1	13.32	21.40	0.2	0.03	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C2	40.92	42.81	0.6	0.05	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C3	10.04	9.99	0.1	0.01	0.01	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C4	33.93	60.22	0.6	0.07	0.04	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C5	54.73	52.54	1.0	0.07	0.07	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C6	26.47	34.57	0.5	0.04	0.03	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C7	29.45	70.75	0.7	0.09	0.04	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C8	48.55	64.90	1.3	0.08	0.06	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C9	24.53	37.76	0.6	0.05	0.03	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C10	41.15	37.25	0.5	0.05	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C11	38.56	36.58	0.5	0.05	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C12	14.26	13.75	0.2	0.02	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C13	25.44	67.70	0.6	0.08	0.03	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C14	21.63	35.57	0.5	0.04	0.03	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C15	30.69	34.38	0.5	0.04	0.04	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C16	41.66	76.15	1.1	0.09	0.05	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C17	79.97	44.22	0.6	0.06	0.10	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C18	14.63	33.64	0.3	0.04	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C19	16.80	36.98	0.3	0.05	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C20	14.97	27.51	0.3	0.03	0.02	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C21	35.54	34.80	0.5	0.04	0.04	0.4	2414.396	315	6300	2414.396	5.341585	6ø24
GR	C22	11.83	13.20	0.2	0.02	0.01	0.4	2414.396	315	6300	2414.396	5.341585	6ø24