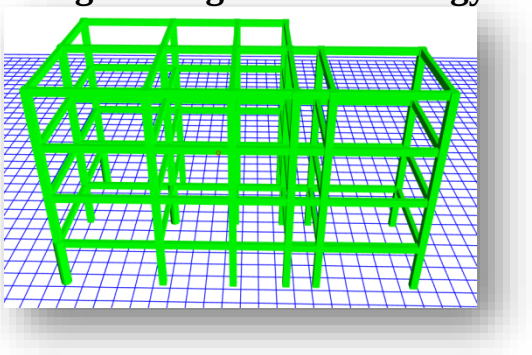
Civil Engineering Department

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G+3 Mixed Used Structural Design

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Preface

This document is a report as a part of fulfillment for Bachelor of Science in civil engineering. The project is conducted by Abiyew AyenewAdege, Mekdelawit Behailu Habte**,**Tamir Kassa Begashaw, Musa kayredin mehammed, Firehiwot Haile Biru and Million Getachow Adugna.

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Abstract/Summary

Our projects are G+3 mixed used structural design. In this project includes roof Analysis and design/wind load, slab analysis and design, Analysis and designearthquake, frame analysis, Analysis and design beam, Analysis and design column analysis and foundation design (ES EN 2015). In addition lateral load and frame analysis by ETABS software.

Acronyms

A- Cross sectional area
Ac-Cross sectional area of concrete
As-Cross sectional area of reinforcement
As-min minimum cross sectional area of reinforcement
Asw-Cross sectional area of shear reinforcement
Fd-Design value of an action
Fk-Characteristic value of an action
Gk -Characteristic permanent action
L-Length
M-Bending moment
N-Axial force
V -Shear force
b-Overall width of a cross-section, or actual flange width in a T or L beam
bw-Width of the web on T, I or L beams
d-Diameter; Depth
d-Effective depth of a cross-section
e-Eccentricity
fc -Compressive strength of concrete
fcd-Design value of concrete compressive strength
fck-Characteristic compressive cylinder strength of concrete at 28 days
fcm-Mean value of concrete cylinder compressive strength
fctk-Characteristic axial tensile strength of concrete
fctm- Mean value of axial tensile strength of concrete
fy-Yield strength of reinforcement
fyd-Design yield strength of reinforcement
fyk-Characteristic yield strength of reinforcement
h-Height
h-Overall depth of a cross-section
r-Radius

Chapter one

1 Introduction

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

A structural design is executed in such a way that the building will remain fit with appropriate degrees of reliability and in an economic way. It should sustain all the actions and influences during execution and use. Therefore, structural design focuses on structural safety and serviceability with due durability. This structural design is executed based on the Ethiopian Building Code of Practice ES EN – 2004: 2015. This code follows the Limit State design approach. Limit state is a state beyond which the structure no longer satisfies the design performance requirements. It consists of two states namely Ultimate Limit and serviceability Limit states. Ultimate Limit states are conditions related with collapse or states prior to structural failure. Its main concern is the safety of structure and people. Serviceability Limit states are those associated to conditions beyond which a structure does not accomplish specified service requirements. It is mainly concerned about the function of construction works, comfort of people, and appearance.

There are many types of buildings which can be classified based on their purpose as

- Residential buildings: buildings that provides the facility for private houses, hotels, dormitories, apartments etc
- Educational buildings: buildings which functions for learning process.
- Institutional buildings: These buildings have built for the purpose of health, recoveries etc
- Assembly building: these buildings are used for accommodation of many peoples together. e.g. recreation, cinema halls, conference halls etc
- Business building: which are used for a business purpose like Banks, private libraries dispensaries etc

- Industrial buildings: are used for fabricating or manufacturing purpose like brick industry, cement factory, steel industry etc
- Storage buildings: buildings which are used for storage purpose.
- Hazard building: are used for to store materials which are dangerous. Like gases, acids, explosive etc

1.1 Back ground of the project

The project is G+ 3 mixed used structural design. This having four floors one ground floor and three other floors. It has three axis in the X direction and six axis in the Y direction. The building comprises shop, bank, bedroom, balcony, bathroom and corridor.

1.2 Objective

The main objective of this B.SC. Project is to:

- Design structure that safe and economical.

This study has the following sub objectives:

- Demonstrate structural analysis and design skill
- Develop structural design software skill
- Understand the new Ethiopian building for structures (ES EN 2015)
- Develop ability to work in diverse team

1.3 Structural Design Philosophy and Method

Over the years, various design philosophies have evolved in different parts of the world, with regard to reinforced concrete design. A 'design philosophy, is built up on a few fundamental premises, and is reflective of the way of thinking.

1) Working stress method This was the traditional method of design not only for reinforced concrete, but also for structural steel and timber design. The conceptual basis of WSM is simple. The method basically assumes that the structure material behaves in a linear elastic manner, and that adequate safety can be ensured by suitably restricting the stress in the material induced by the expected 'working loads' on the structure. As the specified permissible ('allowable') stresses are kept well below the materials strength (i.e., the linear

phase of the stress-strain curve), the assumption of linear elastic behavior is considered justifiable.

2) Ultimate load method (ULM) In this method, the stress condition at the state of impending collapse of the structure is analyzed, and the non-linear stress-strain curves of concrete and steel are made use of. The design stresses used are the ultimate strength of materials and for safety the loads are magnified or scaled up by load factors, defined as the ratio of the ultimate load to working load. The ultimate load method makes it possible for different types of loads to be assigned different load factors under loading conditions, thereby overcoming the related shortcoming of WSM.

3) Limit states method (LSM) Unlike WSM, which based calculations on service load conditions alone, and unlike ULM, which based calculations on ultimate load condition alone, LSM aims for a comprehensive and rational solution to the design problem, by considering safety at ultimate loads and serviceability at working loads. When a structure or structural element becomes unfit for its intended use, it is said to have reached a limit state. The limit states for reinforced concrete structures can be divided into three basic groups.

I. Ultimate limit state. This involves a structural collapse of part or all of the structure. Such limit state should have a very low probability of occurrence, because it may lead to loss of life and major financial losses. The major ultimate limit states are loss of equilibrium of part or all of the structure as rigid body, rupture of critical parts of the structure, progressive collapse, formation of plastic mechanism, instability due to deformations of the structure, and fatigue.

II. Serviceability limits states. These involve disruption of the functional use of the structure, but no collapse occurs. Because there is less danger of loss of life, a higher probability of occurrence is generally tolerated than in case of an ultimate limit state. The major serviceability limit states include excessive deformations, excessive crack widths, and undesirable vibrations. Special limit states. This class of limit states involves damage or failure due to abnormal conditions or abnormal loadings and includes:

a) Damage or collapse in extreme earthquakes,

b) Structural effects of fire, explosions, or vehicular collisions,

c) Structural effects of corrosion or deterioration, and

d) Long term physical or chemical instability (normally not a problem with concrete structures).

Among the three method of design, the limit state method of design (LSM) is the most widely used method of design in many countries building code standard. Our structural building is based on this method of design according to the revised Ethiopian building code standard ES EN 1992:2015adopted from the European building code standard except for typical flat slabs. Flat slab design in ES EN 1992:2015) is based on American concrete institute (ACI).

CHAPTER TWO

2 Roof Analysis and Design

2.1 2.Introduction

Wind produces dynamic loads on a structure at highly variable magnitudes. The variation in pressures at different locations on a building is complex to the point that pressures may become too analytically intensive for precise consideration in design. To simplify the complexity in analysis of wind load different codesprovides specifications for wind load by considering basic static pressure zones on a buildingsrepresentative of peak loads that are likely to be experienced. Wind forces act directly or indirectly on the internal and external surface of structures. Wind loads fluctuate with time. A wind load produces static, dynamic and aerodynamic effects on structures. 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 choice of the two procedures is based on the height of the building and the dynamic coefficient, Cd. Static method or the simplest method of wind load analysis issued for buildings and chimneys less than 200m tall provided that the value of dynamic coefficient. In all other cases the detailed analysis considers the dynamic effect of the load namely dynamic analysis should be applied.

The effect of wind load on the structure depends on many factors such as:

- Wind velocity direction
- The height of the s structure
- Topographic location of the structure
- Shape of the structure
- terrain category
- the roughness of the surrounding

Action of the wind loads on structures is represented either as a wind pressure or a wind force. The action of wind pressure on a structure is assumed to act normal to the surface except otherwise specified; e.g. For tangential friction forces. Wind pressure on the structure may

beexternal wind pressure or internal wind pressure. External wind pressure (Wax) is the wind pressure acting on the external surfaces of a structure and internal wind pressure, is the wind pressure acting on the internal surfaces of a structure.

$Wex = qref * Ce(Ze) * Cpe$

$Wi = qref * Ce(Zi) * Cpi$

Where: = Reference mean wind pressure

Ce(z)= Exposure coefficient that takes into account the influence of terrain roughness.

Cp e = External wind pressure coefficient

Cp i = internal wind pressure coefficient from (Ministry of Ethiopian education, 2015)

First let us determine necessary information

Roof type _ flat roof

Location of the area –W0lkeite

Calculations

Step 1: Determination of basic wind velocity (vb)

$Vb = Cdir * Csea * Vbo$

Cdir=direction factor

Csea=seasonal factor

Vbo=fundamental basic wind velocity

Cdir=Season=1 Vbo=22m/s for Ethiopia

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

Step2: Determination of mean wind velocity, vm (z)

$Vm(z) = Cr(z) * Co(z) * vb$

Where; Co (Z) =orographic factor

Orographic factor is calculated by the following formula under different condition

$(z) = 1 \quad \varphi < 0.05$

$(z) = 1 + 2 ** \varphi for\ 0.05 << 0.3$

$(z) = 1 + 0.6 ** \varphi fo > 0.3$

Where, S – is the Orographic location factor

Φ- Is the upwind slope, Φ=H/LU in the wind direction

LU- is the actual length of the upwind slope

H-is the effective height

Wind pressure Coefficients

External wind pressure coefficient; External pressure coefficients give the effect of the wind on the external surfaces of buildings.

Internal wind pressure Coefficient; internal pressure coefficients give the effect of the wind on the internal surfaces of buildings.

External Pressure coefficients

Note: The loaded area is the area of the structure, which produces the wind action in the section to be calculated. For buildings with the loaded area between lm2and 10m2.the following procedure is recommended for determining the external pressure coefficient.

From ES-EN 1991, 1-4 Variation of external pressure coefficient for buildings with size of the loaded area is based on the following:

A ≤ l m2, Cpe = Cpe1

1 $m2 < A <$ 10$m2$, $Cpe = Cpe1 + (Cpe10 - Cpe1) \log 10A$

$A \geq 10m2$, = $Cpe10$

The values of $Cpel$ and $Cpel0$ are given on (Ministry of Ethiopian education, 2015) orthogonal wind directions 0°, 90°,180° but represent highest values obtained in a range of wind direction 0 = ± 45 °either side of the relevant orthogonal direction. These values are only applicable to buildings.

Internal Pressure coefficients

Internal pressure arises dueto openings, such as windows, doors and vents, in the cladding. In general, if the windward pane has a greater proportion of opening than the leeward panel, then the interior of the structure is subjected to positive (outward) pressure.

Conversely, if the leeward face has more openings, then the interior is subjected to a negative (inward) pressure. Like external pressure internal pressure is considered positive when acting on to the surface of the structure. Thus, internal pressure is positive when the pressure acts outward.

Net Pressure: The net wind pressure across a wall or an element is the difference of the pressures on each surface taking due account of their signs. (Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative.

2.1.1 Wind load analysis of Roof

Generally, we should follow the following steps in analyzing wind loads:

- Determine site design wind speed and basic velocity pressure
- Identify the roof type
- Determine wind pressure coefficients namely external wind pressure coefficient and internal wind pressure coefficients
- Determine net wind pressure coefficient
- Determine the design or net wind pressure from step above
- Determine the wind force
- Load combination

Design information

The building is located in Wolkite city. Therefore, according (Ministry of Ethiopian education, 2015)

the area is categorized according to the following:

Terrain Category: According to (Ministry of Ethiopian education, 2015)the location of the building in wolkite fall under Terrain Category- I I. Category- II is characterized with

- Roughness length, Z0 = 0.05m
- Minimum height, Zmin = 2m

Topography coefficient: no escarpment or hills are located around the building site which means the site is topography unaffected zone and therefore,

Ct (z) = 1 according to terrain condition

Height of the building =15.80m

Width of the roof= 13.961m

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

$Cr\ (z) = krLn\ (z/zo), \quad for\ zmin \leq Z \leq 200m$

$Cr\ (z) = Cr(zmin), \quad forz<zmin$

Where Z0 is the roughness length and Zmin is the minimum height. Both Z0 and Zmin are dependent on the ground roughness and are given in Table 4.1.

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

$Cr\ (z) = ktLn\ (z/zo)$, but

Kr=0.19*(Zo/Z0,I) ^0.07

Calculation of mean wind velocity (Vm(zo))

Vm(z) = Cr(z)*Co(z)*Vb

Z= 15.80

For terrian category II

Zmin= 2

Zo= 0.05

Zo,I= 0.01

Roughness coefficient Cr(z)

Cr (z) = Krln(z/zo) for Zmin< Z <Zmax

Kr = 0.19*(zo/zo,I)^0.07

= 0.19*(0.05/0.01)^0.07

= 0.213

Cr(z)= 0.213ln(15.80/0.05) =1.22

Orographic factor (Co (z)

Co(z)= 1 , Ø<0.05

Basic wind velocity Vb=22m/s,recommended value

Vm(z)= Cr(z)* Co(z)* Vb= 1.22*1*22= 26.84m/s

Altitude

Temperature and Barometer

But ES EN1991-1-4-2015 recommended as 1.25kg/m3

Calculation of peak velocity pressure (qp (z))

qp(z) = [1 + 7Iv(z)] ∗1/2∗ρair∗ vm2(z)

ρair=density of air, it is depend on :- Altitude Temperature and Barometer But ES EN1991-1-4-2015 recommended as 1.25kg/m3

Wind terbulance (lv(zo)

lv= k1/[Co(z)ln(z/zo)] for Zmin< Z <Zmax=200m

Where, K1 is turbulences factor, recommended value is 1.0

Lv = 1/[1*ln(15.80/0.05)]

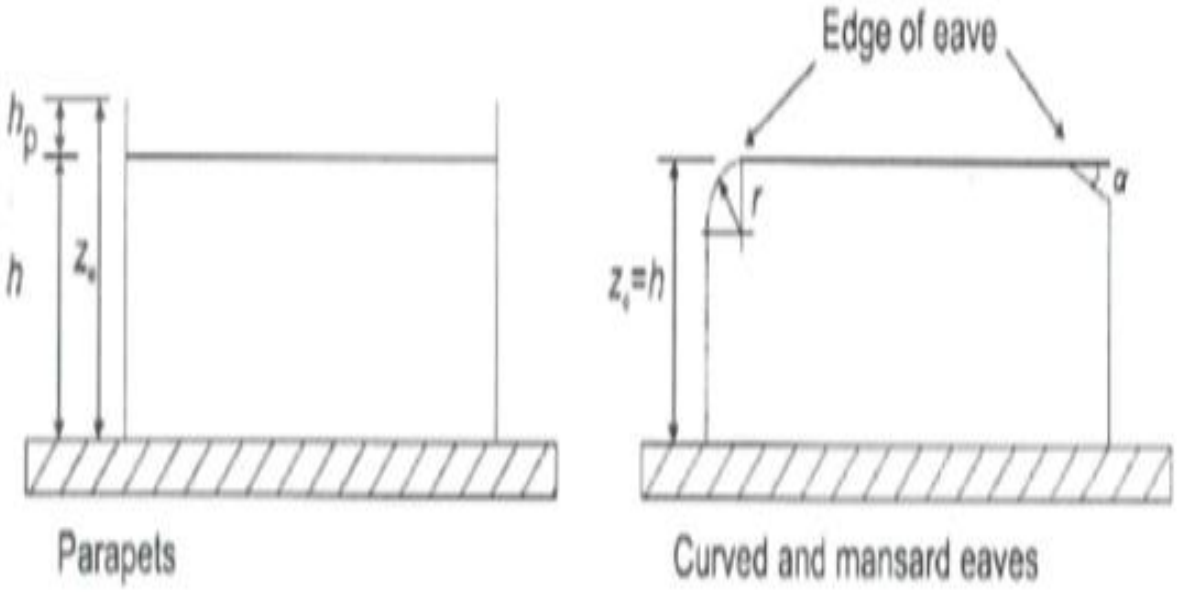
= 0.179

qp(15.80) = [1+7*0.179]*1/2*1.25*26.84^2

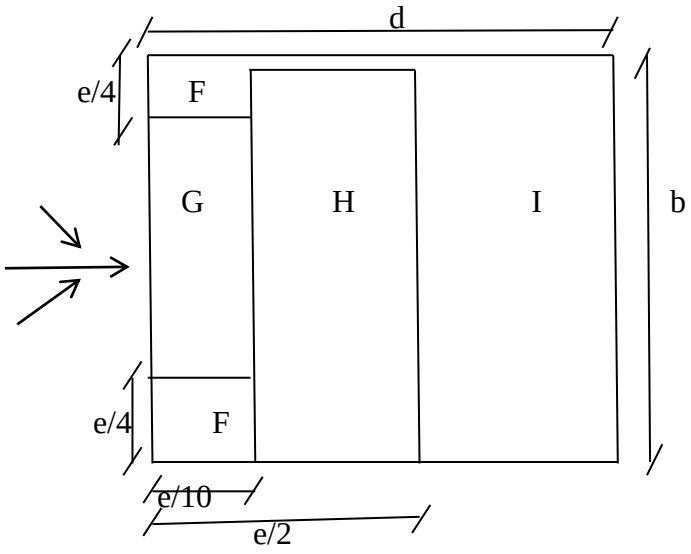
= 0.642 Kpa

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+hp



Case 1



reference height ze=h
e=b or2h
b:crosswind direction
d=22.955
b=13.961
e/10=1.3961

e/2=6.98
e=13.961
e/4=3.49

hp =1.05 hp/h = 0.081 H=13

2.1.2 Wind pressure computation

Table 2.1Determination of External and Internal Pressure Coefficient

Region	F		G		H		I	
hp/h	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10
0.05	-2	-1.4	-1.6	-0.9	-1.2	-0.7	-0.2	-0.2
0.10	-1.2	-1.8	-0.8	-1.4	-0.7	-1.2	-0.2	-0.2
0.081	-1.74	-1.53	-1,34	-1.06	-1.04	-0.86	-0.20	-0.20

We = Qref * Cpe

Table 2.2Computation of External Wind Pressure

Zone	peak velocity Pressur qp(z)	Exter. Press. Coeff. Cpe	Ext. Wind Press. (KN /m2)
F	0.642	-1.53	-0.982
	0.642	0.00	0.00
G	0.642	-1.06	-0.680
	0.642	0.00	0.00

H	0.642	-0.86	-0.552
	0.642	0.00	0.00
I	0.642	-0.20	-0.128
	0.642	0.20	0.128

Critical Pressures are -0.982

0.128

We = Qref * Cpi

Table 2.3Computation of External Wind Pressure

Zone	peak velocity Pressur qp(z)	Inter. Press. Coeff. Cpe	Inter. Wind Press. (KN /m2)
F	0.642	0.8	0.514
	0.642	-0.5	--0.321.
G	0.642	0.8	0.514
	0.642	-0.5	--0.321
H	0.642	0.8	0.514
	0.642	-0.5	--0.321
I	0.642	0.8	0.514
	0.642	-0.5	--0.321

Critical Pressures are -0.321

0.514

Computaion of Net Wind Pressure

The net wind pressure is given by the sum of the internal and external pressure.

Wnet = We + Wi

- from the computation of external pressure above, the critical loads are

Zone	External Pressure
F	-0.982
I	0.128

Hence, the net pressure with reference to the external pressure

Table 2.4Net pressure with reference to the external pressure

Zone	External Press	Internal Press.	Net Press.
	We(KN/m2)	Wi(KN/m2)	Wnet = We - Wi
F	-0.982	-0.321	-0.661
I	0.128	0.514	-0.386

Determination of area for case 1

Area of F=3.49*1.396=4.87m2

Area of G= 1.396*6.981=9.75m2

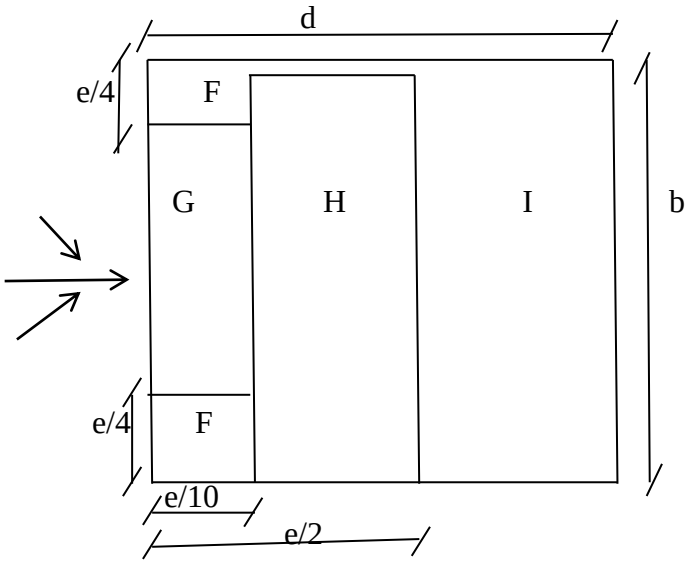
Area of H=5.584*13.961=77.956m2

Area of I=16.035*13.961=223.86m2

Table 2.5Computation of External Wind Pressure

Zone	Area	W net	WL
F	4.87	-o.661	-0.135
G	9.75	-0.661	-0.068
H	77.95	-0.661	-0.0085
I	223.86	-0.661	-0.00295

CASE 2



reference height ze=h
e=b or2h
b:crosswind direction
d=13.961
b=22.955
e/4=5.739

e= 22.955
e/2=11.4775
e/10=2.295

Table 2.6Determination of External and Internal Pressure Coefficient

Region	F		G		H		I	
hp/h	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10	Cpe1	Cpe10
0.05	-2	-1.4	-1.6	-0.9	-1.2	-0.7	-0.2	-0.2
0.10	-1.2	-1.8	-0.8	-1.4	-0.7	-1.2	-0.2	-0.2
0.066	-1.74	-1.53	-1,34	-1.06	-1.04	-0.86	-0.20	-0.20

We = Qref * Cpe

Table 2.7Computation of External Wind Pressure

Zone	peak velocity Pressur	Exter. Press. Coeff.	Ext. Wind Press. (KN /m2)
	qp(z)	Cpe	
F	0.642	-1.53	-0.982
	0.642	0.00	0.00
G	0.642	-1.06	-0.680
	0.642	0.00	0.00
H	0.642	-0.86	-0.552
	0.642	0.00	0.00
I	0.642	-0.20	-0.128
	0.642	0.20	0.128

Critical Pressures are -0.982

0.128

We = Qref * Cpi

Table2. 8Computation of External Wind Pressure

Zone	peak velocity Pressur qp(z)	Inter. Press. Coeff. Cpe	Inter. Wind Press. (KN /m2)
F	0.642	0.8	0.514
	0.642	-0.5	--0.321.
G	0.642	0.8	0.514
	0.642	-0.5	--0.321
H	0.642	0.8	0.514
	0.642	-0.5	--0.321
I	0.642	0.8	0.514
	0.642	-0.5	--0.321

Critical Pressures are -0.321

0.514

Computaion of Net Wind Pressure

The net wind pressure is given by the sum of the internal and external pressure.

Wnet = We + Wi

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

Zone	External Pressure
F	-0.982
I	0.128

Hence, the net pressure with reference to the external pressure

Table 2.9Net pressure with reference to the external pressure

Zone	External Press	Internal Press.	Net Press.
	We(KN/m2)	Wi(KN/m2)	Wnet = We - Wi(KN/m2)
F	-0.982	-0.321	-0.661
I	0.128	0.514	-0.386

Determination of area for case 2

Area of F=5.438*2.2955=12.48m2

Area of G= 2.2955*11.479=26.35m2

Area of H=11.479*9.18=105.377m2

Area of I=2.48*22.922=57.02m2

Table 2.10Determination of wind load

Zone	Area	W net	Wind Load
F	12.48	-o.661	-0.053
G	26.35	-0.661	-0.025
H	105.377	-0.661	-0.0063
I	57.02	-0.661	-0.012

2.1.3 TOTAL DESIGN LOAD

2.1.3.1 DEAD LOAD [GK] FOR ROOF SLAB

RC Slab [150mm]=0.15*25=3.75KN/m2

Cement screed[30mm]=0.03*23=0.69KN/m2

marble[20mm]=0.02*27=0.54KN/m2

Plastering/mortar[20mm]=0.02*23=0.46KN/m2

Total dead load Gk= 5.44 KN/m2

Live load [Qk] for roof terries slab

Qk=0.4 KN/m2 for C 25 – Category Terrase

2.1.4 Roof LOAD COMBINATION

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

= (1.35*5.44) + (1.5*0.4) + (0.9*(-0.135)) =7.82KN/m2

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

= (1.35*5.44) + (1.5*0.4) + (0.9*(-0.068)) =7.88KN/m2

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

= (1.35*5.94) - (1.5*0.4+ (0.9*(-0.0085))=7.41KN/m2

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

=(1.35*5.94)+(1.5*0.4+ (0.9*(-0.012))= 8.6KN/m2

Therefore, the governing combination is combination- I

CHAPTER THREE

3 Analysis and Design of slab

3.1 Overall process in design of slab

A reinforced concrete slab is a broad, flat plate, usually horizontal, with top and bottom surface parallel or nearly so. It is used to provide flat surface mainly for roofs and floors of building parking lots, airfields, road ways, bridges etc. It may be supported by reinforced concrete beam, by masonry or reinforced concrete walls, by structural steel members, directly by columns, or continuously by the ground. But we are intending to design beam supported floor and roofs solid slabs of the building accordingly ESEN1992:2015

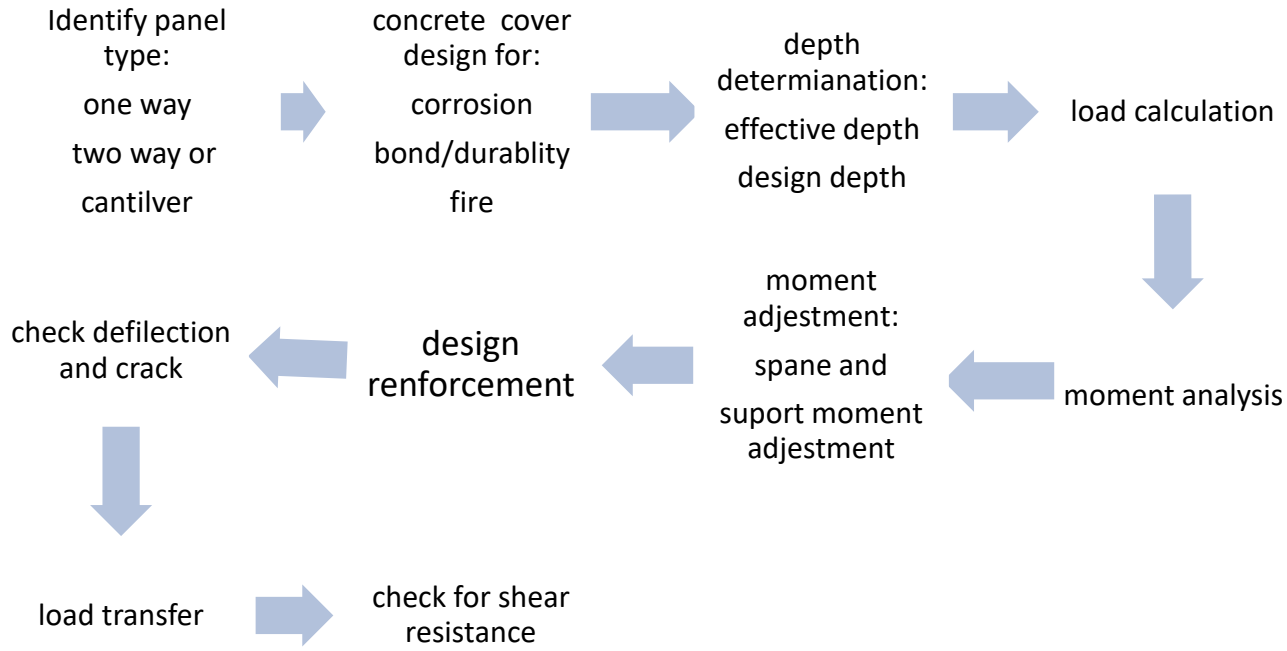


Figure 3.1ver all slab design procurers

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

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

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

Material selected to design the slab structure

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

Design strength value

Concrete

ϒc=1.5

$fcd = \alpha cc * fck/\gamma c$= 0.85∗20/1.5= 11.33MPa

fctd=1.03mpa

Elastic modulus of concrete, Ec=29Gpa

Steel

ϒs=1.15

$fyd = fyk/\gamma s$= 400/1.15= 347.83MPa

Elastic modulus of steel ,Es=200Gpa

Determine Geometry/Dimension

Consider one meter strip width, b=1000mm

D=effective depth (d) +cover + Ø /2

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

3.2 First floor slab

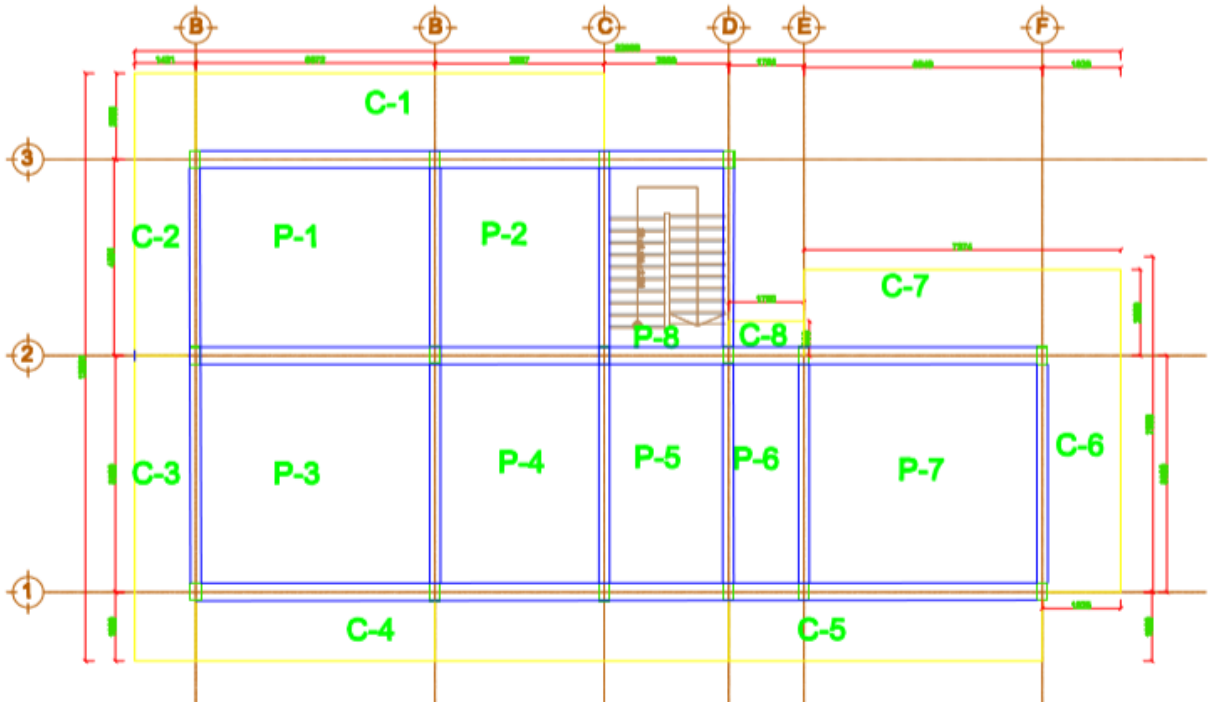


Figure 3.2 First floor plan lay out

Concrete cover design

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

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

Cnom=Cmin+ΔCdev

Where, C min – minimum cover.

ΔCdev is allowance in design for deviation.

Minimum cover, shall provide in order to ensure

- Safe transmission of bond force
- Corrosion resistance/ Durability
- Fire resistance

Cmin, b

Cmin=max { Cmin, dur+ΔCdur, ϒ-ΔCdur, st-ΔCdur,add
10mm

Where;

Cmin, b -minimum cover due to bond requirement,(ministry of Ethiopian education, 2015)

Cmin,dur - minimum cover due to environmental conditions,(ministry of ethiopian education, 2015)

ΔCdur,γ- additive safety element,(ministry of ethiopian education, 2015)

ΔCdur,st -reduction of minimum cover for use of stainless steel,

ΔCdur,add -reduction of minimum cover for use of additional protection(ministry of ethiopian education, 2015)

Cover Design for Corrosion

The condition of exposure is assumed to be XC1

ForXC1 indicative strength class is C-20/25

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 (ministry of ethiopian education, 2015)

Exposure class is reducing by 1 and the structural class would be S3.

Cover Design for Bond /Durability

In order to transmit bond forces safely and to ensure adequate compaction of the Concrete, the minimum cover should not be less than Cumin,

Cmin, b= diameter of bar

Assume Φ10 longitudinal bar and Φ25 nominal maximum aggregate size;

Therefore; Cmin, b=10mm.

Note: If nominal maximum aggregate size is greater than 32mm Cmin, b should be increased by 5mm

The value of minimum cover required for durability of reinforcement steel is determined by using(ministry of ethiopian education, 2015)

Using structural class, S3 and XC1:

Cmin, b=10mm

Cmin= max { Cmin, dur=10mm ; 10mm } Cmin=10mm

ΔCdev (allowance in Design for Variation)

Note: The recommended value of ΔCdev is 10 mm.

Then Cnom=Cmin+ΔCdev=10+10=20mm

Cover Design for Fire

For the slab to sustain fire incident for 120 minutes the required cover and minimum height of the Section can be determined form

For slab P-1

L_Y/L_X =5.572/4.561=1.22, so it is two way and $L_Y/L_X \leq 1.5$

Therefore Cfire =20 mm

Hs = 120mm

Governing concrete Cover for Design

Governing concrete cover accounting for

Corrosion

Bond/Durability

Fire

Cover = 20mm

Similarly, the Governing Concrete Cover for the others slabs……. **Appendix 3A**
From above table the maximum Governing concrete cover is 40mm

3.2.1 Depth Determination (deflection criteria)

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

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

l/d=K(11+1.5$\sqrt{fck}*\rho o/\rho$+3.2*$\sqrt{fck}(\rho o/\rho$-1)^(3/2))*F1*F2*F3 …….if$\rho<\rho o$

art.7.4.2.(7.16a)

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

art.7.4.2.(7.16b)

Where;

l/d - is the limit span/depth

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

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

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

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

For flat slabs with spans ≤ 8.5 m and slab and beams with spans ≤ 7 m:

F1=500/ σs =500/(fyk*As,req/As,pro)

$F2$=As,/As,req< 1.5 $for first trial$: $use F2$=1

$F3$= 1 [section, solid slab and T section with b/bw≤ 3]

$F3$= 0.8 for T section with b/bw> 3

Assumption

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

As,req = As, provided.

l/d=K*N*F1*F2*F3

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

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

N=11+1.5$\sqrt{fck}$=11 + 1.5√20=17.71

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

F2=1(because Assure = As, provided) and F3 = 1 (because our slab is rectangular)

Basic ratios of span/effective depth for reinforced concrete members without axial compression(ministry of Ethiopian education, 2015)

Table3.11Basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural System	K	Concrete highly stressed ρ = 1,5%	Concrete lightly stressed ρ = 0,5%
Simply supported beam, one- or two-way spanning simply supported slab	1,0	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

Example for slab P-1

It is interior span of beam or one-way slab or two-way spanning slab, K=1.5

l/d=K*N*F1*F2*F3=1.5*17.71*1.25*1*1=33.21

Depth, D=d+d' Where: d' = cover +Φ/2 use Φ

d' = cover +Φ/2= 40mm +10mm/2 = 45mm

l/ d =33.21mm but $l = lx$

l=lx=4561 mm

d=4561/33.21=137.34mm

Similarly, for the rest panel's effective depth will be calculated as follows……**Appendix 3B**
From the above effective depth calculation let us take the governing depth for design:

Use diameter of bar Ø10mm

The governing effective depth is d=165.63mm

D=concrete cover + d + Ø/2=40+165.63+10/2=210.63mm

Use the overall depth, D=220mm

3.2.2 Analysis and design of Load

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

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

Dead and live loads are calculated depending on the service of the slabs and self- weight.

Ignoring any localized effects caused by concentrated

Load, the partition loads are distributed over the area of the slab. The design loads are factored according to the following formula.

Pd = 1.35DL + 1.5LL

Where:

Pd - total factored design load

DL - total dead load on slab

LL - total live load on slab

Dead load (DL)

According to Euro code construction material reference is as follows(Ministry of Ethiopian education, 2015)

Table 3.12The nominal density of construction materials

Materials		Density(ϒ)KN/m^3
Concrete		25
Cement screed(mortar)		23
plastering(mortar)		23
Partions wall		14
Floor finish materials	PVC	16
	Ceramic	23
	Porcelain	25

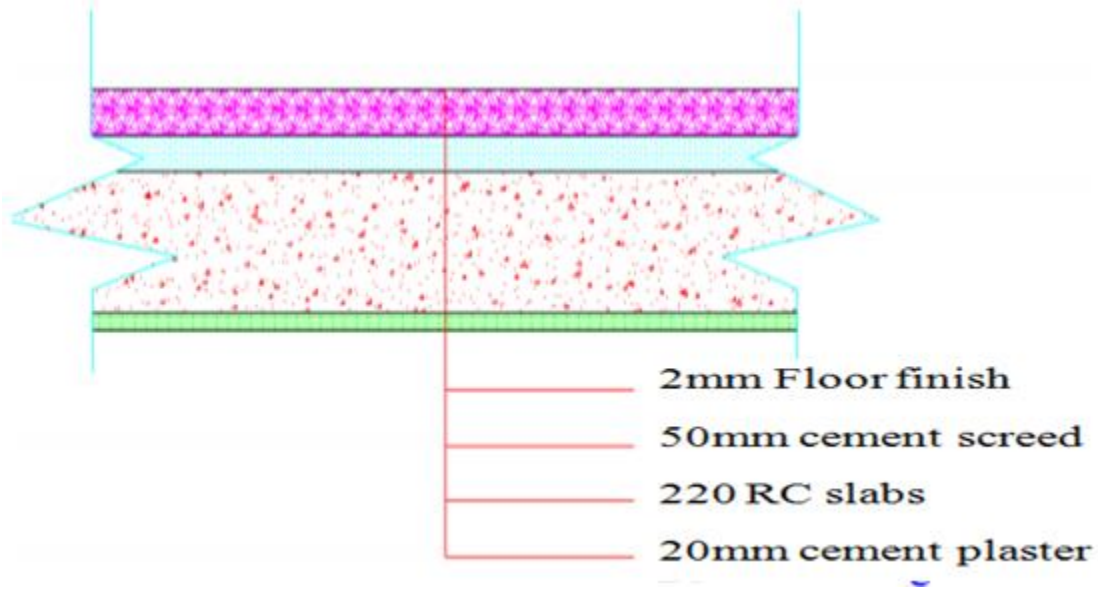


Figure 3.3Slab Crosse section view

Example: - dead load For slab P-1

RC slab=0.22*25=5.5KN/m^2

ff-PVC=0.002*16=0.032KN/m^2

Cement screed=0.05*23=1.15KN/m^2

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

Partition wall = h ∗ l ∗ w ∗ϒc/ A, but this slab have not Partition wall

Total dead load=7.142 KN/m^2

Similarly for the rest panel's dead load (DL)……..**Appendix 3C**

Live load (LL)

The imposed load according to Eur(1995) code

Table3. 13Imposed loads

Function	category	Live load
Hotel	category C3	5
Bath room/toilet	category A	2
Balcony	category A	4
Verandah	category C3	5
Shop	category D1	5
Bed room	category A	2

Kitchen	category A	2
Shower room	category A	2
Corridor	category C3	5

Combining all the above loads, our design load would be:

Pd = 1.35DL + 1.5LL

Example: - design load For slab P1

Pd = 1.35DL + 1.5LL

=1.35(7.142) + 1.5(5)

=17.142 KN / m^2

The same as the other panels design loads for each……**Appendix 3D**

3.2.3 Design Moment 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.

- Dead load to Live load should be less than 1.25 [$DL/LL \leq 1.25$]

Uniformly distributed load

Rectangular slab

No large openings

Supported by beam in directions

Mi=αiPdLx2

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

αi = the coefficient given in

Pd = the design load

Lx = the shorter span of the of the panel

Subscripts for moments and moment coefficients, αi have the following meanings:

S, support

f, field (span)

x -direction of shorter span

y direction of longer span

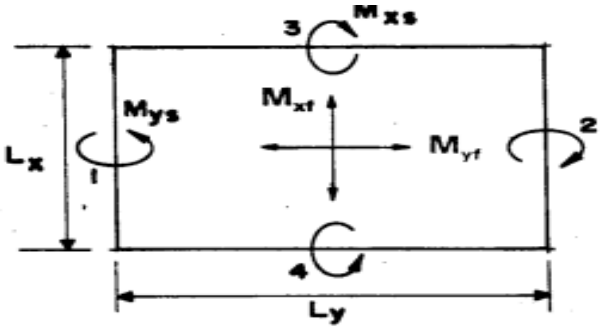
Notations for different critical moments and edge numbers are shown in Figure below

Mxs=αxsPdLx^2

Mxf=αxfPdLx^2

Mys=αysPdLx^2

Myf=αyfPdLx^2



Let us do of slab P-1 or sample and others are done by excel.

Checking if we can use coefficient method for analysis:

Qk/Gk=5/7.142= 0.7 < 1.25……0k!

$Qk \leq 5KN/m2$…….0k!

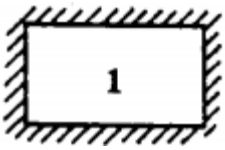
Uniformly distributed load ……0k!

Rectangular slab ……0k!

No large openings ……..0k!

Supported by Beam in all Edges…… 0k!

Support condition one (1)



Ly=5.572m, Lx=4.561m

Ly/Lx=5.572m /4.561m = 1.222 ≤ 2 Two slab

Ly/Lx	αxs	αxf
1.2	0.042	0.032
1.22	αxs	αxf
1.3	0.046	0.035

By using interpolationαxs=0.0428and αxf=0.0326

$$\alpha yf = 0.024 \; and \; \alpha ys = 0.032$$

$Mxs = axs*pd*Lx^2$, $Mxf = axf*pd*Lx^2$

$Mys = ays*pd*Lx^2$, $Myf = ayf*pd*Lx^2$

Mxs=0.0428*17.1417*4.561^2=15.26KNm

Mxf=0.0326*17.1417*4.561^2=11.62KNm

Mys=0.032*17.1417*4.561^2=11.41KNm

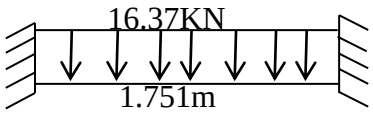
Myf=0.024*17.1417*4.561^2*=8.56KN

Moment analysis for the other two way slabs……**Appendix 3E**

For panel P-6 and P-8, For cantilever C-1,C-2,C-3 ,C-4,C-5,C-6,C-7 and C-8 **:**moment calculation ,LY/LX>2 ,Since the slab is one way it is designed by taking 1m width strip in shorter direction & reinforcement is provided in shorter direction.

3.2.4 Design Moment analysis for one way slab

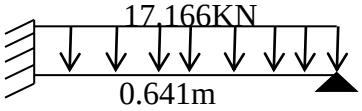
P-6



$Mxs=\frac{pdLx^2}{12}=\frac{16.37*1.751^2}{12}$ =4.18KN/m

$Mxf=\frac{pdLx^2}{24}=\frac{16.37*1.751^2}{24}$ =2.09KN/m

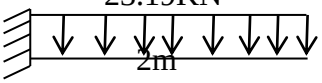
P-8



$Mxs=\frac{9wL^2}{128}=\frac{9*17.166*0.641^2}{128}$ =0.49KN/m

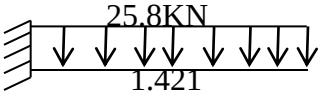
$Mxf=\frac{pdLx^2}{8}$ = $Mxf=\frac{17.166*0.641^2}{8}$ =0.88KN/m

C-1



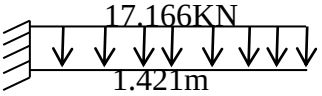
$Mxs=\frac{pdLx^2}{2}=\frac{23.19*2^2}{2}$ =46.38KN/m

C-2



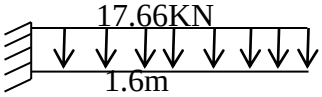
$Mxs=\frac{25.8*1.421^2}{2}$ =26.05KN/m

C-3



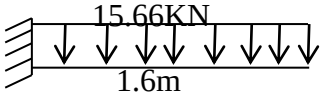
$Mxs=\frac{17.166*1.421^2}{2}$ =17.33KN/m

C-4



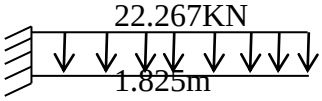
$Mxs=\frac{17.66*1.6^2}{2}$ =21.97KN/m

C-5



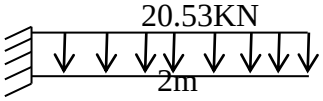
$Mxs=\frac{15.66*1.6^2}{2}$ =20.05KN/m

C-6

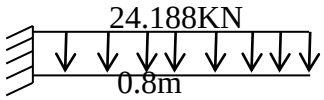


$Mxs=\frac{22.267*1.825^2}{2}$ =37.08KN/m

C-7



$Mxs=\frac{20.53*2^2}{2}$ =41.06KN/m

C-8 

Mxs=$\frac{24.188*0.8^2}{2}$ =7.74KN/m

3.2.5 Adjustment of moment

3.2.5.1 Support moment adjustment

The moment between two panels support should be adjusted in order to decrease the torsionEffect on the beam and to provide the best economical negative reinforcement. To adjust the support Moment there are two cases check.

- Case (1) If $\%\Delta Mxs = Mlarg - Msmall/Mlarge * 100 < 20\%$ then we take

$Md = Mlarge + Msmall/2$……..is called average method

- Case (2) If $\%\Delta M = Mlarg - Msmall/Mlarge * 100 > 20\%$ ……by using kanis method

We did the adjustment with tabula form of table, but as example see the adjustment of support moment between p-1& p-2 slab panels.

Fo r p-1Mxs = 15.26KNM

L_1 = 5.572m

K_1 = 1/ 5.572=0.179

Fo r p-2 Mxs = 15.27KNM

L_2= 3.937m

K_2 = 1/ 3.937=0.254

From the two panels maximum moment isp-1Mxs = 15.27KNM

NOW, ΔMxs = $Mxs_{large} - Mxs_{small}$

= 15.27 –15.26 = 0.01KNM

$\%\Delta Mxs = Mxs_{large} - Mxs_{small}/Mxs_{large}$=0.01/ 15.27* 100 = 0.06% < 20%

So we can take the average to adjestp-1& p-2

Md =$Mxs_{large} + Mxs_{small}/2$=15.27+15.26 /2=15.265KNm

Similarly adjustment between p-1& p-3

Fo r p-1Mxs = 15.26KNM

L_1= 4.561m

K_1 = 1/ 4.561=0.219

Fo r p-3 Mxs = 20.19KNM

L_2= 5.5m

K_2= 1/ =0.18

From the two panels maximum moment isp-3 Mxs = 20.19KNM

NOW, ΔMxs = Mxs_{large}–Mxs_{small}

= 20.19– 15.26= 4.93KNM

%ΔMxs = Mxs_{large}–Mxs_{small}/Mxs_{large}=4.93/20.19∗ 100 = 24.4% >20%

Using relative stiffness

Md =Mxs_{large} – (K_1/K_1+K_2) ∗ ΔMxs

=20.19 – (0.219/0.219+0.18)∗ 3.678 =17.489KNm

Similarly Support moment adjustment for the other panels……**Appendix 3F**

Note that if adjustment is between two-way and cantilever,thenMd = $Mmax$, means the design moment is the maximum moment one of them.

P1andC1:- P1=15.26KNm
C1=46.38KNm
} Mxs(maximum moment) =46.38KNm similarly

P2andC1=46.38KNm

P1andC2=26.05KNm

P3andC3=20.19KNm

P3andC4=21.97KNm

P4andC4=21.97KNm

P5andC5=20.05KNm

P6andC5=20.05KN

P7andC5=20.05KN

P7andC6=37.08KN

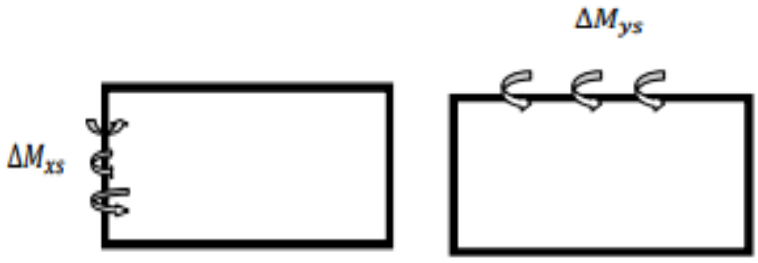
P7andC7=41.06KN

P6andC8=7.74KN

3.2.5.2 Span moment adjustment

Span moments are adjusted only in the direction in which support moment is decreased (reduced) during the support moment adjustment. Because the decreased moment must distributed to the field.

Note that if Ly/Lx≤ 0 neglect spans moment adjustment



$Mxfd = Mxf + Cx\Delta M$ $Myfd = Myf + Cy\Delta M$

$Mxfd$ = C$x1\Delta Mxs + Cx2\Delta Mys + Mxf$, from table A-12

$MYfd = Cy1\Delta Mxs + Cy2\Delta Mys + Myf$, from table A-12,

ΔMxs= Mxs**(**unadjusted**)**-Mxs**(**adjusted**)** ΔMxf= Cx1***ΔMxs+Cx2***ΔMys

ΔMys= Mys**(**unadjusted**)**-Mys**(**adjusted**)**ΔMyf= Cy1***ΔMxs+Cy2***ΔMys

Mxf**(**adjusted**)**=mxf**(**previous**)**+ΔMxfMyf**(**adjusted**)**=Myf**(**previous**)** + ΔMyf

For example for panel (p1)

Mxs (unajustd)= 15.26KNmMys(unadjusted) =11.41KNm

Mxs,(adjusted) = 15.265KNm Mys,(adjusted) = 46.38KNm

Mxf = 11.62KNm Myf = 8.56KNm

ΔMxs= Mxs**(**unadjusted**)**-Mxs**(**adjusted**)**ΔMys=Mys**(**unadjusted**)**-Mys**(**adjusted**)**

= 15.26- 15.265 =11.41-46.38

= -0.005KNm = -34.97KNm

Ly = 5.572m Ly/Lx=1.22

Lx = 4.561m

Cx1=0.335 Cx2=0.349

Cy1=0.165 Cy2=0.361

$Mxfd$ = C$x1\Delta Mxs + Cx2\Delta Mys + Mxf$

= 0.335*(-0.005) +0.349*(-34.97) + 11.62= 0.59KNm

$Myfd$ = Cy1∗ ΔMxs + Cy2∗ ΔMys + = 0.165**(-0.005) +0.361*(-34.97)+8.56 =4.06KNm

Now using similar procedure we can adjust other span moment

Other span moment adjustment……**Appendix 3G**

3.2.6 Slab REINFORCEMENT

Reinforcement design for First floor slab

Designing the slab for flexure

We use the Design Table method to calculate the area of reinforcementrequired for the design moment.

The spacing of bars should not exceed Smax,slab From ES EN 1992 - 1 -1 : 2004

Note: For the value of Smax,slab, The recommended value is:

- For the principal reinforcement, 3D≤400mm, where D is the total depth of the slab;
- For the secondary reinforcement, 3.5D≤450mm

For example: panel (P-1)

Provision of principal reinforcement

,d = D –cover -0.5ϕbar = 220- 40 -0.5(10) = 175 mm

Design moment, Mxf =0.58 KN/m

Km = √$Mmax$* 10^6/$bd2$= √0.58* 10^6/1 ∗ 175^2= 4.35

Using corresponding value of Km take Ks value from design table 1a ES EN -1992-2015

Km=4.35, Ks=2.96

$Ascalc$ =$Ks∗Mmax/d$=2.96∗ 0.58∗ 1000/175=9.81mm^2

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

$fctm$ = 0.3(Fck)(2/3) = 0.3(400)^(2/3) = 2.2

$Asmin$ = Max {0.26$fctmbtd/fck$=0.26 ∗ 2.2 ∗ 1000 ∗ 175/400= 250.25mm2

$ESEN$1992: 2015 Art. 9.2.1.1(1)

0.00013btd = 0.0013 ∗ 1000 ∗ 175 = 227.5mm^2

$Asreq$ = $MaxAscalc$ =9.81mm^2

$Asmin$ =250.25mm^2

Ast, max=0.04*Ac……(ministry of ethiopian education, 2015)

Ast,max=0.04*80*175mm2=560mm2>9.87mm2…….ok

$Asreq$ =250.25mm^2

as= πd^2/4=3.14∗ 10^2/4= 78.5mm^2

$Spacing$ =$b∗as/Asreq$=1000 ∗ 78.5/250.25=310mm

For slab maximum spacing is defined.

$Sprovide = min3D = 3 * 220 = 660mm$

$400mm$

$Spacing$ =310 mm

Provide, Ø10 C/C 310mm

Provision of secondary reinforcement

Section 9.3.1.1(2) and Section 9.3.1.1(3) of ES EN 1992-1-1; provide information about secondary transverse reinforcement.

According to Section 9.3.1.1(2) of ES EN 1992-1-1, Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slab

d = D –cover -3*ϕbar/2 = 220- 40 -3*(10/2) = 165 mm

Design moment, Myf =4.065KN/m

Km = √M*10^6/$bd2$= √ 4.065∗ 10^6/1 ∗ 165^2= 12.22

Using corresponding value of Km take Ks value from design table 1a ES EN -1992-2015

Km=12.22, Ks=2.96

$Ascalc$ =Ks*$Mmax/d$= 2.96∗ 4.065∗ 1000/165= 72.92mm^2

$fctm$ = 0.3(Fck)(2/3) = 0.3(400)^(2/3) = 2.2

$Asmin$ = Max {0.26$fctmbtd/fck$=0.26 ∗ 2.2 ∗ 1000 ∗ 165/400= 235.95mm^2

0.00013btd = 0.0013 ∗ 1000 ∗ 165 = 214.5mm^2

$Asreq$ = $MaxAscalc$ = 72.92mm^2

$Asmin$ =235.95 mm^2

Ast, max=0.04*Ac……ES EN 1992.2015 Art.9.2.1.1 (3)

Ast,max=0.04*80*165mm2=528mm2>389.13mm2…….ok

$Asreq$ =235.95 mm ^2

as =πd^2/4=3.14∗ 10^2/4= 78.5mm^2

$Spacing$=b∗$as/Asreq$=1000 ∗ 78.5/235.95 = 330mm

$Sprovide$ = min3.3D = 3.5 ∗ 220 = 770mm

450mm

$Spacing$ = 330mm

Provide, Ø 10 C/C 330mm

Now using similar procedure we can calculate other panel

Reinforcement design for first floor slab…….**Appendix 3H**

3.2.6.1 Re -check depth for deflection: serviceability requirement

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

As an example For panel-(P1)

Initially we assumed $\rho o = \rho$.but we can calculate them as follows

$\rho o = Ast,pro/bd$

From Reinforcement design Mxf=Φ10 c/c 310

$Ast,\text{pro}=bd/S,prov$= 1000∗175/310= 564.5mm2

$\rho = Ast,\text{pro}/bd$=564.5/1000∗175= 0.32%….. $N1992$: 2015. 7.4.2(7.16a)

ρo =√20/1000= 0.44721% …… 1992: 2015Art. 7.4.2(7.16a)

Now, ρ = 0.32 % <ρo= 0.44721 %

$l/d=K∗N∗F1 ∗F2 ∗F3$ where

N = [11 + 1.5 ∗ √$fck∗\rho0/\rho$+ 32√$fck∗$ ($\rho0/\rho$− 1)3/2]

= [11 + 1.5 ∗ √20 ∗0.44721/0.32+ 3.2√20 ∗ (0.44721/0.32− 1)^3/2] = 20.824

K = 1.5from previous table for interior span

F1 = 500/($fyk∗As,\text{require}/As,prol$) = 500/(400 ∗250.25/564.5) = 2.82

F2 = F3 = 1

$l/d=K∗N∗F1 ∗F2 ∗F3$ = 1.5∗ 20.824∗ 2.82∗ 1 ∗ 1 =88.1 but l=lx=4561 mm

d =4561/88.1= 51.77 < 175 mm ……okay

Now using similar procedure we can check other panel

Depth check of serviceability for first floor……**Appendix 3I**

3.2.7 Load Transferring Mechanism

Loads are transferred to beam by load transferring mechanism. Dea d load with wall Load and live load is present. For the load transfer from the slab to the beams, un - factored slab DL and LL are computed separately.

The design loads on beams supporting solid slabs spanning in two directions at right angles and supporting uniformly distributed loads may be assessed from the following equations obtained from Euro code, BS 8110-1:1997 section 3.5.3.7

$Vx = \beta vx * n * LxVy = \beta vy * n * Lx$

Example for panel (p1) on first floor, the load transferring mechanism is as follow:

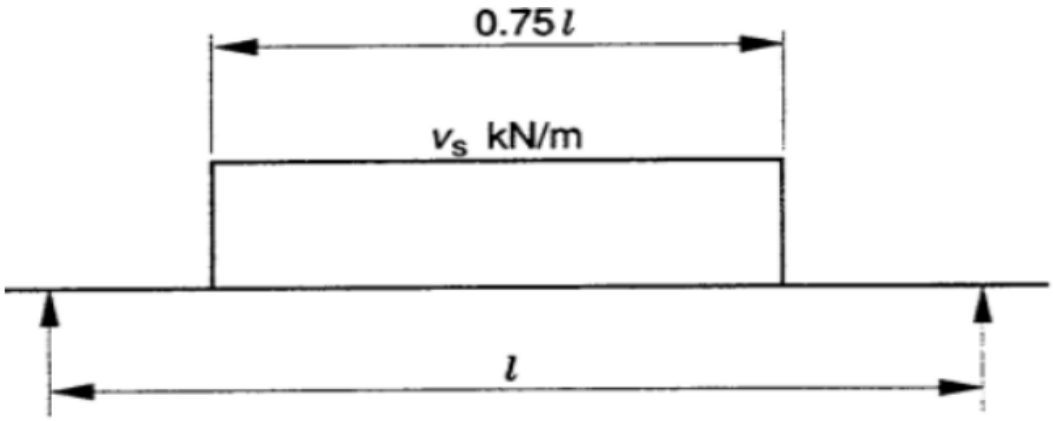


Figure 4 Distribution of load on a beam supporting a two-way spanning slab

βvxc = 0.39, $Vxc = \beta vxc * GK * Lx$ = 0.39∗7.142∗ 4.561 = 12.704KN/m

βvxd = 0, $Vxd = \beta vxd * GK * Lx$ = 0∗7.142∗ 4.561 = 0KN/m

βvyc = 0.33, $Vyc = \beta vyc * GK * Lx$ = 0.33∗7.142∗ 4.561 = 10.75KN/m

βvyd = 0, $Vyc = \beta vy$d$* GK * Lx$ = 0∗7.142∗ 4.561 = 0KN/m

Un-factored live load transferring to beam

βvxc = 0.39, = $\beta vxc * QK * Lx$ = 0.39∗ 5 ∗ 4.561 = 8.89KN/m

βvxd = 0, $Vxd = \beta vxd * QK * Lx$ = 0∗ 5 ∗ 4.561= 0KN/m

βvyc = 0.33, $Vyc = \beta vyc * QK * Lx$ = 0.33∗ 5 ∗ 4.561 = 7.52KN/m

βvyd = 0, $Vyc = \beta vyc * QK * Lx$ = 0∗ 5 ∗ 4.561 = 0KN

Factored Design shear

$Vdsxc$ = 1.35($\beta vxc * GK * Lx$) + 1.5($\beta vxc * QK * Lx$) = 1.35 ∗ 12.704 + 1.5 ∗ 8.89 = 30.49KN/m

$Vdsxd$ = 1.35($\beta vxd * GK * Lx$) + 1.5($\beta vxd * QK * Lx$) = 1.35 ∗ 0 + 1.5 ∗ 0 = 0KN/m

$Vdsyc$ = 1.35($\beta vyc * GK * Lx$) + 1.5($\beta vyc * QK * Lx$) = 1.35 ∗ 10.75+ 1.5 ∗ 7.52 = 25.79KN/m

$Vdsy$d = 1.35(βvyd$*GK*Lx$) + 1.5(βvyd$*QK*Lx$) = 1.35 $*$ 0 + 1.5 $*$ 0 =0KN/m

Now using similar procedure we can calculate other panel……**Appendix 3J**

3.2.8 Check Shear resistance of the section

The design value of the shear resistance is given in ES EN: 1992 section 6.2.2 as follows

Example For panel (P-1)

VRd, c = [$CRd, cK*$ (100 $*\rho 1 fck$)^1/3+ $k1\sigma cp$] ≤ [$Vmin$ + $k1\sigma cp$]

Where:

Fck is in MPa

CRd, c = 0.18/γc=0.18/1.5=0.12

k = 1 + √ (200/d) =1+√ (200/175) =1.08≤2.00 …… OK (Take k=1.08)

$\rho 1$ = Ast/ ($d*bw$) =250.25/ (175∗1000) =1430 ≤ 0.02 ……Not OK (Take ρ1=0.02)

$Vmin$ = 0.035 $*k$^3/2$*fck$^1/2=0.035∗2^3/2∗ 20^1/2=0.443

σcp = NED/AC = 0

Ast is the area of the tensile reinforcement, which extends ≥ (fcd+d) beyond the section considered

bw is the smallest width of the cross-section in the tensile area [mm]

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

Ac is the area of concrete cross-section [mm2]

VRd, c in [N]

Thus the design value for shear resistance will be:

VRd, c = [$CRd, cK*$ (100 $*\rho 1 fck$)^ 1/3+ $k1\sigma cp$] bwd ≤ [$Vmin$ + $k1\sigma cp$]

VRd, c= [0.12∗1.08∗(100∗0.02∗ 20)^1/3+2∗0]∗1000∗175=77.56kN

VRd, (min) = [$Vmin$ + $k1\sigma cp$] bwd=[0.443+2∗0]∗1000∗175=77.53kN<77.56kN...OK

($Take$VRd, c=77.56kN)

Since VRd, c=77.56kN>VED=30.49kN the section doesn't require design for shear reinforcement.

Now using similar procedure, we can check other panel

Check Shear resistance of the section for first floor slab……..**Appendix 3K**

3.3 Second and third floor slab

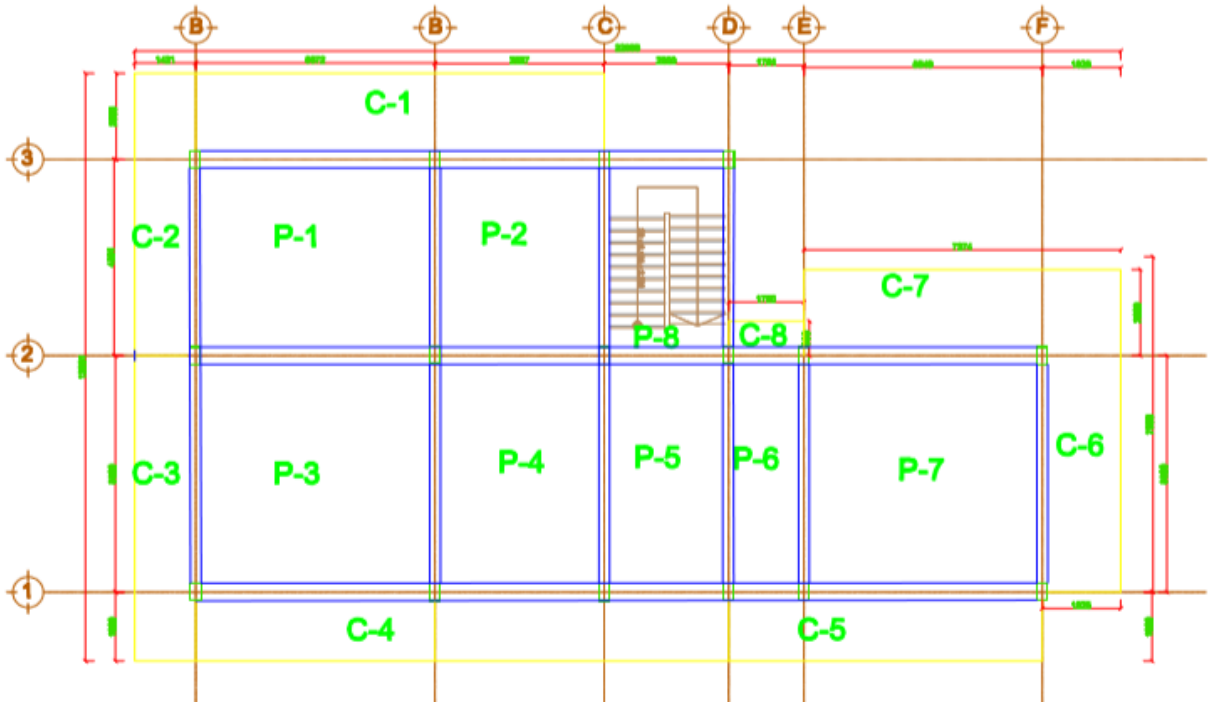


Figure 3.5second and third floor plan lay out

3.3.1 Depth determination (deflection criteria

Example For slab P-1

It is interior span of beam or one way slab or two way spanning slab, K=1.5

l/d=K*N*F1*F2*F3=1.5*17.71*1.25*1*1=33.21

Depth, D=d+d' Where: d' = cover +Φ/2 use Φ

d' = cover +Φ/2= 40mm +10mm/2 = 45mm

l/ d =33.21mm but $l = lx$

l=lx=4561 mm

d=4561/33.21=137.34mm

Other effective Depth Determination……**Appendix 3L**

From the above appendix effective depth calculation let us take the governing depth for design:
Use diameter of bar Ø10mm
The governing effective depth is d=165.63mm
D=concrete cover + d + Ø/2=40+165.63+10/2=210.63mm
Use the overall depth, D=220mm

3.3.2 Analysis and Design Load

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

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

Dead and live loads are calculated depending on the service of the slabs and self- weight.

Ignoring any localized effects caused by concentrated

Example:- dead load For slab P-1

RC slab=0.22*25=5.5KN/m^2

ff-PVC=0.002*16=0.032KN/m^2

Cement screed=0.05*23=1.15KN/m^2

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

Partition wall = h ∗ l ∗ w ∗ϒc/ A, =4.333

Total dead load=11.475 KN/m^2

Other panel dead load……. **Appendix 3M**

Load, the partition loads are distributed over the area of the slab. The design loads are factored according to the following formula.

Example: - design load For slab P1

Pd = 1.35DL + 1.5LL

=1.35(11.5) + 1.5(2)

=18.525 KN / m^2

Pd = 1.35DL + 1.5LL other panels……. **Appendix 3N**

3.3.3 Design Moment 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.

- Dead load to Live load should be less than 1.25

Uniformly distributed load

Rectangular slab

No large openings

Supported by beam in directions

Mi=αiPdLx2……(Ministry of Ethiopian education, 1995)

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

αi = the coefficient given in Table A-1 in EBCS2-1995.

Pd = the design load

Lx = the shorter span of the of the panel

Subscripts for moments and moment coefficients, αi have the following meanings:

Example: panel-1

$Mxs = axs*pd*Lx^2$, $Mxf = axf*pd*Lx^2$

$Mys = ays*pd*Lx^2$, $Myf = ayf*pd*Lx^2$

Mxs=0.0428*18.53*4.561^2=16.49831KNm

Mxf=0.0326*18.53*4.561^2=12.56647KNm

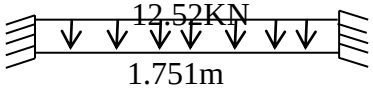
Mys=0.032*18.53*4.561^2=12.33518KNm

Myf=0.024*18.53*4.561^2*=9.251386KN

Other panels moment analysis for two way slabs like panel-1……..**Appendix 3O**

3.3.3.1 Design Moment Analysis for one way slab

P-6 12.52KN

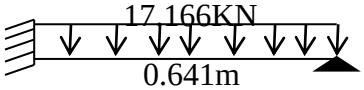


1.751m

$Mxs=\frac{pdLx^2}{12}=\frac{12.52*1.751^2}{12}$ =4.39KN/m

$Mxf=\frac{pdLx^2}{24}=\frac{12.52*1.751^2}{24}$ =2.19KN/m

P-8 17.166KN



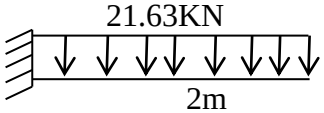
0.641m

$Mxs=\frac{9wL^2}{128}=\frac{9*17.166*0.641^2}{128}$ =0.49KN/m

Mxf $=\frac{pdLx^2}{8}$ =0.88KN/m

=Mxf=$\frac{17.166*0.641^2}{8}$

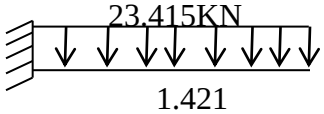
C-1 21.63KN



2m

$Mxs=\frac{pdLx^2}{2}=\frac{21.63*2^2}{2}$ =43.26KN/m

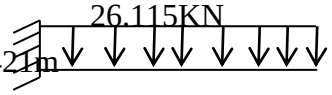
C-2 23.415KN



1.421

$Mxs=\frac{23.4`5*1.421^2}{2}$ =23.65KN/m

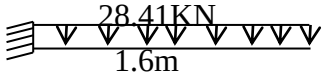
C-3 26.115KN



1.421m

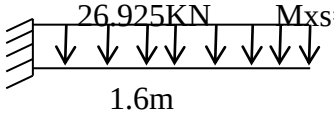
$Mxs=\frac{26.115*1.421^2}{2}$=26.36KN/m

C-4 28.41KN

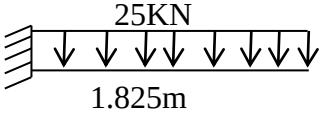


1.6m

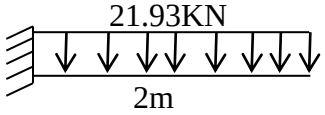
$Mxs=\frac{28.41*1.6^2}{2}$=36.36KN/m

C-5 

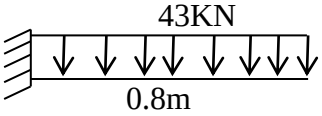
Mxs=$\frac{26.925*1.6^2}{2}$ =34.46KN/m

C-6 

Mxs=$\frac{25*1.825^2}{2}$ =41.63KN/m

C-7 

Mxs=$\frac{21.93*2^2}{2}$ =43.86KN/m

C-8 

Mxs=$\frac{43*0.6^2}{2}$ =13.76KN/m

3.3.4 AdjustmentofMoment

3.3.4.1 Support moment adjustment

The moment between two panels support should be adjusted in order to decrease thetorsion Effect on the beam and to provide the best economical negative reinforcement. To adjust the support Moment there are two cases check.

- Case (1) If $\%\Delta Mxs = Mlarg - Msmall/Mlarge * 100 < 20\%$ then we take

$Md = Mlarge + Msmall/2$…….is called average method

- Case (2) If $\%\Delta M = Mlarg - Msmall/Mlarge * 100 > 20\%$ ……by using kanas method

We did the adjustment with tabula form of table, but as example see the adjustment of support moment between p-1& p-2 slab panels.

For p-1Mxs = 16.5KNM

L_1 = 5.572m

K_1 = 1/ 5.572=0.179

Fo r p-2 Mxs = 11.56KNM

L_2= 3.937m

K_2 = 1/ 3.937=0.254

From the two panels maximum moment isp-1Mxs = 16.5KNM

NOW, ΔMxs = $Mxs_{large} - Mxs_{small}$

= 16.5 –11.56 = 4.94 KNM

$\%\Delta Mxs = Mxs_{large} - Mxs_{small}/Mxs_{large} = 4.94/16.5 * 100 = 30\%$:"20%

So we can Using relative stiffness p-1& p-2

$Md = Mxs_{large} - (K_1/K_1+K_2) * \Delta Mxs$

$= 16.5 - (0.219/0.219+0.18) * 3.678 = 14.44884KNm$

Similarly other support moment adjustment……...**Appendix 3P**

Note that if adjustment is between two-way and cantilever, then

Md = $Mmax$, means the design moment is the maximum moment one of them.

Note that if adjustment is between two-way and cantilever, then

Md = $Mmax$, means the design moment is the maximum moment one of them.

P1andC1:- P1=16.5KNm
C1=43.26KNm
} Mxs(maximum moment) =43.26KNm similarly

P2andC1=43.26KNm

P1andC2=23.65KNm

P3andC3=26.36KNm

P3andC4=36.36KNm

P4andC4=36.36KNm

P5andC5=34.46KNm

P6andC5=34.46KN

P7andC5=43.86KN

P7andC6=41.63KN

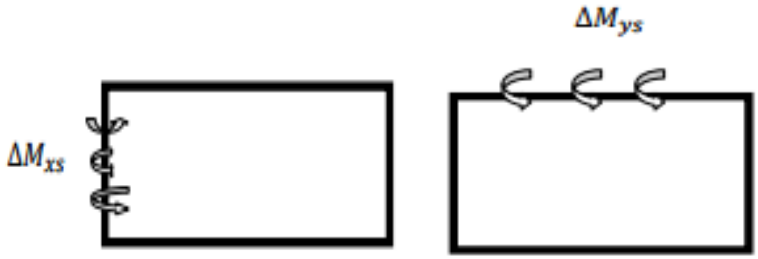
P7andC7=43.86KN

P6andC8=13.76KN

3.3.4.2 Span moment adjustment

Span moments are adjusted only in the direction in which support moment is decreased (reduced) during the support moment adjustment. Because the decreased moment must have distributed to the field.

Note that if Ly/Lx≤ 0 neglect spans moment adjustment



$Mxfd = Mxf + Cx\Delta M$ $Myfd = Myf + Cy\Delta M$

$Mxfd = Cx1\Delta Mxs + Cx2\Delta Mys + Mxf$, from table A-12, page 113, EBCS, 2-1995

$MYfd = Cy1\Delta Mxs + Cy2\Delta Mys + Myf$, from table A-12, page 113, EBCS, 2-1995

ΔMxs= Mxs(unadjusted)-Mxs(adjusted) ΔMxf= Cx1*ΔMxs+Cx2*ΔMys

ΔMys= Mys(unadjusted)-Mys(adjusted)ΔMyf= Cy1*ΔMxs+Cy2*ΔMys

Mxf(adjusted)=mxf(previous)+ΔMxfMyf(adjusted)=Myf(previous) + ΔMyf

For example, for panel (p1)

Mxs (unadjusted)= 16.5KNmMys(unadjusted) =12.33KNm

Mxs,(adjusted) = 14.45KNm Mys,(adjusted) = 43.26KNm

Mxf = 12.56KNm Myf = 9.25KNm

ΔMxs= Mxs(unadjusted)-Mxs(adjusted)ΔMys=Mys(unadjusted)-Mys(adjusted)

= 16.5- 14.45 =12.33-43.26

= 2.05KNm = -30.93KNm

Ly = 5.572m Ly/Lx=1.22

Lx = 4.561m

From EBCS 2-1995 Table A-2 we have coefficients by using interpolation

Cx1=0.335 Cx2=0.349

Cy1=0.165 Cy2=0.361

$Mxfd$ = C$x1\Delta Mxs + Cx2\Delta Mys + Mxf$

= 0.335*(2.05) +0.349*(-30.93) + 12.56= 1.51KNm

$Myfd$ = Cy1$*$ ΔMxs + Cy2$*$ ΔMys +Myf = 0.165**(2.05) +0.361*(-30.37)+9.25 =2.27KNm

Now using similar procedure, we can adjust other span moment

Span moment adjustment……. **Appendix 3Q**

3.3.5 SlabREINFORCEMENT

Designing the slab for flexure

We use the Design Table method to calculate the area of reinforcement required for the design moment.

The spacing of bars should not exceed Smax,slab From ES EN 1992 - 1 -1 : 2004

Reinforcement design for second and third floor slab……. **Appendix 3R**

Re -check depth for deflection: serviceability requirement

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

Initially we assumed $\rho o = \rho$.but we can calculate them as follows

$\rho o = Ast,pro/bd$

Depth check for serviceability……. **Appendix 3S**

3.3.6 Load Transferring Mechanism

Loads are transferred to beam by load transferring mechanism. Dea d load with wall Load and live load is present. For the load transfer from the slab to the beams, un - factored slab DL and LL are computed separately.

The design loads on beams supporting solid slabs spanning in two directions at right angles and supporting uniformly distributed loads may be assessed from the following equations obtained from Eurocode, BS 8110-1:1997 section 3.5.3.7

$Vx = \beta vx * n * LxVy = \beta vy * n * Lx$

Example for panel (p1) on first floor, the load transferring mechanism is as follow:

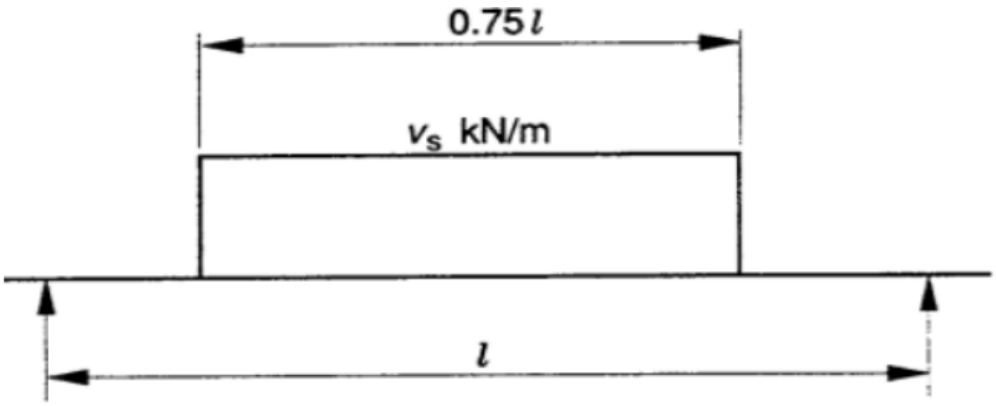


Figure 3.6Distribution of load on a beam supporting a two-way spanning slab

Now we can calculate load transfer mechanism similar to first floor panelstap…**Appendix 3T**

3.3.7 Check Shear resistance of the section

The design value of the shear resistance is given in ES EN: 1992 section 6.2.2 as follows

Example For panel (P-1)

$VRd, c = [CRd, cK * (100 * \rho 1 fck)$^1/3+ $k1\sigma cp] \leq [Vmin + k1\sigma cp]$

Where:

Fck is in MPa

CRd, c = 0.18/γc=0.18/1.5=0.12

Check Shear resistance of the section for second and third floor slab…**Appendix 3U**

3.4 Roof slab

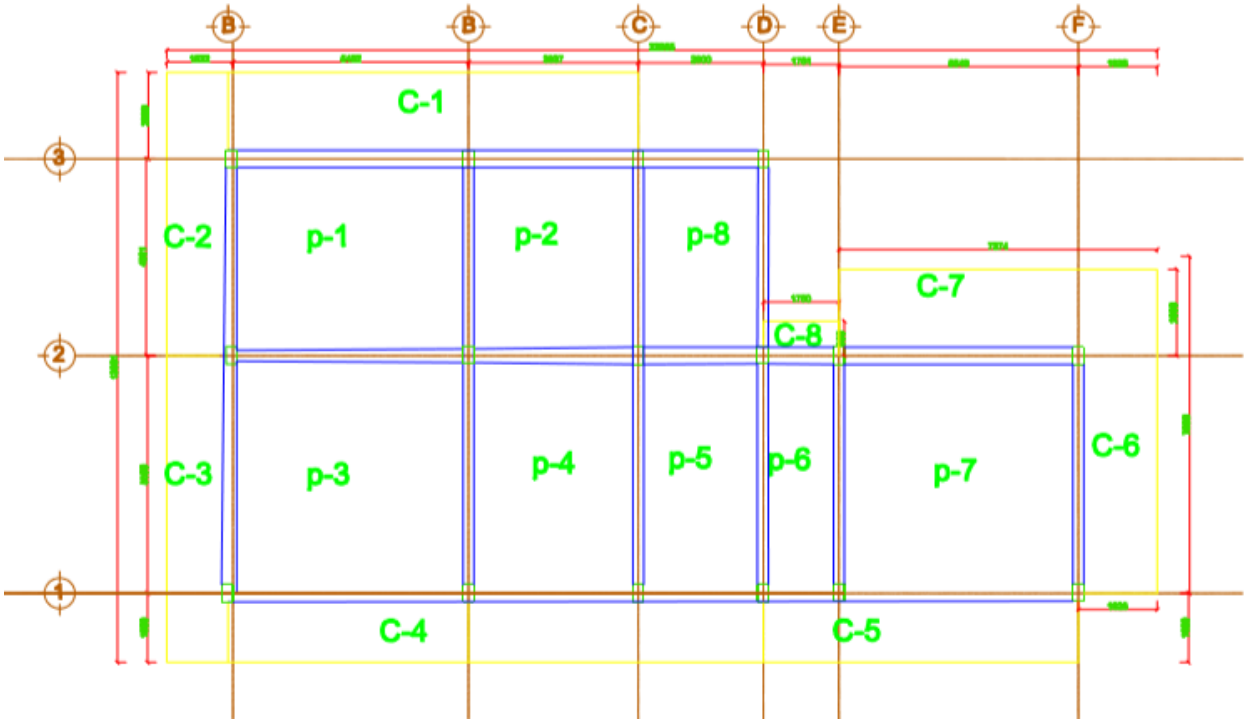


Figure 3.7 Roof slab plan lay out

3.4.1 Depth determination (deflection criteria

Determination of effective depth for roof sold slab to determine producers similarly the above first floor slab in exile.

Depth determination for Roof solid slab……**Appendix 3V**

d=165.63mm

Use diameter of bar Ø10mm

D=220mm

3.4.2 Design Moment 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 slab P-1 or sample and others are done by excel.

$Mxs = axs*pd*Lx^2$, $Mxf = axf*pd*Lx^2$

$Mys = ays*pd*Lx^2$, $Myf = ayf*pd*Lx^2$

Mxs=0.0428*14.83*4.561^2=13.2KNm

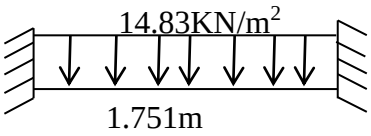
Mxf=0.0326*14.83*4.561^2=10.05KNm

Mys=0.032*14.837*4.561^2=9.9KNm

Myf=0.024*14.83*4.561^2*=7.4KN

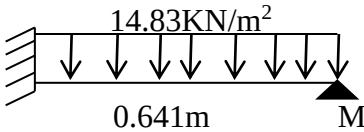
Roof solid slab moment determination other panels……**Appendix 3W**

Moment analysis for one way slab

P-6 

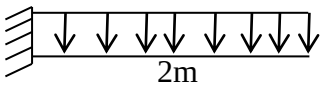
$Mxf=\frac{pdLx^2}{24}=\frac{14.83*1.751^2}{24}$ =1.89KN/m

$Mxs=\frac{pdLx^2}{12}=\frac{16.37*1.751^2}{12}$ =3.78KN/m2

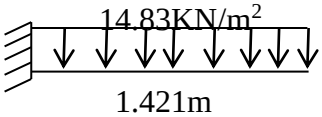
P-8 

$Mxs=\frac{9wL^2}{128}=\frac{9*14.83*0.641^2}{128}$=0.43KN/m

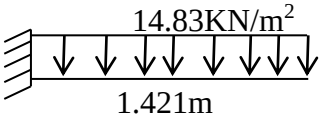
$Mxf=\frac{pdLx^2}{8}$ =Mxf=$\frac{14.83*0.641^2}{8}$ =0.76KN/m

C-1 

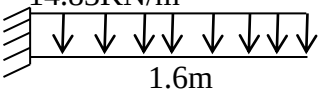
$Mxs=\frac{pdLx^2}{2}=\frac{14.83*2^2}{2}$ =14.83KN/m

C-2 

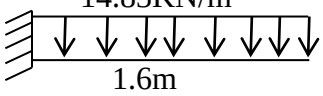
$Mxs=\frac{14.83*1.421^2}{2}$ =14.95KN/m

C-3 

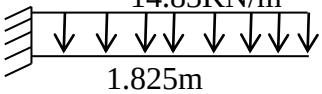
$Mxs=\frac{14.83*1.421^2}{2}$=14.95KN/m

C-4 

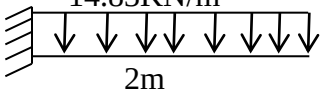
$Mxs=\frac{14.83*1.6^2}{2}$ =18.98KN/m

C-5 

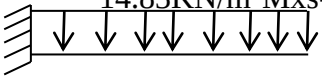
$Mxs=\frac{14.83*1.6^2}{2}$ =18.98KN/m

C-6 

$Mxs=\frac{14.83*1.825^2}{2}$ =24.59KN/m

C-7 

$Mxs=\frac{14.83*2^2}{2}$ =29.66KN/m

C-8 

$Mxs=\frac{14.83*0.8^2}{2}$ =4.75KN/m

3.4.2.1.1.1 0.8m

3.4.3 Adjustment of Moment

Support moment adjustment roof solid calculation similarly first, second and third slab steps…. **Appendix 3X**

3.4.3.1 Span moment adjustment

Span moment adjustment roof solid slab steps similar to first floor format……**Appendix 3Y**

3.4.4 Slab reinforcement

Similar producers that first floor slab Reinforcement design for Roof solid slab ……**Appendix 3Z**

3.4.4.1 Re -check depth for deflection: serviceability requirement

Depth check for serviceability for Roof solid slab as much as the same steps for first floor slab.……**Appendix 3AA**

3.4.5 Load TransferringMechanism

Load transfer for Roof solid slab similar producers the above panels……. **Appendix 3AB**

3.4.6 Check Shear resistance of the section

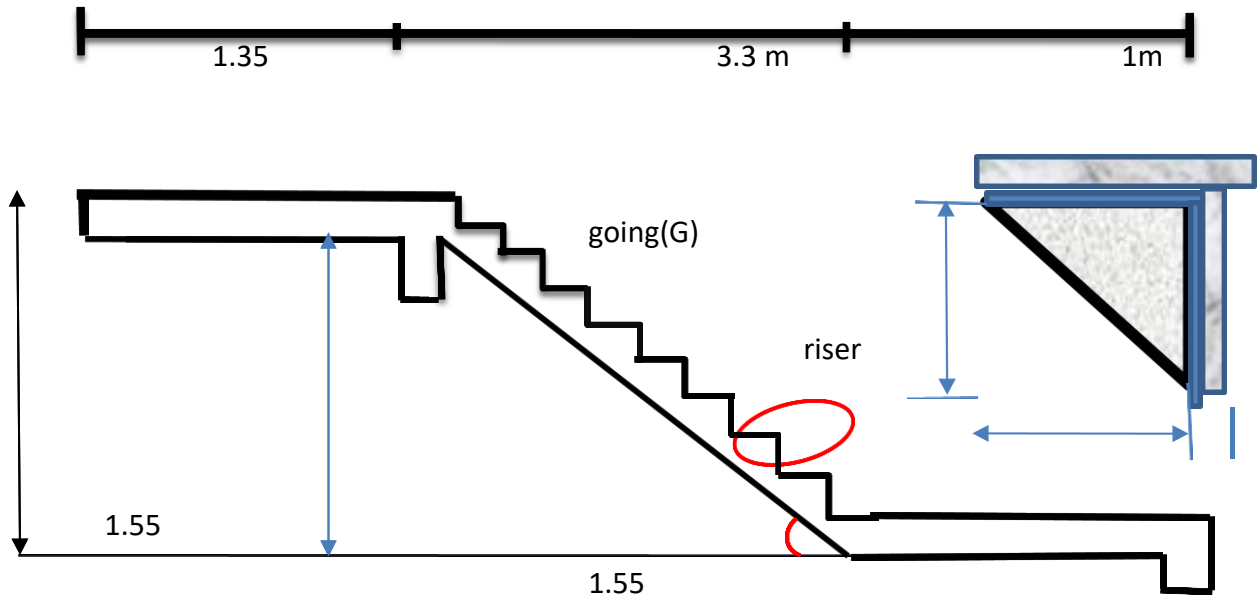
Check Shear resistance of the section for Roof solid slab steps similar to the previous slab ……**Appendix 3AC**

CHAPTER FOUR

4 Staircase Design

Stair case is an important element of a structure which connects places of different levels. Stair case analysis and design is similar to slabs. It involves the analysis steps followed for slabs. The inclined configuration is analyzed by projecting the loads on a horizontal plane. The stair contains two flights with the same configuration.

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



ConcreteclassC25

fck = 20MPa

fcd = 11.33MPa

Steel grade S-400

fyk=400MPa

Step 1-Geometrical data

height of stair in one flight (clear height between landing) = 1.55

number of riser(R) = 10

number of thread (T) = number of riser -1=10-1 =9

height of riser =$\frac{height\ of\ riser(in\ one\ fight)}{number\ of\ riser}$=1.55$m$/0.10$m$=15.5≈ 16

width of thread =$\frac{length\ of\ horizontal\ projecion\ inclind\ length\ of\ one\ fight}{number\ of\ threades}$= 0.367=367$mm$

breadth of step along slope(B) G

B= √ (G^2+R^2)

= √(0.367+0.155)

= 0..398m

take noising =25mm

Width of going=T-N

= 367-25 = 342mm

Step 2 Determination of depth for deflection

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

by limiting the span/depth ratio, according to 7.4.2 or

by comparing a calculated deflection, according to 7.4.3, with a limit value

l/d=K(11+1.5√fck∗ρo/ρ+3.2*√fck(ρo/ρ-1)^(3/2))*F1*F2*F3 …….ifρ<ρo

art.7.4.2.(7.16a)

l/d=K(11+1.5√fck∗ρo/(ρ− ρ‘)+1/12*√fck√(ρo/ρ))*F1*F2*F3 …….ifρ>ρo

art.7.4.2.(7.16b)

Where;

l/d - is the limit span/depth

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

ρo - is the reference reinforcement ratio = 10-3√fck

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

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

F1=300/σs =500/(fyk*As,req/As,pro)

F2=0.8, for flanged sections where the ratio of the flange breadth to the rib breadth exceeds 3. Otherwise; F2=1 for other cases.

F3=7/left,for beams and slabs, other than flat slabs, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflections (left in meters, see Art. 5.3.2.2 (1)). Or F3=8.5/left, for flat slabs where the greater span exceeds 8.5 m, and which support partitions liable to be damaged by excessive deflections (left in meters). Otherwise; F3=1 for both cases.

Table 4.1 Basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural System	*K*	Concrete highly stressed ρ = 1,5%	Concrete lightly stressed ρ = 0,5%
Simply supported beam, one- or two-way spanning simply supported slab	1,0	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

ρ=$\sqrt{\text{fck} * 10^{-3}}$=$\sqrt{20 * 10^{-3}}$=0.00447

For end span of one-way continuous slab

l/d = $1.3 * 11 + 1.5 * 4.47 * 0.89 = 20.26$

K = 1.3 ,l/d =20.26 because we used S400 multiply the value by$\frac{320}{\text{fyk}}$ =0.8

ld =20.26*0.8=16.208, l =lx= 3200mm→d=197.43

Using 12 and cover 15mm H =197.4 3 + 15 + $\frac{12}{2}$ = 218.43 , Use H = 250mm

d = 250-15-6 =229mm

Note: -The stair case slab is designed as one way reinforced slab. The depth of the section for design purpose of longitudinally spanning slab is taken as the minimum thickness perpendicular to soffit of stair case. I.e. the waist (w). For transverse stair case, there is no universal method to determine effective depth.

Step 3-Loading condition on stair

Material data

Material	thickness (m)	unit weight (KN/m^3)
concrete	0.21	24
Marble	0.03	27
cement screed	0.02	23
plastering	0.02	23

Live load

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

qk =3kN/m2 total live load on stair case =3kN/m2*1m=3kN/m

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

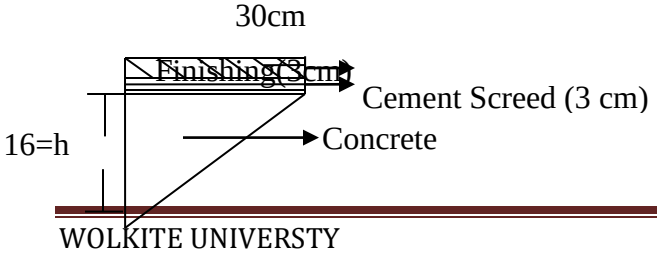
Step 4 Load computation

1- DL for flight (inclined slab)

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

Take 1m width strip

Step dead load



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

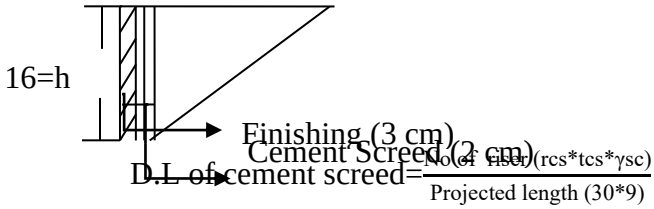
D.L of cement screed=tsc*γsc=0.02*23 KN/m3=0.46 KN/m

D.L of finishing=tfin*γfin=0.03*27 KN/m3=0.81 KN/m

D.L of Concrete=1/2*h*γconc=1/2*0.16m*24 KN/m3 =1.92 KN/m

Therefore, DL of step=0.46 KN/m + 0.81 KN/m + 1.92KN/m=3.19 KN/m

Riser Dead Load



D.L of cement screed=$\frac{\text{No of riser (rcs*tcs*}\gamma\text{sc)}}{\text{Projected length (30*9)}}$

$=\frac{10(0.16*0.02*23KN/m^3}{2.7}$

=0.273KN/m

D.L of Finishing =$\frac{\text{No of riser (rcs*tcs*}\gamma\text{sc)}}{\text{Projected length (3 cm)}}$

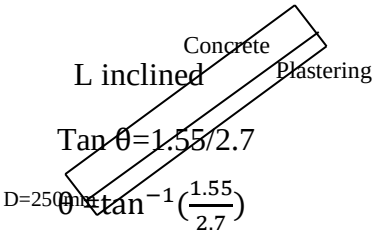
$=\frac{10(0.16*0.03m*23KN/m^3}{2.7}$

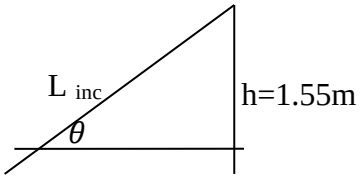
=0.408 KN/m

Therefore D.L of riser (15cm)= 0.273KN/m + 0.408KN/m

= 0.681KN/m

Waist Dead Load





Tan θ=1.55/2.7

$\theta=\tan^{-1}(\frac{1.55}{2.7})$

=29.54°

Lproj=2.7m

Sin θ=2/L_{inc}

$L_{inc\backslash}$=2/sin θ =2m/sin 29.54°

=3.14 m

D.L of concrete $=\frac{D*L\ inc*\gamma conc}{Lprojected}=\frac{0.25m*3.14m*24\ KN/m^3}{2.7\ m}$

=6.97KN/m

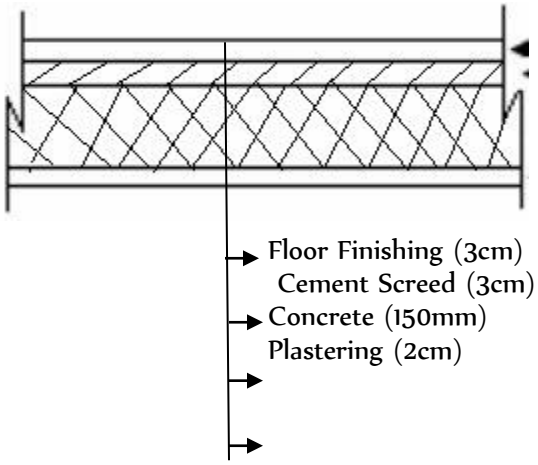
D.L of Plastering $=\frac{tpl*Linc*\gamma pl}{Lprojected}=\frac{0.02m*3.14m*23\ KN/m^3}{2.7\ m}$

=0.53KN/m

Therefore D.L of waist = 6.97KN/m + 0.53KN.m

=7.5 KN/m

Landing Dead Load



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

$=t_{fin}*\gamma_{fin}+t_{cs}*\gamma_{cs}+t_c\gamma_c+t_{pl}*\gamma_{pl}$

=0.03m*27 KN/m^3+0.02*23 KN/m^3 + 0.15m*24 KN/m^3 +0.02m*23 KN/m^3

=5.33KN/m

Therefore, D.L of Landing=5.33KN/m

Total Dead Load and design load

For the inclined slab

Total D. L=D.L of Step+D.L of riser+D.L of waist

=3.19KN/m+0.681KN/m+7.5KN/m

=11.37 KN/m

Live load=3 KN/m^2*1m=3 KN/m

Design Load, pd=1.3 D.L + 1.5 L.L

=1.3*11.37 KN/m + 1.5*3 KN/m

=19.28KN/m

For the landing

D.L=5.33KN/m

L.L=3 KN/m^2 * 1m=3KN/m

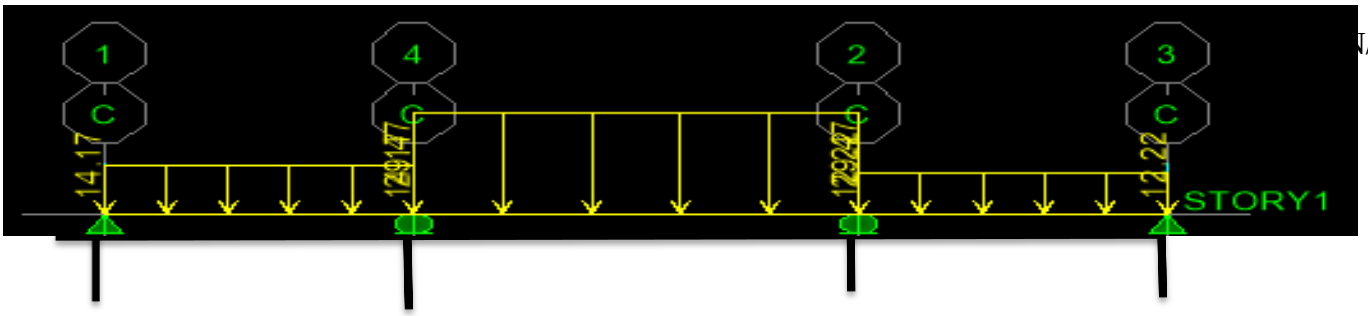
Deisgn load, pd=1.3 D.L + 1.5 L.L

=1.3*5.33KN/m + 1.5*3 KN/m

=11.43 KN/m

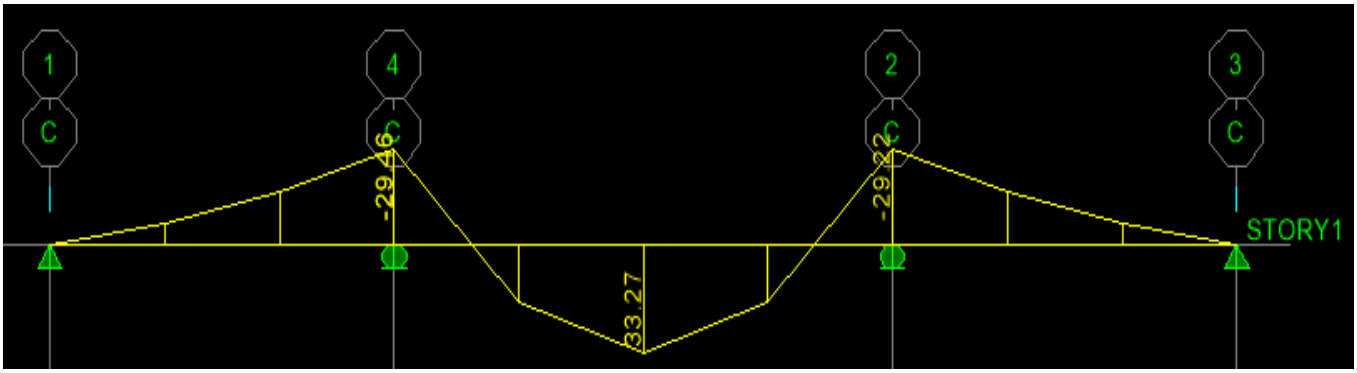
Loading

Moment analysis



N/m

1.35 m 3.3m 1 m



Renforcement calculation

Max design moment at support= 33.27KNm

Max design moment in the span= 29.46KNm

Step-2-check depth for flexure

Considering maximum moment: M =33.27KN.m

Mrd = 0.8kx (1 − 0.4kx)dbd2=0.8*0.448(1-0.448*0.4)*11.33*1000*(229)2

Mrd=54.6KNm/m>30.64KNm/m……OK

For the given Material Data, C-25, S-400 Effective depth, d=140mm Width, b=1000mm

Moments calculated for each panels Using design charts for span moment Using Mu = 30.64KN/m, calculate reinforcement,

µsd = $Mfcd*b*d$^2

= 33.27∗10^6 11.33∗1000∗229^2

= 0.165

using µsd =0.165, read Kz Z Az

Kz=0.90545 Z=Kz*d =0.90545*229 = 115.9mm

As=M/(fyd*z) As = 33.27*10^6/(115.9*347.82) = 825.3mm2

To calculate spacing by selecting diameter of bar as

S=1000as/As where: - as=area of single bar

As=calculated area of steel

S=spacing

S =1000 ∗ 113.1/825=137 use, ϕ12 c/c140mm

Compare the above result with minimum provision given by our code

. Amin= 0.26*2.2Mpa 400Mpa *1000mm*229mm = 183.04mm2

With d = 150-15-7=128mm, for ϕ12main rebar

Asmin=183.04mm2<Ascal=825.3mm2 ok

smax ≤ {2D

400 = 2 ∗ 150 = 300,400mm,140

sacl

Smax = 140mm

Provide ϕ12c/c140mm B) Transverse reinforcement According to Eurocode 2 Part 1,1 - prEN 1992-1-1-2002 Section 9.3.1.1 the ratio of secondary to the main reinforcement shall be at least equal to 20% of the main reinforcement.

- $As,nsv \geq 20\%\ As,calA$s, ≥ 0.2* 825.3 =165mm2<$Asmin=mm2$

Therefore, the minimum reinforcement

d = 250 – 15 – 14 – 4 = 217mm, for Ø8 main transverse rebar

Asmin= 0.26*2.2Mpa 300Mpa *1000mm*217mm

= 223.1mm2

S =1000 ∗ 50.26/223.1 =225.3mm

Provide ϕ8c/c200mm

Step 4-Check for shear Shear Reinforcement

The design value for the shear resistance

VRd,c is given by: VRd,c= [CRd,ck(100 ϱ 1fck)1/3 + k1 σcp] bwd≤ (vmin+ k1 σcp) bwd

Where: Fck is in MPa

K 0 .2$\sqrt{\frac{200}{d}}$<2 with d in mm =1$\sqrt{+}\frac{200}{128}$=2.25

Take, k=2

ϱ=$\frac{As1}{bwd}$<0.02=$\frac{760.09}{1000*128}$=0.006<0.02

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]

σcp = NEd/Ac< 0.2 fcd [MPa],0/Ac=0<0.2*11.33=2.27……ok

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

AC is the area of concrete cross section [mm2]

VRd,c is [N]

Note: For the use of CRD,C, vmin and k1, refer the National Annex. The recommended value for CRd,c is 0.18/□c, that for vmin is given by Expression (6.3N) and that for k1 is 0.15

Vmin = 0.035 $*k\frac{3}{2}*fck\frac{1}{2}$

Fck=20MPa

CRd,c =0.18/γc, =0.18/1.5=0.12

Vmin = =0.035*(2)3/2*(20)1/2=0.443

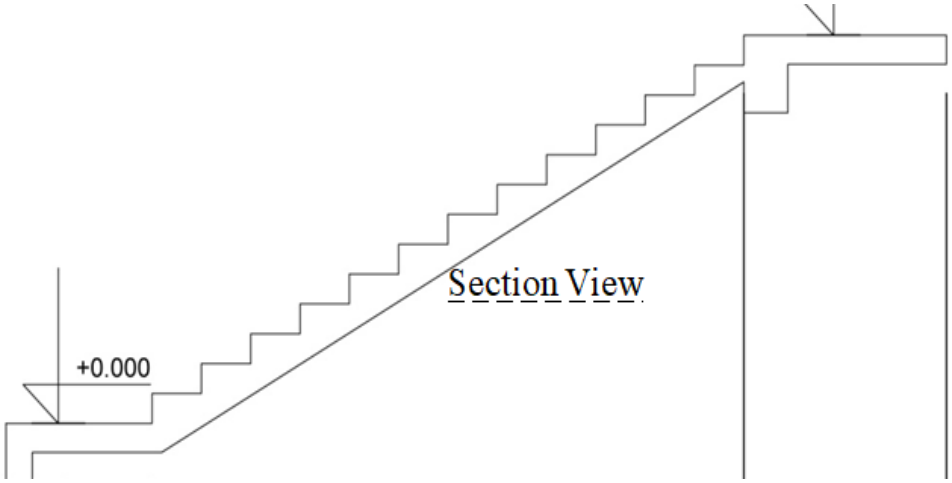
VRd,c= [CRd,ck(100 ϱlfck)1/3 + k1σcp] bwd ≤ (vmin+ k1σcp)

bwdVRd,c=[0.12*2*(100*0.007*20)1/3+0.15*0]1000*229≤ (0.443+0.15*0)*`1000*229

=72.3KN/m >56.7KN/m…NOT OK

So.VRd,c=56.7KN/m>VEd=42.29kN/m……..OK

Stair case 2



concerte class c 25

fck=20Mpa

fcd=11.33Mpa

Steel grade -400

Fyk=400

Step 1-Geometrical data

height of stair in one flight(clear height between landing)= 1.55

number of riser(R) = 10

number of thread (T) = number of riser -1=10-1 =9

height of riser $=\frac{\text{height of riser(in one fight)}}{\text{number of riser}}=1.55m/0.10m=15.5\approx 16$

width of thread $=\frac{\text{length of horizontal projecion inclind length of one fight}}{\text{number of threades}}= 0.367=367mm$

breadth of step along slope(B) G

B= √ (G^2+R^2)

= √(0.367+0.155)

= 0..398m

take noising =25mm

Width of going=T-N

= 367-25 = 342mm

Step 2: 4.3 Determination of depth for deflection

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

by limiting the span/depth ratio, according to 7.4.2 or

by comparing a calculated deflection, according to 7.4.3, with a limit value

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

art.7.4.2.(7.16a)

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

art.7.4.2.(7.16b)

Where;

l/d - is the limit span/depth

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

ρo - is the reference reinforcement ratio = 10-3√fck

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

ρ’ - is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers) fck is in MPa units.

F1=300/σs =500/(fyk*As,req/As,pro)

F2=0.8, for flanged sections where the ratio of the flange breadth to the rib breadth exceeds 3. Otherwise; F2=1 for other cases.

F3=7/leff , For beams and slabs, other than flat slabs, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflections (leff in meters, see Art. 5.3.2.2 (1)). Or

F3=8.5/leff, For flat slabs where the greater span exceeds 8.5 m, and which support partitions liable to be damaged by excessive deflections (leff in meters). Otherwise; F3=1 for both cases.

ρ=$\sqrt{\text{fck} * 10^{-3}}$=$\sqrt{20 * 10^{-3}}$=0.00447

For end span of one-way continuous slab

$l/d = 1.3 * 11 + 1.5 * 4.47 * 0.89 = 20.26$

K = 1.3 ,l/d =20.26 because we used S400 multiply the value by$\frac{320}{fyk}$ =0.8

ld =20.26*0.8=16.208, l =lx= 3200mm→d=197.43

Using 12 and cover 15mm H =197.4 3 + 15 + $\frac{12}{2}$ = 218.43 , Use H = 250mm

d = 250-15-6 =229mm

Note:-The stair case slab is designed as one way reinforced slab. The depth of the section for design purpose of longitudinally spanning slab is taken as the minimum thickness perpendicular to soffit of stair case. I.e. the waist (w). For transverse stair case, there is no universal method to determine effective depth.

Step 3-Loading condition on stair

Material data

Material	thickness (m)	unit weight (KN/m^3)
concrete	0.21	24
Marblr	0.03	27
cement screed	0.02	23
plastering	0.02	23

Live load

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

qk =3kN/m2 total live load on stair case =3kN/m2*1m=3kN/m

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

Step**4 Load computation**

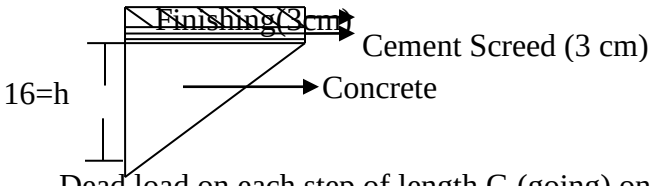
1- DL for flight (inclined slab)

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

Take 1m width strip

Step dead load

30cm



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

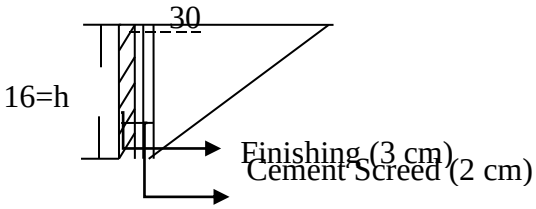
D.L of cement screed=tsc*γsc=0.02*23 KN/m3=0.46 KN/m

D.L of finishing=tfin*γfin=0.03*27 KN/m3=0.81 KN/m

D.L of Concrete=1/2*h*γconc=1/2*0.16m*24 KN/m3 =1.92 KN/m

Therefore DL of step=0.46 KN/m + 0.81 KN/m + 1.92KN/m=3.19 KN/m

Riser Dead Load



D.L of cement screed=$\frac{\text{No of riser (rcs*tcs*}\gamma\text{sc)}}{\text{Projected length (30*9)}}$

=$\frac{10(0.16*0.02*23KN/m^3}{2.7}$

=0.273KN/m

D.L of Finishing =$\frac{\text{No of riser (rcs*tcs*}\gamma\text{sc)}}{\text{Projected length (3 cm)}}$

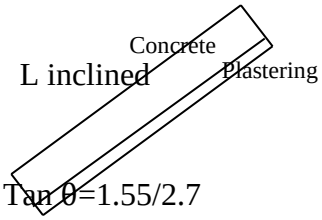
=$\frac{10(0.16*0.03m*23KN/m^3}{2.7}$

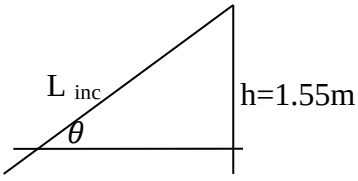
=0.408 KN/m

Therefore D.L of riser (15cm)= 0.273KN/m + 0.408KN/m

= 0.681KN/m

Waist Dead Load





Tan θ=1.55/2.7

D=250mm

$\theta=\tan^{-1}(\frac{1.55}{2.7})$

=29.54°

Sin θ=2/L_{inc}

$L_{inc\backslash}$=2/sin θ =2m/sin 29.54°

=3.14 m

D.L of concrete $=\frac{D*L\ inc*\gamma conc}{Lprojected}=\frac{0.25m*3.14m*24\ KN/m^3}{2.7\ m}$

=6.97KN/m

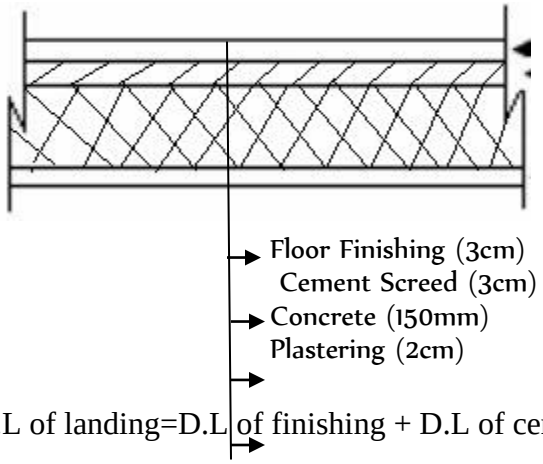
D.L of Plastering $=\frac{tpl*Linc*\gamma pl}{Lprojected}=\frac{0.02m*3.14m*23\ KN/m^3}{2.7\ m}$

=0.53KN/m

Therefore D.L of waist = 6.97KN/m + 0.53KN.m

=7.5 KN/m

Landing Dead Load



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

$=t_{fin}*\gamma_{fin}+t_{cs}*\gamma_{cs}+t_c\gamma_c+t_{pl}*\gamma_{pl}$

=0.03m*27 KN/m^3+0.02*23 KN/m^3 + 0.15m*24 KN/m^3 +0.02m*23 KN/m^3

=5.33KN/m

Therefore, D.L of Landing=5.33KN/m

Total Dead Load and design load

For the inclined slab

Total D. L=D.L of Step+D.L of riser+D.L of waist

=3.19KN/m+0.681KN/m+7.5KN/m

=11.37 KN/m

Live load=3 KN/m^2*1m=3 KN/m (EBSC-2)

Design Load, pd=1.3 D.L + 1.5 L.L

=1.3*11.37 KN/m + 1.5*3 KN/m

=19.04KN/m

For the landing

D.L=5.33KN/m

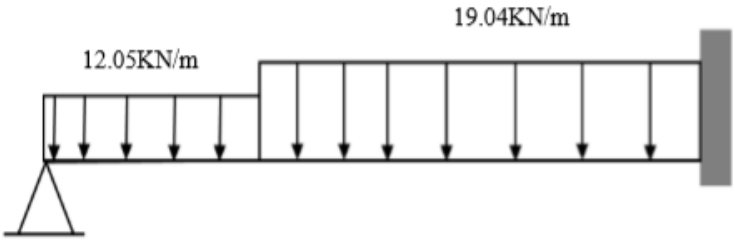
L.L=3 KN/m^2 * 1m=3KN/m

Deisgn load, pd=1.3 D.L + 1.5 L.L

=1.3*5.33KN/m + 1.5*3 KN/m

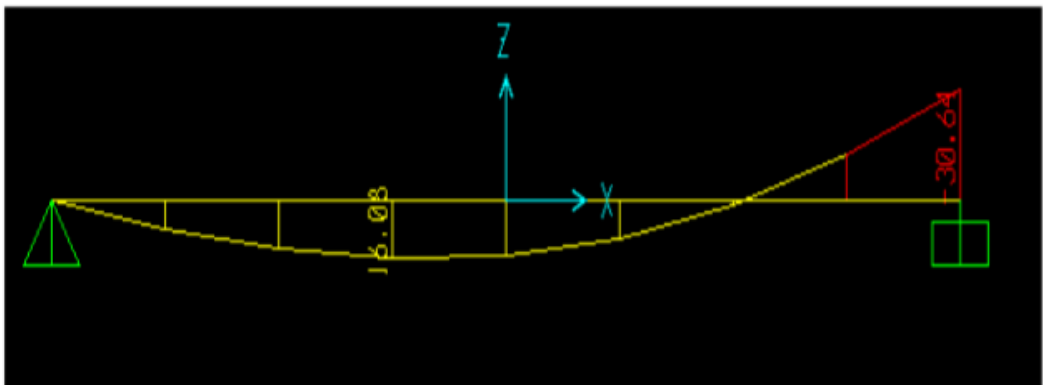
=12.05 KN/m

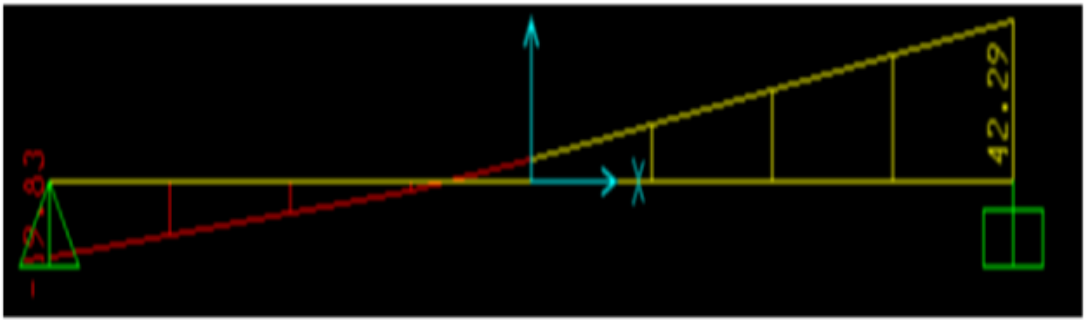
Loading



Step 1-Moment Analysis

Analysis using SAP





Reinforcement calculation

Max design moment at support= 30.64KNm

Max design moment in the span= 16.03KNm

Step-2-check depth for flexure

Considering maximum moment: M =30. 64KN.m

Mrd = 0.8kx (1 − 0.4kx) dbd2=0.8*0.448(1-0.448*0.4)*11.33*1000*(229)2

Mrd=54.6KNm/m>30.64KNm/m……OK

For the given Material Data, C-25, S-400 Effective depth, d=140mm Width, b=1000mm

Moments calculated for each panels Using design charts for span moment Using Mu = 30.64KN/m, calculate reinforcement,

µsd = $Mfcd*b*d$^2

= 30.64∗10^6 11.33∗1000∗229^2

= 0.165

using µsd =0.165, read Kz Z Az

Kz=0.90545 Z=Kz*d =0.90545*229 = 115.9mm

As=M/(fyd*z) As = 30.64*10^6/(115.9*260.27) = 1014mm2

To calculate spacing by selecting diameter of bar as

S=1000as/As where: - as=area of single bar

As=calculated area of steel

S=spacing

S =1000 ∗ 153.94/1014=151.8use, ϕ14 c/c150mm

Compare the above result with minimum provision given by our code

. Asmin= 0.26*2.2Mpa 400Mpa *1000mm*229mm = 183.04mm2

With d = 150-15-7=128mm, for ϕ12main rebar

Asmin=183.04mm2<Ascal=1014mm2 ok

smax ≤ {2D

400 = 2 ∗ 150 = 300,400mm,140

sacl

Smax = 140mm

Provide ϕ12c/c140mm B) Transverse reinforcement According to Eurocode 2 Part 1,1 - prEN 1992-1-1-2002 Section 9.3.1.1 the ratio of secondary to the main reinforcement shall be at least equal to 20% of the main reinforcement.

- $As,nsv \geq 20\%\ As,calA$s, ≥ 0.2* 1014 =202.8mm2<$Asmin=mm2$

Thereforeake the minimum reinforcement

d = 250 – 15 – 14 – 4 = 217mm, for Ø8 main transverse rebar

Asmin= 0.26*2.2Mpa 300Mpa *1000mm*217mm

= 223.1mm2

S =1000 ∗ 50.26/223.1 =225.3mm

Provide ϕ8c/c200mm

Step 4-Check for shear Shear Reinforcement

The design value for the shear resistance

VRd,c is given by: VRd,c= [CRd,ck(100 ϱ 1fck)1/3 + k1 σcp] bwd≤ (vmin+ k1 σcp) bwd

Where: Fck is in MPa

K 0 .2$\sqrt{\frac{200}{d}}$<2 with d in mm =1$\sqrt{+}\frac{200}{128}$=2.25

Take, k=2

ϱ=$\frac{As1}{bwd}$<0.02=$\frac{1014}{1000*128}$=0.007<0.02

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]

σcp = NEd/Ac< 0.2 fcd [MPa],0/Ac=0<0.2*11.33=2.27……ok

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

AC is the area of concrete cross section [mm2]

VRd,c is [N]

Note: For the use of CRD,C, vmin and k1, refer the National Annex. The recommended value for CRd,c is 0.18/γc, that for vmin is given by Expression (6.3N) and that for k1 is 0.15

Vmin = 0.035 $*k\frac{3}{2}*fck\frac{1}{2}$

Fck=20MPa

CRd,c =0.18/γc, =0.18/1.5=0.12

Vmin = =0.035*(2)3/2*(20)1/2=0.443

VRd,c= [CRd,ck(100 ϱlfck)1/3 + k1σcp] bwd ≤ (vmin+ k1σcp)

bwdVRd,c=[0.12*2*(100*0.007*20)1/3+0.15*0]1000*229≤ (0.443+0.15*0)*`1000*229

=72.3KN/m >56.7KN/m…NOT OK

So.VRd,c=56.7KN/m>VEd=42.29kN/m……..OK

CHAPTER FIVE

5 Earthquake Analysis

5.1 INTRODACTION

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 $T1 \leq \begin{cases} 4TC \\ 2Sec \end{cases}$

5.1.1 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)

$$X_m = \frac{\sum W_i X_i}{\sum W_i} \quad X_m = \frac{\sum W_i X_i}{\sum W_i} \quad Y_m = \frac{\sum W_i Y_i}{\sum W_i}$$

Where :Xm, and Ym are the coordinate of the point of application of Fi when the seismic action is

parallel to the Y-direction and X-direction respectively.

5.1.1.1 foundation

Table 14Center of mass calculation for foundation

Column o	Width/Dia	Depth(m)	Height(m)	Weight(KI	Moment Arm		Moment	
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis - A1	0.4	0.4	1.4	5.6	0	0	0	0
Axis - A2	0.4	0.4	1.4	5.6	0	5.5	30.8	0
Axis - A3	0.4	0.4	1.4	5.6	0	10.06	56.336	0
Axis - B1	0.4	0.4	1.4	5.6	5.572	0	0	31.2032
Axis - B2	0.4	0.4	1.4	5.6	5.572	5.5	30.8	31.2032
Axis - B3	0.4	0.4	1.4	5.6	5.572	10.06	56.336	31.2032
Axis - C1	0.4	0.4	1.4	5.6	9.509	0	0	53.2504
Axis - C2	0.4	0.4	1.4	5.6	9.509	5.5	30.8	53.2504
Axis - C3	0.4	0.4	1.4	5.6	9.509	10.06	56.336	53.2504
Axis - D1	0.4	0.4	1.4	5.6	12.409	0	0	69.4904
Axis - D2.	0.4	0.4	1.4	5.6	12.409	5.5	30.8	69.4904
Axis - D3	0.4	0.4	1.4	5.6	12.409	10.06	56.336	69.4904
Axis - E1	0.4	0.4	1.4	5.6	14.16	0	0	79.296
Axis - E2	0.4	0.4	1.4	5.6	14.16	5.5	30.8	79.296
Axis - F1	0.4	0.4	1.4	5.6	19.709	0	0	110.3704
Axis - F2	0.4	0.4	1.4	5.6	19.709	5.5	30.8	110.3704
Total Weight of Footing column				89.6			410.144	841.22

$$X_m = \frac{\sum(W_i * X_i)}{\sum W_i}$$

=410.144/89.6=4.58KNm

$$Y_m = \frac{\sum(W_i * Y_i)}{\sum W_i}$$

=841.22/89.6=9.39KNm

5.1.1.2 Ground floor slab

Table 15Center of mass calculation for ground slab

Column or	Width/Dia	Depth(m)	Height(m)	Weight(KI	Moment Arm		Moment		
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi	
Axis - A1	0.4	0.4	3.3	13.2	0	0	0	0	
Axis - A2	0.4	0.4	3.3	13.2	0	5.5	72.6	0	
Axis - A3	0.4	0.4	3.3	13.2	0	10.06	132.792	0	
Axis - B1	0.4	0.4	3.3	13.2	5.572	0	0	73.5504	
Axis - B2	0.4	0.4	3.3	13.2	5.572	5.5	72.6	73.5504	
Axis - B3	0.4	0.4	3.3	13.2	5.572	10.06	132.792	73.5504	
Axis - C1	0.4	0.4	3.3	13.2	9.509	0	0	125.5188	
Axis - C2	0.4	0.4	3.3	13.2	9.509	5.5	72.6	125.5188	
Axis - C3	0.4	0.4	3.3	13.2	9.509	10.06	132.792	125.5188	
Axis - D1	0.4	0.4	3.3	13.2	12.409	0	0	163.7988	
Axis - D2.	0.4	0.4	3.3	13.2	12.409	5.5	72.6	163.7988	
Axis - D3	0.4	0.4	3.3	13.2	12.409	10.06	132.792	163.7988	
Axis - E1	0.4	0.4	3.3	13.2	14.16	0	0	186.912	
Axis - E2	0.4	0.4	3.3	13.2	14.16	5.5	72.6	186.912	
Axis - F1	0.4	0.4	3.3	13.2	19.709	0	0	260.1588	
Axis - F2	0.4	0.4	3.3	13.2	19.709	5.5	72.6	260.1588	
Total weight				211.2			966.768	1982.746	
				Beam					
Beam	Between /	Width/Dia	Depth(m)	Length	Weight(KI	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis - A	1&3	0.2	0.4	10.06	20.12	0	0	0	0
Axis - B	1&3	0.2	0.4	10.06	20.12	5.572	10.06	202.4072	112.10864
Axis - C	1&3	0.2	0.4	10.06	20.12	9.44	13.66	274.8392	189.9328
Axis - D	1&3	0.2	0.4	10.06	20.12	12.34	10.06	202.4072	248.2808
Axis - E	1&2	0.2	0.4	5.5	11	14.08	5.5	60.5	154.88
Axis - F	1&2	0.2	0.4	5.5	11	19.63	7.5	82.5	215.93
Axis - 1	A&F	0.2	0.4	19.71	39.42	0	13.66	538.4772	0
Axis - 2	A&F	0.2	0.4	19.71	39.42	9.64	6.56	258.5952	380.0088
Axis - 3	A&D	0.2	0.4	12.41	24.82	12.34	2	49.64	306.2788
Total weight					206.14			1669.366	1607.41984

				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	1.6	3.85	6.19	38.1304	1.5	2.8	106.7651	57.1956
Stair 1	Flight 2	1.6	3.85	6.19	38.1304	1.5	1.66	63.29646	57.1956
Total weight					76.2608			170.0616	114.3912
				Partion wall					
Wall	Between /	Width/Dia	Depth(m)	Length	Weight(KI	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
A&B	1	0.15	3	5.572	35.1036	5.572	12.06	423.3494	195.5973
A&B	2	0.15	3	5.572	35.1036	5.572	6.561	230.3147	195.5973
A&B	3	0.15	3	5.572	35.1036	5.572	2	70.2072	195.5973
B&C	1	0.15	3	3.937	24.8031	9.509	12.06	299.1254	235.8527
B&C	2	0.15	3	3.937	24.8031	9.509	6.561	162.7331	235.8527
B&C	3	0.15	3	3.937	24.8031	9.509	2	49.6062	235.8527
C&D	1	0.15	3	2.9	18.27	12.41	12.06	220.3362	226.7307
C&D	2	0.15	3	2.9	18.27	12.41	6.561	119.8695	226.7307
C&D	3	0.15	3	2.9	18.27	12.41	2	36.54	226.7307
D&E	1	0.15	3	1.751	11.0313	14.16	12.06	133.0375	156.2032
D&E	2	0.15	3	1.751	11.0313	14.16	6.561	72.37636	156.2032
E&F	1	0.15	3	5.549	34.9587	19.71	12.06	421.6019	689.036
E&F	2	0.15	3	5.549	34.9587	19.71	6.561	229.364	689.036
1&2	A	0.15	3	5.5	34.65	0	12.06	417.879	0
1&2	B	0.15	3	5.5	34.65	5.572	12.06	417.879	193.0698
1&2	C	0.15	3	5.5	34.65	9.51	12.06	417.879	329.5215
1&2	D	0.15	3	5.5	34.65	12.41	12.06	417.879	430.0065
1&2	E	0.15	3	5.5	34.65	14.16	12.06	417.879	490.644
1&2	F	0.15	3	5.5	34.65	19.71	12.06	417.879	682.9515
2&3	A	0.15	3	4.561	28.7343	0	6.561	188.5257	0
2&3	B	0.15	3	4.561	28.7343	5.572	6.561	188.5257	160.1075
2&3	C	0.15	3	4.561	28.7343	9.51	6.561	188.5257	273.2632
2&3	D	0.15	3	4.561	28.7343	12.41	6.561	188.5257	356.5927
Total weight					649.3473			5729.838	6581.177

$$X_m = \frac{\sum(W_i * X_i)}{\sum W_i} \qquad Y_m = \frac{\sum(W_i * Y_i)}{\sum W_i}$$

=8536.033/1142.74= 7.48KNm

=10315.733/1142.74=9.027KNm

5.1.1.3 First floor slab

Table 16Center of mass calculation for first slab

Column o	Width/Dia	Depth(m)	Height(m)	Weight(KI	Moment Arm		Moment		
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi	
Axis - A1	0.4	0.4	3.3	13.2	0	0	0	0	
Axis - A2	0.4	0.4	3.3	13.2	0	5.5	72.6	0	
Axis - A3	0.4	0.4	3.3	13.2	0	10.06	132.792	0	
Axis - B1	0.4	0.4	3.3	13.2	5.572	0	0	73.5504	
Axis - B2	0.4	0.4	3.3	13.2	5.572	5.5	72.6	73.5504	
Axis - B3	0.4	0.4	3.3	13.2	5.572	10.06	132.792	73.5504	
Axis - C1	0.4	0.4	3.3	13.2	9.509	0	0	125.5188	
Axis - C2	0.4	0.4	3.3	13.2	9.509	5.5	72.6	125.5188	
Axis - C3	0.4	0.4	3.3	13.2	9.509	10.06	132.792	125.5188	
Axis - D1	0.4	0.4	3.3	13.2	12.409	0	0	163.7988	
Axis - D2.	0.4	0.4	3.3	13.2	12.409	5.5	72.6	163.7988	
Axis - D3	0.4	0.4	3.3	13.2	12.409	10.06	132.792	163.7988	
Axis - E1	0.4	0.4	3.3	13.2	14.16	0	0	186.912	
Axis - E2	0.4	0.4	3.3	13.2	14.16	5.5	72.6	186.912	
Axis - F1	0.4	0.4	3.3	13.2	19.709	0	0	260.1588	
Axis - F2	0.4	0.4	3.3	13.2	19.709	5.5	72.6	260.1588	
Total weight				211.2			966.768	1982.746	
				Beam					
Beam	Between /	Width/Dia	Depth(m)	Length	Weight(KI	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis - A	1&3	0.2	0.4	10.06	20.12	0	0	0	0
Axis - B	1&3	0.2	0.4	10.06	20.12	5.572	10.06	202.4072	112.1086

Axis - C	1&3	0.2	0.4	10.06	20.12	9.44	13.66	274.8392	189.9328
Axis - D	1&3	0.2	0.4	10.06	20.12	12.34	10.06	202.4072	248.2808
Axis - E	1&2	0.2	0.4	5.5	11	14.08	5.5	60.5	154.88
Axis - F	1&2	0.2	0.4	5.5	11	19.63	7.5	82.5	215.93
Axis - 1	A&F	0.2	0.4	19.71	39.42	0	13.66	538.4772	0
Axis - 2	A&F	0.2	0.4	19.71	39.42	9.64	6.56	258.5952	380.0088
Axis - 3	A&D	0.2	0.4	12.41	24.82	12.34	2	49.64	306.2788
Total weight					206.14			1669.366	1607.42
				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	1.6	3.85	6.19	38.1304	1.5	2.8	106.7651	57.1956
Stair 1	Flight 2	1.6	3.85	6.19	38.1304	1.5	1.66	63.29646	57.1956
Total weight					76.2608			170.0616	114.3912
				Partion wall					
Wall	Between /	Width/Dia	Depth(m)	Length	Weight(KI	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
A&B	1	0.15	3	5.572	35.1036	5.572	12.06	423.3494	195.5973
A&B	2	0.15	3	5.572	35.1036	5.572	6.561	230.3147	195.5973
A&B	3	0.15	3	5.572	35.1036	5.572	2	70.2072	195.5973
B&C	1	0.15	3	3.937	24.8031	9.509	12.06	299.1254	235.8527
B&C	2	0.15	3	3.937	24.8031	9.509	6.561	162.7331	235.8527
B&C	3	0.2	3	3.937	33.0708	9.509	2	66.1416	314.4702
C&D	1	0.15	3	2.9	18.27	12.41	12.06	220.3362	226.7307
C&D	2	0.15	3	2.9	18.27	12.41	6.561	119.8695	226.7307
C&D	3	0.15	3	2.9	18.27	12.41	2	36.54	226.7307
D&E	1	0.15	3	1.751	11.0313	14.16	12.06	133.0375	156.2032
D&E	2	0.2	3	1.751	14.7084	14.16	6.561	96.50181	208.2709
E&F	1	0.15	3	5.549	34.9587	19.71	12.06	421.6019	689.036
E&F	2	0.15	3	5.549	34.9587	19.71	6.561	229.364	689.036
1&2	A	0.15	3	5.5	34.65	0	12.06	417.879	0
1&2	B	0.2	3	5.5	46.2	5.572	12.06	557.172	257.4264
1&2	C	0.15	3	5.5	34.65	9.51	12.06	417.879	329.5215
1&2	D	0.15	3	5.5	34.65	12.41	12.06	417.879	430.0065
1&2	E	0.2	3	5.5	46.2	14.16	12.06	557.172	654.192
1&2	F	0.15	3	5.5	34.65	19.71	12.06	417.879	682.9515

2&3	A	0.15	3	4.561	28.7343	0	6.561	188.5257	0
2&3	B	0.2	3	4.561	38.3124	5.572	6.561	251.3677	213.476693
2&3	C	0.15	3	4.561	28.7343	9.51	6.561	188.5257	273.263193
2&3	D	0.15	3	4.561	28.7343	12.41	6.561	188.5257	356.592663
Total weight					693.9702			6111.927	6993.13603
				Slab					
Panel	Lx(m)	Ly(m)	DL(KN/m2	Weight(KI	Moment Arm		Moment		
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi	
P1	4.561	5.572	7.14	181.4552	0	2	362.9104	0	
P2	3.937	4.561	10.15	182.2601	5.572	2	364.5201	1015.553	
P3	5.5	5.572	9.43	288.9918	0	6.561	1896.075	0	
P4	3.937	5.5	8.87	192.0665	5.572	6.561	1260.149	1070.195	
P5	2.9	5.5	7.61	121.3795	9.509	6.561	796.3709	1154.198	
P6	1.751	5.5	9.91	95.43826	12.409	6.561	626.1704	1184.293	
P7	5.5	5.549	8.39	256.0586	14.16	6.561	1680.001	3625.79	
P8	0.8	2.9	7.16	16.6112	9.509	5.92	98.3383	157.9559	
C1	2	9.509	11.62	220.9892	0	0	0	0	
C2	1.421	9.561	13.56	184.2286	0	0	0	0	
C3	1.421	7.1	7.16	72.23796	0	6.561	473.9532	0	
C4	2	3.909	7.16	55.97688	0	0	0	0	
C5	2	10.2	7.16	146.064	9.509	12.06	1761.532	1388.923	
C6	1.825	5.5	10.94	109.8103	19.71	6.561	720.4651	2164.36	
C7	2	7.37	9.66	142.3884	14.16	4.706	670.0798	2016.22	
C8	6	1.751	15.7	164.9442	12.41	5.05	832.9682	2046.958	
Total weight				**2430.901**			**11543.53**	**15824.44**	

$$X_m = \frac{\sum(W_i * X_i)}{\sum W_i} \qquad Y_m = \frac{\sum(W_i * Y_i)}{\sum W_i}$$

=18792.29/3618.47=

5.19KNm=24539.38/3618.47=6.78KNm

5.1.1.4 Second and third floor slab

Table 17Center of mass calculation for second and third slab

Column or	Width/Dia	Depth(m)	Height(m)	Weight(Kl	Moment Arm		Moment		
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi	
Axis - A1	0.4	0.4	3.3	13.2	0	0	0	0	
Axis - A2	0.4	0.4	3.3	13.2	0	5.5	72.6	0	
Axis - A3	0.4	0.4	3.3	13.2	0	10.06	132.792	0	
Axis - B1	0.4	0.4	3.3	13.2	5.572	0	0	73.5504	
Axis - B2	0.4	0.4	3.3	13.2	5.572	5.5	72.6	73.5504	
Axis - B3	0.4	0.4	3.3	13.2	5.572	10.06	132.792	73.5504	
Axis - C1	0.4	0.4	3.3	13.2	9.509	0	0	125.5188	
Axis - C2	0.4	0.4	3.3	13.2	9.509	5.5	72.6	125.5188	
Axis - C3	0.4	0.4	3.3	13.2	9.509	10.06	132.792	125.5188	
Axis - D1	0.4	0.4	3.3	13.2	12.409	0	0	163.7988	
Axis - D2.	0.4	0.4	3.3	13.2	12.409	5.5	72.6	163.7988	
Axis - D3	0.4	0.4	3.3	13.2	12.409	10.06	132.792	163.7988	
Axis - E1	0.4	0.4	3.3	13.2	14.16	0	0	186.912	
Axis - E2	0.4	0.4	3.3	13.2	14.16	5.5	72.6	186.912	
Axis - F1	0.4	0.4	3.3	13.2	19.709	0	0	260.1588	
Axis - F2	0.4	0.4	3.3	13.2	19.709	5.5	72.6	260.1588	
Total weight				**211.2**			**966.768**	**1982.746**	
				Beam					
Beam	Between /	Width/Dia	Depth(m)	Length	Weight(Kl	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis - A	1&3	0.2	0.4	10.06	20.12	0	0	0	0
Axis - B	1&3	0.2	0.4	10.06	20.12	5.572	10.06	202.4072	112.1086
Axis - C	1&3	0.2	0.4	10.06	20.12	9.44	13.66	274.8392	189.9328
Axis - D	1&3	0.2	0.4	10.06	20.12	12.34	10.06	202.4072	248.2808
Axis - E	1&2	0.2	0.4	5.5	11	14.08	5.5	60.5	154.88
Axis - F	1&2	0.2	0.4	5.5	11	19.63	7.5	82.5	215.93
Axis - 1	A&F	0.2	0.4	19.71	39.42	0	13.66	538.4772	0
Axis - 2	A&F	0.2	0.4	19.71	39.42	9.64	6.56	258.5952	380.0088
Axis - 3	A&D	0.2	0.4	12.41	24.82	12.34	2	49.64	306.2788
Total weight					**206.14**			**1669.366**	**1607.42**

				Partion wall					
Wall	Between A>	Width/Diamte	Depth(m)	Length	Weight(KI	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
A&B	1	0.15	3	5.572	35.1036	5.572	12.06	423.3494	195.5973
A&B	2	0.15	3	5.572	35.1036	5.572	6.561	230.3147	195.5973
A&B	3	0.2	3	5.572	46.8048	5.572	2	93.6096	260.7963
B&C	1	0.15	3	3.937	24.8031	9.509	12.06	299.1254	235.8527
B&C	2	0.15	3	3.937	24.8031	9.509	6.561	162.7331	235.8527
B&C	3	0.2	3	3.937	33.0708	9.509	2	66.1416	314.4702
C&D	1	0.15	3	2.9	18.27	12.41	12.06	220.3362	226.7307
C&D	2	0.15	3	2.9	18.27	12.41	6.561	119.8695	226.7307
C&D	3	0.15	3	2.9	18.27	12.41	2	36.54	226.7307
D&E	1	0.15	3	1.751	11.0313	14.16	12.06	133.0375	156.2032
D&E	2	0.2	3	1.751	14.7084	14.16	6.561	96.50181	208.2709
E&F	1	0.15	3	5.549	34.9587	19.71	12.06	421.6019	689.036
E&F	2	0.15	3	5.549	34.9587	19.71	6.561	229.364	689.036
1&2	A	0.15	3	5.5	34.65	0	12.06	417.879	0
1&2	B	0.2	3	5.5	46.2	5.572	12.06	557.172	257.4264
1&2	C	0.15	3	5.5	34.65	9.51	12.06	417.879	329.5215
1&2	D	0.15	3	5.5	34.65	12.41	12.06	417.879	430.0065
1&2	E	0.2	3	5.5	46.2	14.16	12.06	557.172	654.192
1&2	F	0.15	3	5.5	34.65	19.71	12.06	417.879	682.9515
2&3	A	0.15	3	4.561	28.7343	0	6.561	188.5257	0
2&3	B	0.2	3	4.561	38.3124	5.572	6.561	251.3677	213.4767
2&3	C	0.15	3	4.561	28.7343	9.51	6.561	188.5257	273.2632
2&3	D	0.15	3	4.561	28.7343	12.41	6.561	188.5257	356.5927
Total weight					**705.6714**			**6135.33**	**7058.335**
				Slab					
Panel	Lx(m)	Ly(m)	DL(KN/m2	Weight(KI	Moment Arm		Moment		
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi	
P1	4.561	5.572	11.475	291.6244	0	2	583.2488	0	
P2	3.937	4.561	7.142	128.2464	5.572	2	256.4929	714.5892	
P3	5.5	5.572	12.933	396.3447	0	6.561	2600.418	0	
P4	3.937	5.5	11.635	251.9385	5.572	6.561	1652.968	1403.801	

P5	2.9	5.5	11.51	183.5845	9.509	6.561	1204.498	1745.705
P6	1.751	5.5	7.16	68.95438	12.409	6.561	452.4097	855.6549
P7	5.5	5.549	14.079	429.684	14.16	6.561	2819.157	6084.326
P8	0.8	2.9	7.16	16.6112	9.509	5.92	98.3383	157.9559
C1	2	9.509	13.849	263.3803	0	0	0	0
C2	1.421	9.561	12.93	175.6693	0	0	0	0
C3	1.421	7.1	14.871	150.035	0	6.561	984.3797	0
C4	2	3.909	16.597	129.7553	0	0	0	0
C5	2	10.2	15.492	316.0368	9.509	12.06	3811.404	3005.194
C6	1.825	5.5	16.265	163.2599	19.71	6.561	1071.148	3217.853
C7	2	7.37	11.798	173.9025	14.16	4.706	818.3853	2462.46
C8	6	1.751	26.312	276.4339	12.41	5.05	1395.991	3430.544
Total weight				**3415.461**			**17748.84**	**23078.08**

$$X_m = \frac{\sum(W_i * X_i)}{\sum W_i} \qquad Y_m = \frac{\sum(W_i * Y_i)}{\sum W_i}$$

=26690.36/4538.43= 5.88KNm

=33840.229/4538.43=7.45KNm

5.1.1.5 Roof slab

Table 18Center of mass calculation for roof slab

Column or	Width/Dia	Depth(m)	Height(m)	Weight(Kl	Moment Arm		Moment	
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis - A1	0.4	0.4	3.3	13.2	0	0	0	0
Axis - A2	0.4	0.4	3.3	13.2	0	5.5	72.6	0
Axis - A3	0.4	0.4	3.3	13.2	0	10.06	132.792	0
Axis - B1	0.4	0.4	3.3	13.2	5.572	0	0	73.5504
Axis - B2	0.4	0.4	3.3	13.2	5.572	5.5	72.6	73.5504
Axis - B3	0.4	0.4	3.3	13.2	5.572	10.06	132.792	73.5504
Axis - C1	0.4	0.4	3.3	13.2	9.509	0	0	125.5188
Axis - C2	0.4	0.4	3.3	13.2	9.509	5.5	72.6	125.5188
Axis - C3	0.4	0.4	3.3	13.2	9.509	10.06	132.792	125.5188

Axis - D1	0.4	0.4	3.3	13.2	12.409	0	0	163.7988	
Axis - D2.	0.4	0.4	3.3	13.2	12.409	5.5	72.6	163.7988	
Axis - D3	0.4	0.4	3.3	13.2	12.409	10.06	132.792	163.7988	
Axis - E1	0.4	0.4	3.3	13.2	14.16	0	0	186.912	
Axis - E2	0.4	0.4	3.3	13.2	14.16	5.5	72.6	186.912	
Axis - F1	0.4	0.4	3.3	13.2	19.709	0	0	260.1588	
Axis - F2	0.4	0.4	3.3	13.2	19.709	5.5	72.6	260.1588	
Total weight				**211.2**			**966.768**	**1982.746**	
				Beam					
Beam	Between /	Width/Dia	Depth(m)	Length	Weight(KI	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis - A	1&3	0.2	0.3	10.06	15.09	0	0	0	0
Axis - B	1&3	0.2	0.3	10.06	15.09	5.572	10.06	151.8054	84.08148
Axis - C	1&3	0.2	0.3	10.06	15.09	9.44	13.66	206.1294	142.4496
Axis - D	1&3	0.2	0.3	10.06	15.09	12.34	10.06	151.8054	186.2106
Axis - E	1&2	0.2	0.3	5.5	8.25	14.08	5.5	45.375	116.16
Axis - F	1&2	0.2	0.3	5.5	8.25	19.63	7.5	61.875	161.9475
Axis - 1	A&F	0.2	0.3	19.71	29.565	0	13.66	403.8579	0
Axis - 2	A&F	0.2	0.3	19.71	29.565	9.64	6.56	193.9464	285.0066
Axis - 3	A&D	0.2	0.3	12.41	18.615	12.34	2	37.23	229.7091
Total weight					**154.605**			**1252.025**	**1205.565**
				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	1.6	3.85	6.19	38.1304	1.5	2.8	106.7651	57.1956
Stair 1	Flight 2	1.6	3.85	6.19	38.1304	1.5	1.66	63.29646	57.1956
Total weight					**76.2608**			**170.0616**	**114.3912**

				Partion wall					
Wall	Between /	Width/Dia	Depth(m)	Length	Weight(KI	Moment Arm		Moment	
						X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
A&B	1	0.15	3	5.572	35.1036	5.572	12.06	423.3494	195.5973
A&B	2	0.15	3	5.572	35.1036	5.572	6.561	230.3147	195.5973
A&B	3	0.15	3	5.572	35.1036	5.572	2	70.2072	195.5973
B&C	1	0.15	3	3.937	24.8031	9.509	12.06	299.1254	235.8527
B&C	2	0.15	3	3.937	24.8031	9.509	6.561	162.7331	235.8527
B&C	3	0.15	3	3.937	24.8031	9.509	2	49.6062	235.8527
C&D	1	0.15	3	2.9	18.27	12.41	12.06	220.3362	226.7307
C&D	2	0.15	3	2.9	18.27	12.41	6.561	119.8695	226.7307
C&D	3	0.15	3	2.9	18.27	12.41	2	36.54	226.7307
D&E	1	0.15	3	1.751	11.0313	14.16	12.06	133.0375	156.2032
D&E	2	0.15	3	1.751	11.0313	14.16	6.561	72.37636	156.2032
E&F	1	0.15	3	5.549	34.9587	19.71	12.06	421.6019	689.036
E&F	2	0.15	3	5.549	34.9587	19.71	6.561	229.364	689.036
1&2	A	0.15	3	5.5	34.65	0	12.06	417.879	0
1&2	B	0.15	3	5.5	34.65	5.572	12.06	417.879	193.0698
1&2	C	0.15	3	5.5	34.65	9.51	12.06	417.879	329.5215
1&2	D	0.15	3	5.5	34.65	12.41	12.06	417.879	430.0065
1&2	E	0.15	3	5.5	34.65	14.16	12.06	417.879	490.644
1&2	F	0.15	3	5.5	34.65	19.71	12.06	417.879	682.9515
2&3	A	0.15	3	4.561	28.7343	0	6.561	188.5257	0
2&3	B	0.15	3	4.561	28.7343	5.572	6.561	188.5257	160.1075
2&3	C	0.15	3	4.561	28.7343	9.51	6.561	188.5257	273.2632
2&3	D	0.15	3	4.561	28.7343	12.41	6.561	188.5257	356.5927
Total weight					**649.3473**			**5729.838**	**6581.177**

				Slab				
Panel	Lx(m)	Ly(m)	DL(KN/m2	Weight(KI	Moment Arm		Moment	
					X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
P1	4.561	5.572	5.44	138.2516	0	2	276.5031	0
P2	3.937	4.561	5.44	97.68421	5.572	2	195.3684	544.2964
P3	5.5	5.572	5.44	166.7142	0	6.561	1093.812	0
P4	3.937	5.5	5.44	117.795	5.572	6.561	772.8533	656.354
P5	2.9	5.5	5.44	86.768	9.509	6.561	569.2848	825.0769
P6	1.751	5.5	5.44	52.38992	12.409	6.561	343.7303	650.1065
P7	5.5	5.549	5.44	166.0261	14.16	6.561	1089.297	2350.929
P8	0.8	2.9	5.44	12.6208	9.509	5.92	74.71514	120.0112
C1	2	9.509	5.44	103.4579	0	0	0	0
C2	1.421	9.561	5.44	73.90882	0	0	0	0
C3	1.421	7.1	5.44	54.8847	0	6.561	360.0985	0
C4	2	3.909	5.44	42.52992	0	0	0	0
C5	2	10.2	5.44	110.976	9.509	12.06	1338.371	1055.271
C6	1.825	5.5	5.44	54.604	19.71	6.561	358.2568	1076.245
C7	2	7.37	5.44	80.1856	14.16	4.706	377.3534	1135.428
C8	6	1.751	5.44	57.15264	12.41	5.05	288.6208	709.2643
Total weight				**1415.949**			**7138.265**	**9122.982**

$X_m = \frac{\sum(W_i * X_i)}{\sum W_i}$ $Y_m = \frac{\sum(W_i * Y_i)}{\sum W_i}$

=15256.9475/2507.35= 6.08 =19006.85/2507.35=7.58

Base shear determination

$$F_b = S_d(T_1) \cdot m \cdot \lambda \quad (4.5)$$

where

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

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

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

λ is the correction factor, the value of which is equal to: $\lambda = 0{,}85$ if $T_1 \leq 2\ T_C$ and the building has more than two storeys, or $\lambda = 1{,}0$ otherwise.

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

Activate Win

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

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

$$T_1 = C_t \cdot H^{3/4} \quad (4.6)$$

where

C_t is 0,085 for moment resistant space steel frames, 0,075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0,050 for all other structures;

H is the height of the building, in m, from the foundation or from the top of a rigid basement.

For concrete moment resisting frame Ct=0.075, H=15.80m

$T1=0.075*15.80^{0.75}=0.59sec$

4Tc(s) =4*0.25=1sec

T1≤ min 2sec

T1=0.59sec < 1sec….. ok

Take T1=0.59sec

Table 19 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 20Values 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 21Values 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 22Recommended 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 23Ground 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			

Table 24 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:

$$0 \le T \le T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3}\right)\right] \quad (3.13)$$

$$T_B \le T \le T_C : \; S_d(T) = a_g \cdot S \cdot \frac{2,5}{q} \quad (3.14)$$

$$T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T}\right] \\ \ge \beta \cdot a_g \end{cases} \quad (3.15)$$

$$T_D \le T : \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2}\right] \\ \ge \beta \cdot a_g \end{cases} \quad (3.16)$$

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 αu/α1

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

qo = 3 αu/α1 - 0.2*3 αu/α1 = 0.8 * 3 αu/α1

- α1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant;
- αu is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor αu may be obtained from a nonlinear static (pushover) global analysis.

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

Frames or frame-equivalent dual systems.

− One-storey buildings: αu/α1=1,1;

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

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

αu/α1 = 1.3

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

$$q = q_o k_w \geq 1{,}5$$

Where K_w =1

q_o=3 αu/α1

q=0.8*3*1.3*1=3.12

$$T_C \leq T \leq T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \dfrac{2{,}5}{q} \cdot \left[\dfrac{T_C}{T}\right] \\ \geq \beta \cdot a_g \end{cases}$$

Where T_c=0.8 ,T=0.59

$\propto_g$ =3.5

S=1.35

Sd(T) = Max 5.133

0.700

Sd(T)=5.133

$$F_b = S_d(T_1) * W_{Tot}$$

=5.133*9389.24=48,199.10

$$F_t = 0.07 T_1 F_b$$

=0.07*0.59*48,199.10=1,990.62

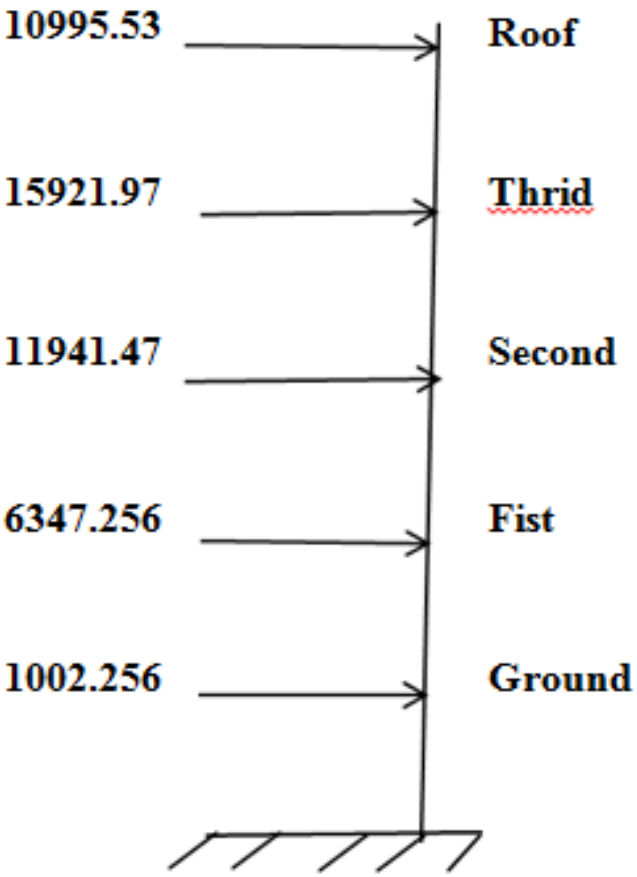
5.2 Determination of D-Values of Frames

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 25Computation of Story Shear Force(Fi)

				BASE SHEAR				
Story	Remark	Wi	hi	Wihi	Fb	Ft	Fb-Ft	Fi
Ground Fl	W1	1142.74	3	3428.22	48,199.10	1,990.62	46,208.48	1002.256
First Floor	W2	3618.47	6	21710.82	48,199.10	1,990.62	46,208.48	6347.256
Second Fl	W3	4538.43	9	40845.87	48,199.10	1,990.62	46,208.48	11941.47
Third Floo	W4	4538.43	12	54461.16	48,199.10	1,990.62	46,208.48	15921.97
roof Floor	W5	2507.35	15	37610.25	48,199.10	1,990.62	46,208.48	10995.53
			Total Wihi	**158056.3**				


10995.53
Roof
15921.97
Thrid
11941.47
Second
6347.256
Fist
1002.256
Ground

CHAPTER SIX

6 FrameAnalysis

6.1 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

Plotgridcoordinatesthatrepresentthegiven 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: Uniaxial

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

6.2 Analysis output of ETABS 2016 beam shear force diagram

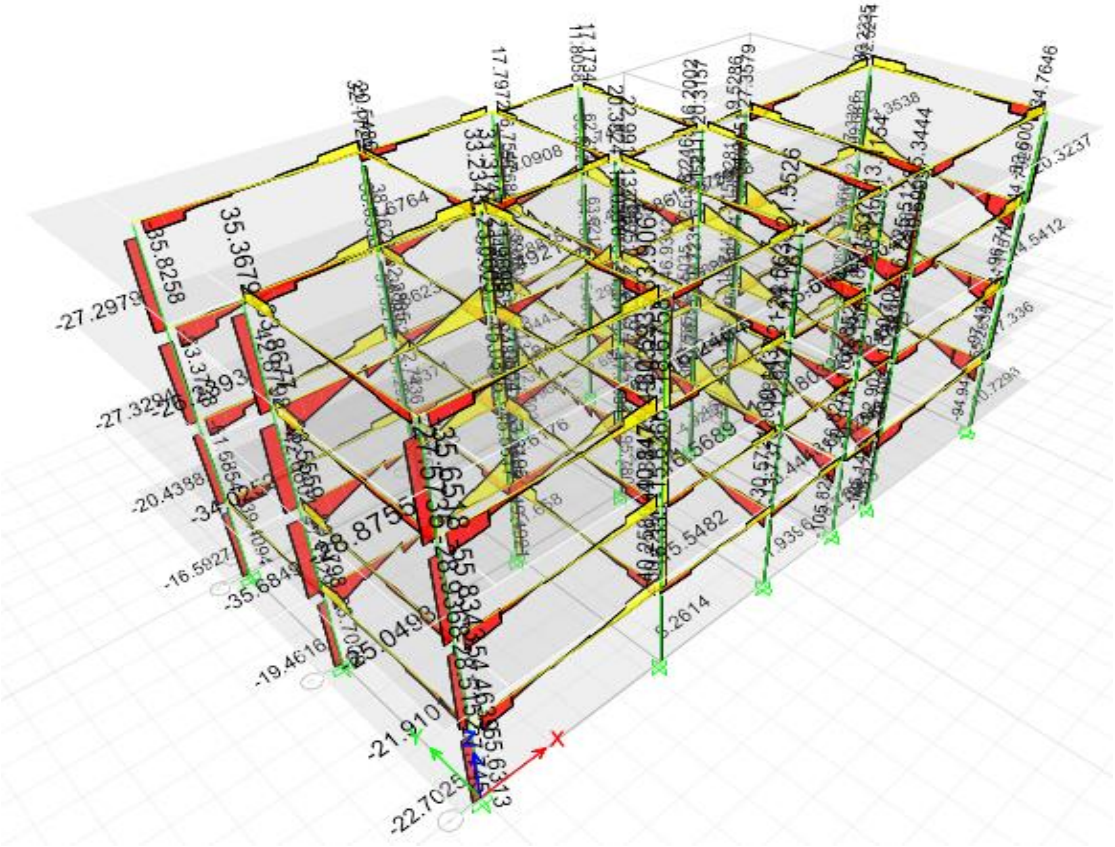


Figure 6.8Beam sheer force of ETABS output

6.3 Analysis output of ETABS 2016 beam bending moment diagram

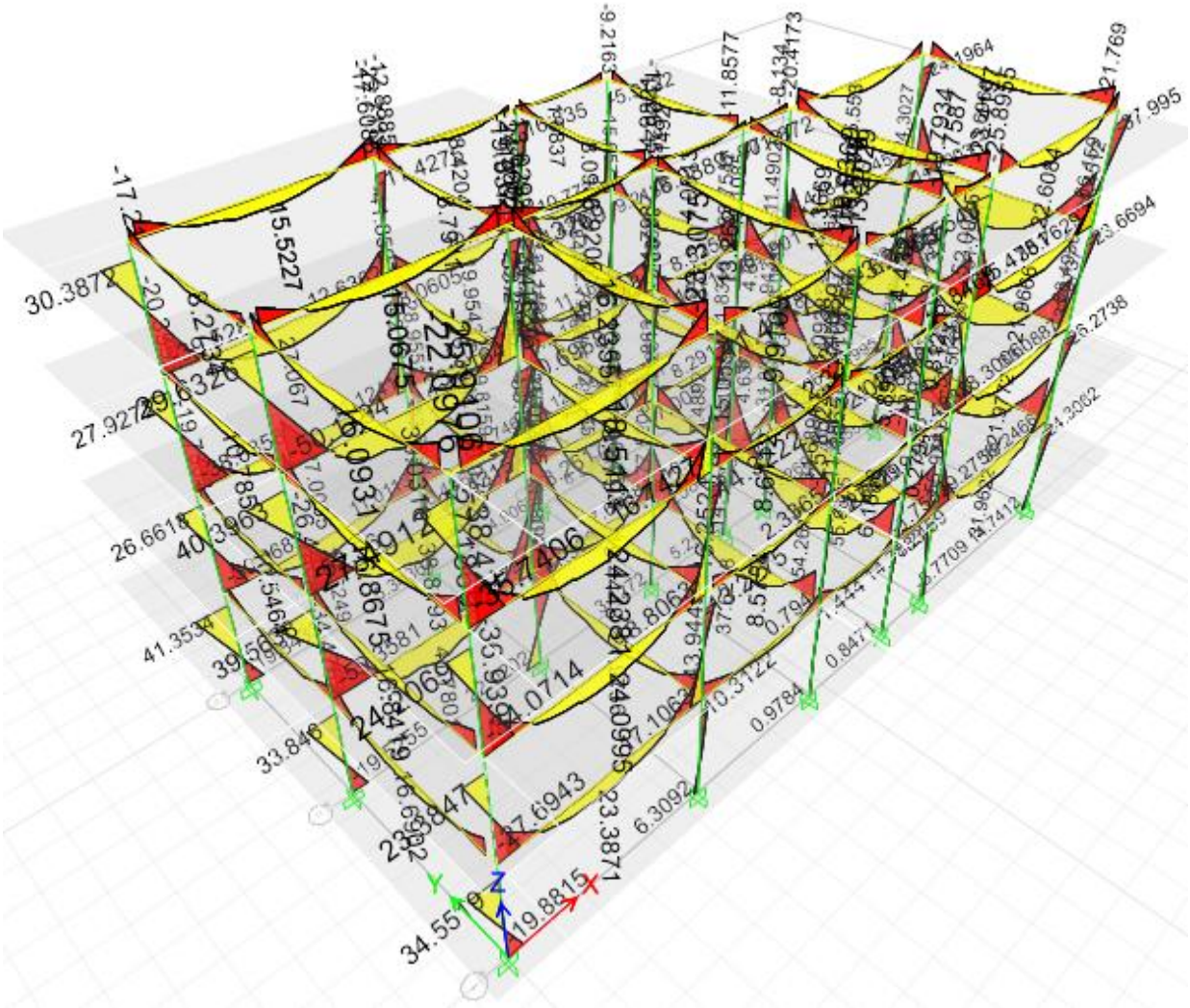


Figure 6.9ETABS output of beam bending moment

6.4 Analysis output of ETABS 2016 column axial force and moment diagram

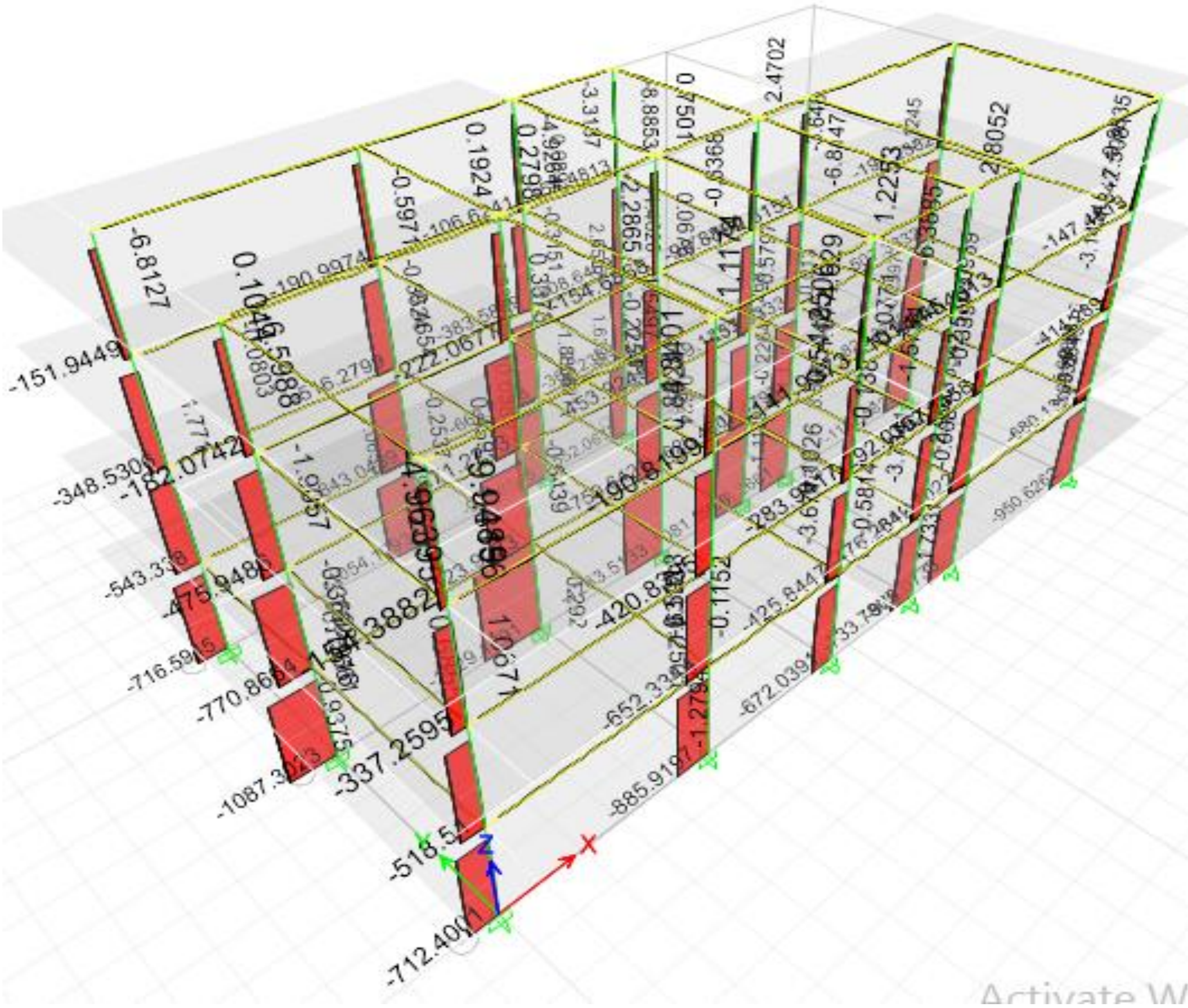


Figure6. 10ETABS output of column axial force

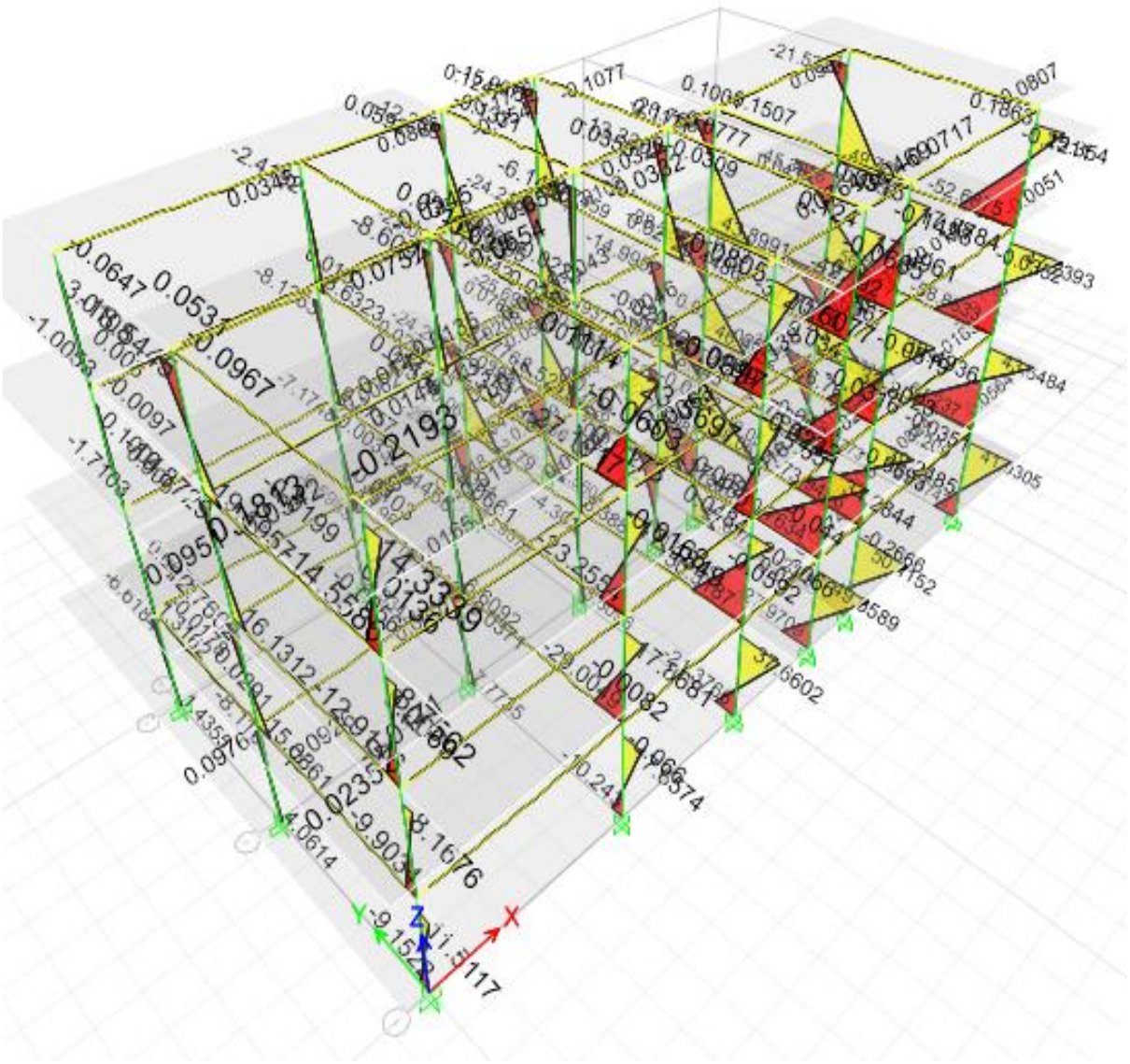


Figure 6.11Moment output of ETABS

6.5 Analysis output of ETBS 2016 3D deflection analysis

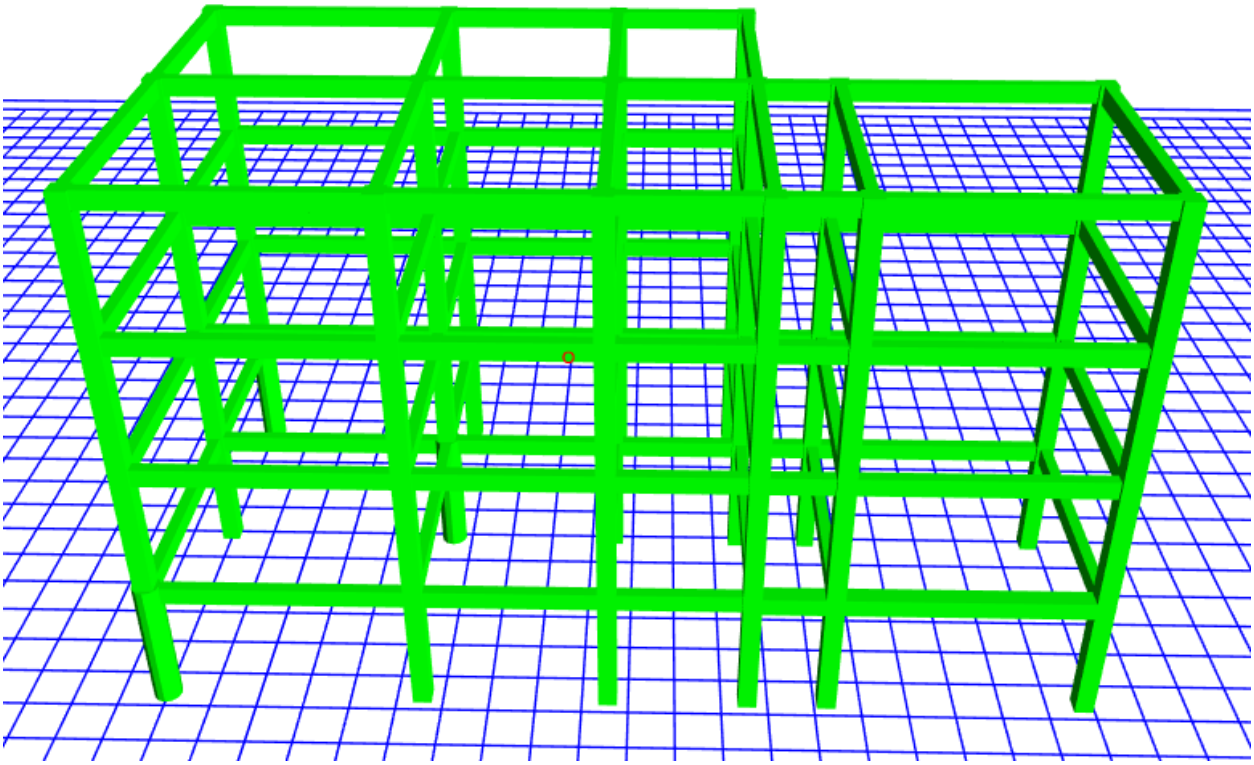


Figure 6.123D deflection analysis

CHAPTER SEVEN

7 Analysis and Design of Beam

7.1 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 themagnitude 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 – 400x250mm

A. Concrete
   - Grade of concrete C-25
   - Concrete cube strength, $f$cu=25MPA
   - Concrete characteristic strength, , $f$ck=0.8*$f$cu=0.8*25=20MPA
   - Concrete design strength, $f$cd=0.85*$f$ck/ϒc=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

7.1.1 Flexural Design

STEP 1: Concrete Cover

The concrete cover is the distance between the surface of the reinforcement closestto 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.

$Cmin = Max\ \{Cmin,,+ \Delta Cdur,\gamma - \Delta Cdur,st - \Delta Cdur,add 10mm$

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 7.26Value of minimum cover, Cmin,dur, requirements with regard to durability for reinforcement steel

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

Cmin,dur=10mm

$Cmin,=Cmin=20mm$

$10mm$

$Therefore;\quad Cmin= 20mm$

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

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

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

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{\frac{3}{2}}\right] * F1 * F2 * F3 \dots\dots\dots\dots if\ \rho \le \rho_o$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho-\rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho}}\right] * F1 * F2 * F3 \dots\dots\dots\dots if\ \rho \geq \rho_o$$

Where;

l/d - is the limit span/depth

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

ρo - is the reference reinforcement ratio

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) fckis in MPa units.

$$F1 = \frac{310}{\sigma_s} = \frac{500}{(f_{yk} * {}^{A_{s,req}}/_{A_{s,prov}})}$$

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.

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

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

$$\rho_o = \sqrt{f_{ck}} * 10^{-3} = \sqrt{20} * 10^{-3} = 0.447\%$$

$$N = 11 + 1.5\sqrt{f_{ck}} = 11 + 1.5\sqrt{20} = 17.71$$

$$F1 = \frac{310}{\sigma_s} = \frac{500}{(f_{yk} * {}^{A_{s,req}}/_{A_{s,prov}})} = \frac{500}{(400)} = 1.75$$

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

Now: *determineKusingtable* 7.4*Nof* (ministry of ethiopian education, 2015)

So K = 1.3, F3 =1, and F2 = 1

$l/d = K*N*F1*F2*F3 = 1 * 17.71 * 1.75 * 1 * 1 = 31$

L/d =31 Where; l = 5572 mm, this is the maximum span length.

d = 5572/31 = 179.74mm

So Effective depth d, = 180mm

Depth, D=d+d'

Where: d' = cover + Φstr+Φlon/2$use\Phi str = 8mmand\Phi long = 20mm$

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

Example from story-4 Axis-A

Suport moment=22.09KNm

b =0.25m

d=0.4m

$fck = 20MPA$

Where:$K=M_{sd}/b*d^2*fck=22.09/0.25*0.4*20=0.138<0.167$……**single reinforced beam**

STEP 4: Provide reinforcement

$Z=d/2(1+\sqrt{1-3.53k})=352/2(1+\sqrt{1-3.53*0.138})=302.04 \leq 0.95d = 334.4$

$As,calc=Msd/fyd*z=22.09/347.83*302.04=210.26mm^2$

STEP 5: Check for minimum & maximum reinforcement area

$A_{s,min} = \frac{0.26 f_{ctm} b_t d}{f_{yk}} = 0.0013 b_t d$ =130mm^2

$A_{s,max} = 0.04bd$ =4000mm^2

Therefore, $A_{s,min} \leq A_{s,calc} \leq A_{s,max}$

$A_{s,provided} = A_{s,calc}$ =210.26mm^2

STEP 6: No of steel, n

$a_s = \frac{\pi D^2}{4}$ =3.14*20^2/4=314mm^2

$$n = \frac{A_{s,calc}}{a_s} =$$

=210.26/314=0.669≈4Ø20

So,use 4Ø20

Then, the other Storys in for each axis the same as the above step to determine by exile.

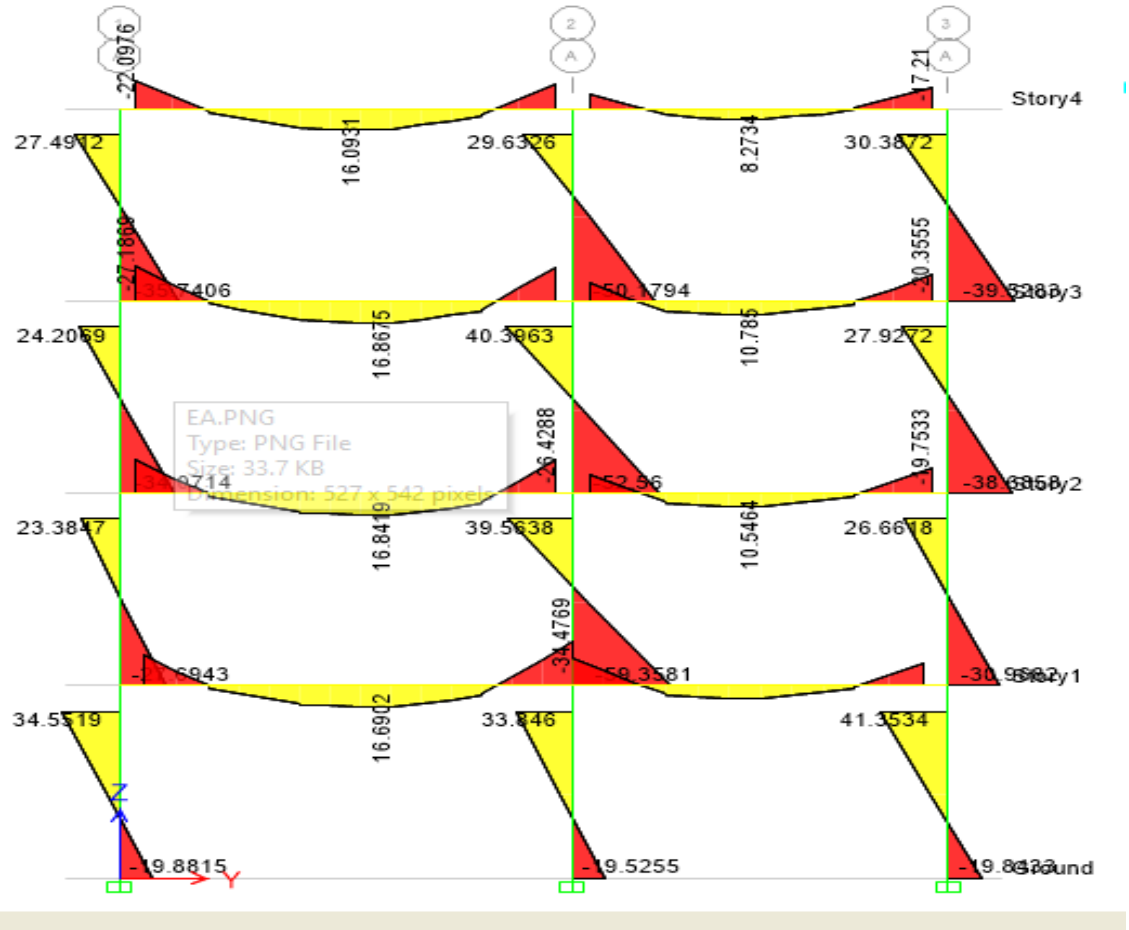


Figure7.13Bending moment diagram on Axis A

Reinforcement of a beam on axis A……**Appendix 6A**

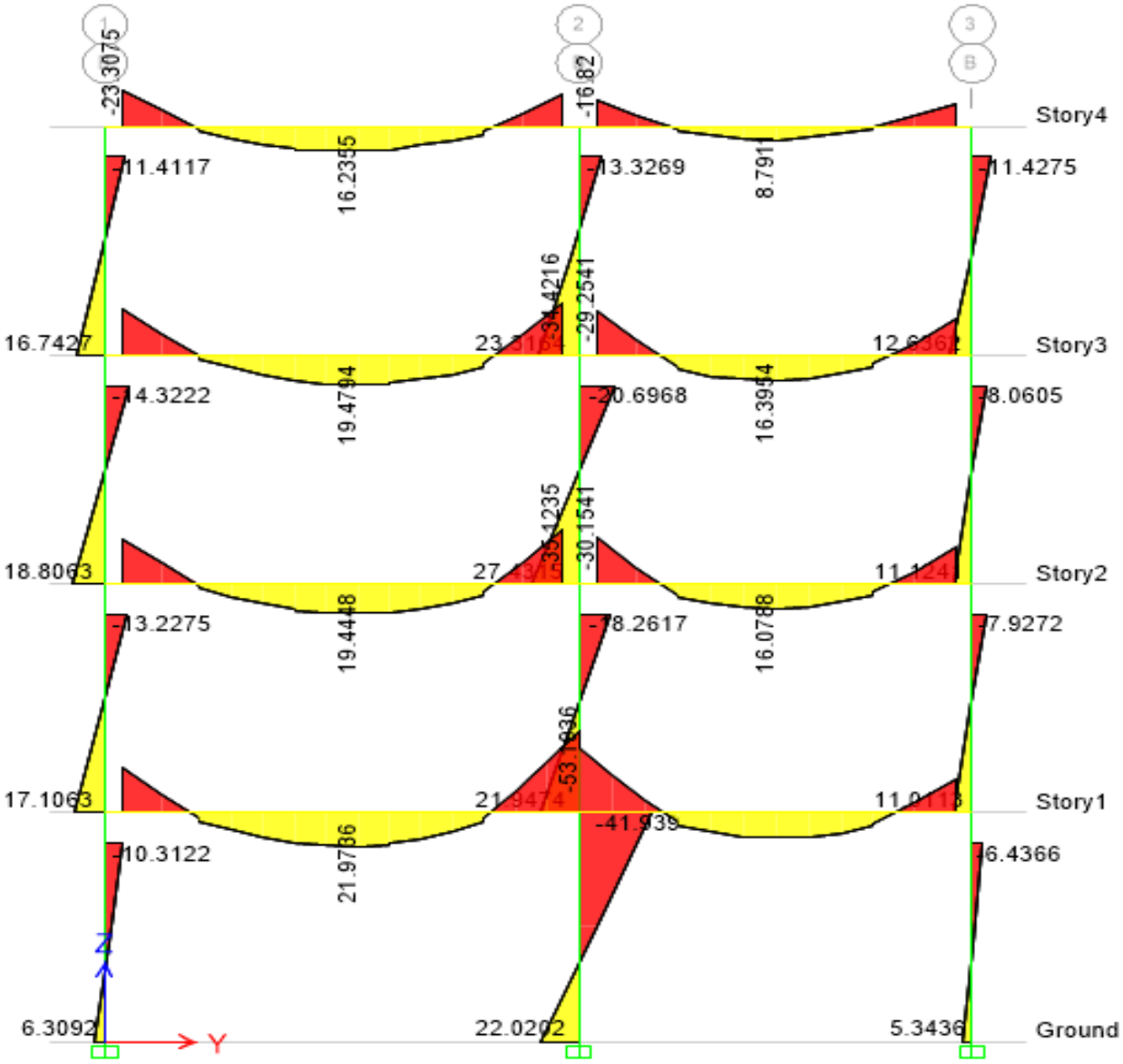


Figure 7.14Bending moment diagram on Axis B

Reinforcement of a beam on axis B**……. Appendix 6B**

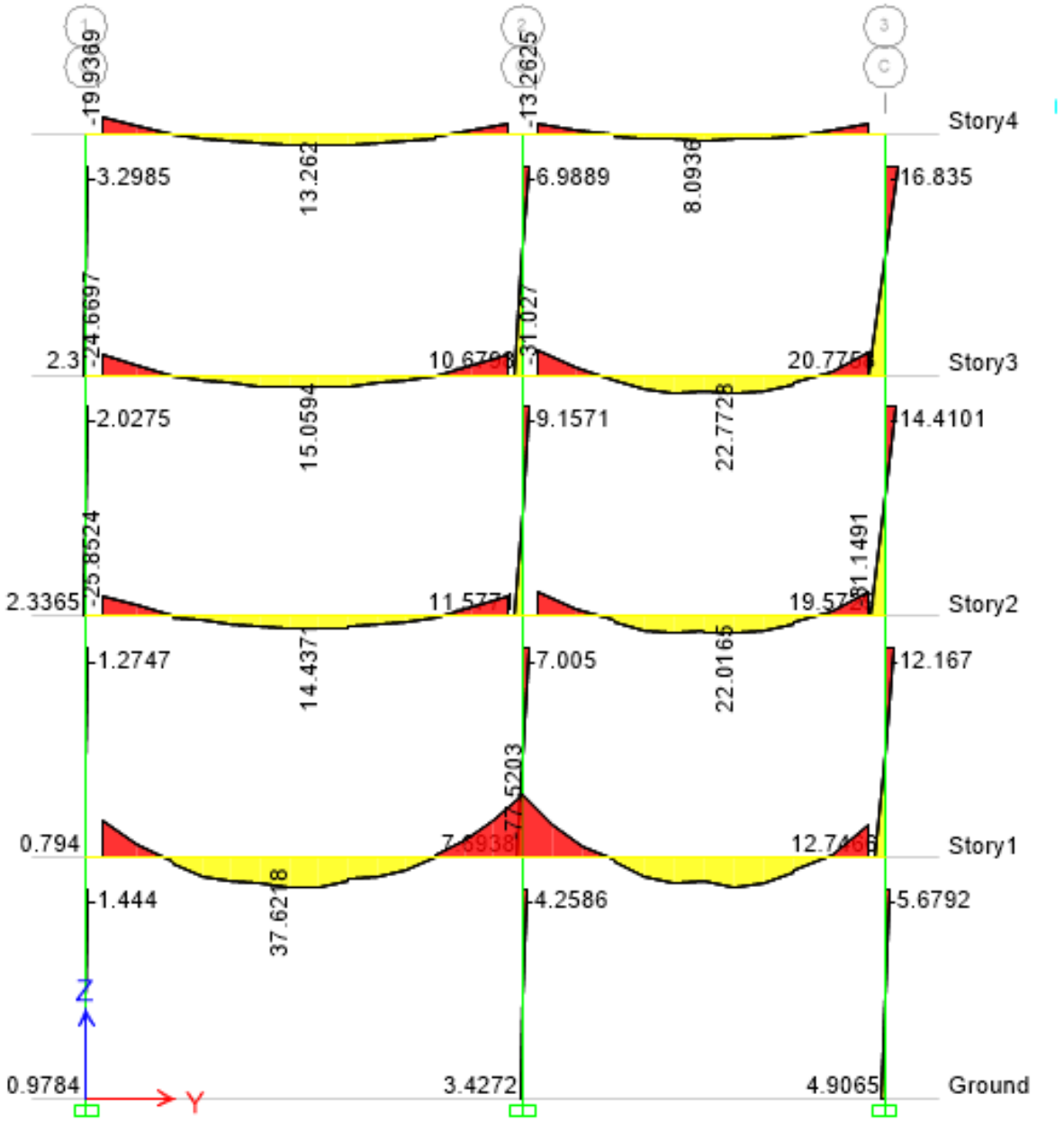


Figure 7.15Bending moment diagram on Axis C

Reinforcement of a beam on axis C……. **Appendix 6C**

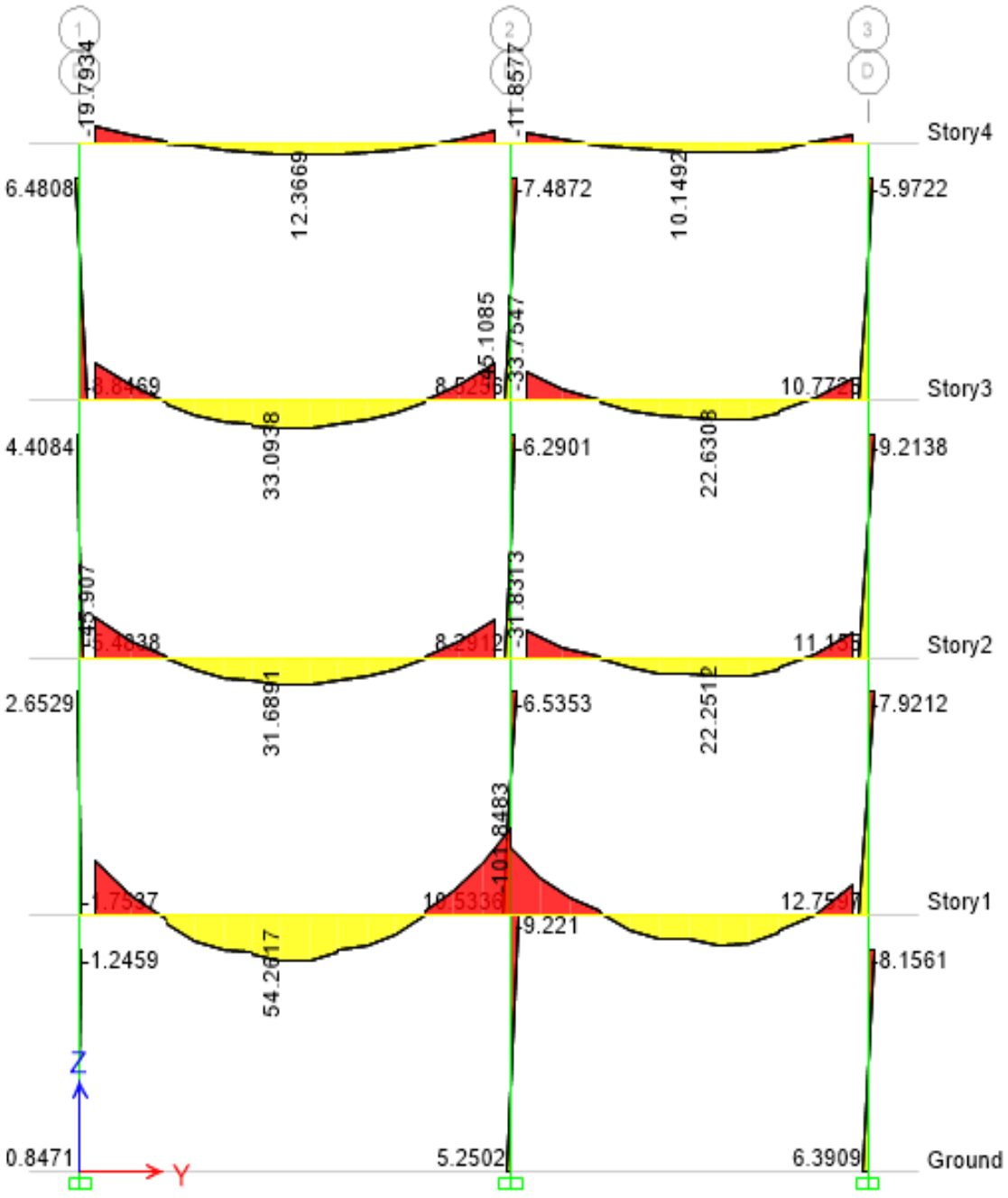


Figure7.16Bending moment diagram on Axis D

Reinforcement of a beam on axis D……. **Appendix 6D**

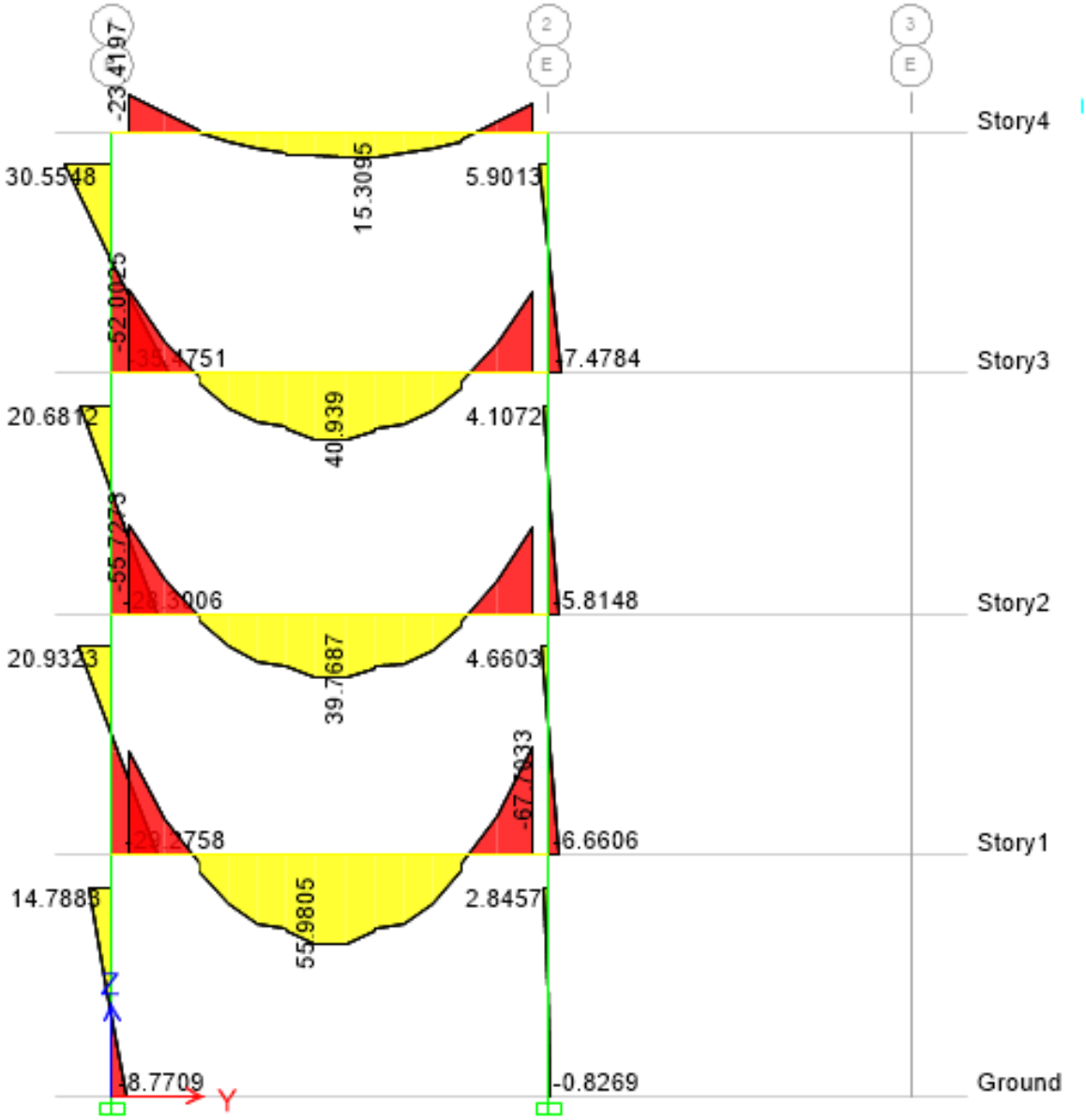


Figure 7.17Bending moment diagram on Axis E

Reinforcement of a beam on axis E……. **Appendix 6E**

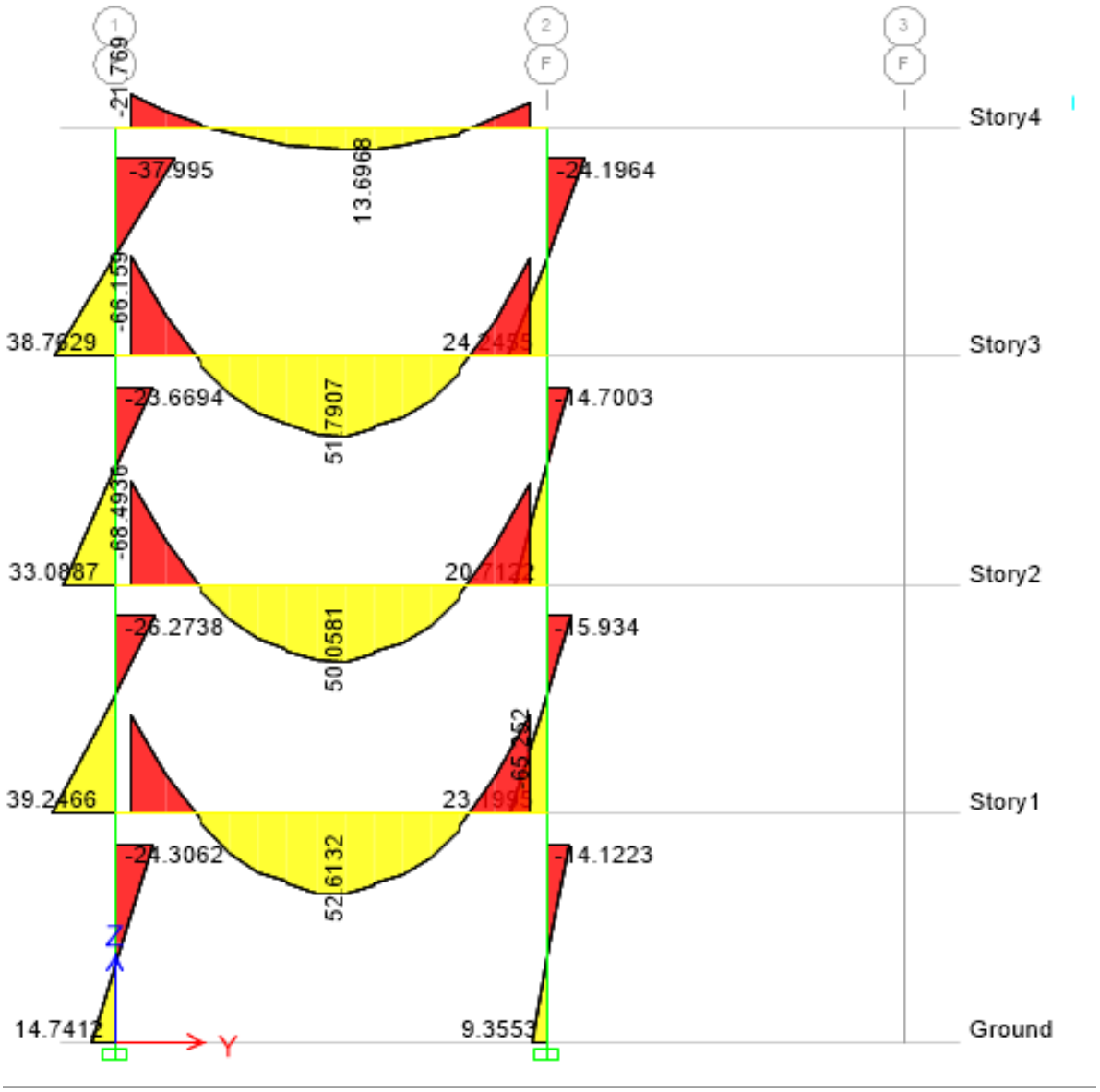


Figure7.18Bending moment diagram on Axis F

Reinforcement of a beam on axis F……**Appendix 6F**

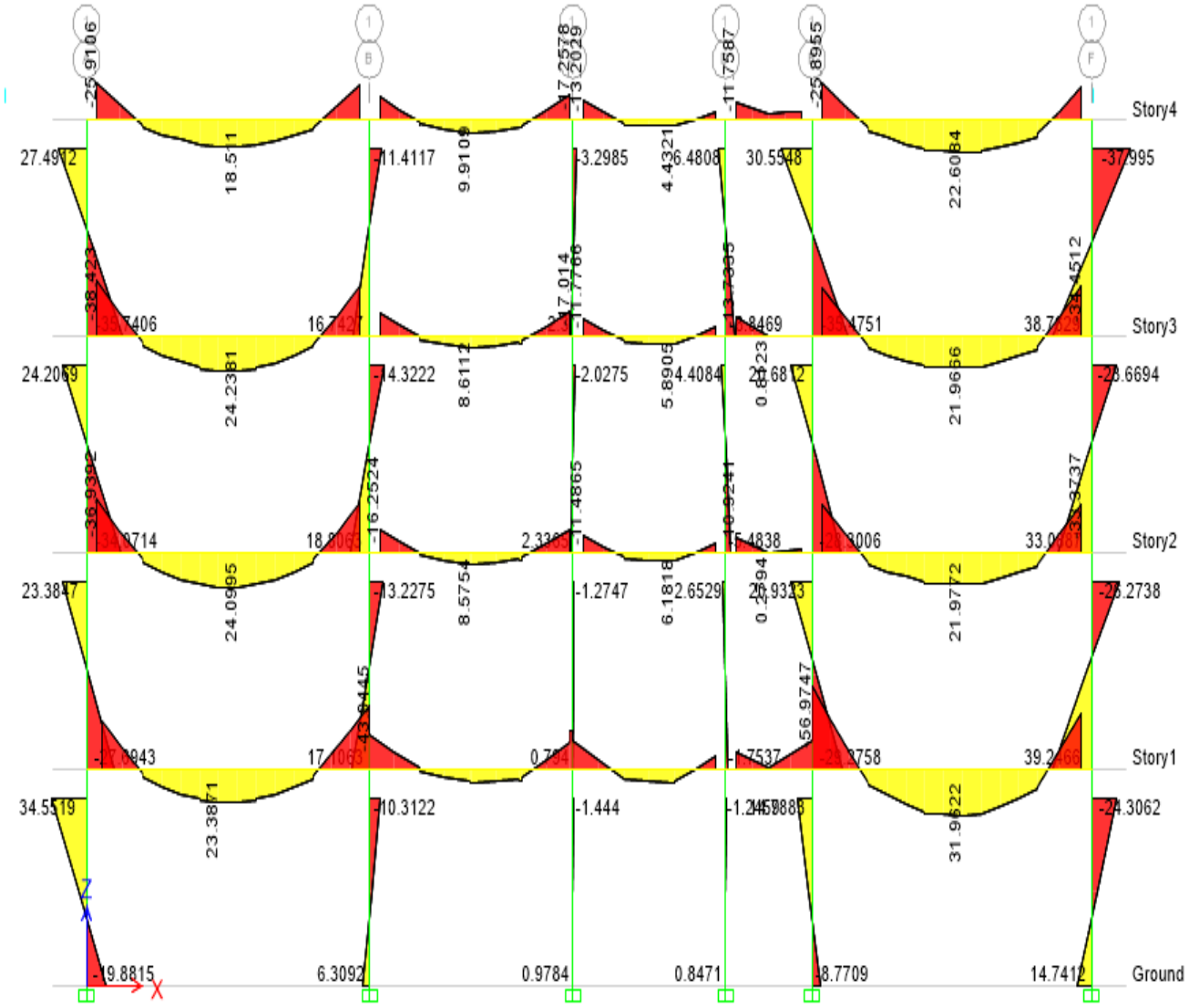


Figure 7.19Bending moment diagram on Axis1

Reinforcement of a beam on axis 1……. **Appendix 61**

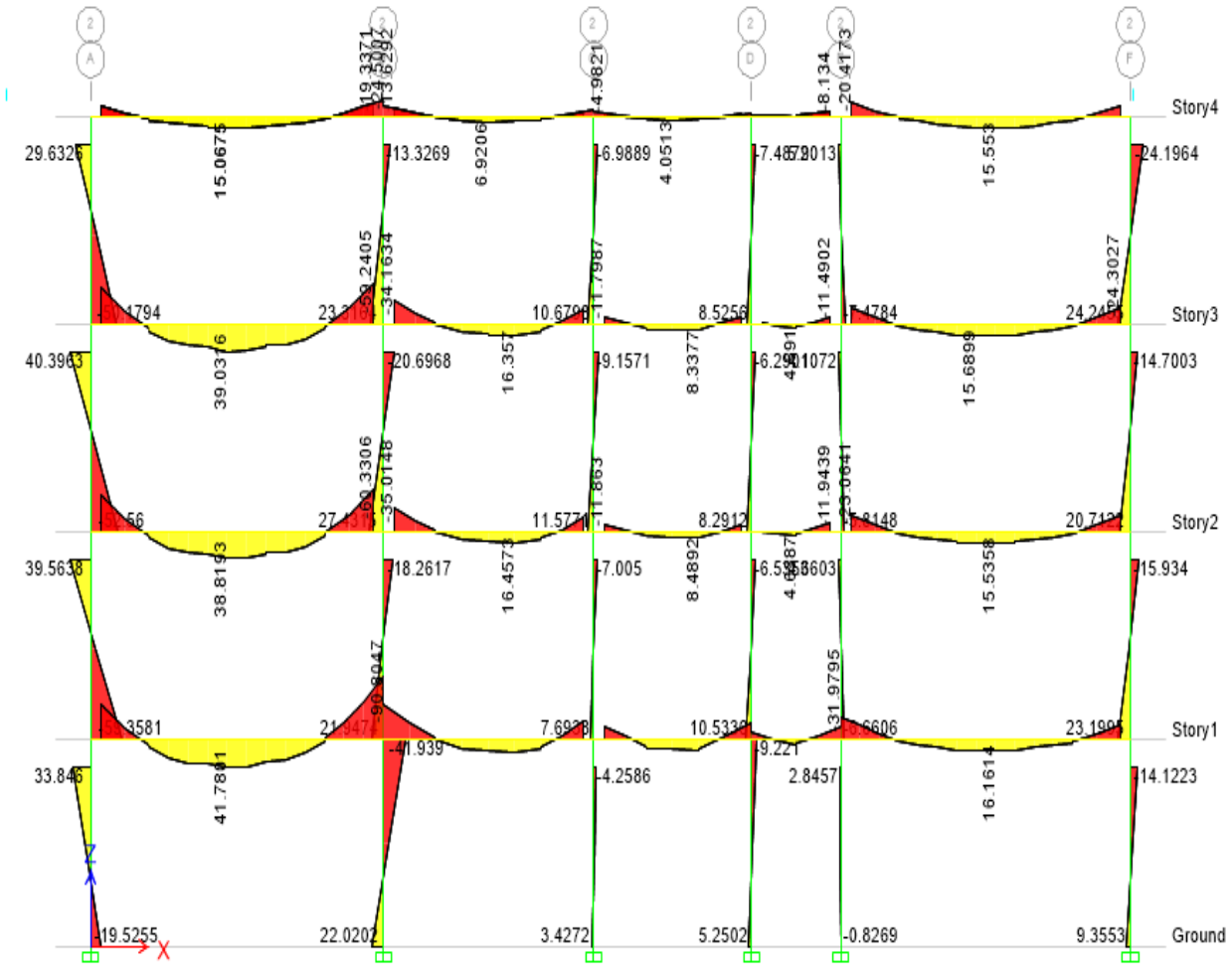


Figure 7..20Bending moment diagram on Axis 2

Reinforcement of a beam on axis 2……. **Appendix 62**

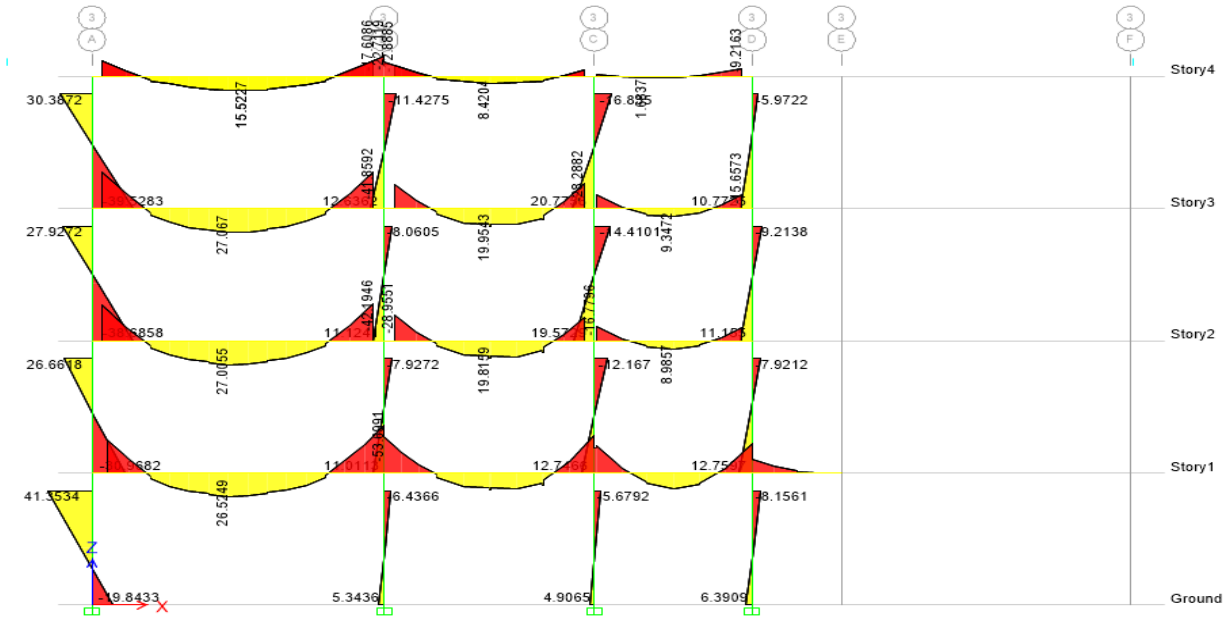


Figure 7.21Bending moment diagram on Axis 3

Reinforcement of a beam on axis 3……. **Appendix 63**

7.1.2 Shear Design

The concrete itself can resist shear by a combination ofuncrackedconcretein thecompression zone. The dowelling action of the bending reinforcement and aggregateinterlocking across tension crack but, because concrete is weak in tension, the shear reinforcement is designed to resist all the tensile stress caused by the sheer force. Even where the shear force is small near the Centre span of beam a minimum amount of reinforcement in the form of links must be provided in order to form a cage supporting the longitudinal reinforcement and to resist any tensile stresses due to factors such as thermal movement and shrinkage of concrete. Behavior of beams without web reinforcement the forces transferring shear across an inclined crack in a beam without web reinforcements are illustrated in below Figure

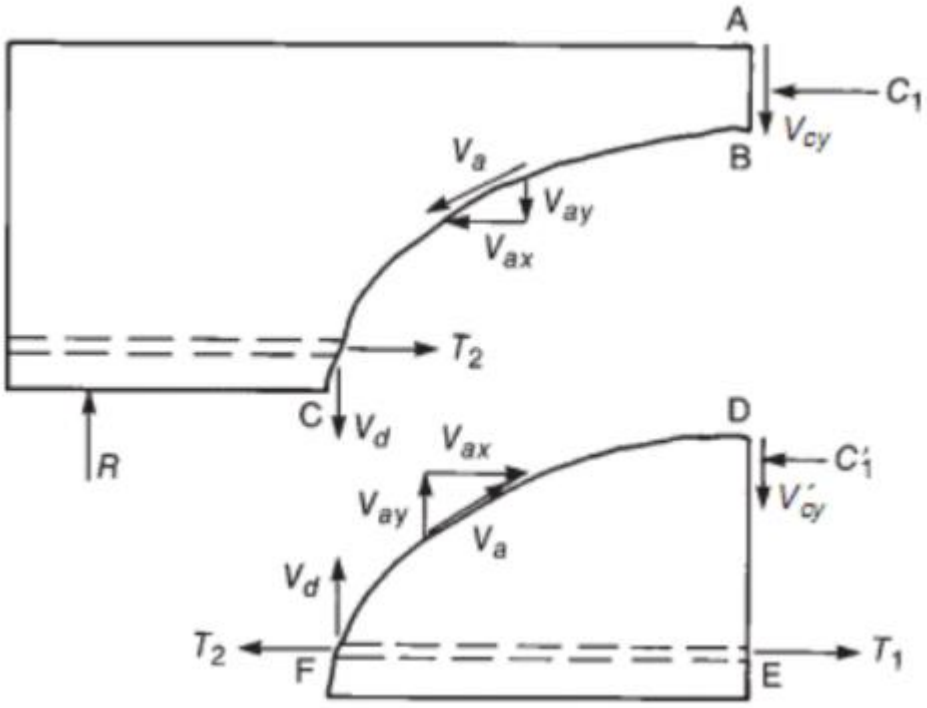


Figure 7.22Internal forces in a cracked beam without web reinforcement

Behavior of beams with web reinforcement

Inclined cracking causes the shear strength of beams to drop below the flexural capacity. The purpose of web reinforcement is to ensure that the full flexural capacity can be developed.

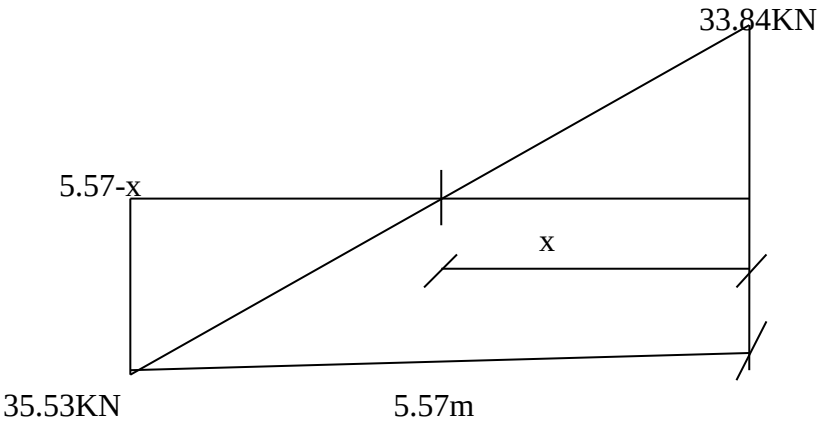
For the verification of the shear resistance the following symbols are defined:

- VRd,cis the design shear resistance of the member without shear reinforcement.
- VRd,sis the design value of the shear force which can be sustained by the yielding shear reinforcement.
- VRd,maxis the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.

  In regions of the member where VEd≤VRd,c,no calculated shear reinforcement is necessary. VEdis the design shear force in the section considered resulting from external loading and prestressing (bonded or unbounded)

  In regions where VEd>VRd,, sufficient shear reinforcement should be provided in order that VEd≤VRd

  Sample Beam Design for Shear



33.84/x=35.53/5.57-x

X=2.72

33.84/2.72=VEd/2.72-0.352

VEd=29.46KN

A. Concrete capacity

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

$$V_{Rd,c} = [C_{Rd,c} k (100\,\rho_l f_{ck})^{1/3} + k_1 \sigma_{cp}]\, b_w d$$

with a minimum of $V_{Rd,c} = (V_{min} + k_1 \sigma_{cp})\, b_w d$

Where;

fck = 20MPa

$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{352}} = 1.75 \leq 2$

$\rho 1 = A_{s1}/b_w * d = 314/250*352 = 0.035 \leq 0.02$

Therefore;ρ1=0.02

$\sigma_{cp} = N_{Ed}/A_{C=}0/250*400=0$

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

ACis the area of concrete cross section .

VRd,c is

CRd,c=0.18/γc=0.18/1.5=0.12

K_1=1.5

$$V_{Rd,c} = [C_{Rd,c}K(100\rho_1 f_{ck})^{1/3} + K_1\sigma_{cp}]b_w d$$

=[0.12*1.75(100*0.02*20)$^{1/3}$+1.5*0]*250*352

VRd, c≤VEd=24.64KN≤29.46KN….Shear reinforcement is required.

B. Diagonal compression check of concrete

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

$$v = 0.6\ for\ f_{ck} \leq 60MPa\ from\ ESEN\ 1992-1:2015$$

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

Where,

$$\alpha_{cw} = 1\ and\ cot\theta = 2.5,$$

b_w=250mm

Z=0.9d=0.9*352=316.8

$f$cd=11.33MPa

$$v_1 = 0.6\left(1 - \frac{f_{ck}}{250}\right)$$

=0.6*(1-20/250)=0.552

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

=2.5*250*316.8*0.552*11.33(1⁄2.5+0.4) =162.64KN≥VEd=33.84KN

C.Calculate the required shear reinforcement

The ratio of shear reinforcement is given by;

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

Where:

$\rho_w$is the shear reinforcement ratio,$\rho_w$should not be less thanρ_w ,min

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

s is the spacing of the shear reinforcement measured along the longitudinal axis of the memberb$_w$is the breadth of the web of the member

$\alpha$is the angle between shear reinforcement and the longitudinal axis.

$\rho_{w,min=0.08}\sqrt{fck}/fyk=0.08\sqrt{20}/400=0.00089$

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

$S_{l,max}=0.75d(1+\cot\propto)=0.75*352(1+\cot 90)=238.55mm$

Where:

$\alpha$is the inclination of the shear reinforcement to the longitudinal axis of the beam.

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

$S_{b,max}=0.6d(1+\cot\propto)=0.6*352(1+\cot 90)=190.85mm$

- The transverse spacing of the legs in a series of shear links should not exceed $S_{t,max}$

$S_{t,max}=0.75d\ 0.75*352=264\leq 600mm$

$S_{min}=Asw/\rho w*b_w*\sin\propto=50.24/0.00089*250*1=225.79mm$

- Required shear reinforcement

  $Asw/S=VEd/0.78df yk\cot\theta$

  $S_{cal}=Astri*0.78*d*fyk*\cot\theta/VEd$

  $=50.24*0.78*352*400*2.5/29.46=468.22mm$

  $S_{prov}=\min S_{l,max}, S_{b,max}, S_{ca}$ but $S_{pro}\geq S_{min}$

  $S_{prov}=\min\ 238.55mm, 190.85mm, 468.22mm$

  $S_{pro}\geq S_{min}=190.85mm\geq 225.79$….NOT OK

  Therefore:$S_{prov}=225.79mm$

  Therefore, Use Φ8mm c/c 220mm

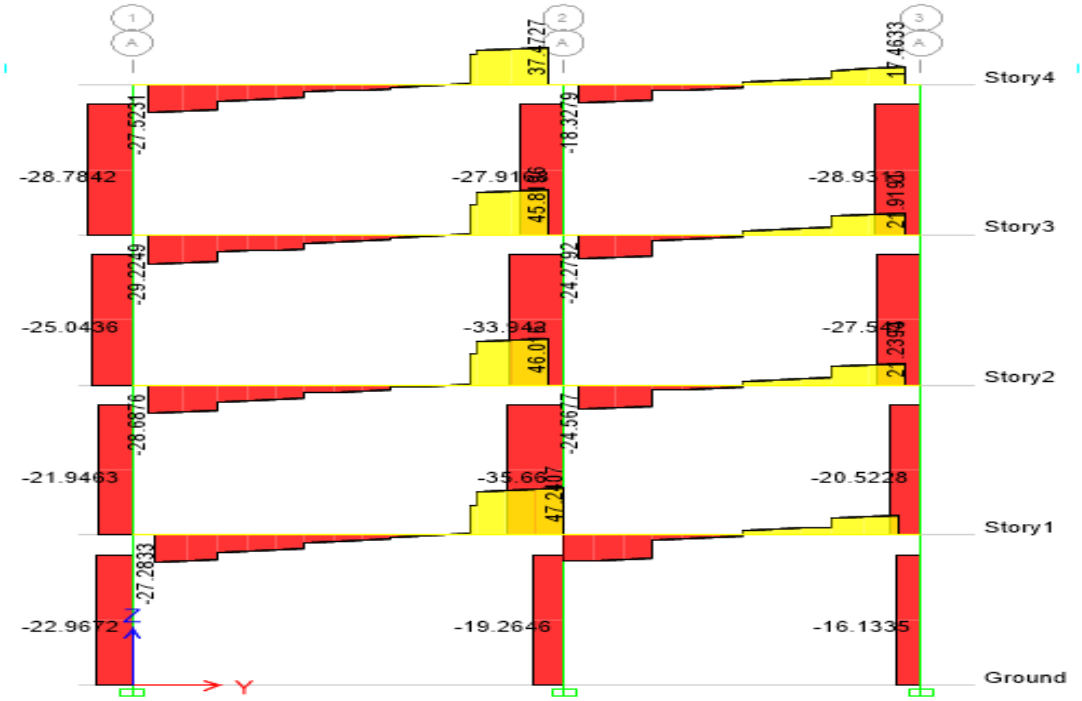


Figure 7.23 One sample example of Shear force diagram of beam in axis A

Anchorage of longitudinal reinforcement

Reinforcing bars, wires or welded mesh fabrics shall be so anchored that the bond forces are safely transmitted to the concrete avoiding longitudinal cracking or spalling. Transverse reinforcement shall be provided if necessary.

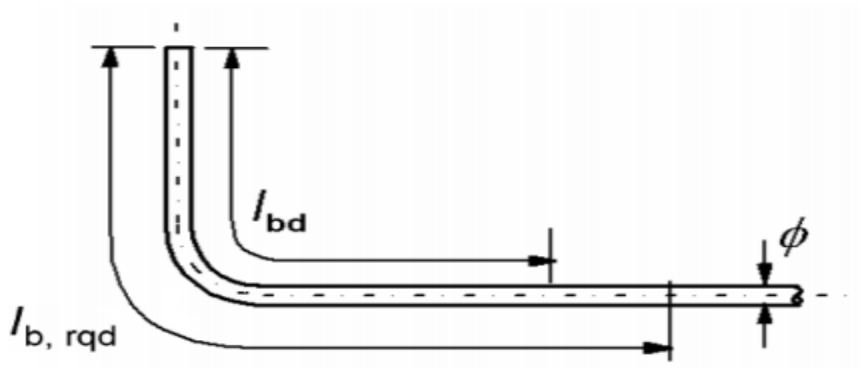


Figure 7.24Basic tension anchorage length, lb, rqd, for any shape measured along the centerline

$$l_{b,rqd} = (\phi/4)\,(\sigma_{sd}/f_{bd})$$

For bent bars the basic required anchorage length, lb,rqd, and the design length, lbd, should be measured along the centre-line of the bar.

The calculation of the required anchorage length shall take into consideration the type of steel and bond properties of the bars. The ultimate bond strength shall be sufficient to prevent bond failure.

The design value of the ultimate bond stress, fbd, may be taken as:

$$fbd = 2.25\eta_1\eta_2 fctd$$

Where:

Fctd is the design value of concrete tensile strength, Fctd = Fctk/1.5=1.0315

η1 is a coefficient related to the quality of the bond condition and the position of the bar during concreting.

η_1 = when 'good' conditions areobtained and

η_1= 0.7 for all other cases and for bars in structural elements built with slip forms, unless it can be shown that good bondconditions exist.

Therefore,η_1 = 1.0

$\eta_2$is related to the bar diameter:

η_2 = 1.0 for $\phi \leq 32$mm

η_2 = (132 - ϕ)/100 forϕ> 32 mm

Therefore, η_2 = 1.0 sinceϕ =20mm ≤32 mm

fbd = 2.25 η1 η_2 fctd = 2.25x1.0x1.0x1.0315 = 2.32MPa

lb,rqd = (ϕ/ 4) (σsd / fbd) = (20/4)(347.83/2.32) = 749.6mm

The design anchorage length, lbd, is:

lbd = α1 α2α 3 α4α5 lb,rqd≥ lb,min

Where:α1, α2, α3,α4and α5 are coefficients given in Table 8.2:

α1is for the effect of the form of the bars assuming adequate cover

α2 is for the effect of concrete minimum cover

α3is for the effect of confinement by transverse reinforcement

α4 is for the influence of one or more welded transverse bars (ϕt>0.6ϕ) along the design anchorage length lbd

α5is for the effect of the pressure transverse to the plane of splitting along the design anchorage length

The product (α2 α3 α5) ≥ 0.7

lb,rqdis basic anchorage length, obtained earlier.

lb,minis the minimum anchorage length if no other limitation is applied:

For anchorages in tension: lb,min≥ max{0.3lb,rqd; 10ϕ; 100 mm}

For anchorages in compression: lb,min≥ max{0.6lb,rqd ; 10ϕ; 100 mm}

Table 7.27Value of α_1, α_2, α_3, α_4 and α_5 coefficients

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Shape of bars	Straight	$\alpha_1 = 1.0$	$\alpha_1 = 1.0$
	Other than straight (see Figure 8.1 (b), (c) and (d)	$\alpha_1 = 0.7$ if $c_d > 3\phi$ otherwise $\alpha_1 = 1.0$ (see Figure 8.3 for values of c_d)	$\alpha_1 = 1.0$
Concrete cover	Straight	$\alpha_2 = 1 - 0.15\,(c_d - \phi)/\phi$ ≥ 0.7 ≤ 1.0	$\alpha_2 = 1.0$
	Other than straight (see Figure 8.1 (b), (c) and (d))	$\alpha_2 = 1 - 0.15\,(c_d - 3\phi)/\phi$ ≥ 0.7 ≤ 1.0 (see Figure 8.3 for values of c_d)	$\alpha_2 = 1.0$
Confinement by transverse reinforcement not welded to main reinforcement	All types	$\alpha_3 = 1 - K\lambda$ ≥ 0.7 ≤ 1.0	$\alpha_3 = 1.0$
Confinement by welded transverse reinforcement*	All types, position and size as specified in Figure 8.1 (e)	$\alpha_4 = 0.7$	$\alpha_4 = 0.7$
Confinement by transverse pressure	All types	$\alpha_5 = 1 - 0.04p$ ≥ 0.7 ≤ 1.0	-

lbd = α1 α2 α3 α4 α5 l b,rqd = 1x0.7x0.7x0.7x0.7x749.6 = 180mm

But, lbd ≥lb,min

For anchorages in tension:

lb,min≥ max{0.3lb,rqd; 10ϕ; 100 mm}

lb,min≥ max{0.3x749.6=224.89mm ; 10ϕ =200mm; 100 mm}

lb,min≥ max{225mm; 200mm; 100 mm}

lb,min≥225mm

For anchorages in compression:

lb,min≥ max{0.3lb,rqd; 10ϕ; 100 mm}

lb,min ≥ max{0.3x749.6=224.89mm ; 10ϕ =200mm; 100 mm}

lb,min≥ max{225mm; 200mm; 100 mm}

lb,min ≥225mm

The design anchorage length, lbd, is:

- For tension, lbd =230mm
- For compression, lbd =230mm

Anchorage of bottom reinforcement at intermediate supports

The anchorage length should not be less than 10ϕ (for straight bars) or not less than the diameter of the mandrel (for hooks and bends with bar diameters at least equal to 16 mm) or twice the diameter of the mandrel. This reinforcement should be continuous which may be achieved by means of lapped bars.

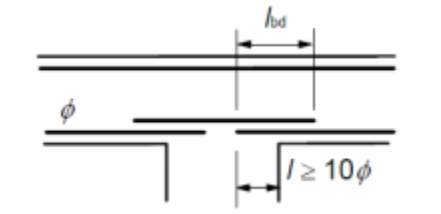


Figure 7.25Anchorage at intermediate supports

Laps

Forces are transmitted from one bar to another by:

- Lapping of bars, with or without bends or hooks;
- Welding;
- Mechanical devices assuring load transfer in tension-compression or in compression only.

The detailing of laps between bars shall be such that:

- the transmission of the forces from one bar to the next is assured;
- spalling of the concrete in the neighborhood of the joints does not occur;

- large cracks which affect the performance of the structure do not occur.
- between bars should normally be staggered and not located in areas of high moments /forces (e.g. plastic hinges).
- at any section should normally be arranged symmetrically.

The arrangement of lapped bars should comply with Figure 8.7:

- the clear distance between lapped bars should not be greater than 4ϕ or 50 mm, otherwise the lap length should be increased by a length equal to the clear space where it exceeds 4 ϕor 50 mm;
- the longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length,l_0;
- In case of adjacent laps, the clear distance between adjacent bars should not be less than2ϕ or 20 mm.

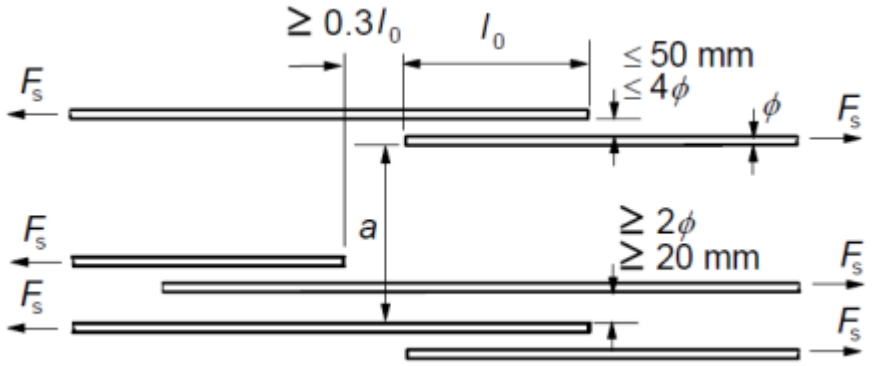


Figure 7.26Adjacent laps

Lap Length

The design lap length is:

$l_0 = \alpha1\ \alpha2\ \alpha3\alpha\ 5\ \alpha6\ l_{b,rqd} \geq l_{0,min}$

Where:

$l_{b,rqd}$is basic anchorage length

$l_{0,min} \geq \max \{0.3\ \alpha6\ l_{b,rqd};\ 15\phi;\ 200\ mm\}$

Values of α1α 2α3andα 5may be taken from table.

α6 = Values ofα6are given under table……

Table 6.28Value of the coefficient α_6

Percentage of lapped bars relative to the total cross-section area	< 25%	33%	50%	> 50%
α_6	1	1.15	1.4	1.5
Note: Intermediate values may be determined by interpolation.				

l_0=α1 α2α 3α 5 α6 lb,rqd= 1x0.7x0.7x0.7x1.4x749.6 = 359.86mm ≥ l_0,min

l_0, min≥max {0.3 α6 lb,rqd; 15ϕ; 200 mm}≥ max {0.3x1.4x749.6=314.83mm; 15x20=300mm; 200 mm}

Therefore, Take l_0 = 360mm

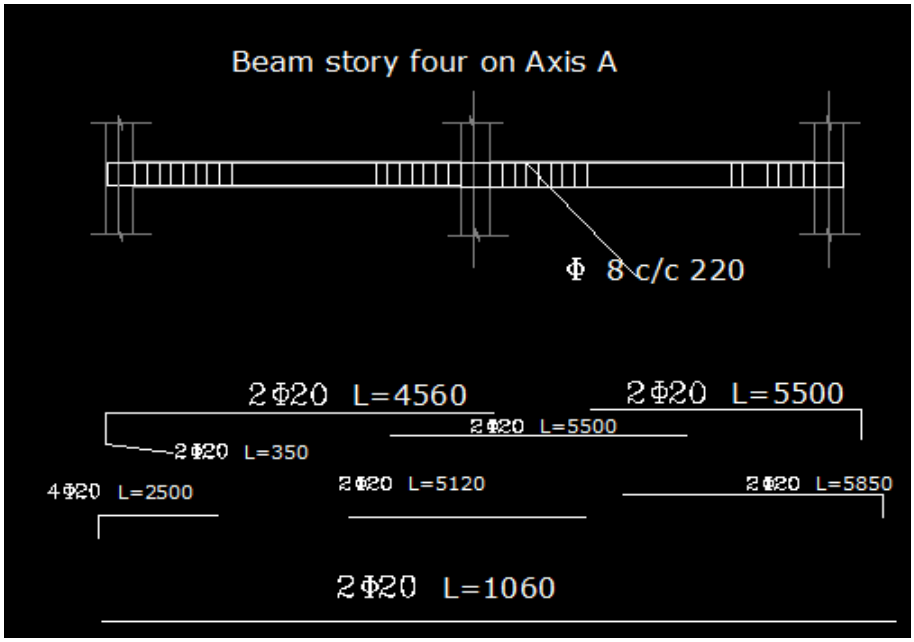


Figure 7.17 Reinforcement detail for column axis on A

CHAPTER EIGHT

8 ANALYSIS ANDDESIGN OF COLUMN

8.1 INTRODUCTION

A column is a vertical structural member transmitting axial compression loads with or without moments. The cross sectional dimensions of a column are generally considerably less than its height. Column support mainly vertical loads from the floors and roof and transmit these loads to the foundation.

The strength of a column depends on many factors including the following:

- The strength of the material
- Shape and size of the cross section
- Length
- The degree of positional and directional restraints at its end

Classification of columns

- Classification on the basis of geometry; rectangular, square, circular, etc. depending on the structural or architectural requirements.
- Classification on the basis of degree of slenderness; short column, slender(long) column.
- Classification on the basis of loading: axially loaded column, columns under uni-axial bending or columns under biaxial bending.
- Classification on the basis of lateral reinforcement; tied columns, spiral columns.
- Classification on the basis of lateral stability is provided to the structure as a whole; braced or un braced column

Axial column; column subjected to axial loads accompanied by bending about one axis whereas biaxial, for column support axial force and bending about two perpendicular axes. A

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

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

Rectangular column sample calculation for C-3 3rd or story4 floor column

Given data

Table 8.1 : Dimensions of top and bottom beams 3rd floor column

Top beam	depth(m)	width(m)	length(m)
b1(x,x)	0.4	0.25	5.572
b2(x,x)	0.4	0.25	3.937
b1(y,y)	0.4	0.25	4.561
b2(y,y)	0.4	0.25	5.5

Bottom beam	depth(m)	width(m)	length(m)
b3(x,x)	0.4	0.25	5.572
b4(x,x)	0.4	0.25	3.937
b3(y,y)	0.4	0.25	4.561
b4(y,y)	0.4	0.25	5.5

Column	depth(m)	width(m)	length(m)
	0.4	0.4	3.03

Mtopx-x=8.69KNm Mtopy-y=13.32KNm

Mbotx-x=7.43KNm Mboty-y=23.31KNm

NED=240.43KN

Where: NED is axial compression

Step 1: determination of concrete cover

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

$Cnom = Cmin + \Delta Cdev$(ministry of ethiopian education, 2015)

Where; $Cmin - minimumcoverand\ \Delta Cdevisallowanceindesignfordeviation$

Minimum cover, shall provide in order to ensure

- Safe transmission of bond force
- Corrosion resistance/ Durability
- Fire resistance

$Cmin = \begin{cases} c\ \min, b \\ Cmin, + \Delta Cdur, - \Delta Cdur, - \Delta Cdur, \\ 10mm \end{cases}$

Where;

- Cmin,b -minimum cover due to bond requirement
- Cmin, dur - minimum cover due to environmental conditions
- ΔCdur, γ- additive safety element
- ΔCdur,st -reduction of minimum cover for use of stainless steel
- ΔCdur,add -reduction of minimum cover for use of additional protection

But; the recommended value of ΔCdur,γ , ΔCdur,st , and ΔCdur,add is zero

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

Table 8.29Bond requirement

Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n) (see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $C_{min,b}$, should be increased by 5 mm.	

- Arrangement of Bars we selected separated

Assume Φ24 longitudinal bar and Φ20 nominal maximum aggregate size;

Therefore; Cmin,b=24mm.

Cover Design for Corrosion/Durability

- The condition of exposure is given to be XC2

Table8.30Class designation

Class Designation	Description of the Environment	Informative examples where exposure classes may occur
1. No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2. Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2

Activate Windows

- Exposure Class to determine Cmin,dur

Table 8.31Structural class

Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2/ XS1	XD3/ XS2/ XS3
Design working life of 100 years	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2
Strength Class[1)2)]	≥C30/37 reduce class by 1	≥C30/37 reduce class by 1	≥C35/45 reduce class by 1	≥C40/50 reduce class by 1	≥C40/50 reduce class by 1	≥C40/50 reduce class by 1	≥C45/55 reduce class by 1
Member with slab geometry (Position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Table 8.32Indicative minimum strength class

Table E.1N: Indicative minimum strength class

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

Note: The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Ann ex E and the recommended modifications to the structural class is given in Table 4.3N. But based on the above table the exposure cl ass 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 (ministry of Ethiopian education, 2015)

Table 8.33Values of minimum cover

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

Cmin, dur=10mm.

Then; $Cmin = Cmin, = 24mm$

$Cmin, =$ $10mmTherefore$; $Cmin = 10mm$

ΔCdev (allowance in Design for Variation)

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

$Cnom$=$Cmin$,b+$\Delta Cdev$=24mm+10mm=34mm

Cover Design for Fire

For the slab to sustain fire incident for 60 minutes the required cover and minimum height of the section can be determine d form Table 5.8 0f EN 1992-1-2.

Cfire = 10mm ,hs = 80mm

Governing Cover for Design

Governing cover accounting for

- Corrosion
- Bond/Durability
- Fire

$Cover$=34mm

fcd=$\boldsymbol{\alpha cc * fck/\Upsilon mc}$

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

fyd=$\boldsymbol{fyk/\Upsilon ms}$

fyd=$\mathbf{400/1.15}$= $\mathbf{347.8}$Mpa

Step 2 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 1992-1-1:2014 (5:15)

$$lo=0.5 * l\sqrt{\left(\left(1+\frac{K1}{0.45+k1}\right)\left(1+\frac{K2}{0.45+k2}\right)\right)}$$

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) =column stiffness/∑beam stiffness

$$K1x\text{-}x = \left(\frac{\left(\frac{EI}{l}\right)c}{\left(\frac{2*EI}{l}\right)b}\right) \geq \mathbf{0.1}$$

$I_{Column}=bh^3/12 =400*400^3/12=2,133,333,333$

$I_{beam} =bh^3/12=250*400^3/12=1,333,333,333$

$K1x\text{-}x=\frac{2,133,333,333E/4.75}{2*\left(\frac{1,333,333,333E}{5.57}\right)+2*\left(\frac{1,333,333,333}{3.937}\right)}=\frac{449,122.8}{1,155,918.1}=0.388$

$=bh^3/12 =400*400^3/12=2,133,333.3$

$I_{beam} =bh^3/12=250*400^3/12=1,333,333.3$

$K2x\text{-}x=\frac{2,133,333,333E/4.75}{2*\left(\frac{893,229.17E}{5.57}\right)+2*\left(\frac{893,229.17}{3.937}\right)}=\frac{449,122.8}{774,374.81}=0.58$

$K1y\text{-}y=\frac{2,133,333,333E/5.03}{2*\left(\frac{1,333,333333E}{5.5}\right)+2*\left(\frac{1,333,333,333}{4.56}\right)}=\frac{424121.92}{1,069,643.77}=0.39$

$K2y\text{-}y=\frac{2,133,333,333E/5.03}{2*\left(\frac{893,229.17E}{5.5}\right)+2*\left(\frac{893,229.17}{4.56}\right)}=\frac{424121.92}{716,577.79}=0.61$

$$Lox\text{-}x=0.5 *l\sqrt{\left(\left(1+\frac{K1x-x}{0.45+k1x-x}\right)\left(1+\frac{K2x-x}{0.45+k2x-x}\right)\right)}$$

$Lox\text{-}x= 0.5 * 3.03 * \frac{\sqrt{((1+0.388}}{0.45+0.388})(1 + \frac{0.58}{0.45+0.58})) =2.74m$

$$Loy\text{-}y=0.5 *l\sqrt{\left(\left(1+\frac{K1y-y}{0.45+k1y-y}\right)\left(1+\frac{K2y-y}{0.45+k2y-y}\right)\right)}$$

$Loy\text{-}y= 0.5 * 3.03 * \frac{\sqrt{((1+0.39}}{0.45+0.39})(1 + \frac{0.61}{0.45+0.61})) =2.69m$

Step 3 maximum and minimum reinforcement calculations

$As,min=max\ (0.1 * NED/fyd) = 0.1 * 240.43/347.8= 691.13mm^2$

$0.002 * Ac = 0.002 * 400 * 400 = 320mm^2$

$As,min=691.13mm^2$

$As,max=0.04 * Ac = 0.04 * 400 * 400 = 6400mm^2$

Step 4 design of eccentricity

The effect of imperfection taken from According to ES EN 1992: Art 5.2.7

$lo/400$=2690/400= 6.72mm

Eix-x=max $h/30$=303/30= 10.1mm

20mm

eix-x=20mm

$lo/400$=2740/400= 6.85mm

eiy-y=max $h/30$=303/30= 10.1mm

20mm

eiy-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

MED=max {MO2, Moe+M2, Mo1+0.5M2}

M01=min {|Mtop|,|Mbottom|+ei*NED}

M02=max {|Mtop|,|Mbottom|+ei*NED}

Moe=0.4*M01+0.6*M02≥0.4*M02

For first order moment M2=0

In the X-dirn

M01x-x=min {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M01x-x=min {|8.69|, |-7.34|+0.02*688.18}

M01x-x=8.69KNm

M02x-x=max {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M02x-x=max {|8.69|, |-7.34|+0.02*240.43}

M02x-x=12.15KNm

Moex-x=0.4*M01x-x+0.6*M02x-x≥0.4*M02x-x

Moex-x=0.4*8.69+0.6*12.15≥0.4*12.15

Moex-x=10.77KNm

MEDx-x=10.77KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|13.32|,|-23.31|+0.02*240.43}

M01y-y=13.32KNm

M02y-y=max {|Mtopy-y|, |Mbotty-y|+ei*NED}

M02y-y=max {|13.32|,|-23.31|+0.02*240.43}

M02y-y=28.12KNm

Moey-y=0.4*M01y-y+0.6*M02y-y≥0.4*M02y-y

Moey-y=0.4*13.32+0.6*28.12≥0.4*28.12

Moey-y=22.2KNm

MEDy-y=22.2KNm

Step 5: Check slenderness

Slenderness ratio λ =lo/i

Where lo-effective length of column

i-is radius of gyration of the uncracked concrete

λx-x=$lox-x/i$ but i=$\sqrt{Ic/Ac}$

i=$\sqrt{2133333333/160000}$= 115.47mm

λx-x=2740/115.47=23.72

λy-y=$loy-y/i$=2690/115.47= 22.98

The minimum limiting value of slenderness is

λlim=20*A*B*C/$\sqrt{n}$

λlimx-x=20*A*B*C/$\sqrt{n}$

Where, A – 1/ (1 + 0.2φef), φef-effective creep ratio

B - √1 + 2ω,-Asfyd/Acfcd, mechanical reinforcement ratio

C = 1.7-rm

n=NEd/Acfcd, relative normal force

rm,moment ratio=Mo1x-x/Mo2x-x=8.69/12.15=0.715

Mo1, Mo2 are the first order end moments, |Mo2|≥|Mo1|

If φef is not known, A=0.7

If ω is not known, B=1.1

C=1.7-0.715=0.985

Ac=$b* h$ = 400 ∗ 400 = 160000mm^2

for rectangular column

n=NEd/Acfcd=(240.43*1000)/(160000*11.33)=0.133

Therefore, slenderness limit (ƛlimx-x)

=20*A*B*C/√n=20*0.7*1.1*0.985*/√0.133=41.59mm

ƛlimy-y=20*A*B*C/√n

A=0.7

B=1.1

C=1.7-(13.32/28.12)=1.22

n=NEd/Acfcd=(240.43*1000/(160000*11.33)=0.133

λlimy-y =20*A*B*C/√n=51.52mm

λx-x >λ limx-x, slender column otherwise short column

λy- y>λ limy-y, slender column otherwise short column

λx-x < λ limx-x= 23.72< 41.59, short column

λy- y< λ limy-y= 22.98< 51.52,short column

vsd=Nsd/(Ac*fcd,) =240.43*1000/ (400*400*11.33) =0.38

μsdx-x=MEDx-x/(fcd*Ac*h)=10.77*1000000/ (11.33*160000*400) =0.015

μsdy-y= MEDy-y/(fcd*Ac*h) =28.12*1000000/ (11.33*160000*400) =0.038

Using biaxial rectangular chart number

ω=0.1

Step 5 reinforcement calculation

- Longitudinal reinforcement

As,tot=ω*Ac*fcd/fyd

As,tot=0.1*160000*11.33/347.8

As,tot=521.23mm^2

As min=691.13mm² and As max=6400mm²

As min <As<As max……NOT OK

Take ∅24mm, as=3.14*(24)²/4=452mm²

Number of bar=As/as=691.13mm²/452mm²=1.53≈4bar

Provide 4Ø24mm

- For Lateral(shear) reinforcement

8.2 Functions of Lateral Reinforcement.

- they hold the longitudinal bars in position in the forms while the concrete is being placed

- They prevent the slender longitudinal bars from buckling out ward by bursting the thin concrete cover.

For reinforced concrete section to resist shear force ties are used as transverse reinforcement. The diameter of the transverse reinforcement should not be less than 6mm or one quarter of the maximum diameter of the longitudinal bars.

¼*longitudinal bar diameter=$\boldsymbol{24/4}$=6

Diameter of bar=max 6mm

Diameter of stirrup = 8mm

From above take Ø8mm

20*$\boldsymbol{\emptyset long}$, =20*24mm=480mm

Spacing of shear reinforcement= min lesser dimension of column =303mm

400mm

Finally, provideØ8 c /c 303mm

Rectangular column sample calculation for C-3 2nd or story3 floor column

Given data

Table8.7 Dimensions of top and bottom beams 2nd floor column

Top beam	depth(m)	width(m)	length(m)
b1(x,x)	0.4	0.25	5.572
b2(x,x)	0.4	0.25	3.937
b1(y,y)	0.4	0.25	4.561
b2(y,y)	0.4	0.25	5.5

Bottom beam	depth(m)	width(m)	length(m)
b3(x,x)	0.4	0.25	5.572

b4(x,x)	0.4	0.25	3.937
b3(y,y)	0.4	0.25	4.561
b4(y,y)	0.4	0.25	5.5

Column	depth(m)	width(m)	length(m)
	0.4	0.4	3.03

Mtopx-x=2.56KNm Mtopy-y=20.69KNm

Mbotx-x=4.14KNm Mboty-y=27.43KNm

NED=688.18KN

Step 2 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 1992-1-1:2014 (5:15)

$$lo=0.5 * l\sqrt{((1+\frac{K1}{0.45+k1})\ (1+\frac{K2}{0.45+k2}))}$$

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) =column stiffness/∑beam stiffness

$$K1x\text{-}x = (\frac{(\frac{EI}{l})c}{(\frac{2*EI}{l})b}) \geq 0.1$$

$I_{Column}=bh^3/12 =400*400^3/12=2,133,333,333$

$I_{beam} =bh^3/12=250*400^3/12=1,333,333,333$

$K1x\text{-}x=\frac{2,133,333,333E/4.75}{2*(\frac{1,333,333,333E}{5.57})+2*(\frac{1,333,333,333}{3.937})}=\frac{449,122.8}{1,155,918.1}=0.388$

$=bh^3/12 =400*400^3/12=2,133,333.3$

I_{beam} =$bh^3/12$=$250*400^3/12$=1,333,333.3

K2x-x=$\frac{2,133,333,333E/4.75}{2*(\frac{893,229.17E}{5.57})+2*(\frac{893,229.17}{3.937})}=\frac{449,122.8}{774,374.81}$=0.58

K1y-y=$\frac{2,133,333,333E/5.03}{2*(\frac{1,333,333333E}{5.5})+2*(\frac{1,333,333,333.}{4.56})}=\frac{424121.92}{1,069,643.77}$=0.39

K2y-y=$\frac{2,133,333,333E/5.03}{2*(\frac{893,229.17E}{5.5})+2*(\frac{893,229.17}{4.56})}=\frac{424121.92}{716,577.79}$=0.61

$$Lox\text{-}x=0.5*l\sqrt{((1+\frac{K1x-x}{0.45+k1x-x})(1+\frac{K2x-x}{0.45+k2x-x}))}$$

Lox-x= $0.5*3.03*\frac{\sqrt{((1+0.388}}{0.45+0.388})(1+\frac{0.58}{0.45+0.58}))$ =2.74m

$$Loy\text{-}y=0.5*l\sqrt{((1+\frac{K1y-y}{0.45+k1y-y})(1+\frac{K2y-y}{0.45+k2y-y}))}$$

Loy-y= $0.5*3.03*\frac{\sqrt{((1+0.39}}{0.45+0.39})(1+\frac{0.61}{0.45+0.61}))$ =2.69m

Step 3 maximum and minimum reinforcement calculations

As,min=max $(0.1*NED/fyd) = 0.1*688.18/347.8= 197.86mm^2$

$0.002*Ac = 0.002*400*400 = 320mm^2$

As,min=$320mm^2$

As,max=$0.04*Ac = 0.04*400*400 = 6400mm^2$

Step 4 design of eccentricity

The effect of imperfection taken from According to ES EN 1992: Art 5.2.7

$lo/400$=2690/400= $6.72mm$

Eix-x=max h/30=303/30= $10.1mm$

20mm

eix-x=20mm

$lo/400$=2740/400= $6.85mm$

eiy-y=max h/30=303/30= $10.1mm$

20mm

eiy-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

MED=max {MO2, Moe+M2, Mo1+0.5M2}

M01=min {|Mtop|,|Mbottom|+ei*NED}

M02=max {|Mtop|,|Mbottom|+ei*NED}

Moe=0.4*M01+0.6*M02≥0.4*M02

For first order moment M2=0

In the X-dirn

M01x-x=min {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M01x-x=min {|-2.56|, |4.14|+0.02*688.18}

M01x-x=2.56KNm

M02x-x=max {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M02x-x=max {|-2.56|, |4.14|+0.02*688.18}

M02x-x=17.9KNm

Moex-x=0.4*M01x-x+0.6*M02x-x≥0.4*M02x-x

Moex-x=0.4*2.56+0.6*17.9≥0.4*17.9

Moex-x=11.76KNm

MEDx-x=11.76KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|20,69|,|-27.43|+0.02*688.18}

M01y-y=20.69KNm

M02y-y=max {|Mtopy-y|, |Mbotty-y|+ei*NED}

M02y-y=max {|20.69|,|-27.43|+0.02*688.18}

M02y-y=41.19KNm

Moey-y=0.4*M01y-y+0.6*M02y-y≥0.4*M02y-y

Moey-y=0.4*20.69+0.6*41.19≥0.4*41.19

Moey-y=32.99KNm

MEDy-y=32.99KNm

Step 5: Check slenderness

Slenderness ratio λ =lo/i

Where lo-effective length of column

i-is radius of gyration of the uncracked concrete

λx-x=$lox-x/i$ but i=$\sqrt{Ic/Ac}$

i=$\sqrt{2133333333/160000}$= 115.47mm

λx-x=2740/115.47=23.72

λy-y=$loy-y/i$=2690/115.47= 22.98

The minimum limiting value of slenderness is

λlim=20*A*B*C/$\sqrt{n}$

λlimx-x=20*A*B*C/$\sqrt{n}$

Where, A – 1/ (1 + 0.2φef), φef-effective creep ratio

B - √1 + 2ω,-Asfyd/Acfcd, mechanical reinforcement ratio

C = 1.7-rm

n=NEd/Acfcd, relative normal force

rm,moment ratio=Mo1x-x/Mo2x-x=2.56/17.9=0.139

Mo1, Mo2 are the first order end moments, |Mo2|≥|Mo1|

If φef is not known, A=0.7

If ω is not known, B=1.1

C=1.7-0.139=1.56

Ac=$b* h$ = 400 ∗ 400 = 160000mm^2

for rectangular column

n=NEd/Acfcd=(688.18*1000)/(160000*11.33)=0.379

Therefore, slenderness limit (λlimx-x)

=20*A*B*C/$\sqrt{n}$=20*0.7*1.1*1.56*/√0.379=39.02mm

λlimy-y=20*A*B*C/$\sqrt{n}$

A=0.7

B=1.1

C=1.7-(20.69/41.19)=1.19

n=NEd/Acfcd=(688.18*1000/(160000*11.33)=0.379

λlimy-y =20*A*B*C/√n=29.76mm

λx-x >λ limx-x, slender column otherwise short column

λy- y>λ limy-y, slender column otherwise short column

λx-x < λ limx-x= 23.72< 39.02, short column

λy- y< λ limy-y= 22.98< 29.76,short column

vsd=Nsd/(Ac*fcd,) =688.18*1000/ (400*400*11.33) =0.379

μsdx-x=MEDx-x/(fcd*Ac*h)=11.76*1000000/ (11.33*160000*400) =0.016

μsdy-y= MEDy-y/(fcd*Ac*h) =32.99*1000000/ (11.33*160000*400) =0.045

Using biaxial rectangular chart number

ω=0.15

Step 5 reinforcement calculation

- Longitudinal reinforcement

As,tot=ω*Ac*fcd/fyd

As,tot=0.15*160000*11.33/347.8

As,tot=781.83mm^2

As min=320mm² and As max=6400mm²

As min <As<As max…… OK

Take Ø24mm, as=3.14*(24)²/4=452mm²

Number of bar=As/as=781.83mm²/452mm²=1.79≈4bar

Provide 4Ø24mm

- For Lateral(shear) reinforcement

¼*longitudinal bar diameter=$\boldsymbol{24/4}$=6

Diameter of bar=max 6mm

Diameter of stirrup = 8mm

From above take Ø8mm

20*$\boldsymbol{\emptyset long}$, =20*24mm=480mm

Spacing of shear reinforcement= min lesser dimension of column =303mm

400mm

Finally, provideØ8C/C 303mm

Rectangular column sample calculation for C-3 1st or story2 floor column

Given data

Table 8.34 Dimensions of top and bottom beams 1rd floor column

Top beam	depth(m)	width(m)	length(m)
b1(x,x)	0.4	0.25	5.572
b2(x,x)	0.4	0.25	3.937
b1(y,y)	0.4	0.25	4.561
b2(y,y)	0.4	0.25	5.5

Bottom beam	depth(m)	width(m)	length(m)
b3(x,x)	0.4	0.25	5.572
b4(x,x)	0.4	0.25	3.937
b3(y,y)	0.4	0.25	4.561
b4(y,y)	0.4	0.25	5.5

Column	depth(m)	width(m)	length(m)
	0.4	0.4	3.03

Mtopx-x=4.92KNm Mtopy-y=18.28KNm

Mbotx-x=6.39KNm Mboty-y=21.95KNm

NED=1139.9KN

Step 2 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 1992-1-1:2014 (5:15)

$$lo=0.5 * l\sqrt{((1+\frac{K1}{0.45+k1})\ (1+\frac{K2}{0.45+k2}))}$$

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) =column stiffness/∑beam stiffness

$$K1x\text{-}x = \frac{\left(\frac{EI}{l}\right)c}{\left(\frac{2*EI}{l}\right)b} \geq 0.1$$

$I_{Column}=bh^3/12 =400*400^3/12=2,133,333,333$

$I_{beam} =bh^3/12=250*400^3/12=1,333,333,333$

$$K1x\text{-}x=\frac{2,133,333,333E/4.75}{2*\left(\frac{1,333,333,333E}{5.57}\right)+2*\left(\frac{1,333,333,333}{3.937}\right)}=\frac{449,122.8}{1,155,918.1}=0.388$$

$=bh^3/12 =400*400^3/12=2,133,333.3$

$I_{beam} =bh^3/12=250*400^3/12=1,333,333.3$

$$K2x\text{-}x=\frac{2,133,333,333E/4.75}{2*\left(\frac{893,229.17E}{5.57}\right)+2*\left(\frac{893,229.17}{3.937}\right)}=\frac{449,122.8}{774,374.81}=0.58$$

$$K1y\text{-}y=\frac{2,133,333,333E/5.03}{2*\left(\frac{1,333,333333E}{5.5}\right)+2*\left(\frac{1,333,333,333}{4.56}\right)}=\frac{424121.92}{1,069,643.77}=0.39$$

$$K2y\text{-}y=\frac{2,133,333,333E/5.03}{2*\left(\frac{893,229.17E}{5.5}\right)+2*\left(\frac{893,229.17}{4.56}\right)}=\frac{424121.92}{716,577.79}=0.61$$

$$Lox\text{-}x=0.5 * l\sqrt{((1+\frac{K1x-x}{0.45+k1x-x})\ (1+\frac{K2x-x}{0.45+k2x-x}))}$$

$$Lox\text{-}x= 0.5 * 3.03 * \frac{\sqrt{((1+0.388}}{0.45+0.388})(1 + \frac{0.58}{0.45+0.58})) =2.74m$$

$$Loy\text{-}y=0.5 * l\sqrt{((1+\frac{K1y-y}{0.45+k1y-y})\ (1+\frac{K2y-y}{0.45+k2y-y}))}$$

$$Loy\text{-}y= 0.5 * 3.03 * \frac{\sqrt{((1+}0.39}{0.45+0.39})(1 + \frac{0.61}{0.45+0.61})) =2.69m$$

Step 3 maximum and minimum reinforcement calculations

As,min=max $(0.1 * NED/fyd) = 0.1 * 1139.9/347.8 = 319.78 mm^2$

$0.002 * Ac = 0.002 * 400 * 400 = 320 mm^2$

As,min=$320 mm^2$

As,max=$0.04 * Ac = 0.04 * 400 * 400 = 6400 mm^2$

Step 4 design of eccentricity

The effect of imperfection taken from According to ES EN 1992: Art 5.2.7

$lo/400 = 2690/400 = 6.72mm$

Eix-x=max $h/30 = 303/30 = 10.1mm$

20mm

eix-x=20mm

$lo/400 = 2740/400 = 6.85mm$

eiy-y=max $h/30 = 303/30 = 10.1mm$

20mm

eiy-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

MED=max {MO2, Moe+M2, Mo1+0.5M2}

M01=min {|Mtop|,|Mbottom|+ei*NED}

M02=max {|Mtop|,|Mbottom|+ei*NED}

Moe=0.4*M01+0.6*M02≥0.4*M02

For first order moment M2=0

In the X-dirn

M01x-x=min {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M01x-x=min {|-4.29|, |6.39|+0.02*1139.9}

M01x-x=4.29KNm

M02x-x=max {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M02x-x=max {|-4.29|, |6.39|+0.02*1139.9}

M02x-x=29.19KNm

Moex-x=0.4*M01x-x+0.6*M02x-x≥0.4*M02x-x

Moex-x=0.4*4.29+0.6*29.19≥0.4*29.19

Moex-x=19.65KNm

MEDx-x=19.65KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|18.28|,|-21.94|+0.02*1139.9}

M01y-y=18.28KNm

M02y-y=max {|Mtopy-y|, |Mbotty-y|+ei*NED}

M02y-y=max {|18.28|,|-21.95|+0.02*1139.9}

M02y-y=44.75KNm

Moey-y=0.4*M01y-y+0.6*M02y-y≥0.4*M02y-y

Moey-y=0.4*18.28+0.6*44.75≥0.4*44.75

Moey-y=34.16KNm

MEDy-y=34.16KNm

Step 5: Check slenderness

Slenderness ratio λ =lo/i

Where lo-effective length of column

i-is radius of gyration of the uncracked concrete

λx-x=$lox-x/i$ but i=$\sqrt{Ic/Ac}$

i=$\sqrt{2133333333/160000}$= 115.47mm

λx-x=2740/115.47=23.72

λy-y=$loy-y/i$=2690/115.47= 22.98

The minimum limiting value of slenderness is

λlim=20*A*B*C/$\sqrt{n}$

λlimx-x=20*A*B*C/$\sqrt{n}$

Where, A – 1/ (1 + 0.2φef), φef-effective creep ratio

B - √1 + 2ω,-Asfyd/Acfcd, mechanical reinforcement ratio

C = 1.7-rm

n=NEd/Acfcd, relative normal force

rm, moment ratio=Mo1x-x/Mo2x-x=4.29/29.19=0.147

Mo1, Mo2 are the first order end moments, |Mo2|≥|Mo1|

If φef is not known, A=0.7

If ω is not known, B=1.1

C=1.7-0.147=1.55

Ac=$b* h$ = 400 ∗ 400 = 160000mm^2

for rectangular column

n=NEd/Acfcd=(1139.9*1000)/(160000*11.33)=0.628

Therefore, slenderness limit (ƛlimx-x)

=20*A*B*C/√n=20*0.7*1.1*1.55*/√0.628=30.12mm

ƛlimy-y=20*A*B*C/√n

A=0.7

B=1.1

C=1.7-(18.28/44.75)=1.29

n=NEd/Acfcd=(1139.9*1000/(160000*11.33)=0.628

λlimy-y =20*A*B*C/√n=25.07mm

λx-x >λ limx-x, slender column otherwise short column

λy- y>λ limy-y, slender column otherwise short column

λx-x < λ limx-x= 23.72< 30.12, short column

λy- y< λ limy-y= 22.98< 25.07,short column

vsd=Nsd/(Ac*fcd,) =1139.9*1000/ (400*400*11.33) =0.628

μsdx-x=MEDx-x/(fcd*Ac*h)=19.65*1000000/ (11.33*160000*400) =0.027

μsdy-y= MEDy-y/(fcd*Ac*h) =34.16*1000000/ (11.33*160000*400) =0.047

Using biaxial rectangular chart number

ω=0.1

Step 5 reinforcement calculation

- Longitudinal reinforcement

As,tot=ω*Ac*fcd/fyd

As,tot=0.1*160000*11.33/347.8

As,tot=521.17mm^2

As min=320mm² and As max=6400mm²

As min <As<As max…… OK

Take Ø24mm, as=3.14*(24)²/4=452mm²

Number of bar=As/as=521.17mm²/452mm²=1.15≈4bar

Provide 4Ø24mm

- For Lateral(shear) reinforcement

¼*longitudinal bar diameter=$\boldsymbol{24/4}$=6

Diameter of bar=max 6mm

Diameter of stirrup = 8mm

From above take Ø8mm

20*$\emptyset\boldsymbol{long}$, =20*24mm=480mm

Spacing of shear reinforcement= min lesser dimension of column =303mm

400mm

Finally, provideØ8C/C 303mm

Rectangular column sample calculation for C-3 ground or story 1 floor column

Given data

Table 8.35 Dimensions of top and bottom beams ground floor column

Top beam	depth(m)	width(m)	length(m)
b1(x,x)	0.4	0.25	5.572
b2(x,x)	0.4	0.25	3.937
b1(y,y)	0.4	0.25	4.561
b2(y,y)	0.4	0.25	5.5

Bottom beam	depth(m)	width(m)	length(m)
b3(x,x)	0.4	0.25	5.572
b4(x,x)	0.4	0.25	3.937
b3(y,y)	0.4	0.25	4.561

b4(y,y)	0.4	0.25	5.5

Column	depth(m)	width(m)	length(m)
	0.4	0.4	3.03

Mtopx-x=14.16KNm Mtopy-y=41.93KNm

Mbotx-x=7.14KNm Mboty-y=22.02KNm

NED=1644.7KN

Step 2 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 1992-1-1:2014 (5:15)

$$lo=0.5 * l\sqrt{((1+\frac{K1}{0.45+k1})\ (1+\frac{K2}{0.45+k2}))}$$

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) =column stiffness/∑beam stiffness

$$K1x\text{-}x = (\frac{(\frac{EI}{l})c}{(\frac{2*EI}{l})b}) \geq 0.1$$

$I_{Column}=bh^3/12 =400*400^3/12=2,133,333,333$

$I_{beam} =bh^3/12=250*400^3/12=1,333,333,333$

$K1x\text{-}x=\frac{2,133,333,333E/4.75}{2*(\frac{1,333,333,333E}{5.57})+2*(\frac{1,333,333,333}{3.937})}=\frac{449,122.8}{1,155,918.1}=0.388$

$=bh^3/12 =400*400^3/12=2,133,333.3$

$I_{beam} =bh^3/12=250*400^3/12=1,333,333.3$

$K2x\text{-}x=\frac{2,133,333,333E/4.75}{2*(\frac{893,229.17E}{5.57})+2*(\frac{893,229.17}{3.937})}=\frac{449,122.8}{774,374.81}=0.58$

$$K1y\text{-}y=\frac{2,133,333,333E/5.03}{2*\left(\frac{1,333,333333E}{5.5}\right)+2*\left(\frac{1,333,333,333}{4.56}\right)}=\frac{424121.92}{1,069,643.77}=0.39$$

$$K2y\text{-}y=\frac{2,133,333,333E/5.03}{2*\left(\frac{893,229.17E}{5.5}\right)+2*\left(\frac{893,229.17}{4.56}\right)}=\frac{424121.92}{716,577.79}=0.61$$

$$Lox\text{-}x=0.5*l\sqrt{\left(\left(1+\frac{K1x-x}{0.45+k1x-x}\right)\left(1+\frac{K2x-x}{0.45+k2x-x}\right)\right)}$$

$$Lox\text{-}x=0.5*3.03*\frac{\sqrt{((1+0.388}}{0.45+0.388})(1+\frac{0.58}{0.45+0.58}))=2.74m$$

$$Loy\text{-}y=0.5*l\sqrt{\left(\left(1+\frac{K1y-y}{0.45+k1y-y}\right)\left(1+\frac{K2y-y}{0.45+k2y-y}\right)\right)}$$

$$Loy\text{-}y=0.5*3.03*\frac{\sqrt{((1+0.39}}{0.45+0.39})(1+\frac{0.61}{0.45+0.61}))=2.69m$$

Step 3 maximum and minimum reinforcement calculations

$As,min=\max\begin{cases}0.1*NED/fyd=0.1*1644.7/347.8=472.85mm^2\\0.002*Ac=0.002*400*400=320mm^2\end{cases}$

$As,min=472.85mm^2$

$As,max=0.04*Ac=0.04*400*400=6400mm^2$

Step 4 design of eccentricity

The effect of imperfection taken from According to ES EN 1992: Art 5.2.7

$lo/400=2690/400=6.72mm$

Eix-x= max $h/30=303/30=10.1mm$

20mm

eix-x=20mm

$lo/400=2740/400=6.85mm$

eiy-y=max $h/30=303/30=10.1mm$

20mm

eiy-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

MED=max {MO2, Moe+M2, Mo1+0.5M2}

M01=min {|Mtop|,|Mbottom|+ei*NED}

M02=max {|Mtop|,|Mbottom|+ei*NED}

Moe=0.4*M01+0.6*M02≥0.4*M02

For first order moment M2=0

In the X-dirn

M01x-x=min {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M01x-x=min {|-14.16|, |7.14|+0.02*1644.7}

M01x-x=14.16KNm

M02x-x=max {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M02x-x=max {|-14.16|, |7.14|+0.02*1644.7}

M02x-x=40KNm

Moex-x=0.4*M01x-x+0.6*M02x-x≥0.4*M02x-x

Moex-x=0.4*14.16+0.6*40≥0.4*40

Moex-x=29.66KNm

MEDx-x=29.66KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|41.93|,|-22.02|+0.02*1644.7}

M01y-y=41.93KNm

M02y-y=max {|Mtopy-y|, |Mbotty-y|+ei*NED}

M02y-y=max {|41.93|,|-22.02|+0.02*1644.7}

M02y-y=54.89KNm

Moey-y=0.4*M01y-y+0.6*M02y-y≥0.4*M02y-y

Moey-y=0.4*41.93+0.6*54.89≥0.4*54.89

Moey-y=49.71KNm

MEDy-y=49.71KNm

Step 5: Check slenderness

Slenderness ratio λ =lo/i

Where lo-effective length of column

i-is radius of gyration of the uncracked concrete

λx-x=$lox-x/i$ but i=$\sqrt{Ic/Ac}$

i=$\sqrt{2133333333/160000}$= 115.47mm

λx-x=2740/115.47=23.72

λy-y=$loy-y/i$=2690/115.47= 22.98

The minimum limiting value of slenderness is

λlim=20*A*B*C/$\sqrt{n}$

λlimx-x=20*A*B*C/$\sqrt{n}$

Where, A – 1/ (1 + 0.2φef), φef-effective creep ratio

B - √1 + 2ω,-Asfyd/Acfcd, mechanical reinforcement ratio

C = 1.7-rm

n=NEd/Acfcd, relative normal force

rm,moment ratio=Mo1x-x/Mo2x-x=14.16/40=0.35

Mo1, Mo2 are the first order end moments, |Mo2|≥|Mo1|

If φef is not known, A=0.7

If ω is not known, B=1.1

C=1.7-0.35=1.35

Ac=$b* h$ = 400 ∗ 400 = 160000mm^2

for rectangular column

n=NEd/Acfcd=(1644.7*1000)/(160000*11.33)=0.71

Therefore, slenderness limit (λlimx-x) =20*A*B*C/$\sqrt{n}$=20*0.7*1.1*1.35*/√0.71=26.67mm

λlimy-y=20*A*B*C/$\sqrt{n}$

A=0.7

B=1.1

C=1.7-(41.93/54.81)=0.93

n=NEd/Acfcd=(1644.7*1000/(160000*11.33)=0.71

λlimy-y =20*A*B*C/$\sqrt{n}$=24.87mm

λx-x >λ limx-x, slender column otherwise short column

λy- y>λ limy-y, slender column otherwise short column

λx-x < λ limx-x= 23.72< 26.67, short column

λy- y< λ limy-y= 22.98< 24.87,short column

vsd=Nsd/(Ac*fcd,) =1644.7*1000/ (400*400*11.33) =0.91

μsdx-x=MEDx-x/(fcd*Ac*h)=49.7*1000000/ (11.33*160000*400) =0.068

μsdy-y= MEDy-y/(fcd*Ac*h) =29.9*1000000/ (11.33*160000*400) =0.041

Using biaxial rectangular chart number

ω=0.2

Step 5 reinforcement calculation

- Longitudinal reinforcement

As,tot=ω*Ac*fcd/fyd

As,tot=0.2*160000*11.33/347.8

As,tot=1042.35mm^2

As min=320mm² and As max=6400mm²

As min <As<As max…… OK

Take Ø24mm, as=3.14*(24)²/4=452mm²

Number of bar=As/as=1042.35mm²/452mm²=2.31≈4bar

Provide 4Ø24mm

- For Lateral(shear) reinforcement

¼*longitudinal bar diameter=$\boldsymbol{24/4}$=6

Diameter of bar=max 6mm

Diameter of stirrup = 8mm

From above take Ø8mm

20*$\boldsymbol{\emptyset long}$, =20*24mm=480mm

Spacing of shear reinforcement= min lesser dimension of column =303mm

400mm

Finally, provideØ8C/C 303mm

In similar procedure the other column determination of effective length, slenderness check and design of enforcementcalculated by excel template……..**Appendix 8A,Appendix 8B**and **Appendix 8C** respectively.

Design of lap length

Lapping of reinforcement is necessary to transfer the force from one bar to another. Between bar should be staggered and should not occur in regions of high stress. The length of the lap should be based on the minimum anchorage length modified to take into account factors.

Lo=α1*α2*α3*α5*α6*lbrq lomin

Lomin>max{0.3α6lbrq, 15ø, 200mm}

The value of α1,α2,α3,α5 can be taken from table 8.2 of ESEN2.1

α6=(ϱ/25)0.5 ≤and 1 ≤ α6

lb,rqd = (ø*σsd)/(4*fbd), where σsd is design stress of bar

fbd=2.25*η1*η2*fctd, η1=1 when good condition are obtained and 0.7 for all other cases

η2=1 for ø<32

rqd = (ø*σsd)/(4 *fbd)= (24* 347.83/(4 *2.32) =899.6

α1*α2*α3*α5*α6*lbrqd=1*0.7*0.7*0.7*1.15*899.6=354.84

lo=max 0.3α6lbrq=0.3*1.15*899.6=310.36

15ø= 15*24=360

200mm

Take lo=360mm

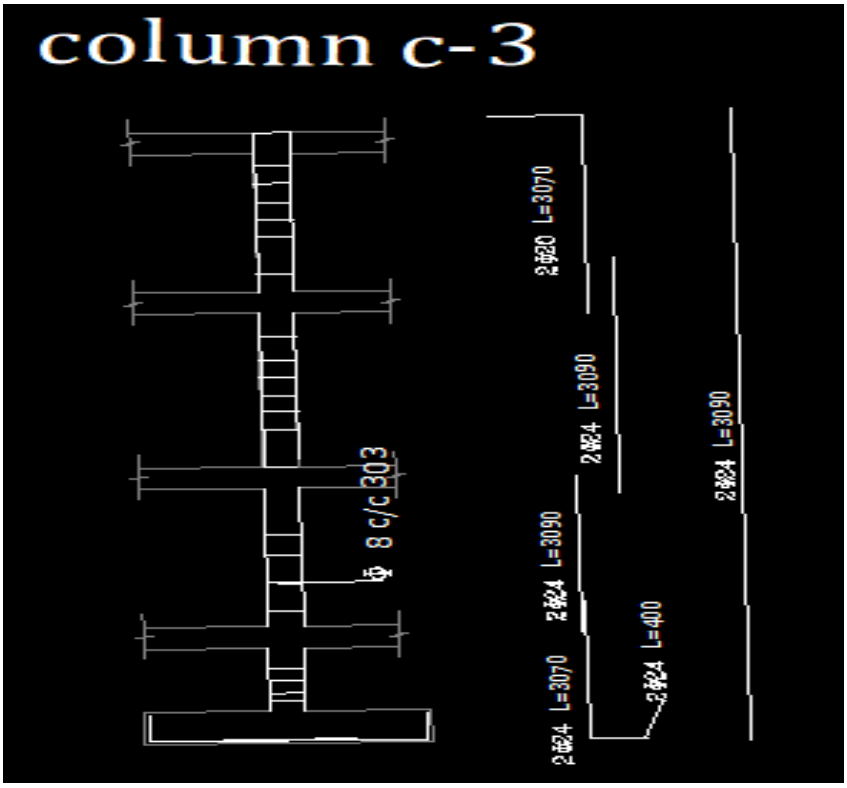


Figure 27 Reinforcement detail for column

CHAPTER NINE

9 Foundation Analysis and Design

9.1 Introduction

The foundation is the part of an engineered system that transmits to, and into, the underlying soil or rock the loads supported by the foundation and it's self-weight.
Purposes of foundations to distribute the load of the structure over a large bearing area so as to bring intensity of loading within the safe bearing capacity of the soil lying underneath. To load the bearing surface at a uniform rate so as to prevent unequal settlement.
The main purpose of footings and other foundation systems is to transfer column loads safely to the soil. Since the soil bearing capacity is much lower than the concrete columns; the loads need to be transferred safely to the to the soil by using larger areas usually called shallow foundations. If the soil has low bearing capacity, or the applied loads are very large, it may be necessary to transfer the load to a deeper soil through the use of piles or caissons usually called deep foundation

- To prevent the lateral movement of the supporting material.
- To secure a level and firm bed for building operations.
- To increase the stability of the structure as a whole.

The foundation should be design such that

- The soil below doesn't fail in shear
- The settlement is within the safe limit

Table 9.36The values of axial load, FY, Mx and My are obtained from ETABS 2016 analysis result as follows

Story	Joint Label	Unique Name	Load Case/Combo	FX	FY	FZ	MX	MY	MZ
				kN	kN	kN	kN-m	kN-m	kN-m
Ground	1	83	Comb2	16.822	5.8976	527.7085	-6.7766	12.646	-0.3381
Ground	2	101	Comb2	14.413	-3.4709	811.6182	3.0143	13.462	-0.1078
Ground	3	98	Comb2	-7.9459	18.1748	704.1208	-16.195	-8.929	-0.0954
Ground	4	99	Comb2	12.308	-2.3101	537.3602	1.0911	11.352	-0.3479
Ground	5	103	Comb2	-4.6937	-18.052	838.6667	19.1769	-5.217	-0.1089
Ground	6	86	Comb2	-6.1163	7.9563	656.2962	-7.5901	-7.142	-0.1111
Ground	7	87	Comb2	-1.4336	16.8266	498.0239	-15.837	-2.571	-0.1049
Ground	8	91	Comb2	-2.0887	-13.808	679.7074	14.0564	-2.221	-0.1131
Ground	9	100	Comb2	-2.8076	-3.1107	791.8492	3.2113	-2.908	-0.1095
Ground	10	93	Comb2	-4.0546	-8.4726	806.5839	9.1372	-4.585	-0.1074
Ground	11	94	Comb2	-1.9216	22.1205	543.6934	-20.718	-3.039	-0.1071
Ground	12	96	Comb2	3.3394	-19.974	786.9334	20.5148	2.6249	-0.1066
Ground	13	95	Comb2	4.5312	22.2271	725.2034	-20.66	3.2538	-0.1047
Ground	14	92	Comb2	-2.0057	-16.231	409.4103	16.7271	-2.147	-0.1158
Ground	15	102	Comb2	-16.028	-5.4622	1219.595	5.7593	-17.22	-0.3557
Ground	16	90	Comb2	-2.3173	-2.3114	942.8562	2.8273	-2.893	-0.1073

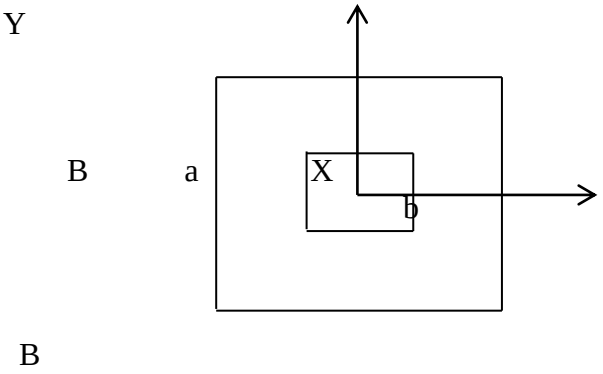


Figure9.28Footing layout

Using of Material

Concret:- fcu(Mpa)=C25/30 Reinforcement:- Fyk(Mpa)= S-400

Fck=20 Fyd(Mpa)=347.8

Fcd(Mpa)=11.13 gms=1.5

Fctm(Mpa)=2.2104 main bar(mm)=Ø20

Gmc=1.5 Es(Gpa)=200

Ecm(30)

Concrete cover=50

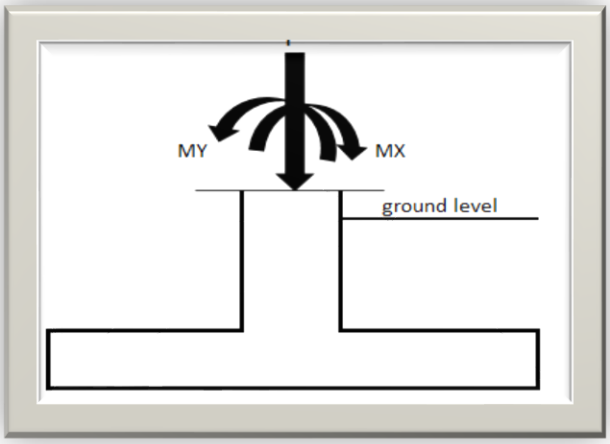


Figure9.29Sample of isolated footing

First let us assume that the footing pad is square
Area (A)=B*L=B*B

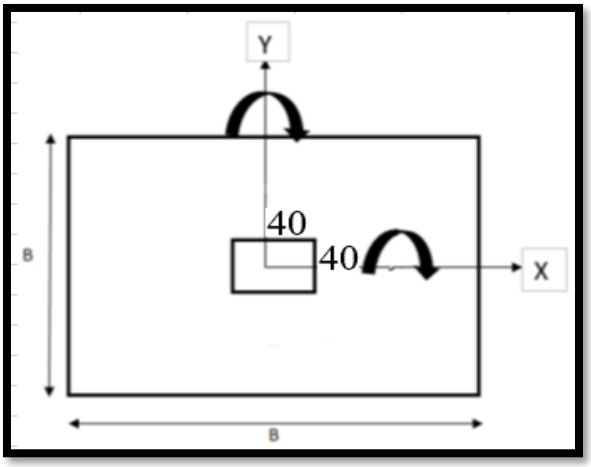


Figure 9.30Footing pad layout

BEARING CAPACITY

The soil must be capable of carrying the loads from the structure placed up on it without a shear failure and with the resulting settlements being tolerable for the structure. The design bearing resistance can be taken from the presumed design bearing pressure for different soils type.

Footing 1

Dimension of column is 400x400mm
From bearing capacity of the soil using the theoretical method
FZ=P=527.7085
M_Y=12.6457
M_X=6.7766

Area of footing req

A=$\frac{(N+W)}{\tau soil}$ =$\frac{527.7085+52.77085}{150}$ = **3.87** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6=**6.76m^2**

Weight =6.76*0.7*2 =118.3KN

δ=$\frac{P}{A}$ *(1 ± $\frac{6ey}{B}$ ± $\frac{6eX}{B}$) where ex=$\frac{My}{P}$ =$\frac{12.6457}{527.7085}$ =0.0239

ey=$\frac{Mx}{P}$ =$\frac{6.7766}{527.7085}$ =0.0128

δ_{max}=$\frac{527.7085}{6.76}$ *(1 + $\frac{6(0.0128)}{2.6}$ + $\frac{6(0.0239)}{2.6}$) = 84.67

δ_{min}=$\frac{527.7085}{6.76}$ *(1 − $\frac{6(0.0128)}{2.6}$ − $\frac{6(0.0239)}{2.6}$) = 71.44

σmax=84.64<71.44 σ(allowable)=350

Punching shear resistance

M_{xx}=79.07*$\frac{1.1^2}{2}$+(84.67-71.44)($\frac{1.1}{2}$)*(1.1 ∗ $\frac{2}{3}$=47.84+5.336= **53.173**

M_{yy}=($\frac{84.67+71.44}{2}$)*$\frac{1.1^2}{2}$**= 47.22**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{53.173}{20*2.6*70^2}$ = 0.0021< 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= **0.174**

  Z= $\frac{d}{2}$(1+$\sqrt{1-3.53k}$ ≤ 0.665

  Z= $\frac{0.7}{2}$(1+$\sqrt{1-3.53(0.0021)}$ ≤ 0.665

  Z= 0.699≤ 0.665

  Z=**0.665**

  A_S= $\frac{M}{fydz}$ = $\frac{53.173}{347.82*0.665}$ =230 mm^2

  Min & max area of reinforcement

$As_{min} = \frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm²**

As_{max}=0.04*Ac=0.04*2.6*0.7

As_{max} =**72800mm²**

Use As_{min}=2614mm²

UsProvide **6Ø20(As=2614mm²)**

Main reinforcement-transvers bar

$$k = \frac{M_{yy}}{fck * b * d^2} = \frac{47.22}{20 * 2.6 * 629^2} = 0.0023 < Kbal = 0.167$$

There for compression reinforcement is not required.

$$z = d\left(0.25 - \frac{k}{1.134}\right) = 0.23d < 0.95d$$

$$Asreq = \frac{Myy}{0.87fyk * z} = \frac{47.22 * 10^6}{0.87 * 400 * 0.239 * 629} = 902.6mm^2$$

Minimum and maximum area of reinforcement

$$Asmin = 0.26 * \left(\frac{fctm}{fyk}\right) * bd = 0.26 * \left(\frac{2.5}{400}\right) * 2.6 * 629 = \mathbf{2657.5mm^2}$$

$$Asmax = 0.04 * Ac = 0.04 * 2.6 * 0.629 = \mathbf{65416mm^2}$$

Since A_s<A_{min} use A_{smin}= $\mathbf{2657.5mm^2}$

Provide **5Ø20(As=2657.5mm²)**

1.Vertical shear

Critical shear at load from face of column

Average pressure critical section

=71.44+($\frac{2.15}{2.6}$)*13.23 =82.38

- Design shear force

$V_{€d}$= 82.38*0.35*2.6=74.97KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$\delta_1=\frac{Asl}{bd}$ =0

V_{RdC}=0

$V_{min}=[0.035K^{3/2}\sqrt{fck}]bd$

$V_{min}=[0.035* 1.532^{3/2}\sqrt{20}]2600*700$

V_{min}= 540.185KN

$V_{€d}<V_{min}$ **OK**

1. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2]

=7.895m^2

Average punching shear strees

$V_{€d}=(\frac{84.67+71.44}{2})[(2.6*2.6)-7.895]$= **886** KN

Punching shear stress

$V_{€d}=\frac{BV€d}{Ud}$, $B=1+K(\frac{M€d}{V€d})(\frac{U1}{W1})$

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

$B=1+0.659(\frac{6.05*10^6}{886*10^3})(\frac{9984}{10113660})$=1.004

$V_{€d}=\frac{1.004*886*10^3}{9984*700}$ =0.127N/mm^2

Punching shear resistance

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

$V_{Rdc}=V_{min}=0.035K^{3/2}fck^{1/2}$

$=0.035(1.535)^{3/2}(20)^{1/2}=0.298$

$0.298>V_{€d}(0.127)$ **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=527.7085

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ $B=1+K(\frac{M€d}{V€d})(\frac{Uo}{w1})$

$W1=0.5*400^2+400*400$

$W1=240000mm^2$

$B=1+0.65(\frac{6.05*10^6}{527.7085*10^3})(\frac{1600}{240000})$

B=1.05

$V_{€d}=(\frac{1.05*527.7085*10^3}{1600*886})=0.4N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}$

$=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

$=3.68>V_{€d}$**OK**

Footing 2

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=811.618

$M_Y=13.4617$

$M_X=3.0143$

Area of footing req

$A=\frac{(N+W)}{\gamma soil}=\frac{811.618+81.168}{150}=$ **5.95** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6=**6.76m²**

Weight =6.76*0.7*25=118.3KN

$\delta=\frac{P}{A}*(1\pm\frac{6ey}{B}\pm\frac{6eX}{B})$ where ex=$\frac{My}{P}=\frac{13.4617}{811.618}$ =0.0166

ey=$\frac{Mx}{P}=\frac{3.0143}{811.618}$ =0.0037

$\delta_{max}=\frac{811.618}{6.76}*(1+\frac{6(0.0037)}{2.6}+\frac{6(0.0166)}{2.6})$ = 125.681

$\delta_{min}=\frac{811.618}{6.76}*(1-\frac{6(0.0037)}{2.6}-\frac{6(0.0166)}{2.6})$ = 114.47

σmax=125.681<114.47 σ(allowable)=350

Punching shear resistance

M_{xx}=120.94*$\frac{1.1^2}{2}$+(125.681-114.47)($\frac{1.1}{2}$)*(1.1 * $\frac{2}{3}$ =73.1687+4.5 = **77.69**

M_{yy}=($\frac{125.681+114.47}{2}$)*$\frac{1.1^2}{2}$= **72.65**

- 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= $\frac{Mxx}{fck*bd^2}=\frac{77.69}{20*2.6*70^2}$ = 0.0003< 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-($\frac{0.0003}{1.134}$))

  Z= **0.175**

  Z= $\frac{d}{2}(1+\sqrt{1-3.53k}\leq 0.665$

  Z= $\frac{0.7}{2}(1+\sqrt{1-3.53(0.0003)}\leq 0.665$

  Z= 0.699≤ 0.665

  Z=**0.665**

  $A_S=\frac{M}{fydz}=\frac{77.69}{347.82*0.665}$ =336 mm²

  Min & max area of reinforcement

$As_{min}=\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

$= \frac{0.26*2.21*2.6*0.7}{400}$

As_{min}= **2614mm²**

As_{max}=0.04*Ac =0.04*2.6*0.7

As_{max} =**72800mm²**

Use As_{min}=2614mm²

2. Vertical shear

Critical shear at load from face of column

Average pressure critical section

=114.47+($\frac{2.15}{2.6}$)*11.211=123.74

↳ Design shear force

$V_{€d}$= 123.74*0.35*2.6=112.604KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm²

$ð_1$=$\frac{Asl}{bd}$ =0

V_{RdC}=0

V_{min}=[0.035$K^{3/2}\sqrt{fck}$]bd

V_{min}=[0.035∗ $1.532^{3/2}\sqrt{20}$]2600*700

V_{min}= 540.185KN

$V_{€d}$<V_{min} **OK**

3. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2]

=7.895m^2

Average punching shear strees

$V_{€d}$=($\frac{125.681+114.47}{2}$)[(2.6*2.6)-7.895]= **136** KN

Punching shear stress

$V_{€d}$=$\frac{BV€d}{Ud}$, B=1+K($\frac{M€d}{V€d}$)($\frac{U1}{W1}$)

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

B=1+0.659($\frac{6.05*10^6}{136*10^3}$)($\frac{9984}{10113660}$)=1.029

$V_{€d}$=$\frac{1.029*136*10^3}{9984*700}$ =0.02N/mm^2

Punching shear resistance

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

V_{Rdc}=V_{min}=0.035$K^{3/2}fck^{1/2}$=0.035$(1.535)^{3/2}(20)^{1/2}$ =0.298

0.298 >$V_{€d}$(0.02) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=811.618

U=2(400+400)=1600

$V_{€d}$=($\frac{BV€d}{Uod}$) B=1+K($\frac{M€d}{V€d}$)($\frac{Uo}{w1}$)

W1=0.5*400^2+400*400

W1=240000mm^2

B=1+0.65($\frac{6.05*10^6}{811.618*10^3}$)($\frac{1600}{240000}$)

B=1.03

$V_{€d}$=($\frac{1.03*811.618*10^3}{1600*136}$)=0.4N/mm^2

Maximum shear resistance

$V_{ed.max}$=0.5[0.6(1- $\frac{fck}{250}$)]$\frac{fck}{1.5}$

=0.5[0.6(1- $\frac{20}{250}$)]($\frac{20}{1.5}$)

=3.68>$V_{€d}$**Ok**

Footing 3

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=704.1208

M_Y=8.9286

M_X=16.1947

Area of footing req

A=$\frac{(N+W)}{\tau soil}$ =$\frac{704.1208+70.41208}{150}$ = **5.16** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6 **=6.76m²**

Weight =6.76*0.7*25=118.3KN

δ=$\frac{P}{A}$ *(1 ± $\frac{6ey}{B}$ ± $\frac{6eX}{B}$) where ex=$\frac{My}{P}$ =$\frac{8.9286}{704.1208}$ =0.0127

ey=$\frac{Mx}{P}$ =$\frac{16.1947}{704.1208}$ =0.0229

δ_{max}=$\frac{704.1208}{6.76}$ *(1 + $\frac{6(0.0229)}{2.6}$ + $\frac{6(0.0127)}{2.6}$) = 117.62

δ_{min}=$\frac{704.1208}{6.76}$ *(1 − $\frac{6(0.0229)}{2.6}$ − $\frac{6(0.0127)}{2.6}$) = 90.7

σmax=117.62<90.7 σ(allowable)=350

Punching shear resistance

M_{xx}=106.23*$\frac{1.1^2}{2}$+(117.62-90.7)($\frac{1.1}{2}$)*(1.1 ∗ $\frac{2}{3}$)=64.27+10.86= **75.13**

M_{yy}=($\frac{117.62+90.7}{2}$)*$\frac{1.1^2}{2}$**= 63.02**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{75.13}{20*2.6*70^2}$ = 0.000295< 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-($\frac{0.000295}{1.134}$))

Z= **0.175**

Z= $\frac{d}{2}$(1+$\sqrt{1-3.53k}$ ≤ 0.665

Z= $\frac{0.7}{2}$(1+$\sqrt{1-3.53(0.000295)}$ ≤ 0.665

Z= 0.699≤ 0.665

Z=**0.665**

A_S= $\frac{M}{fydz}$ = $\frac{75.13}{347.82*0.665}$ =325 mm^2

Min & max area of reinforcement

As_{min}= $\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm^2**

As_{max}=0.04*Ac=0.04*2.6*0.7

As_{max} =**72800mm^2**

Use As_{min}=2614mm^2

4. Vertical shear

Critical shear at load from face of column

Average pressure critical section

=90.7+($\frac{2.15}{2.6}$)*26.9 =112.96

↳ Design shear force

$V_{€d}$= 112.96*0.35*2.6=102.79KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$\delta_1=\frac{Asl}{bd}$ =0

V_{RdC}=0

V_{min}=[0.035$K^{3/2}\sqrt{fck}$]bd

V_{min}=[0.035∗ $1.532^{3/2}\sqrt{20}$]2600*700

V_{min}= 540.185KN

$V_{€d}$<V_{min} **OK**

5. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2]

=7.895m^2

Average punching shear strees

$V_{€d}$=($\frac{117.62+90.7}{2}$)[(2.6*2.6)-7.895]= **118.2** KN

Punching shear stress

$V_{€d}$=$\frac{BV€d}{Ud}$, B=1+K($\frac{M€d}{V€d}$)($\frac{U1}{W1}$)

W1=0.5$c1^2$+c1c2+4c2d+$16d^2$+2πdc1

=0.5$(400)^2$+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

B=1+0.659($\frac{6.05*10^6}{118.2*10^3}$)($\frac{9984}{10113660}$)=1.03

$V_{€d}$=$\frac{1.03*118.2*10^3}{9984*700}$ =0.017N/mm^2

Punching shear resistance

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

V_{Rdc}=V_{min}=0.035$K^{3/2}fck^{1/2}$

$=0.035(1.535)^{3/2}(20)^{1/2}=0.298$

$0.298 > V_{€d}(0.02)$ **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=704.1208

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ $B=1+K(\frac{M€d}{V€d})(\frac{Uo}{w1})$

W1=0.5*400^2+400*400

W1=240000mm^2

$B=1+0.65(\frac{6.05*10^6}{704.1208*10^3})(\frac{1600}{240000})$

B=1.037

$V_{€d}=(\frac{1.037*704.1208*10^3}{1600*118.2})=0.4N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

$=3.68>V_{€d}$**Ok**

Footing 4

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=537.3602

M_Y=11.3516

M_X=1.0911

Area of footing req

$A=\frac{(N+W)}{\tau soil}=\frac{537.3602+53.73602}{150}=$ **3.9** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6=**6.76m^2**

Weight =6.76*0.7*25=118.3KN

$\delta=\frac{P}{A}*(1\pm\frac{6ey}{B}\pm\frac{6eX}{B})$ where $ex=\frac{My}{P}=\frac{11.3516}{537.3602}=0.021$

$ey=\frac{Mx}{P}=\frac{1.0911}{537.3602}=0.002$

$\delta_{max}=\frac{537.3602}{6.76}*(1+\frac{6(0.002)}{2.6}+\frac{6(0.021)}{2.6})$ = 83.7

$\delta_{min}=\frac{537.3602}{6.76}*(1-\frac{6(0.002)}{2.6}-\frac{6(0.021)}{2.6})$ = 75.27

σmax=83.7<75.27 σ(allowable)=350

Punching shear resistance

M_{xx}=80.13*$\frac{1.1^2}{2}$+(83.7-75.27)($\frac{1.1}{2}$)*(1.1 =48.48+3.4= **51.9**

M_{yy}=($\frac{83.7+75.27}{2}$)*$\frac{1.1^2}{2}$=**48.1**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{51.9}{20*2.6*70^2}$ = 0.0002< 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-($\frac{0.0002}{1.134}$))

  Z= **0.175**

  Z= $\frac{d}{2}(1+\sqrt{1-3.53k}\leq 0.665$

  Z= $\frac{0.7}{2}(1+\sqrt{1-3.53(0.0002)}\leq 0.665$

  Z= 0.699≤ 0.665

  Z=**0.665**

  A_S= $\frac{M}{fydz}$ = $\frac{51.9}{347.82*0.665}$ =224 mm^2

  Min & max area of reinforcement

As_{min}= $\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm^2**

As_{max}=0.04*Ac=0.04*2.6*0.7

As_{max} =**72800mm^2**

Use $As_{min}=2614mm^2$

6. Vertical shear

Critical shear at load from face of column

Average pressure critical section

$=75.27+(\frac{2.15}{2.6})*8.43=82.2$

↳ Design shear force

$V_{€d}= 82.2*0.35*2.6=74.8KN$

$K=1+\sqrt{\frac{200}{d}}= 1+\sqrt{\frac{200}{700}} = 1.535< 2.0$

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$ð_1=\frac{Asl}{bd}=0$

$V_{RdC}=0$

$V_{min}=[0.035K^{3/2}\sqrt{fck}]bd$

$V_{min}=[0.035* 1.532^{3/2}\sqrt{20}]2600*700$

$V_{min}= 540.185KN$

$V_{€d}<V_{min}$ **OK**

7. Punching shear

Critical shear at 2.0d from face of column

Average $d=\frac{700+635}{2} = 667.5$

2d=1335

Control perimeter

$U=2(400+400)+(2\pi*1335)=9984mm$

Area within perimeter

$A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[\pi*1.335^2]=7.895m^2$

Average punching shear strees

$V_{€d}=(\frac{83.7+75.27}{2})[(2.6*2.6)-7.895]=$ **90.2** KN

Punching shear stress

$V_{€d}=\frac{BV€d}{Ud}$, $B=1+K(\frac{M€d}{V€d})(\frac{U1}{W1})$

$W1=0.5c1^2+c1c2+4c2d+16d^2+2\pi dc1$

$=0.5(400)^2+400*400+4*400*667.5+16*667.5^2+2*\pi*667.5*400$

$W1=\underline{10113660mm^2}$

$B=1+0.659(\frac{6.05*10^6}{90.2*10^3})(\frac{9984}{10113660})=1.03$

$V_{€d}=\frac{1.03*90.2*10^3}{9984*700}=0.013N/mm^2$

Punching shear resistance

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

$V_{Rdc}=V_{min}=0.035K^{3/2}fck^{1/2}=0.035(1.535)^{3/2}(20)^{1/2}=0.298$

$0.298 > V_{€d}(0.02)$ **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=537.3602

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ $B=1+K(\frac{M€d}{V€d})(\frac{Uo}{w1})$

$W1=0.5*400^2+400*400$

$W1=240000mm^2$

$B=1+0.65(\frac{6.05*10^6}{537.3602*10^3})(\frac{1600}{240000})$

$B=1.049$

$V_{€d}=(\frac{1.049*537.3602*10^3}{1600*90.2})=0.4N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

$=3.68>V_{€d}$ **Ok**

Footing 5

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=838.6667

M_Y=5.2172

M_X=19.1769

Area of footing req

A=$\frac{(N+W)}{\tau soil}$ =$\frac{838.6667+83.86667}{150}$ = **6.15** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6 **=6.76m²**

Weight =6.76*0.7*25=118.3KN

δ=$\frac{P}{A}$ *(1 ± $\frac{6ey}{B}$ ± $\frac{6eX}{B}$) where ex=$\frac{My}{P}$ =$\frac{5.2172}{838.6667}$ =0.0062

ey=$\frac{Mx}{P}$ =$\frac{19.1769}{838.6667}$ =0.023

δ_{max}=$\frac{838.6667}{6.76}$ *(1 + $\frac{6(0.023)}{2.6}$ + $\frac{6(0.0062)}{2.6}$) = 132.4

δ_{min}=$\frac{838.6667}{6.76}$ *(1 − $\frac{6(0.023)}{2.6}$ − $\frac{6(0.0062)}{2.6}$) = 115.7

σmax=132.4<115.7 σ(allowable)=350

Punching shear resistance

M_{xx}=125.3*$\frac{1.1^2}{2}$+(132.4-115.7)($\frac{1.1}{2}$)*(1.1 * $\frac{2}{3}$=75.81+6.74 = **82.5**

M_{yy}=($\frac{132.4+115.7}{2}$)*$\frac{1.1^2}{2}$**= 75.1**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{82.5}{20*2.6*70^2}$ = 0.00032< 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-($\frac{0.00032}{1.134}$))

  Z= **0.175**

  Z= $\frac{d}{2}$(1+$\sqrt{1-3.53k}$ ≤ 0.665

  Z= $\frac{0.7}{2}$(1+$\sqrt{1-3.53(0.00032)}$ ≤ 0.665

Z= 0.699≤ 0.665

Z=**0.665**

$A_S = \frac{M}{fydz} = \frac{82.5}{347.82*0.665}$ =357 mm^2

Min & max area of reinforcement

$As_{min} = \frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

$= \frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm²**

As_{max}=0.04*Ac=0.04*2.6*0.7

As_{max} =**72800mm²**

Use As_{min}=2614mm^2

8. Vertical shear

Critical shear at load from face of column

Average pressure critical section=115.7+($\frac{2.15}{2.6}$)*16.7 =129.5

↳ Design shear force

$V_{€d}$= 82.2*0.35*2.6=74.8KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$ð_1 = \frac{Asl}{bd}$ =0

V_{RdC}=0

V_{min}=[0.035$K^{3/2}\sqrt{fck}$]bd

V_{min}=[0.035∗ $1.532^{3/2}\sqrt{20}$]2600*700

V_{min}= 540.185KN

$V_{€d}$<V_{min} **OK**

9. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2] =7.895m^2

Average punching shear strees

$V_{€d}$=($\frac{115.7+132.4}{2}$)[(2.6*2.6)-7.895]= **140.8** KN

Punching shear stress

$V_{€d}$=$\frac{BV€d}{Ud}$, B=1+K($\frac{M€d}{V€d}$)($\frac{U1}{W1}$)

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

B=1+0.659($\frac{6.05*10^6}{140.8*10^3}$)($\frac{9984}{10113660}$)=1.03

$V_{€d}$=$\frac{1.03*140.8*10^3}{9984*700}$ =0.013N/mm^2

Punching shear resistance

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

V_{Rdc}=V_{min}=0.035$K^{3/2}fck^{1/2}$ =0.035$(1.535)^{3/2}(20)^{1/2}$ =0.298

0.298 >$V_{€d}$(0.02) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=838.6667

U=2(400+400)=1600

$V_{€d}$=($\frac{BV€d}{Uod}$) B=1+K($\frac{M€d}{V€d}$)($\frac{Uo}{w1}$)

W1=0.5*400^2+400*400

W1=240000mm^2

$B=1+0.65(\frac{6.05*10^6}{838.6667*10^3})(\frac{1600}{240000})$

$B=1.03$

$V_{€d}=(\frac{1.03*838.6667*10^3}{1600*140.8})=0.4N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

$=3.68>V_{€d}$ **Ok**

Footing 6

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=656.2962

$M_Y=7.1418$

$M_X=7.5901$

Area of footing req

$A=\frac{(N+W)}{\tau soil}=\frac{656.2962+65.62962}{150}=$ **4.8** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6=**$6.76m^2$**

Weight =6.76*0.7*25 =118.3KN

$\delta=\frac{P}{A}*(1\pm\frac{6ey}{B}\pm\frac{6eX}{B})$ where $ex=\frac{My}{P}=\frac{7.1418}{656.2962}=0.01$

$ey=\frac{Mx}{P}=\frac{7.5901}{656.2962}=0.011$

$\delta_{max}=\frac{656.2962}{6.76}*(1+\frac{6(0.011)}{2.6}+\frac{6(0.01)}{2.6})=101.76$

$\delta_{min}=\frac{656.2962}{6.76}*(1-\frac{6(0.011)}{2.6}-\frac{6(0.01)}{2.6})=92.44$

σmax=101.76<92.44 σ(allowable)=350

Punching shear resistance

$M_{xx}=97.8*\frac{1.1^2}{2}+(101.76-92.44)(\frac{1.1}{2})*(1.1*\frac{2}{3})=59.169+3.759=$ **62.93**

$M_{yy}=(\frac{101.76+92.44}{2})*\frac{1.1^2}{2}=$ **58.74**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{62.93}{20*2.6*70^2}$ = 0.00025< 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-($\frac{0.00025}{1.134}$))

Z= **0.175**

Z= $\frac{d}{2}$(1+$\sqrt{1-3.53k}$ ≤ 0.665

Z= $\frac{0.7}{2}$(1+$\sqrt{1-3.53(0.00025)}$ ≤ 0.665

Z= 0.699≤ 0.665

Z=**0.665**

A_S= $\frac{M}{fydz}$ = $\frac{62.93}{347.82*0.665}$ =272 mm^2

Min & max area of reinforcement

As_{min}= $\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm^2**

As_{max}=0.04*Ac=0.04*2.6*0.7

As_{max} =**72800mm^2**

Use As_{min}=2614mm^2

10. Vertical shear

Critical shear at load from face of column

Average pressure critical section =92.44+($\frac{2.15}{2.6}$)*9.3 =100.1

- Design shear force

$V_{€d}$= 100.1*0.35*2.6=91.11KN

$K=1+\sqrt{\frac{200}{d}}= 1+\sqrt{\frac{200}{700}} = 1.535< 2.0$

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$ƍ_1=\frac{Asl}{bd}$ =0

V_{RdC}=0

$V_{min}=[0.035K^{3/2}\sqrt{fck}]bd$

$V_{min}=[0.035* 1.532^{3/2}\sqrt{20}]2600*700$

V_{min}= 540.185KN

$V_{€d}<V_{min}$ **OK**

11. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2] =7.895m^2

Average punching shear strees

$V_{€d}=(\frac{101.76+92.44}{2})[(2.6*2.6)-7.895]=$ **110.2** KN

Punching shear stress

$V_{€d}=\frac{BV€d}{Ud}$, B=1+K$(\frac{M€d}{V€d})(\frac{U1}{W1})$

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

$B=1+0.659(\frac{6.05*10^6}{110.2*10^3})(\frac{9984}{10113660})=1.03$

$V_{€d}=\frac{1.03*110.2*10^3}{9984*700}$ =0.013N/mm^2

Punching shear resistance

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

$V_{Rdc}=V_{min}=0.035K^{3/2}fck^{1/2}=0.035(1.535)^{3/2}(20)^{1/2}=0.298$

0.298 >$V_{€d}$(0.02) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=537.3602

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ $B=1+K(\frac{M€d}{V€d})(\frac{Uo}{w1})$

$W1=0.5*400^2+400*400$

$W1=240000mm^2$

$B=1+0.65(\frac{6.05*10^6}{656.2962*10^3})(\frac{1600}{240000})$

B=1.039

$V_{€d}=(\frac{1.039*656.2962*10^3}{1600*110.2})=0.4N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}$

$=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

$=3.68>V_{€d}$**Ok**

Footing 7

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=498.0239

M_Y=2.5706

M_X=15.8365

Area of footing req

$A=\frac{(N+W)}{\tau soil}=\frac{489.0239+48.90239}{150}=$ **3.6** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6 **=6.76m²**

Weight =6.76*0.7*25 =118.3KN

$\delta=\frac{P}{A}*(1\pm\frac{6ey}{B}\pm\frac{6eX}{B})$ where ex=$\frac{My}{P}=\frac{2.5706}{498.0239}$ =0.0052

ey=$\frac{Mx}{P}=\frac{15.8365}{498.0239}$ =0.032

$\delta_{max}=\frac{498.0239}{6.76}*(1+\frac{6(0.0052)}{2.6}+\frac{6(0.032)}{2.6})$ = 80.3

$\delta_{min}=\frac{498.0239}{6.76}*(1-\frac{6(0.052)}{2.6}-\frac{6(0.032)}{2.6})$ =67

σmax=101.76<92.44 σ(allowable)=350

Punching shear resistance

M_{xx}=74.67*$\frac{1.1^2}{2}$+(80.3-67)($\frac{1.1}{2}$)*(1.1 * $\frac{2}{3}$ =45.175+5.36 = **50.5**

M_{yy}=($\frac{80.3+67}{2}$)*$\frac{1.1^2}{2}$**= 44.56**

- 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= $\frac{Mxx}{fck*bd^2}=\frac{50.5}{20*2.6*70^2}$ = 0.000198< 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-($\frac{0.000198}{1.134}$))

  Z= **0.175**

  Z= $\frac{d}{2}(1+\sqrt{1-3.53k}\leq 0.665$

  Z= $\frac{0.7}{2}(1+\sqrt{1-3.53(0.00025)}\leq 0.665$

  Z= 0.699≤ 0.665

  Z=**0.665**

  $A_S=\frac{M}{fydz}=\frac{50.5}{347.82*0.665}$ =218 mm^2

  Min & max area of reinforcement

$As_{min}=\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

$= \frac{0.26*2.21*2.6*0.7}{400}$

As_{min}= **2614mm²**

As_{max}=0.04*Ac =0.04*2.6*0.7

As_{max} =**72800mm²**

Use As_{min}=2614mm²

UsProvide **6Ø20(As=2614mm²)**

Main reinforcement-transvers bar

$$k = \frac{M_{yy}}{fck * b * d^2} = \frac{44.56}{20 * 2.6 * 629^2} = 0.0021 < Kbal = 0.167$$

There for compression reinforcement is not required.

$$z = d\left(0.25 - \frac{k}{1.134}\right) = 0.23d < 0.95d$$

$$Asreq = \frac{Myy}{0.87fyk * z} = \frac{44.56 * 10^6}{0.87 * 400 * 0.239 * 629} = 851.76mm^2$$

Minimum and maximum area of reinforcement

$$Asmin = 0.26 * \left(\frac{fctm}{fyk}\right) * bd = 0.26 * \left(\frac{2.5}{400}\right) * 2.6 * 629 = \mathbf{2657.5mm^2}$$

$$Asmax = 0.04 * Ac = 0.04 * 2.6 * 0.629 = \mathbf{65416mm^2}$$

Since A_s<A_{min} use A_{smin}= $\mathbf{2657.5mm^2}$

Provide **5Ø20(As=2657.5mm²)**

12. Vertical shear

Critical shear at load from face of column

Average pressure critical section

=67+($\frac{2.15}{2.6}$)*13.3 =77.99

↳ Design shear force

$V_{€d}$= 77.99*0.35*2.6=70.9KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$\delta_1=\frac{Asl}{bd}$ =0

V_{RdC}=0

$V_{min}=[0.035K^{3/2}\sqrt{fck}]bd$

$V_{min}=[0.035* 1.532^{3/2}\sqrt{20}]2600*700$

V_{min}= 540.185KN

$V_{€d}<V_{min}$ **OK**

13. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2] =7.895m^2

Average punching shear strees

$V_{€d}=(\frac{80.3+67}{2})[(2.6*2.6)-7.895]=$ **83.59** KN

Punching shear stress

$V_{€d}=\frac{BV€d}{Ud}$, $B=1+K(\frac{M€d}{V€d})(\frac{U1}{W1})$

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

$B=1+0.659(\frac{6.05*10^6}{83.59*10^3})(\frac{9984}{10113660})=1.047$

$V_{€d}=\frac{1.047*83.59*10^3}{9984*700}$ =0.013N/mm^2

Punching shear resistance

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

$V_{Rdc}=V_{min}=0.035K^{3/2}fck^{1/2}$

$=0.035(1.535)^{3/2}(20)^{1/2}=0.298$

$0.298 > V_{€d}(0.02)$ **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=498.0239

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ $B=1+K(\frac{M€d}{V€d})(\frac{Uo}{w1})$

$W1=0.5*400^2+400*400$

$W1=240000mm^2$

$B=1+0.65(\frac{6.05*10^6}{498.0239*10^3})(\frac{1600}{240000})$

B=1.039

$V_{€d}=(\frac{1.039*656.2962*10^3}{1600*110.2})=0.4N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}$

$=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

$=3.68>V_{€d}$**Ok**

Footing 8

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=679.7074

M_Y=2.2209

M_X=14.0564

Area of footing req

$A=\frac{(N+W)}{\gamma soil}=\frac{679.7074+67.9707}{150}=$ **4.9**where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6 **=6.76m²**

Weight =6.76*0.7*25=118.3KN

$\delta=\frac{P}{A}*(1\pm\frac{6ey}{B}\pm\frac{6eX}{B})$ where ex=$\frac{My}{P}=\frac{2.2209}{679.7074}$ =0.00327

ey=$\frac{Mx}{P}=\frac{14.0564}{679.7074}$ =0.021

$\delta_{max}=\frac{679.7074}{6.76}*(1+\frac{6(0.00327)}{2.6}+\frac{6(0.021)}{2.6})$ = 106.2

$\delta_{min}=\frac{679.7074}{6.76}*(1-\frac{6(0.00327)}{2.6}-\frac{6(0.021)}{2.6})$ =104.67

σmax=106.2<104.67 σ(allowable)=350

Punching shear resistance

M_{xx}=105.6*$\frac{1.1^2}{2}$+(106.2-104.67)($\frac{1.1}{2}$)*(1.1 * $\frac{2}{3}$ =63.89+0.617 = **64.51**

$M_{yy}=(\frac{106.2+104.67}{2})*\frac{1.1^2}{2}$ **= 63.79**

- 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= $\frac{Mxx}{fck*bd^2}=\frac{64.51}{20*2.6*70^2}$ = 0.00025< 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-($\frac{0.00025}{1.134}$))

  Z= **0.175**

  Z= $\frac{d}{2}(1+\sqrt{1-3.53k}\leq 0.665$

  Z= $\frac{0.7}{2}(1+\sqrt{1-3.53(0.00025)}\leq 0.665$

  Z= 0.699≤ 0.665

  Z=**0.665**

  $A_S=\frac{M}{fydz}=\frac{64.51}{347.82*0.665}$ =279mm²

  Min & max area of reinforcement

$As_{min}=\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

$= \frac{0.26*2.21*2.6*0.7}{400}$

As_{min}= **2614mm²**

As_{max}=0.04*Ac

=0.04*2.6*0.7

As_{max} =**72800mm²**

UsProvide **6Ø20(As=2614mm²)**

Main reinforcement-transvers bar

$$k = \frac{M_{yy}}{fck * b * d^2} = \frac{63.79}{24 * 2.5 * 629^2} = 0.0031 < Kbal = 0.167$$

There for compression reinforcement is not required.

$$z = d\left(0.25 - \frac{k}{1.134}\right) = 0.23d < 0.95d$$

$$Asreq = \frac{Myy}{0.87fyk * z} = \frac{63.79 * 10^6}{0.87 * 400 * 0.239 * 629} = 1219.3mm^2$$

Minimum and maximum area of reinforcement

$$Asmin = 0.26 * \left(\frac{fctm}{fyk}\right) * bd = 0.26 * \left(\frac{2.5}{400}\right) * 2.5 * 629 = \mathbf{2657.5mm^2}$$

$$Asmax = 0.04 * Ac = 0.04 * 2.6 * 0.629 = \mathbf{65416mm^2}$$

Since A_s<A_{min} use A_{smin}= $\mathbf{2657.5mm^2}$

Provide **5Ø20(As=2657.5mm²)**

1.Vertical shear

Critical shear at load from face of column

Average pressure critical section

=104.67+($\frac{2.15}{2.6}$)*1.53=105.9

↳ Design shear force

$V_{€d}$= 105.9*0.35*2.6=96.4KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm²

$\eth_1=\frac{Asl}{bd}$ =0

V_{RdC}=0

V_{min}=[0.035$K^{3/2}\sqrt{fck}$]bd

V_{min}=[0.035∗ $1.532^{3/2}\sqrt{20}$]2600*700

V_{min}= 540.185KN

$V_{€d}$<V_{min} **OK**

1. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2] =7.895m^2

Average punching shear strees

$V_{€d}$=($\frac{106.2+104.67}{2}$)[(2.6*2.6)-7.895]= **119.7** KN

Punching shear stress

$V_{€d}$=$\frac{BV€d}{Ud}$, B=1+K($\frac{M€d}{V€d}$)($\frac{U1}{W1}$)

W1=0.5$c1^2$+c1c2+4c2d+$16d^2$+2πdc1

=0.5$(400)^2$+400*400+4*400*667.5+$16*667.5^2$ +2*π*667.5*400

W1= <u>10113660mm^2</u>

B=1+0.659($\frac{6.05*10^6}{119.7*10^3}$)($\frac{9984}{10113660}$)=1.032

$V_{€d}$=$\frac{1.032*119.7*10^3}{9984*700}$ =0.0177N/mm^2

Punching shear resistance

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

V_{Rdc}=V_{min}=0.035$K^{3/2}fck^{1/2}$=0.035$(1.535)^{3/2}(20)^{1/2}$ =0.298

0.298 >$V_{€d}$(0.02) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=498.0239

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ $B=1+K(\frac{M€d}{V€d})(\frac{Uo}{w1})$

W1=0.5*400^2+400*400

W1=240000mm^2

$B=1+0.65(\frac{6.05*10^6}{679.7074*10^3})(\frac{1600}{240000})$

B=1.039

$V_{€d}=(\frac{1.039*679.7074*10^3}{1600*110.2})$=0.4N/mm^2

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

=3.68>$V_{€d}$**O**

Footing 9

P=791.8492

M_Y=2.9081

M_X=3.2113

Area of footing req

$A=\frac{(N+W)}{\tau soil}=\frac{791.8492+791.8492}{150}$ = **10.56** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6 **=6.76m^2**

Weight =6.76*0.7*25=118.3KN

$\delta=\frac{P}{A}*(1\pm\frac{6ey}{B}\pm\frac{6eX}{B})$ where $ex=\frac{My}{P}=\frac{2.9081}{791.8492}$ =0.0037

$ey=\frac{Mx}{P}=\frac{3.2113}{791.8492}$ =0.0041

$\delta_{max}=\frac{791.8492}{6.76}*(1+\frac{6(0.0041)}{2.6}+\frac{6(0.0037)}{2.6})$ = 119.25

$\delta_{min}=\frac{791.8492}{6.76}*(1-\frac{6(0.0041)}{2.6}-\frac{6(0.0037)}{2.6})$ = 115.03

σmax=115.03<119.25 σ(allowable)=350

Punching shear resistance

M_{xx}=117.46*$\frac{1.1^2}{2}$+(119.25-115.03)($\frac{1.1}{2}$)*(1.1 * $\frac{2}{3}$) =71.063+1.702 = **72.765**

M_{yy}=($\frac{119.25+115.03}{2}$)*$\frac{1.1^2}{2}$= **70.870**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{72.765}{20*2.6*70^2}$ = 0.00029< 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-($\frac{0.00029}{1.134}$))

  Z= **0.175**

  Z= $\frac{d}{2}$(1+$\sqrt{1-3.53k}$ ≤ 0.665

  Z= $\frac{0.7}{2}$(1+$\sqrt{1-3.53(0.00025)}$ ≤ 0.665

  Z= 0.699≤ 0.665

  Z=**0.665**

  A_S= $\frac{M}{fydz}$ = $\frac{63.27}{347.82*0.665}$ =274 mm^2

  Min & max area of reinforcement

As_{min}= $\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm^2**

As_{max}=0.04*Ac =0.04*2.6*0.7

As_{max} =**72800mm^2**

Use As_{min}=2

614mm^2

14. Vertical shear

Critical shear at load from face of column

Average pressure critical section

=115.03+($\frac{2.15}{2.6}$)*4.22 =118.52

- Design shear force

$V_{€d}$= 118.52*0.35*2.6=107.85KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$ð_1$=$\frac{Asl}{bd}$ =0

V_{RdC}=0

V_{min}=[0.035$K^{3/2}\sqrt{fck}$]bd

V_{min}=[0.035∗ $1.532^{3/2}\sqrt{20}$]2500*700

V_{min}= 540.773KN

$V_{€d}$<V_{min} **OK**

15. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2] =7.895m^2

Average punching shear strees

$V_{€d}$=($\frac{102.13+92.04}{2}$)[(2.6*2.6)-7.895]= **110.19** KN

Punching shear stress

$V_{€d}$=$\frac{BV€d}{Ud}$, B=1+K($\frac{M€d}{V€d}$)($\frac{U1}{W1}$)

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400)^2+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= 10113660mm^2

B=1+0.659($\frac{6.05*10^6}{110.19*10^3}$)($\frac{9984}{10113660}$)=1.036

$V_{€d}$=$\frac{1.036*110.19*10^3}{9984*700}$ =0.016N/mm^2

Punching shear resistance

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

$V_{Rdc}=V_{min}=0.035K^{3/2}fck^{1/2}=0.035(1.535)^{3/2}(20)^{1/2}$ =0.298

0.298 >$V_{€d}$(0.127) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=656.2962

U=2(400+400)=1600

$V_{€d}$=($\frac{BV€d}{Uod}$) B=1+K($\frac{M€d}{V€d}$)($\frac{Uo}{w1}$)

W1=0.5*400^2+400*400

W1=240000mm^2

B=1+0.65($\frac{6.05*10^6}{656.2962*10^3}$)($\frac{1600}{240000}$)

B=1.04

$V_{€d}$=($\frac{1.04*656.2962*10^3}{1600*886}$)=0.5N/mm^2

Maximum shear resistance

$V_{ed.max}$=0.5[0.6(1- $\frac{fck}{250}$)]$\frac{fck}{1.5}$ =0.5[0.6(1- $\frac{20}{250}$)]($\frac{20}{1.5}$)

=3.68>$V_{€d}$**OK**

Footing 10

P=806.5839

M_Y=4.5851

M_X=9.1372

Area of footing req

A=$\frac{(N+W)}{\gamma soil}$ =$\frac{806.5839+806.5839}{150}$ = **10.75** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6 **=6.76m^2**

Weight =6.76*0.7*25 =118.3KN

δ=$\frac{P}{A}$ *(1 ± $\frac{6ey}{B}$ ± $\frac{6eX}{B}$) where ex=$\frac{My}{P}$ =$\frac{4.5851}{806.5839}$ =0.0057

ey=$\frac{Mx}{P}$ =$\frac{9.1372}{806.5839}$ =0.0113

δ_{max}=$\frac{806.5839}{6.76}$ *(1 + $\frac{6(0.0113)}{2.6}$ + $\frac{6(0.0057)}{2.6}$) = 124.01

δ_{min}=$\frac{806.5839}{6.76}$ *(1 − $\frac{6(0.0113)}{2.6}$ − $\frac{6(0.0057)}{2.6}$) = 114.63

σmax=114.63<124.01 σ(allowable)=350

Punching shear resistance

M_{xx}=120.04*$\frac{1.1^2}{2}$+(124.01-114.63)($\frac{1.1}{2}$)*(1.1 * $\frac{2}{3}$) =72.6+3.7833 = **76.38**

M_{yy}=($\frac{124.01+114.63}{2}$)*$\frac{1.1^2}{2}$**= 72.19**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{76.38}{20*2.6*70^2}$ = 0.00029< 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-($\frac{0.00025}{1.134}$))

  Z= **0.174**

  Z= $\frac{d}{2}$(1+$\sqrt{1-3.53k}$ ≤ 0.665

  Z= $\frac{0.7}{2}$(1+$\sqrt{1-3.53(0.00025)}$ ≤ 0.665

  Z= 0.699≤ 0.665

  Z=**0.665**

  A_S= $\frac{M}{fydz}$ = $\frac{63.27}{347.82*0.665}$ =274 mm^2

  Min & max area of reinforcement

$As_{min}= \frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm²**

As_{max}=0.04*Ac=0.04*2.6*0.7

As_{max} =**72800mm²**

Use As_{min}=2614mm²

16. Vertical shear

Critical shear at load from face of column

Average pressure critical section

=114.01+($\frac{2.15}{2.6}$)*9.36=121.77

↳ Design shear force

$V_{€d}$= 121.77*0.35*2.6=110.81KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm²

$ƍ_1$=$\frac{Asl}{bd}$ =0

V_{RdC}=0

V_{min}=[0.035$K^{3/2}\sqrt{fck}$]bd

V_{min}=[0.035∗ $1.532^{3/2}\sqrt{20}$]2500*700

V_{min}= 540.773KN

$V_{€d}$<V_{min} **OK**

17. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2] =7.895m^2

Average punching shear strees

$V_{€d}$=($\frac{124.01+114.63}{2}$)[(2.6*2.6)-7.895]= **135.43** KN

Punching shear stress

$V_{€d}$=$\frac{BV€d}{Ud}$, B=1+K($\frac{M€d}{V€d}$)($\frac{U1}{W1}$)

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

B=1+0.659($\frac{6.05*10^6}{110.19*10^3}$)($\frac{9984}{10113660}$)=1.036

$V_{€d}$=$\frac{1.036*110.19*10^3}{9984*700}$ =0.016N/mm^2

Punching shear resistance

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

V_{Rdc}=V_{min}=0.035$K^{3/2}fck^{1/2}$

=0.035$(1.535)^{3/2}(20)^{1/2}$ =0.298

0.298 >$V_{€d}$(0.127) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=656.2962

U=2(400+400)=1600

$V_{€d}$=($\frac{BV€d}{Uod}$) B=1+K($\frac{M€d}{V€d}$)($\frac{Uo}{w1}$)

W1=0.5*400^2+400*400

W1=240000mm^2

B=1+0.65($\frac{6.05*10^6}{656.2962*10^3}$)($\frac{1600}{240000}$)

B=1.04

$V_{€d}=(\frac{1.04*656.2962*10^3}{1600*886})=0.5N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

=3.68>$V_{€d}$**OK**

Footing 11

P=543.6934

M_Y=3.0391

M_X=20.7178

Area of footing req

$A=\frac{(N+W)}{\tau soil}=\frac{543.6934+543.6934}{150}$ = **7.25** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6

=6.76m²

Weight =6.76*0.7*25 =118.3KN

$\delta=\frac{P}{A}*(1\pm\frac{6ey}{B}\pm\frac{6eX}{B})$ where $ex=\frac{My}{P}=\frac{3.0391}{543.6934}$ =0.0056

$ey=\frac{Mx}{P}=\frac{20.7178}{543.6934}$ =0.0381

$\delta_{max}=\frac{543.6934}{6.76}*(1+\frac{6(0.0381)}{2.6}+\frac{6(0.0056)}{2.6})$ = 88.54

$\delta_{min}=\frac{543.6934}{6.76}*(1-\frac{6(0.0381)}{2.6}-\frac{6(0.0056)}{2.6})$ = 72.32

σmax=72.32<88.54 σ(allowable)=350

Punching shear resistance

$M_{xx}=81.68*\frac{1.1^2}{2}+(88.54-72.32)(\frac{1.1}{2})*(1.1*\frac{2}{3})=49.42+6.5421$ = **55.96**

$M_{yy}=(\frac{88.54+72.32}{2})*\frac{1.1^2}{2}$ **= 48.66**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{55.96}{20*2.6*70^2}$ = 0.00022< 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-($\frac{0.00025}{1.134}$))

Z= **0.174**

Z= $\frac{d}{2}(1+\sqrt{1-3.53k} \leq 0.665$

Z= $\frac{0.7}{2}(1+\sqrt{1-3.53(0.00025)} \leq 0.665$

Z= 0.699≤ 0.665

Z=**0.665**

A_S= $\frac{M}{fydz}$ = $\frac{63.27}{347.82*0.665}$ =274 mm^2

Min & max area of reinforcement

As_{min}= $\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm^2**

As_{max}=0.04*Ac

=0.04*2.6*0.7

As_{max} =**72800mm^2**

Use As_{min}=2614mm^2

18. Vertical shear

Critical shear at load from face of column

Average pressure critical section=72.32+($\frac{2.15}{2.6}$)*9.36 =80.06

- Design shear force

$V_{€d}$= 80.06*0.35*2.6=72.8546KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$\delta_1=\frac{Asl}{bd}$ =0

V_{RdC}=0

$V_{min}=[0.035K^{3/2}\sqrt{fck}]bd$

$V_{min}=[0.035* 1.532^{3/2}\sqrt{20}]2500*700$

V_{min}= 540.773KN

$V_{€d}<V_{min}$ **OK**

19. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2] =7.895m^2

Average punching shear strees

$V_{€d}=(\frac{88.54+72.32}{2})[(2.6*2.6)-7.895]$= **91.288** KN

Punching shear stress

$V_{€d}=\frac{BV€d}{Ud}$, $B=1+K(\frac{M€d}{V€d})(\frac{U1}{W1})$

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

$B=1+0.659(\frac{6.05*10^6}{110.19*10^3})(\frac{9984}{10113660})$=1.036

$V_{€d}=\frac{1.036*110.19*10^3}{9984*700}$ =0.016N/mm^2

Punching shear resistance

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

$V_{Rdc}=V_{min}=0.035K^{3/2}fck^{1/2}$ $=0.035(1.535)^{3/2}(20)^{1/2}$ $=0.298$

0.298 $>V_{€d}$(0.127) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=656.2962

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ B=1+K$(\frac{M€d}{V€d})(\frac{Uo}{w1})$

W1=0.5*400^2+400*400

W1=240000mm^2

B=1+0.65$(\frac{6.05*10^6}{656.2962*10^3})(\frac{1600}{240000})$

B=1.04

$V_{€d}=(\frac{1.04*656.2962*10^3}{1600*886})$=0.5N/mm^2

Maximum shear resistance

$V_{ed.max}$=0.5[0.6(1- $\frac{fck}{250}$)]$\frac{fck}{1.5}$ =0.5[0.6(1- $\frac{20}{250}$)]$(\frac{20}{1.5})$

=3.68>$V_{€d}$**OK**

Footing 12

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=786.933

M_Y=2.624

M_X=20.5148

Area of footing req

A=$\frac{(N+W)}{\tau soil}$ =$\frac{786.933+78.6933}{150}$ = **5.77** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6 **=6.76m^2**

Weight =6.76*0.7*25 =118.3KN

δ=$\frac{P}{A}$ *(1 $\pm \frac{6ey}{B} \pm \frac{6eX}{B}$) where ex=$\frac{My}{P}$ =$\frac{2.6249}{786.933}$ =0.0033

ey=$\frac{Mx}{P}$ =$\frac{20.5148}{786.933}$ =0.0026

$\delta_{max}=\frac{786.933}{6.76}*(1+\frac{6(0.0026)}{2.6}+\frac{6(0.0033)}{2.6})$ = 117.98

$\delta_{min}=\frac{786.933}{6.76}*(1-\frac{6(0.0226)}{2.6}-\frac{6(0.0033)}{2.6})$ = 114.81

σmax=117.98<114.81 σ(allowable)=350

Punching shear resistance

M_{xx}=116.6*$\frac{1.1^2}{2}$+(117.98-114.81)($\frac{1.1}{2}$)*(1.1 ∗ $\frac{2}{3}$) =70.543+1..5 = **71.82**

M_{yy}=($\frac{117.98+114.81}{2}$)*$\frac{1.1^2}{2}$= **70.4**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{71.82}{20*2.6*70^2}$ = 0.00028< 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-($\frac{0.00028}{1.134}$))

  Z= **0.175**

  Z= $\frac{d}{2}$(1+$\sqrt{1-3.53k}$ ≤ 0.665

  Z= $\frac{0.7}{2}$(1+$\sqrt{1-3.53(0.000295)}$ ≤ 0.665

  Z= 0.699≤ 0.665

  Z=**0.665**

  A_S= $\frac{M}{fydz}$ = $\frac{71.82}{347.82*0.665}$ =310 mm^2

  Min & max area of reinforcement

As_{min}= $\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}= **2614mm^2**

As_{max}=0.04*Ac=0.04*2.6*0.7

As_{max} =**72800mm^2**

Use $As_{min}=2614mm^2$

20. Vertical shear

Critical shear at load from face of columnAverage pressure critical section

$=107.28+(\frac{2.15}{2.6})*1.42 =108.5$

↳ Design shear force

$V_{€d}$= 108.5*0.35*2.6=98.69KN

$K=1+\sqrt{\frac{200}{d}}= 1+\sqrt{\frac{200}{700}} = 1.535< 2.0$

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$ð_1=\frac{Asl}{bd}$ =0

V_{RdC}=0

$V_{min}=[0.035K^{3/2}\sqrt{fck}]bd$

$V_{min}=[0.035* 1.532^{3/2}\sqrt{20}]2600*700$

V_{min}= 540.185KN

$V_{€d}<V_{min}$ **OK**

21. Punching shear

Critical shear at 2.0d from face of column

Average $d=\frac{700+635}{2}$ = 667.5

2d=1335

Control perimeter

$U=2(400+400)+(2\pi*1335)=9984mm$

Area within perimeter

$A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[\pi*1.335^2] =7.895m^2$

Average punching shear strees

$V_{€d}=(\frac{108.7+107.28}{2})[(2.6*2.6)-7.895]=$ **118.2** KN

Punching shear stress

$V_{€d}=\frac{BV€d}{Ud}$, $B=1+K(\frac{M€d}{V€d})(\frac{U1}{W1})$

$W1=0.5c1^2+c1c2+4c2d+16d^2+2\pi dc1$

$=0.5(400)^2+400*400+4*400*667.5+16*667.5^2+2*\pi*667.5*400$

W1= $\underline{10113660mm^2}$

$B=1+0.659(\frac{6.05*10^6}{118.2*10^3})(\frac{9984}{10113660})=1.03$

$V_{€d}=\frac{1.03*118.2*10^3}{9984*700}=0.017N/mm^2$

Punching shear resistance

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

$V_{Rdc}=V_{min}=0.035K^{3/2}fck^{1/2}$

$=0.035(1.535)^{3/2}(20)^{1/2}=0.298$

$0.298 > V_{€d}(0.02)$ **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=786.9334

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ $B=1+K(\frac{M€d}{V€d})(\frac{Uo}{w1})$

$W1=0.5*400^2+400*400$

$W1=240000mm^2$

$B=1+0.65(\frac{6.05*10^6}{942.8562*10^3})(\frac{1600}{240000})$

B=1.037

$V_{€d}=(\frac{1.037*786.933*10^3}{1600*118.2})=0.5N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

$=3.68>V_{€d}$**Ok**

Footing 13

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=725.2034

M_Y=2.1472

M_X=20.6603

Area of footing req

A=$\frac{(N+W)}{\tau soil}$ =$\frac{725.2034+72.52034}{150}$ = **5.32** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6

=6.76m²

Weight =6.76*0.7*25

=118.3KN

δ=$\frac{P}{A}$ *(1 ± $\frac{6ey}{B}$ ± $\frac{6eX}{B}$) where ex=$\frac{My}{P}$ =$\frac{2.1472}{725.2034}$ =0.0029

ey=$\frac{Mx}{P}$ =$\frac{20.6603}{725.2034}$ =0.0028

δ_{max}=$\frac{725.2034}{6.76}$ *(1 + $\frac{6(0.0029)}{2.6}$ + $\frac{6(0.0028)}{2.6}$) = 108.7

δ_{min}=$\frac{942.8562}{6.76}$ *(1 − $\frac{6(0.0229)}{2.6}$ − $\frac{6(0.0031)}{2.6}$) = 107.3

σmax=108.7<107.3 σ(allowable)=350

Punching shear resistance

M_{xx}=108.1*$\frac{1.1^2}{2}$+(108.7-107.3)($\frac{1.1}{2}$)*(1.1 * $\frac{2}{3}$)=65.4+0.791= **66.19**

M_{yy}=($\frac{108.7+107.3}{2}$)*$\frac{1.1^2}{2}$

= 65.34

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{66.19}{20*2.6*70^2}$ = 0.00026< 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-($\frac{0.00026}{1.134}$))

  Z= **0.175**

Z= $\frac{d}{2}(1+\sqrt{1-3.53k} \leq 0.665$

Z= $\frac{0.7}{2}(1+\sqrt{1-3.53(0.000295)} \leq 0.665$

Z= 0.699≤ 0.665

Z=**0.665**

$A_S = \frac{M}{fydz} = \frac{66.19}{347.82*0.665}$ =286 mm^2

Min & max area of reinforcement

$As_{min} = \frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm^2**

As_{max}=0.04*Ac

=0.04*2.6*0.7

As_{max} =**72800mm^2**

Use As_{min}=2614mm^2

22. Vertical shear

Critical shear at load from face of columnAverage pressure critical section

=107.3+($\frac{2.15}{2.6}$)*1.4=108.5

↳ Design shear force

$V_{€d}$= 108.5*0.35*2.6=98.74KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$ð_1=\frac{Asl}{bd}$ =0 ,V_{RdC}=0

V_{min}=[0.035$K^{3/2}\sqrt{fck}$]bd

V_{min}=[0.035∗ $1.532^{3/2}\sqrt{20}$]2600*700

V_{min}= 540.185KN

$V_{€d} < V_{min}$ **OK**

23. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5= 2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2] =7.895m^2

Average punching shear strees

$V_{€d}$=($\frac{108.7+107.28}{2}$)[(2.6*2.6)-7.895]= **122.6** KN

Punching shear stress

$V_{€d}$=$\frac{BV€d}{Ud}$, B=1+K($\frac{M€d}{V€d}$)($\frac{U1}{W1}$)

W1=0.5$c1^2$+c1c2+4c2d+16d^2+2πdc1

=0.5$(400)^2$+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

B=1+0.659($\frac{6.05*10^6}{122.6*10^3}$)($\frac{9984}{10113660}$)=1.03

$V_{€d}$=$\frac{1.03*122.6*10^3}{9984*700}$ =0.017N/mm^2

Punching shear resistance

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

V_{Rdc}=V_{min}=0.035$K^{3/2}fck^{1/2}$ =0.035$(1.535)^{3/2}(20)^{1/2}$ =0.298

0.298 >$V_{€d}$(0.02) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=725.2034

U=2(400+400)=1600

$V_{€d}$=($\frac{BV€d}{Uod}$) B=1+K($\frac{M€d}{V€d}$)($\frac{Uo}{w1}$)

W1=0.5*400^2+400*400

W1=240000mm^2

B=1+0.65($\frac{6.05*10^6}{942.8562*10^3}$)($\frac{1600}{240000}$)

B=1.037

$V_{€d}$=($\frac{1.037*725.2034*10^3}{1600*118.2}$)=0.5N/mm^2

Maximum shear resistance

$V_{ed.max}$=0.5[0.6(1- $\frac{fck}{250}$)]$\frac{fck}{1.5}$=0.5[0.6(1- $\frac{20}{250}$)]($\frac{20}{1.5}$)

=3.68>$V_{€d}$**Ok**

Footing 14

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=409.4103

M_Y=2.147

M_X=16.7271

Area of footing req

A=$\frac{(N+W)}{\tau soil}$ =$\frac{409.4103+40.94103}{150}$ = **3** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6=**6.76m^2**

Weight =6.76*0.7*25=118.3KN

δ=$\frac{P}{A}$ *(1 ± $\frac{6ey}{B}$ ± $\frac{6eX}{B}$) where ex=$\frac{My}{P}$ =$\frac{2.147}{409.4103}$ =0.00524

ey=$\frac{Mx}{P}$ =$\frac{16.7271}{409.4103}$ =0.041

δ_{max}=$\frac{409.4103}{6.76}$ *(1 + $\frac{6(0.00524)}{2.6}$ + $\frac{6(0.041)}{2.6}$) = 76.196

δ_{min}=$\frac{409.4103}{6.76}$ *(1 − $\frac{6(0.00524)}{2.6}$ − $\frac{6(0.041)}{2.6}$) = 44.9

σmax=76.196<44.9 σ(allowable)=350

Punching shear resistance

M_{xx}=62.96*$\frac{1.1^2}{2}$+(76.196-44.9)($\frac{1.1}{2}$)*(1.1 * $\frac{2}{3}$) =38.1+12.62= **50.72**

M_{yy}=($\frac{76.196+44.9}{2}$)*$\frac{1.1^2}{2}$**= 36.64**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{50.72}{20*2.6*70^2}$ = 0.000199< 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-($\frac{0.000199}{1.134}$))

Z= **0.174**

Z= $\frac{d}{2}$(1+$\sqrt{1-3.53k}$ ≤ 0.665

Z= $\frac{0.7}{2}$(1+$\sqrt{1-3.53(0.000199)}$ ≤ 0.665

Z= 0.699≤ 0.665

Z=**0.665**

A_S= $\frac{M}{fydz}$ = $\frac{50.72}{347.82*0.665}$ =219mm^2

Min & max area of reinforcement

As_{min}= $\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

= $\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm^2**

As_{max}=0.04*Ac =0.04*2.6*0.7

As_{max} =**72800mm^2**

Use As_{min}=2614mm^2

1.Vertical shear

Critical shear at load from face of column

Average pressure critical section

=44.9+($\frac{2.15}{2.6}$)*31.296=70.78

- Design shear force

$V_{€d}$= 70.78*0.35*2.6=64.41KN

$K=1+\sqrt{\frac{200}{d}}= 1+\sqrt{\frac{200}{700}} = 1.535< 2.0$

Note 9bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$ƍ_1=\frac{Asl}{bd}$ =0

V_{RdC}=0

$V_{min}=[0.035K^{3/2}\sqrt{fck}]$bd

$V_{min}=[0.035* 1.532^{3/2}\sqrt{20}]$2600*700

V_{min}= 540.185KN

$V_{€d}<V_{min}$ **OK**

24. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5= 2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2]

=7.895m^2

Average punching shear strees

$V_{€d}=(\frac{76.196+44.9}{2})$[(2.6*2.6)-7.895]= **68** KN

Punching shear stress

$V_{€d}=\frac{BV€d}{Ud}$, B=1+K$(\frac{M€d}{V€d})(\frac{U1}{W1})$

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

$B=1+0.659(\frac{6.05*10^6}{68*10^3})(\frac{9984}{10113660})=1.06$

$V_{€d}=\frac{1.06*68*10^3}{9984*700}$ =0.01N/mm^2

Punching shear resistance

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

$V_{Rdc}=V_{min}=0.035K^{3/2}fck^{1/2}=0.035(1.535)^{3/2}(20)^{1/2}=0.298$

0.298 >$V_{€d}$(0.127) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=409.4103

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ $B=1+K(\frac{M€d}{V€d})(\frac{Uo}{w1})$

W1=0.5*400^2+400*400

W1=240000mm^2

$B=1+0.65(\frac{6.05*10^6}{409.4103*10^3})(\frac{1600}{240000})$

B=1.06

$V_{€d}=(\frac{1.06*409.4103*10^3}{1600*68})=0.4N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

=3.68>$V_{€d}$**OK**

Footing 15

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=1219.595

M_Y=17.22

M_X=5.7593

Area of footing req

$A=\frac{(N+W)}{\tau soil}=\frac{1219.595+121.96}{150}=$ **8.94** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6 **=6.76m^2**

Weight =6.76*0.7*25=118.3KN

$\delta=\frac{P}{A}*(1\pm\frac{6ey}{B}\pm\frac{6eX}{B})$ where ex=$\frac{My}{P}=\frac{17.22}{1219.595}$ =0.014

ey=$\frac{Mx}{P}=\frac{5.7593}{1219.595}$ =0.0047

$\delta_{max}=\frac{1219.595}{6.76}*(1+\frac{6(0.0047)}{2.6}+\frac{6(0.014)}{2.6})$ = 188.17

$\delta_{min}=\frac{1219.595}{6.76}*(1-\frac{6(0.0047)}{2.6}-\frac{6(0.014)}{2.6})$ = 184.2

σmax=125.681<114.47 σ(allowable)=350

Punching shear resistance

M_{xx}=186.49*$\frac{1.1^2}{2}$+(188.17-184.2)($\frac{1.1}{2}$)*(1.1 * $\frac{2}{3}$) =112.8+1.6 = **114.4**

M_{yy}=($\frac{188.17+184.2}{2}$)*$\frac{1.1^2}{2}$= **112.6**

- 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= $\frac{Mxx}{fck*bd^2}=\frac{114.4}{20*2.6*70^2}$ = 0.00045< 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-($\frac{0.00045}{1.134}$))

  Z= **0.175**

  Z= $\frac{d}{2}(1+\sqrt{1-3.53k}\leq 0.665$

  Z= $\frac{0.7}{2}(1+\sqrt{1-3.53(0.00045)}\leq 0.665$

  Z= 0.699≤ 0.665

  Z=**0.665**

  $A_S=\frac{M}{fydz}=\frac{114.4}{347.82*0.665}$ =495 mm^2

  Min & max area of reinforcement

$As_{min}=\frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

$=\frac{0.26*2.21*2.6*0.7}{400}$

As_{min}= **2614mm^2**

As_{max}=0.04*Ac=0.04*2.6*0.7

As_{max} =**72800mm²**

Use As_{min}=2614mm²

25. Vertical shear

Critical shear at load from face of column

Average pressure critical section

=184.2+($\frac{2.15}{2.6}$)*3.97 =187.5

↳ Design shear force

$V_{€d}$= 187.5*0.35*2.6=170.61KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm²

δ_1=$\frac{Asl}{bd}$ =0 ,V_{RdC}=0

V_{min}=[0.035$K^{3/2}\sqrt{fck}$]bd

V_{min}=[0.035∗ $1.532^{3/2}\sqrt{20}$]2600*700

V_{min}= 540.185KN

$V_{€d}$<V_{min} **OK**

26. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5=2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2]=7.895m²

Average punching shear strees

$V_{€d}$=($\frac{188.17+184.2}{2}$)[(2.6*2.6)-7.895]= **211** KN

Punching shear stress

$V_{€d}=\frac{BV€d}{Ud}$, $B=1+K(\frac{M€d}{V€d})(\frac{U1}{W1})$

$W1=0.5c1^2+c1c2+4c2d+16d^2+2\pi dc1$

$=0.5(400)^2+400*400+4*400*667.5+16*667.5^2+2*\pi*667.5*400$

$W1=\underline{10113660mm^2}$

$B=1+0.659(\frac{6.05*10^6}{211*10^3})(\frac{9984}{10113660})=1.019$

$V_{€d}=\frac{1.019*211*10^3}{9984*700}=0.031N/mm^2$

Punching shear resistance

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

$V_{Rdc}=V_{min}=0.035K^{3/2}fck^{1/2}=0.035(1.535)^{3/2}(20)^{1/2}=0.298$

$0.298 > V_{€d}(0.02)$ **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=1219.595

U=2(400+400)=1600

$V_{€d}=(\frac{BV€d}{Uod})$ $B=1+K(\frac{M€d}{V€d})(\frac{Uo}{w1})$

$W1=0.5*400^2+400*400$

$W1=240000mm^2$

$B=1+0.65(\frac{6.05*10^6}{1219.595*10^3})(\frac{1600}{240000})$

$B=1.02$

$V_{€d}=(\frac{1.02*1219.595*10^3}{1600*211})=0.4N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

$=3.68>V_{€d}$ **Ok**

Footing 16

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

P=942.8562

M_Y=2.893

M_X=2.8273

Area of footing req

A=$\frac{(N+W)}{\tau soil}$ =$\frac{942.8562+94.28562}{150}$ = **6.9** where W=10%N

B*H*h =2.6*2.6*0.7

Area =2.6*2.6 **=6.76m²**

Weight =6.76*0.7*25 =118.3KN

δ=$\frac{P}{A}$ *(1 ± $\frac{6ey}{B}$ ± $\frac{6eX}{B}$) where ex=$\frac{My}{P}$ =$\frac{2.893}{942.8562}$ =0.0031

ey=$\frac{Mx}{P}$ =$\frac{2.8273}{942.8562}$ =0.0029

δ_{max}=$\frac{942.8562}{6.76}$ *(1 + $\frac{6(0.0029)}{2.6}$ + $\frac{6(0.0031)}{2.6}$) = 141.4

δ_{min}=$\frac{942.8562}{6.76}$ *(1 − $\frac{6(0.0229)}{2.6}$ − $\frac{6(0.0031)}{2.6}$) = 137.5

σmax=141.4<137.5 σ(allowable)=350

Punching shear resistance

M_{xx}=139.75*$\frac{1.1^2}{2}$+(141.4-137.5)($\frac{1.1}{2}$)*(1.1 ∗ $\frac{2}{3}$)=84+1..573 = **86.123**

M_{yy}=($\frac{141.4+137.5}{2}$)*$\frac{1.1^2}{2}$**= 84.4**

- 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= $\frac{Mxx}{fck*bd^2}$ =$\frac{86.123}{20*2.6*70^2}$ = 0.00034< 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-($\frac{0.00034}{1.134}$))

  Z= **0.175**

  Z= $\frac{d}{2}$(1+$\sqrt{1-3.53k}$ ≤ 0.665

  Z= $\frac{0.7}{2}$(1+$\sqrt{1-3.53(0.000295)}$ ≤ 0.665

  Z= 0.699≤ 0.665

Z=**0.665**

$A_S = \frac{M}{fydz} = \frac{75.13}{347.82*0.665}$ =325 mm^2

Min & max area of reinforcement

$As_{min} = \frac{0.26*fctm*b*d}{fyk}$ where fctm=0.3*$fck^{2/3}$,fck=$\frac{400}{1.15}$ = 347.82

=0.3*$20^{2/3}$ =**2.21**

$= \frac{0.26*2.21*2.6*0.7}{400}$

As_{min}**= 2614mm²**

As_{max}=0.04*Ac =0.04*2.6*0.7

As_{max} =**72800mm²**

Use As_{min}=2614mm^2

27. Vertical shear

Critical shear at load from face of columnAverage pressure critical section

=137.5+($\frac{2.15}{2.6}$)*3.9 =140.7

↳ Design shear force

$V_{€d}$= 140.7*0.35*2.6=128.1KN

K=1+$\sqrt{\frac{200}{d}}$= 1+$\sqrt{\frac{200}{700}}$ = 1.535< 2.0

Note bar extend beyond critical section at 350-35 =315>(bdfd)=36ø+d

36*20+700=1420

Asl =Dmm^2

$ϑ_1$=$\frac{Asl}{bd}$, V_{RdC}=0

V_{min}=[0.035$K^{3/2}\sqrt{fck}$]bd

V_{min}=[0.035∗ $1.532^{3/2}\sqrt{20}$]2600*700

V_{min}= 540.185KN

$V_{€d}$<V_{min} **OK**

28. Punching shear

Critical shear at 2.0d from face of column

Average d=$\frac{700+635}{2}$ = 667.5= 2d=1335

Control perimeter

U=2(400+400)+(2π*1335)=9984mm

Area within perimeter

A=[0.4*0.4]+[2*0.4*1.335]+[2*0.4*1.335]+[π*1.335^2 =7.895m^2

Average punching shear strees

$V_{€d}$=($\frac{117.62+90.7}{2}$)[(2.6*2.6)-7.895]= **118.2** KN

Punching shear stress

$V_{€d}$=$\frac{BV€d}{Ud}$, B=1+K($\frac{M€d}{V€d}$)($\frac{U1}{W1}$)

W1=0.5c1^2+c1c2+4c2d+16d^2+2πdc1

=0.5(400) ^{2}+400*400+4*400*667.5+16*667.5^2 +2*π*667.5*400

W1= <u>10113660mm^2</u>

B=1+0.659($\frac{6.05*10^6}{118.2*10^3}$)($\frac{9984}{10113660}$)=1.03

$V_{€d}$=$\frac{1.03*118.2*10^3}{9984*700}$ =0.017N/mm^2

Punching shear resistance

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

V_{Rdc}=V_{min}=0.035$K^{3/2}fck^{1/2}$ =0.035$(1.535)^{3/2}(20)^{1/2}$ =0.298

0.298 >$V_{€d}$(0.02) **OK**

3. Maximum punching shear at column perimeter

Max punching shear force

Vedmax=942.8562

U=2(400+400)=1600

$V_{€d}$=($\frac{BV€d}{Uod}$) B=1+K($\frac{M€d}{V€d}$)($\frac{Uo}{w1}$)

W1=0.5*400^2+400*400

W1=240000mm^2

B=1+0.65($\frac{6.05*10^6}{942.8562*10^3}$)($\frac{1600}{240000}$)

B=1.037

$V_{€d}=(\frac{1.037*942.8562*10^3}{1600*118.2})=0.5N/mm^2$

Maximum shear resistance

$V_{ed.max}=0.5[0.6(1-\frac{fck}{250})]\frac{fck}{1.5}=0.5[0.6(1-\frac{20}{250})](\frac{20}{1.5})$

$=3.68>V_{€d}$**Ok**

Conclusion

This final year project aims to provide a complete structural analysis and design of G+3 mixed used building. The building is located in Wolkite. In this project design includeswind load, Solid slab and roof, stair case, earthquake, frame analysis by ETABS 2016 software, beam, column andfoundation.

This structural design concerned to ensure economical, safe and adequate design that able to stand safely and function without excessive deflection in having excellent serviceability.

The design is carried out based on building standard code, and published reference book.

Deferent software analysis is also used to simplify and precise design results. Microsoft Excel sheet and ETABS software programs are include.

Generally; the project express full and step wise procedures that provide an excellent structural design of mixed used building

Recommendation

During done this project has brought a great benefit in making all of us fully qualified for the responsibilities we are expected to face ahead us. So, we would like to appreciate all the deeds and efforts that the university and the department of civil engineering has been done.

We hope that the purpose of learning this degree program is to be able to create a qualified civil engineer who builds a better Ethiopia.

The material used during construction should be satisfied the quality of material used in design

The selected site soil investigation should be performed in detail, by using the required lab and field test, because the uncertainty in foundation work is very high.

During construction every stage of construction should be controlled by skilled person due to the reason of construction complexity.

Everything should be performed or done as per design specification.

If any confusion will happen, anyone can check or refer the new Ethiopian building and amend the design.

Reference

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Appendix

Appendix 3A**Governing concrete Cover for first floor slab**

spanel	L_Y/L_X	type	Concrete Cover for Bond/Durability	Concrete Cover for Fire			Governing concrete cover
				One way	two way		
					$L_Y/L_X \leq 1.5$	$1.5 \leq L_Y/L_X \leq 2$	
P-1	1.220	two way	20		20		20
P-2	1.585	two way	20			25	25
P-3	0.131	two way	20		20		20
P-4	1.397	two way	20		20		20
P-5	1.897	two way	20			25	25
P-6	3. 141	One way	20	40			40
P-7	1.009	two way	20		20		20
P-8	4.524	One way	20	40			40
C-1	4.754	One way	20	40			40
C-2	4.617	One way	20	40			40
C-3	4.996	One way	20	40			40
C-4	5.943	One way	20	40			40
C-5	6.375	One way	20	40			40
C-6	2.992	One way	20	40			40
C-7	3.687	One way	20	40			40
C-8	2.188	One way	20	40			40

Appendix 3B**effective depth will be calculated for first floor slab**

panel	Ly (mm)	Lx (mm)	Ly/Lx(mm	Type of p	support cc	K	N	F1	F2	F3	L/d (mm)	eff. Depth (mm)
P1	5572	4561	1.221662	Two way	Interior	1.5	17.71	1.25	1	1	33.20625	137.3537
P2	4561	3937	1.158496	Two way	Exterior	1.3	17.71	1.25	1	1	28.77875	136.8023
P3	5572	5500	1.013091	Two way	Interior	1.5	17.71	1.25	1	1	33.20625	165.6315
P4	5500	3937	1.397003	Two way	Interior	1.5	17.71	1.25	1	1	33.20625	118.562
P5	5500	2900	1.896552	Two way	Interior	1.5	17.71	1.25	1	1	33.20625	87.33296
P6	5500	1751	3.141062	One way	Interior	1.5	17.71	1.25	1	1	33.20625	52.73104
P7	5549	5500	1.008909	Two way	Interior	1.5	17.71	1.25	1	1	33.20625	165.6315
P8	2900	610	4.754098	One way	Exterior	1.3	17.71	1.25	1	1	28.77875	21.1962
C1	9509	2000	4.7545	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174
C2	9561	1421	6.72836	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	45.8498
C3	7100	1421	4.996481	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	45.8498
C4	3909	1600	2.443125	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	51.62539
C5	10200	1600	6.375	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	51.62539
C6	5500	1825	3.013699	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	58.88521
C7	7374	2000	3.687	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174
C8	1751	800	2.18875	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	25.8127

Appendix 3C**dead load calculation for first floor slab**

pannel	materials	Thickness	UnitWeigl	dead load	Total dead load	
P1	**RC Slab**	0.22	25	5.50		
	ff pvc	0.002	16	0.03		
	Cement sc	0.05	23	1.15	7.14	
	Plastering	0.02	23	0.46		
P2	**RC Slab**	0.22	25	5.50		
	ceramic	0.02	21	0.42		
	Cement sc	0.05	23	1.15	10.15	
	Plastering	0.02	23	0.46		
	partion wall			2.623		

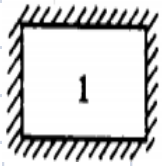
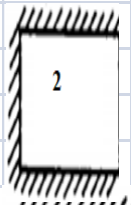
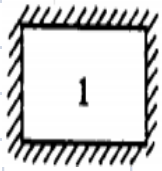
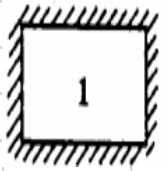
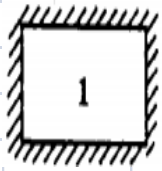
P3	**RC Slab**	0.22	25	5.50		
	ff procelia	0.02	25	0.50		
	Cement sc	0.05	23	1.15	9.43	
	Plastering	0.02	23	0.46		
	partion wall			1.822		
P4	**RC Slab**	0.22	25	5.50		
	ff procelia	0.02	25	0.50		
	Cement sc	0.05	23	1.15	8.87	
	Plastering	0.02	23	0.46		
	partion wall			1.26		
P5	**RC Slab**	0.22	25	5.50		
	ff procelia	0.02	25	0.50		
	Cement sc	0.05	23	1.15	7.61	
	Plastering	0.02	23	0.46		
	partion wall			0.00		
P6	**RC Slab**	0.22	25	5.50		
	ceramic	0.02	21	0.42		
	Cement sc	0.05	23	1.15	9.91	
	Plastering	0.02	23	0.46		
	partion wall			2.376		
P7	**RC Slab**	0.22	25	5.50		
	ff pvc	0.002	16	0.03		
	Cement sc	0.05	23	1.15	8.39	
	Plastering	0.02	23	0.46		
	partion wall			1.2488		
P8	**RC Slab**	0.22	25	5.50		
	ff procelia	0.002	25	0.05		
	Cement sc	0.05	23	1.15	7.16	
	Plastering	0.02	23	0.46		
	partion wall			0.00		
C1	**RC Slab**	0.22	25	5.50		
	ceramic	0.02	21	0.42		
	Cement sc	0.05	23	1.15	11.62	
	Plastering	0.02	23	0.46		
	partion wall			4.093		
C2	**RC Slab**	0.22	25	5.50		
	ceramic	0.02	21	0.42		
	Cement sc	0.05	23	1.15	13.56	
	Plastering	0.02	23	0.46		
	partion wall			6.026		
C3	**RC Slab**	0.22	25	5.50		
	ff procelia	0.002	25	0.05		
	Cement sc	0.05	23	1.15	7.16	
	Plastering	0.02	23	0.46		
	partion wall			0.00		
C4	**RC Slab**	0.22	25	5.50		
	ff procelia	0.002	25	0.05		
	Cement sc	0.05	23	1.15	7.16	
	Plastering	0.02	23	0.46		
	partion wall			0.00		
C5	**RC Slab**	0.22	25	5.50		
	ff procelia	0.002	25	0.05		
	Cement sc	0.05	23	1.15	7.16	
	Plastering	0.02	23	0.46		
	partion wall			0.00		
C6	**RC Slab**	0.22	25	5.50		
	ff pvc	0.002	16	0.03		
	Cement sc	0.05	23	1.15	10.94	
	Plastering	0.02	23	0.46		
	partion wall			3.797		

C7	**RC Slab**	0.22	25	5.50		
	ff pvc	0.002	16	0.03		
	Cement sc	0.05	23	1.15	9.66	
	Plastering	0.02	23	0.46		
	partion wall			2.513		
C8	**RC Slab**	0.22	25	5.50		
	ceramic	0.02	21	0.42		
	Cement sc	0.05	23	1.15	15.70	
	Plastering	0.02	23	0.46		
	partion wall			8.168		

Appendix 3D**design loads for first floor slab**

Panel	Dead Loac	Live load	Design Load(KN/m2
P-1	7.142	5	17.1417
P-2	10.153	2	16.70655
P-3	9.432	5	20.2332
P-4	8.87	5	19.4745
P-5	7.61	5	17.7735
P-6	9.906	2	16.3731
P-7	8.391	5	18.82785
P-8	7.16	5	17.166
C-1	11.623	5	23.19105
C-2	13.556	5	25.8006
C-3	7.16	5	17.166
C-4	7.16	5	17.166
C-5	7.16	4	15.666
C-6	10.939	5	22.26765
C-7	9.655	5	20.53425
C-8	15.698	2	24.1923

Appendix 3E**Moment analysis forthe two way slabs first floor**

panel	panel cnd	Lx		Lx*Lx	design loa	cofficent		moment analysis	
			4.561	20.80272	17.14	αxs=	0.0428	Mxs=	15.26071
P-1						αxf=	0.0326	Mxf=	11.62381
						αys=	0.032	Mys=	11.40988
						αyf=	0.024	Myf=	8.557407
			3.937	15.49997	16.7	αxs=	0.059	Mxs=	15.27212
P-2						αxf=	0.04	Mxf=	10.35398
						αys=	0.039	Mys=	10.09513
						αyf=	0.029	Myf=	7.506635
P-3			5.5	30.25	20.23	αxs=	0.033	Mxs=	20.1946
						αxf=	0.025	Mxf=	15.29894
						αys=	0.032	Mys=	19.58264
						αyf=	0.024	Myf=	14.68698
			3.937	15.49997	19.47	αxs=	0.049	Mxs=	14.78744
P-4						αxf=	0.036	Mxf=	10.86424
						αys=	0.032	Mys=	9.657101
						αyf=	0.024	Myf=	7.242826
			2.9	8.41	17.77	αxs=	0.061	Mxs=	9.116188
						αxf=	0.046	Mxf=	6.874502
P-5						αys=	0.032	Mys=	4.782262
						αyf=	0.024	Myf=	3.586697
			5.5	30.25	18.83	αxs=	0.0324	Mxs=	18.45528
P-7						αxf=	0.024	Mxf=	13.67058
						αys=	0.032	Mys=	18.22744
						αyf=	0.024	Myf=	13.67058

Appendix 3F **Support moment adjustment for first floor slab**

Pannel	support moment		Length		valueof K					
	Mxs(max)	Mxs(min)	L_1	L_2	K_1	K_2	Mxs(max)-Mxs(min)	average	stifness method	
p1and p2	15.27	15.26	5.572	3.937	0.18	0.25	0.01	15.27		
p1and p3	20.19	15.26	4.561	5.5	0.22	0.18	4.93		17.49	
p2and p4	15.27	14.78	5.5	4.561	0.18	0.22	0.49	15.03		
p3and p4	20.19	14.78	5.572	3.937	0.18	0.25	5.41		17.95	
p4and p5	14.78	9.12	3.937	2.9	0.25	0.34	5.66		12.38	
p5and p6	9.12	4.18	2.9	1.75	0.34	0.57	4.94		7.26	
p6and p7	18.45	4.18	1.75	5.549	0.57	0.18	14.27		7.60	
p5and p8	9.12	0.46	5.5	0.46	0.18	2.17	8.66		8.45	
p2and p8	15.27	0.46	5.5	0.46	0.18	2.17	14.81		14.13	

Appendix 3G span **moment adjustment for first floor slab**

pannel	unadjusted moment		adjusted moment		ΔMxs	ΔMys	previous moment		Ly/Lx	cofficeints				ΔMxf	ΔMyf	Mxfd	Myfd
	Mxs	Mys	Mxs	Mys			Mxf	Myf		Cx1	Cy1	Cx2	Cy2				
p1	15.26	11.41	15.265	46.38	-0.01	-34.97	11.62	8.56	1.22	0.335	0.165	0.349	0.361	-12.21	-12.62	0.586	4.065
p2	15.27	10.09	46.38	15.26	-31.11	-5.17	10.35	7.51	1.158	0.346	0.192	0.331	0.368	-12.48	-7.88	2.125	0.365
p3	20.19	19.58	20.19	17.49	0.00	2.09	12.29	14.68	1.013	0.376	0.272	0.284	0.379	0.59	0.79	12.88	15.47
p4	14.78	9.65	15.025	17.95	-0.25	-8.30	10.86	7.24	1.397	0.315	0.111	0.397	0.332	-3.37	-2.78	7.49	4.46
p5	9.12	4.78	8.45	12.38	0.67	-7.60	6.87	3.58	1.896	0.259	0.06	0.481	0.251	-3.48	-1.87	3.39	1.71
p7	18.22	18.22	14.27	41.06	3.95	-22.84	13.67	13.67	1	0.38	0.28	0.28	0.38	-4.89	-7.57	8.78	6.10

Appendix 3H**Reinforcement design for first floor slab**

Panel	Adjusted Mom	Φ	D	Km	Ks	Ascalc	Asmin	As require	as	Spacing	St provide	Spacing
P-1	Mxf=0.58	10	175	4.35	2.96	9.81	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=46.38	10	175	38.92	3.11	824.24	250.25	824.24	78.5	95	95	Φ10 c/c 95
	Myf=4.065	10	165	12.22	2.96	72.92	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=26.05	10	165	30.93	3.04	536.79	235.95	536.79	78.5	140	140	Φ10 c/c 140
P2	Mxf=2.125	10	175	8.32	2.96	35.94	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=46.38	10	175	38.92	3.11	824.24	250.25	824.24	78.5	95	95	Φ10 c/c 95
	Myf=0.365	10	165	3.66	2.96	6.55	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=15.265	10	165	23.68	2.98	275.69	235.95	275.69	78.5	280	280	Φ10 c/c 280
P3	Mxf=12.88	10	175	20.51	2.97	218.59	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=21.97	10	175	26.78	3	376.63	250.25	376.63	78.5	200	200	Φ10 c/c 200
	Myf=15.47	10	165	23.84	2.98	279.39	235.95	279.39	78.5	280	280	Φ10 c/c 280
	Mys=20.19	10	165	27.23	3.01	368.31	235.95	368.31	78.5	210	210	Φ10 c/c 210
P4	Mxf=7.49	10	175	15.64	2.96	126.69	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=21.97	10	175	26.78	3	376.63	250.25	376.63	78.5	200	200	Φ10 c/c 200
	Myf=4.46	10	165	12.79	2.96	80	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=17.95	10	165	25.68	2.99	325.28	235.95	325.28	78.5	240	240	Φ10 c/c 240
P5	Mxf=3.39	10	175	10.52	2.96	57.34	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=20.05	10	175	25.59	2.99	342.57	250.25	342.57	78.5	220	220	Φ10 c/c 220
	Myf=1.71	10	165	7.93	2.96	30.68	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=12.38	10	165	21.32	2.97	222.84	235.95	235.95	78.5	330	330	Φ10 c/c 330

p6	Mxf=2.09	10	175	8.26	2.96	35.35	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=20.05	10	175	25.59	2.99	342.57	250.25	342.57	78.5	220	220	Φ10 c/c 220
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=7.60	10	165	16.71	2.96	136.34	235.95	235.95	78.5	330	330	Φ10 c/c 330
p7	Mxf=8.78	10	175	16.93	2.96	148.51	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=41.06	10	175	36.62	3.08	722.66	250.25	722.66	78.5	100	100	Φ10 c/c 100
	Myf=6.09	10	165	14.95	2.96	109.25	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=37.06	10	165	36.89	3.08	691.79	235.95	691.79	78.5	110	110	Φ10 c/c 110
p8	Mxf=0.88	10	175	5.36	2.96	14.88	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=8.45	10	175	16.61	2.96	142.93	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=14.13	10	165	22.78	2.97	254.34	235.95	254.34	78.5	300	300	Φ10 c/c 300
C1	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=46.38	10	175	38.92	3.11	824.34	250.25	824.34	78.5	95	95	Φ10 c/c 95
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C2	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=26.05	10	165	30.93	3.04	479.95	235.95	479.95	78.5	160	160	Φ10 c/c 160
C3	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=20.19	10	165	27.23	3.01	368.31	235.95	368.31	78.5	210	210	Φ10 c/c 210
C4	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=21.97	10	175	26.78	3	376.63	250.25	376.63	78.5	200	200	Φ10 c/c 200
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C5	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=20.05	10	175	25.59	2.99	342.57	250.25	342.57	78.5	220	220	Φ10 c/c 220
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C6	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=37.08	10	165	36.91	3.08	692.16	235.95	692.16	78.5	110	110	Φ10 c/c 110
C7	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=41.06	10	175	36.62	3.08	722.66	250.25	722.66	78.5	100	100	Φ10 c/c 100
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C8	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=7.74	10	175	15.89	2.96	130.92	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330

Appendix 3I Depth check of serviceability for first floor

Panel	D	b	St provi	As provi	ρo	ρ	Criteria	As requ	F1	K	N	Lx	F2&F3	lx/d	d	depth check
P-1	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.82	4561	1	88.08	51.78	OK
	175	1000	95	1842.11	0.4472	1.12	ρ>ρo	824.24	2.79	1.5	12.15	4561	1	50.91	89.60	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	4561	1	86.11	52.97	OK
	165	1000	140	1178.57	0.4472	0.67	ρ>ρo	536.79	2.74	1.5	15.19	4561	1	62.52	72.95	OK
P2	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	3937	1	87.53	44.98	OK
	175	1000	95	1842.11	0.4472	1.12	ρ>ρo	824.24	2.79	1.5	12.15	3937	1	50.91	77.34	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	3937	1	86.11	45.72	OK
	165	1000	280	589.29	0.4472	0.34	ρ<=ρo	275.69	2.67	1.5	29.16	3937	1	116.87	33.69	OK
P3	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	5500	1	87.53	62.83	OK
	175	1000	200	875.00	0.4472	0.53	ρ>ρo	376.63	2.90	1.5	16.68	5500	1	72.66	75.70	OK
	165	1000	280	589.29	0.4472	0.36	ρ<=ρo	279.39	2.64	1.5	18.37	5500	1	72.63	75.72	OK
	165	1000	210	785.71	0.4472	0.45	ρ<=ρo	368.31	2.67	1.5	17.68	5500	1	70.73	77.76	OK
C6	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	1825	1	87.53	20.85	OK
	175	1000	310	564.52	0.4472	0.34	ρ<=ρo	250.25	2.82	1.5	17.53	1825	1	74.16	24.61	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	1825	1	86.11	21.19	OK
	165	1000	110	1500.00	0.4472	0.86	ρ>ρo	692.16	2.71	1.5	13.72	1825	1	55.74	32.74	OK
C7	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	2000	1	87.53	22.85	OK
	175	1000	100	1750.00	0.4472	1.06	ρ>ρo	722.66	3.03	1.5	16.32	2000	1	74.12	26.98	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	2000	1	86.11	23.23	OK
	165	1000	330	500.00	0.4472	0.29	ρ<=ρo	235.95	2.65	1.5	22.79	2000	1	90.56	22.08	OK
C8	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	800	1	87.53	9.14	OK
	175	1000	310	564.52	0.4472	0.34	ρ<=ρo	250.25	2.82	1.5	17.53	800	1	74.16	10.79	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	800	1	86.11	9.29	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	800	1	86.11	9.29	OK

P4	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	3937	1	87.53	44.98	OK
	175	1000	200	875.00	0.4472	0.53	ρ>ρo	376.63	2.90	1.5	16.68	3937	1	72.66	54.18	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	3937	1	86.11	45.72	OK
	165	1000	240	687.50	0.4472	0.39	ρ<=ρo	325.28	2.64	1.5	18.65	3937	1	73.92	53.26	OK
P5	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	2900	1	87.53	33.13	OK
	175	1000	220	795.45	0.4472	0.48	ρ>ρo	342.57	2.90	1.5	17.22	2900	1	74.97	38.68	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	2900	1	86.11	33.68	OK
	165	1000	330	500.00	0.4472	0.29	ρ<=ρo	235.95	2.65	1.5	22.79	2900	1	90.56	32.02	OK
p6	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	1751	1	87.53	20.00	OK
	175	1000	220	795.45	0.4472	0.48	ρ>ρo	342.57	2.90	1.5	17.22	1751	1	74.97	23.36	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	1751	1	86.11	20.34	OK
	165	1000	330	500.00	0.4472	0.29	ρ<=ρo	235.95	2.65	1.5	22.79	1751	1	90.56	19.33	OK
p7	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	5500	1	87.53	62.83	OK
	175	1000	100	1750.00	0.4472	1.06	ρ>ρo	722.66	3.03	1.5	16.32	5500	1	74.12	74.20	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	5500	1	86.11	63.88	OK
	165	1000	110	1500.00	0.4472	0.86	ρ>ρo	691.79	2.71	1.5	13.72	5500	1	55.77	98.62	OK
p8	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	641	1	87.53	7.32	OK
	175	1000	310	564.52	0.4472	0.34	ρ<=ρo	250.25	2.82	1.5	17.53	641	1	74.16	8.64	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	641	1	86.11	7.44	OK
	165	1000	300	550.00	0.4472	0.31	ρ<=ρo	254.34	2.70	1.5	21.08	641	1	85.49	7.50	OK
C1	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	2000	1	87.53	22.85	OK
	175	1000	95	1842.11	0.4472	1.12	ρ>ρo	824.34	2.79	1.5	12.15	2000	1	50.90	39.29	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	2000	1	86.11	23.23	OK
	165	1000	330	500.00	0.4472	0.29	ρ<=ρo	235.95	2.65	1.5	22.79	2000	1	90.56	22.08	OK
C2	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	1421	1	87.53	16.23	OK
	175	1000	310	564.52	0.4472	0.34	ρ<=ρo	250.25	2.82	1.5	17.53	1421	1	74.16	19.16	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	1421	1	86.11	16.50	OK
	165	1000	160	1031.25	0.4472	0.59	ρ>ρo	479.95	2.69	1.5	15.99	1421	1	64.42	22.06	OK
C3	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	1421	1	87.53	16.23	OK
	175	1000	310	564.52	0.4472	0.34	ρ<=ρo	250.25	2.82	1.5	17.53	1421	1	74.16	19.16	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	1421	1	86.11	16.50	OK
	165	1000	210	785.71	0.4472	0.45	ρ<=ρo	368.31	2.67	1.5	17.68	1421	1	70.73	20.09	OK
C4	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	1600	1	87.53	18.28	OK
	175	1000	200	875.00	0.4472	0.53	ρ>ρo	376.63	2.90	1.5	16.68	1600	1	72.66	22.02	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	1600	1	86.11	18.58	OK
	165	1000	330	500.00	0.4472	0.29	ρ<=ρo	235.95	2.65	1.5	22.79	1600	1	90.56	17.67	OK
C5	175	1000	310	564.52	0.4472	0.32	ρ<=ρo	250.25	2.82	1.5	20.7	1600	1	87.53	18.28	OK
	175	1000	220	795.45	0.4472	0.48	ρ>ρo	342.57	2.90	1.5	17.22	1600	1	74.97	21.34	OK
	165	1000	330	500.00	0.4472	0.30	ρ<=ρo	235.95	2.65	1.5	21.67	1600	1	86.11	18.58	OK
	165	1000	330	500.00	0.4472	0.29	ρ<=ρo	235.95	2.65	1.5	22.79	1600	1	90.56	17.67	OK

Appendix 3J Load **Transferring Mechanism for first floor slab**

Panel	Lx	GK	QK	Coefficient		Shear due to dead		Shear due to live		Design shear	
P-1	4.561	7.142	5	βvx,c=	0.39	Vx,c=	12.70	Vx,c=	8.89	30.49	
	4.561	7.142	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00	
	4.561	7.142	5	βvy,c=	0.33	Vy,c=	10.75	Vy,c=	7.53	25.80	
	4.561	7.142	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00	
P2	3.937	10.153	2	βvx,c=	0.41	Vx,c=	16.39	Vx,c=	3.23	26.97	
	3.937	10.153	2	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00	
	3.937	10.153	2	βvy,c=	0.36	Vy,c=	14.39	Vy,c=	2.83	23.68	
	3.937	10.153	2	βvy,d=	0.24	Vy,d=	9.59	Vy,d=	1.89	15.79	
P3	5.5	9.432	5	βvx,c=	0.33	Vx,c=	17.12	Vx,c=	9.08	36.72	
	5.5	9.432	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00	
	5.5	9.432	5	βvy,c=	0.33	Vy,c=	17.12	Vy,c=	9.08	36.72	
	5.5	9.432	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00	
P4	3.937	8.87	5	βvx,c=	0.42	Vx,c=	14.67	Vx,c=	8.27	32.20	
	3.937	8.87	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00	
	3.937	8.87	5	βvy,c=	0.33	Vy,c=	11.52	Vy,c=	6.50	25.30	
	3.937	8.87	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00	
P5	2.9	7.61	5	βvx,c=	0.49	Vx,c=	10.81	Vx,c=	7.11	25.26	
	2.9	7.61	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00	
	2.9	7.61	5	βvy,c=	0.33	Vy,c=	7.28	Vy,c=	4.79	17.01	
	2.9	7.61	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00	

p6	1.751	9.906	2	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	1.751	9.906	2	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	1.751	9.906	2	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	1.751	9.906	2	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
p7	5.5	8.391	5	βvx,c=	0.33	Vx,c=	15.23	Vx,c=	9.08	34.17
	5.5	8.391	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	5.5	8.391	5	βvy,c=	0.33	Vy,c=	15.23	Vy,c=	9.08	34.17
	5.5	8.391	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
p8	0.641	7.16	5	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	0.641	7.16	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	0.641	7.16	5	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	0.641	7.16	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
C1	2	11.623	5	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	2	11.623	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	2	11.623	5	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	2	11.623	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
C2	1.421	13.556	5	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	1.421	13.556	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	1.421	13.556	5	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	1.421	13.556	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
C3	1.421	7.16	5	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	1.421	7.16	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	1.421	7.16	5	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	1.421	7.16	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
C4	1.6	7.16	5	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	1.6	7.16	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	1.6	7.16	5	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	1.6	7.16	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
C5	1.6	7.16	4	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	1.6	7.16	4	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	1.6	7.16	4	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	1.6	7.16	4	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
C6	1.825	10.939	5	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	1.825	10.939	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	1.825	10.939	5	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	1.825	10.939	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
C7	2	9.655	5	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	2	9.655	5	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	2	9.655	5	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	2	9.655	5	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00
C8	0.8	9.655	2	βvx,c=	0	Vx,c=	0.00	Vx,c=	0.00	0.00
	0.8	9.655	2	βvx,d=	0	Vx,d=	0.00	Vx,d=	0.00	0.00
	0.8	9.655	2	βvy,c=	0	Vy,c=	0.00	Vy,c=	0.00	0.00
	0.8	9.655	2	βvy,d=	0	Vy,d=	0.00	Vy,d=	0.00	0.00

Appendix 3K Check **Shear resistance of the section for first floor slab**

Panel	Design sh	D	K1	CRD,C	Vmin	σcp	fck	$\rho 1$	$VRd,c(m$	VRd,c	check for	Asprovide
P-1	30.49	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	824.24
	25.8	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	536.79
P2	26.97	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	824.24
	23.68	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	15.79	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	275.69
C6	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	692.16
C7	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	722.66
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
C8	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95

P3	36.72	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	376.63
	36.72	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	279.39
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	368.31
P4	32.2	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	376.63
	25.3	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	325.28
P5	25.26	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	342.57
	17.01	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
p6	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	342.57
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
p7	34.17	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	722.66
	34.17	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	691.79
p8	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	254.34
C1	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	824.34
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
C2	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	479.95
C3	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	368.31
C4	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	376.63
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
C5	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.53	77.5656	SAFE	342.57
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.10	73.13328	SAFE	235.95

Appendix 3L**effective Depth Determination for second and thrid floor slab**

pannel	Ly(m)	Lx(m)	Ly/Lx	type	suppor	K	N	F1	F2	F3	L/d(mn	eff/ de	(m)
P1	5572	4561	1.221662	two way	Interior	1.5	17.71	1.25	1	1	33.20625	137.3537	
P2	4561	3937	1.158496	two way	Exterior	1.3	17.71	1.25	1	1	28.77875	136.8023	
P3	5572	5500	1.013091	two way	Interior	1.5	17.71	1.25	1	1	33.20625	165.6315	
P4	5500	3937	1.397003	two way	Interior	1.5	17.71	1.25	1	1	33.20625	118.562	
P5	5500	2900	1.896552	two way	Interior	1.5	17.71	1.25	1	1	33.20625	87.33296	
P6	5500	1751	3.141062	One way	Interior	1.5	17.71	1.25	1	1	33.20625	52.73104	
P7	5549	5500	1.008909	two way	Interior	1.5	17.71	1.25	1	1	33.20625	165.6315	
P8	2900	610	4.754098	One way	Exterior	1.3	17.71	1.25	1	1	28.77875	21.1962	
C1	9509	2000	4.7545	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174	
C2	9561	1421	6.72836	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	45.8498	
C3	7100	1421	4.996481	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	45.8498	
C4	3909	2000	1.9545	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174	
C5	1020	2000	0.51	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174	
C6	5500	1825	3.013699	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	58.88521	
C7	7374	2000	3.687	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174	
C8	1751	600	2.918333	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	19.35952	

Appendix 3Mdead load for second third floor slab

pannel	materials	Thickness	UnitWeigh	dead load	Total dead load	
P1	**RC Slab**	0.22	25	5.5		
	ff pvc	0.002	16	0.032		
	Cement sc	0.05	23	1.15		
	Plastering	0.02	23	0.46	11.475	
	partion wall			4.333		
P2	**RC Slab**	0.22	25	5.5		
	ff pvc	0.002	16	0.032		
	Cement sc	0.05	23	1.15	7.142	
	Plastering	0.02	23	0.46		
	partion wall			0		
P3	**RC Slab**	0.22	25	5.5		
	ff pvc	0.002	16	0.032		
	Cement sc	0.05	23	1.15	12.933	
	Plastering	0.02	23	0.46		
	partion wall			5.791		
P4	**RC Slab**	0.22	25	5.5		
	ff pvc	0.002	16	0.032		
	Cement sc	0.05	23	1.15	11.635	
	Plastering	0.02	23	0.46		
	partion wall			4.493		
P5	**RC Slab**	0.22	25	5.5		
	ff pvc	0.002	16	0.032		
	Cement sc	0.05	23	1.15	11.51	
	Plastering	0.02	23	0.46		
	partion wall			4.368		
P6	**RC Slab**	0.22	25	5.5		
	ff procelia	0.002	25	0.05		
	Cement sc	0.05	23	1.15	7.16	
	Plastering	0.02	23	0.46		
	partion wall			0		

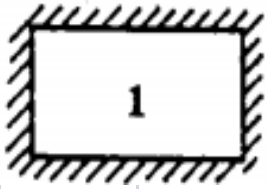
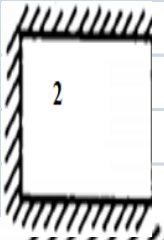
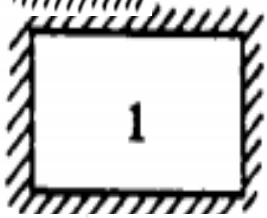
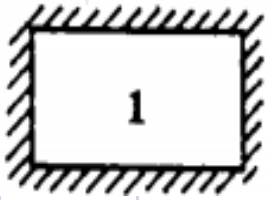
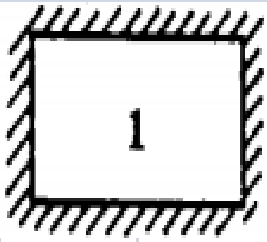
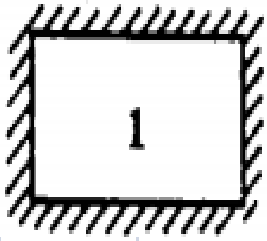
P7	**RC Slab**	0.22	25	5.5		
	ff pvc	0.002	16	0.032		
	Cement sc	0.05	23	1.15	14.079	
	Plastering	0.02	23	0.46		
	partion wall			6.937		
P8	**RC Slab**	0.22	25	5.5		
	ff procelia	0.002	25	0.05		
	Cement sc	0.05	23	1.15	7.16	
	Plastering	0.02	23	0.46		
	partion wall			0		
C1	**RC Slab**	0.22	25	5.5		
	ceramic	0.02	21	0.42		
	Cement sc	0.05	23	1.15	13.849	
	Plastering	0.02	23	0.46		
	partion wall			6.319		
C2	**RC Slab**	0.22	25	5.5		
	ff pvc	0.002	16	0.032		
	Cement sc	0.05	23	1.15	12.93	
	Plastering	0.02	23	0.46		
	partion wall			5.788		
C3	**RC Slab**	0.22	25	5.5		
	ff pvc	0.002	16	0.032		
	Cement sc	0.05	23	1.15	14.871	
	Plastering	0.02	23	0.46		
	partion wall			7.729		
C4	**RC Slab**	0.22	25	5.5		
	ceramic	0.02	21	0.42		
	Cement sc	0.05	23	1.15	16.597	
	Plastering	0.02	23	0.46		
	partion wall			9.067		
C5	**RC Slab**	0.22	25	5.5		
	ceramic	0.02	21	0.42		
	Cement sc	0.05	23	1.15	15.492	
	Plastering	0.02	23	0.46		
	partion wall			7.962		
C6	**RC Slab**	0.22	25	5.5		
	ceramic	0.02	21	0.42		
	Cement sc	0.05	23	1.15	16.265	
	Plastering	0.02	23	0.46		
	partion wall			8.735		

C7	**RC Slab**	0.22	25	5.5		
	ff pvc	0.002	16	0.032		
	Cement sc	0.05	23	1.15	11.798	
	Plastering	0.02	23	0.46		
	partion wall			4.656		
C8	**RC Slab**	0.22	25	5.5		
	ff procelia	0.002	25	0.05		
	Cement sc	0.05	23	1.15	26.312	
	Plastering	0.02	23	0.46		
	partion wall			19.152		

Appendix 3N Pd **= 1.35DL + 1.5LL for second third floor slab**

pannel	Dead Loac	Live load	Design Load(KN/m2	
P1	11.5	2	18.525	
P2	7.14	2	12.639	
P3	12.9	2	20.415	
P4	11.6	2	18.66	
P5	11.5	2	18.525	
P6	7.16	5	17.166	
P7	14.1	2	22.035	
P8	7.16	5	17.166	
C1	13.8	2	21.63	
C2	12.9	4	23.415	
C3	14.9	4	26.115	
C4	16.6	4	28.41	
C5	15.5	4	26.925	
C6	16.3	2	25.005	
C7	11.8	4	21.93	
C8	26.3	5	43.005	

Appendix 30**moment analysis for two way slabs for second and thrid floor slab**

panel	panel cnd	Lx	Lx*Lx	design loa	cofficent		moment analysis	
			0.80272	18.53	αxs=	0.0428	Mxs=	16.49831
P-1					αxf=	0.0326	Mxf=	12.56647
					αys=	0.032	Mys=	12.33518
					αyf=	0.024	Myf=	9.251386
		3.937	15.49997	12.64	αxs=	0.059	Mxs=	11.55926
P-2					αxf=	0.04	Mxf=	7.836784
					αys=	0.039	Mys=	7.640865
					αyf=	0.029	Myf=	5.681669
P-3			30.25	20.42	αxs=	0.033	Mxs=	20.38427
					αxf=	0.025	Mxf=	15.44263
					αys=	0.032	Mys=	19.76656
					αyf=	0.024	Myf=	14.82492
			5.49997	18.66	αxs=	0.049	Mxs=	14.17224
P-4					αxf=	0.036	Mxf=	10.41226
					αys=	0.032	Mys=	9.255341
					αyf=	0.024	Myf=	6.941506
			8.41	18.53	αxs=	0.061	Mxs=	9.506075
					αxf=	0.046	Mxf=	7.168516
P-5					αys=	0.032	Mys=	4.986794
					αyf=	0.024	Myf=	3.740095
			30.25	22.04	αxs=	0.0324	Mxs=	21.6014
P-7					αxf=	0.024	Mxf=	16.00104
					αys=	0.032	Mys=	21.33472
					αyf=	0.024	Myf=	16.00104

Appendix 3p support **moment adjustment for second and third floor slab**

Pannel	support moment		Length		valueof K							
	Mxs(max)	Mxs(min)	L_1	L_2	K_1	K_2	Mxs(max)-Mxs(min)		average	stifness method		
p1and p2	16.49	11.56	5.572	3.937	0.179469	0.254001	4.93			14.44884		
p1and p3	20.38	16.49	4.561	5.5	0.21925	0.181818	3.89		18.435			
p2and p4	14.17	11.56	5.5	4.561	0.181818	0.21925	2.61		12.865			
p3and p4	20.38	14.17	5.572	3.937	0.179469	0.254001	6.21			17.80888		
p4and p5	14.17	9.51	3.937	2.9	0.254001	0.344828	4.66			12.1934		
p5and p6	9.51	4.39	2.9	1.75	0.344828	0.571429	5.12			7.583118		
p6and p7	21.6	4.39	1.75	5.549	0.571429	0.180213	17.21			8.51625		
p5and p8	9.51	0.49	5.5	0.46	0.181818	2.173913	9.02			8.813826		
p2and p8	11.56	0.49	5.5	0.46	0.181818	2.173913	11.07			10.7056		

Appendix 3Q**Span moment adjustment**

pannel	unadjusted moment		adjusted moment		ΔMxs	ΔMys	previous moment		Ly/Lx	cofficeints				ΔMxf	ΔMyf	Mxfd	Myfd
	Mxs	Mys	Mxs	Mys			Mxf	Myf		Cx1	Cy1	Cx2	Cy2				
p1	16.498	12.335	14.445	43.26	2.053	-30.925	11.62	8.56	1.22	0.335	0.165	0.349	0.361	-10.1051	-10.8252	1.51493	2.265
p2	11.559	7.641	43.26	14.445	-31.701	-6.804	10.35	7.51	1.158	0.346	0.192	0.331	0.368	-13.2207	-8.59046	2.871	1.08
p3	20.384	19.766	26.36	18.435	-5.976	1.331	12.29	14.68	1.013	0.376	0.272	0.284	0.379	-1.86897	-1.12102	10.42103	13.55898
p4	14.172	9.255	17.81	12.865	-3.638	-3.61	10.86	7.24	1.397	0.315	0.111	0.397	0.332	-2.57914	-1.60234	8.28086	5.637662
p5	9.51	4.987	12.583	8.813	-3.073	-3.826	6.87	3.58	1.896	0.259	0.06	0.481	0.251	-2.63621	-1.14471	4.233787	2.435294
p7	21.601	21.335	41.63	41.06	-20.029	-19.725	13.67	13.67	1	0.38	0.28	0.28	0.38	-13.134	-13.1036	0.53598	0.56638

Appendix 3R**Reinforcement design for second and third floor slab**

Panel	Adjusted I	Φ	D	Km	Ks	Ascalc	Asmin	As require	as	Spacing	St provide	Spacing
P-1	Mxf=1.52	10	175	7.05	2.96	25.71	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=43.26	10	175	37.58	3.1	766.32	250.25	766.32	78.5	100	100	Φ10 c/c 100
	Myf=2.27	10	165	9.13	2.96	40.72	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=23.65	10	165	29.47	3.03	434.3	235.95	434.3	78.5	180	180	Φ10 c/c 180
P2	Mxf=2.87	10	175	9.68	2.96	48.54	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=43.26	10	175	37.58	3.1	766.32	250.25	766.32	78.5	100	100	Φ10 c/c 100
	Myf=1.08	10	165	6.29	2.96	19.37	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=14.45	10	165	23.04	2.98	260.98	235.95	260.98	78.5	300	300	Φ10 c/c 300
P3	Mxf=10.42	10	175	18.45	2.96	176.25	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=37.85	10	175	35.16	3.07	663.99	250.25	663.99	78.5	110	110	Φ10 c/c 110
	Myf=13.56	10	165	22.32	2.97	244.08	235.95	244.08	78.5	320	320	Φ10 c/c 320
	Mys=26.36	10	165	31.11	3.05	487.26	235.95	487.26	78.5	160	160	Φ10 c/c 160

P4	Mxf=8.28	10	175	16.44	2.96	140.05	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=37.85	10	175	35.16	3.07	663.99	250.25	663.99	78.5	110	110	Φ10 c/c 110
	Myf=5.64	10	165	14.39	2.96	101.18	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=17.81	10	165	25.58	2.99	322.74	235.95	322.74	78.5	240	240	Φ10 c/c 240
P5	Mxf=4.23	10	175	11.75	2.96	71.55	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=36.31	10	175	34.43	3.07	636.98	250.25	636.98	78.5	120	120	Φ10 c/c 120
	Myf=2.44	10	165	9.46	2.96	43.77	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=12.19	10	165	21.16	2.97	219.42	235.95	235.95	78.5	330	330	Φ10 c/c 330
p6	Mxf=2.19	10	175	8.46	2.96	37.04	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=36.31	10	175	34.43	3.07	636.98	250.25	636.98	78.5	120	120	Φ10 c/c 120
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=8.52	10	165	17.69	2.96	152.84	235.95	235.95	78.5	330	330	Φ10 c/c 330
p7	Mxf=0.54	10	175	4.19	2.96	9.13	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=43.86	10	175	37.84	3.1	776.95	250.25	776.95	78.5	100	100	Φ10 c/c 100
	Myf=0.57	10	165	4.57	2.96	10.23	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=41.63	10	165	39.1	3.12	787.19	235.95	787.19	78.5	99	99	Φ10 c/c 99
p8	Mxf=0.88	10	175	5.36	2.96	14.88	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=8.81	10	175	16.96	2.96	149.01	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=10.71	10	165	19.83	2.96	192.13	235.95	235.95	78.5	330	330	Φ10 c/c 330
C1	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=43.26	10	175	37.58	3.1	766.32	250.25	766.32	78.5	100	100	Φ10 c/c 100
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C2	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=23.65	10	165	19.47	2.96	424.27	235.95	424.27	78.5	180	180	Φ10 c/c 180
C3	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=26.36	10	165	31.11	3.05	487.26	235.95	487.26	78.5	160	160	Φ10 c/c 160
C4	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=37.85	10	175	35.16	3.07	663.99	250.25	663.99	78.5	110	110	Φ10 c/c 110
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C5	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=36.31	10	175	34.43	3.07	636.98	250.25	636.98	78.5	330	330	Φ10 c/c 330
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330

C6	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=41.63	10	165	39.1	3.08	777.09	235.95	777.09	78.5	100	100	Φ10 c/c 100
C7	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=43.86	10	175	37.84	3.1	776.95	250.25	776.95	78.5	100	100	Φ10 c/c 100
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C8	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=13.76	10	175	21.19	2.97	233.53	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330

Appendix 3S**Depth check for serviceability**

Panel	D	b	St provi	As provid	ρo	ρ	Criteria	As req	F1	K	N	Lx	F2&F3	lx/d	d	depth check
P-1	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	4561	1	87.6	52.07	OK
	175	1000	100	1750	0.447	1.061	ρ>ρo	766.3	2.9	1.5	12.444	4561	1	53.28	85.6	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	4561	1	86.11	52.97	OK
	165	1000	180	916.6667	0.447	0.524	ρ>ρo	434.3	2.6	1.5	16.704	4561	1	66.11	68.99	OK
P2	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	3937	1	87.6	44.94	OK
	175	1000	100	1750	0.447	1.061	ρ>ρo	766.3	2.9	1.5	12.444	3937	1	53.28	73.89	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	3937	1	86.11	45.72	OK
	165	1000	300	550	0.447	0.314	ρ<=ρo	261	2.6	1.5	21.086	3937	1	83.32	47.25	OK
P3	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	5500	1	87.6	62.79	OK
	175	1000	110	1590.909	0.447	0.964	ρ>ρo	664	3	1.5	13.007	5500	1	58.43	94.12	OK
	165	1000	320	515.625	0.447	0.313	ρ<=ρo	244.1	2.6	1.5	22.433	5500	1	88.86	61.9	OK
	165	1000	160	1031.25	0.447	0.589	ρ>ρo	487.3	2.6	1.5	15.991	5500	1	63.46	86.67	OK

P4	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	3937	1	87.6	44.94	OK
	175	1000	110	1590.909	0.447	0.964	ρ>ρo	664	3	1.5	13.007	3937	1	58.43	67.38	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	3937	1	86.11	45.72	OK
	165	1000	240	687.5	0.447	0.393	ρ<=ρo	322.7	2.7	1.5	18.654	3937	1	74.51	52.84	OK
P5	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	2900	1	87.6	33.1	OK
	175	1000	120	1458.333	0.447	0.884	ρ>ρo	637	2.9	1.5	13.531	2900	1	58.08	49.93	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	2900	1	86.11	33.68	OK
	165	1000	330	500	0.447	0.286	ρ<=ρo	236	2.6	1.5	22.793	2900	1	90.56	32.02	OK
p6	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	1751	1	87.6	19.99	OK
	175	1000	120	1458.333	0.447	0.884	ρ>ρo	637	2.9	1.5	13.531	1751	1	58.08	30.15	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	1751	1	86.11	20.34	OK
	165	1000	330	500	0.447	0.286	ρ<=ρo	236	2.6	1.5	22.793	1751	1	90.56	19.33	OK
p7	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	5500	1	87.6	62.79	OK
	175	1000	100	1750	0.447	1.061	ρ>ρo	777	2.8	1.5	12.444	5500	1	52.55	104.7	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	5500	1	86.11	63.88	OK
	165	1000	99	1666.667	0.447	0.952	ρ>ρo	787.2	2.6	1.5	13.082	5500	1	51.93	105.9	OK
p8	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	641	1	87.6	7.317	OK
	175	1000	310	564.5161	0.447	0.342	ρ<=ρo	250.3	2.8	1.5	19.976	641	1	84.49	7.587	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	641	1	86.11	7.444	OK
	165	1000	330	500	0.447	0.286	ρ<=ρo	236	2.6	1.5	22.793	641	1	90.56	7.078	OK
C1	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	2000	1	87.6	22.83	OK
	175	1000	100	1750	0.447	1.061	ρ>ρo	766.3	2.9	1.5	12.444	2000	1	53.28	37.54	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	2000	1	86.11	23.23	OK
	165	1000	330	500	0.447	0.286	ρ<=ρo	236	2.6	1.5	22.793	2000	1	90.56	22.08	OK
C2	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	1421	1	87.6	16.22	OK
	175	1000	310	564.5161	0.447	0.342	ρ<=ρo	250.3	2.8	1.5	19.976	1421	1	84.49	16.82	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	1421	1	86.11	16.5	OK
	165	1000	180	916.6667	0.447	0.524	ρ>ρo	424.3	2.7	1.5	16.704	1421	1	67.67	21	OK
C3	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	1421	1	87.6	16.22	OK
	175	1000	310	564.5161	0.447	0.342	ρ<=ρo	250.3	2.8	1.5	19.976	1421	1	84.49	16.82	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	1421	1	86.11	16.5	OK
	165	1000	160	1031.25	0.447	0.589	ρ>ρo	487.3	2.6	1.5	15.991	1421	1	63.46	22.39	OK
C4	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	2000	1	87.6	22.83	OK
	175	1000	110	1590.909	0.447	0.964	ρ>ρo	664	3	1.5	13.007	2000	1	58.43	34.23	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	2000	1	86.11	23.23	OK
	165	1000	330	500	0.447	0.286	ρ<=ρo	236	2.6	1.5	22.793	2000	1	90.56	22.08	OK
C5	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	2000	1	87.6	22.83	OK
	175	1000	330	530.303	0.447	0.321	ρ<=ρo	637	1	1.5	20.763	2000	1	32.41	61.71	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	2000	1	86.11	23.23	OK
	165	1000	330	500	0.447	0.286	ρ<=ρo	236	2.6	1.5	22.793	2000	1	90.56	22.08	OK

C6	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	1825	1	87.6	20.83	OK
	175	1000	310	564.5161	0.447	0.342	ρ<=ρo	250.3	2.8	1.5	19.976	1825	1	84.49	21.6	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	1825	1	86.11	21.19	OK
	165	1000	100	1650	0.447	0.943	ρ>ρo	777.1	2.7	1.5	13.142	1825	1	52.32	34.88	OK
C7	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	2000	1	87.6	22.83	OK
	175	1000	100	1750	0.447	1.061	ρ>ρo	777	2.8	1.5	12.444	2000	1	52.55	38.06	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	2000	1	86.11	23.23	OK
	165	1000	330	500	0.447	0.286	ρ<=ρo	236	2.6	1.5	22.793	2000	1	90.56	22.08	OK
C8	175	1000	310	564.5161	0.447	0.323	ρ<=ρo	250.3	2.8	1.5	20.71	600	1	87.6	6.849	OK
	175	1000	310	564.5161	0.447	0.342	ρ<=ρo	250.3	2.8	1.5	19.976	600	1	84.49	7.101	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	600	1	86.11	6.968	OK
	165	1000	330	500	0.447	0.303	ρ<=ρo	236	2.6	1.5	21.671	600	1	86.11	6.968	OK

Appendix 3T calculate**load transfer mechanism**

Panel	Lx	GK	QK	Coefficient		Shear due to dead		Shear due to live		Design shear	
P-1	4.561	11.5	2	βvx,c=	0.39	Vx,c=	20.45609	Vx,c=	3.55758	32.95208	
	4.561	11.5	2	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	4.561	11.5	2	βvy,c=	0.33	Vy,c=	17.309	Vy,c=	3.01026	27.88253	
	4.561	11.5	2	βvy,d=	0	Vy,d=	0	Vy,d=	0	0	
P2	3.937	7.14	2	βvx,c=	0.41	Vx,c=	11.52517	Vx,c=	3.22834	20.40149	
	3.937	7.14	2	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	3.937	7.14	2	βvy,c=	0.36	Vy,c=	10.11966	Vy,c=	2.83464	17.91351	
	3.937	7.14	2	βvy,d=	0.24	Vy,d=	6.746443	Vy,d=	1.88976	11.94234	
P3	5.5	12.9	2	βvx,c=	0.33	Vx,c=	23.4135	Vx,c=	3.63	37.05323	
	5.5	12.9	2	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	5.5	12.9	2	βvy,c=	0.33	Vy,c=	23.4135	Vy,c=	3.63	37.05323	
	5.5	12.9	2	βvy,d=	0	Vy,d=	0	Vy,d=	0	0	
P4	3.937	11.6	2	βvx,c=	0.42	Vx,c=	19.18106	Vx,c=	3.30708	30.85506	
	3.937	11.6	2	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	3.937	11.6	2	βvy,c=	0.33	Vy,c=	15.07084	Vy,c=	2.59842	24.24326	
	3.937	11.6	2	βvy,d=	0	Vy,d=	0	Vy,d=	0	0	
P5	2.9	11.5	2	βvx,c=	0.49	Vx,c=	16.3415	Vx,c=	2.842	26.32403	
	2.9	11.5	2	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	2.9	11.5	2	βvy,c=	0.33	Vy,c=	11.0055	Vy,c=	1.914	17.72843	
	2.9	11.5	2	βvy,d=	0	Vy,d=	0	Vy,d=	0	0	
p6	1.751	7.16	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0	
	1.751	7.16	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	1.751	7.16	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0	
	1.751	7.16	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0	
p7	5.5	14.1	2	βvx,c=	0.33	Vx,c=	25.5915	Vx,c=	3.63	39.99353	
	5.5	14.1	2	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	5.5	14.1	2	βvy,c=	0.33	Vy,c=	25.5915	Vy,c=	3.63	39.99353	
	5.5	14.1	2	βvy,d=	0	Vy,d=	0	Vy,d=	0	0	
p8	0.8	7.16	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0	
	0.8	7.16	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	0.8	7.16	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0	
	0.8	7.16	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0	
C1	2	13.8	2	βvx,c=	0	Vx,c=	0	Vx,c=	0	0	
	2	13.8	2	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	2	13.8	2	βvy,c=	0	Vy,c=	0	Vy,c=	0	0	
	2	13.8	2	βvy,d=	0	Vy,d=	0	Vy,d=	0	0	
C2	1.421	12.9	4	βvx,c=	0	Vx,c=	0	Vx,c=	0	0	
	1.421	12.9	4	βvx,d=	0	Vx,d=	0	Vx,d=	0	0	
	1.421	12.9	4	βvy,c=	0	Vy,c=	0	Vy,c=	0	0	
	1.421	12.9	4	βvy,d=	0	Vy,d=	0	Vy,d=	0	0	

C3	1.421	14.9	4	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	1.421	14.9	4	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	1.421	14.9	4	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	1.421	14.9	4	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C4	2	16.6	4	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	2	16.6	4	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	2	16.6	4	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	2	16.6	4	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C5	2	15.5	4	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	2	15.5	4	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	2	15.5	4	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	2	15.5	4	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C6	1.825	16.3	2	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	1.825	16.3	2	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	1.825	16.3	2	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	1.825	16.3	2	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C7	2	11.8	4	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	2	11.8	4	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	2	11.8	4	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	2	11.8	4	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C8	0.6	26.3	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	0.6	26.3	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	0.6	26.3	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	0.6	26.3	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0

Appendix 3U **Check Shear resistance of the section for second and third floor slab**

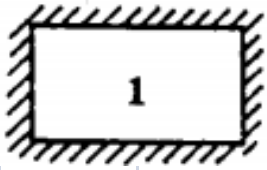
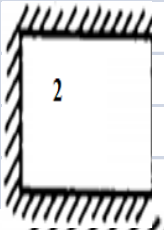
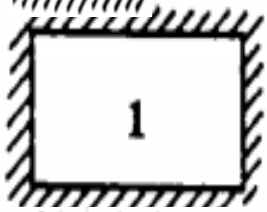
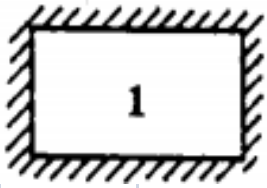
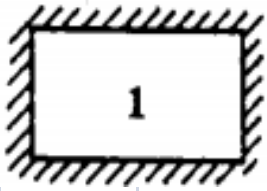
Panel	Design sh	D	K1	CRD,C	Vmin	σcp	fck	$\rho 1$	$VRd,c(m$	VRd,c	check for	Asprovide
P-1	32.952	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	766.32
	27.88	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	434.3
P2	20.4	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	766.32
	17.91	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	11.94	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	260.98
P3	37.05	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	663.99
	37.05	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	244.08
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	487.26
P4	30.86	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	663.99
	24.24	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	322.74
P5	26.32	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	636.98
	17.73	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
p6	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	636.98
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95

p7	39.99	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	776.95	
	39.99	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	787.19	
p8	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
C1	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	766.32	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
C2	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	424.27	
C3	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	487.26	
C4	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	663.99	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
C5	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	636.98	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
C6	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	777.09	
C7	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	776.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
C8	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95	

Appendix 3V Depth **determination for Roof solid slab**

pannel	Ly(m)	Lx(m)	Ly/Lx	type	suppor	K	N	F1	F2	F3	L/d(mn	Eff/dep	(mm)
P1	5572	4561	1.221662	two way	Interior	1.5	17.71	1.25	1	1	33.20625	137.3537	
P2	4561	3937	1.158496	two way	Exterior	1.3	17.71	1.25	1	1	28.77875	136.8023	
P3	5572	5500	1.013091	two way	Interior	1.5	17.71	1.25	1	1	33.20625	165.6315	
P4	5500	3937	1.397003	two way	Interior	1.5	17.71	1.25	1	1	33.20625	118.562	
P5	5500	2900	1.896552	two way	Interior	1.5	17.71	1.25	1	1	33.20625	87.33296	
P6	5500	1751	3.141062	One way	Interior	1.5	17.71	1.25	1	1	33.20625	52.73104	
P7	5549	5500	1.008909	two way	Interior	1.5	17.71	1.25	1	1	33.20625	165.6315	
P8	2900	610	4.754098	One way	Exterior	1.3	17.71	1.25	1	1	28.77875	21.1962	
C1	9509	2000	4.7545	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174	
C2	9561	1421	6.72836	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	45.8498	
C3	7100	1421	4.996481	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	45.8498	
C4	3909	2000	1.9545	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174	
C5	1020	2000	0.51	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174	
C6	5500	1825	3.013699	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	58.88521	
C7	7374	2000	3.687	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	64.53174	
C8	1751	600	2.918333	One way	Cantilever	1.4	17.71	1.25	1	1	30.9925	19.35952	

Appendix 3W **Roof solid slab moment determinatio**

panel	panel cnd	Lx	Lx*Lx	design loa	cofficent		moment analysis	
			0.80272	14.83	αxs=	0.0428	Mxs=	13.20399
P-1					αxf=	0.0326	Mxf=	10.05724
					αys=	0.032	Mys=	9.872139
					αyf=	0.024	Myf=	7.404104
		3.937	15.49997	14.83	αxs=	0.059	Mxs=	13.56201
P-2					αxf=	0.04	Mxf=	9.194582
					αys=	0.039	Mys=	8.964717
					αyf=	0.029	Myf=	6.666072
P-3			30.25	14.83	αxs=	0.033	Mxs=	14.80405
					αxf=	0.025	Mxf=	11.21519
					αys=	0.032	Mys=	14.35544
					αyf=	0.024	Myf=	10.76658
			5.49997	14.83	αxs=	0.049	Mxs=	11.26336
P-4					αxf=	0.036	Mxf=	8.275123
					αys=	0.032	Mys=	7.355665
					αyf=	0.024	Myf=	5.516749
			8.41	14.83	αxs=	0.061	Mxs=	7.607938
					αxf=	0.046	Mxf=	5.737134
P-5					αys=	0.032	Mys=	3.99105
					αyf=	0.024	Myf=	2.993287
			30.25	14.83	αxs=	0.0324	Mxs=	14.53488
P-7					αxf=	0.024	Mxf=	10.76658
					αys=	0.032	Mys=	14.35544
					αyf=	0.024	Myf=	10.76658

Appendix 3X Support **moment adjustment roof solid calculation**

Pannel	support moment		Length		valueof K						
	Mxs(max)	Mxs(min)	L_1	L_2	K_1	K_2	Mxs(max)-Mxs(min)		average	stifness method	
p1and p2	13.56	13.203	5.572	3.937	0.179469	0.254001	0.357			13.41219	
p1and p3	14.83	13.203	4.561	5.5	0.21925	0.181818	1.627		14.0165		
p2and p4	13.56	11.26	5.5	4.561	0.181818	0.21925	2.3		12.41		
p3and p4	14.83	11.26	5.572	3.937	0.179469	0.254001	3.57			13.35192	
p4and p5	11.26	7.61	3.937	2.9	0.254001	0.344828	3.65			9.711806	
p5and p6	7.61	3.78	2.9	1.75	0.344828	0.571429	3.83			6.168602	
p6and p7	14.53	3.78	1.75	5.549	0.571429	0.180213	10.75			6.357408	
p5and p8	7.61	0.43	5.5	0.46	0.181818	2.173913	7.18			7.055839	
p2and p8	13.56	0.43	5.5	0.46	0.181818	2.173913	13.13			12.54661	

Appendix 3Y**Span moment adjustment roof solid slab**

pannel	unadjusted moment		adjusted moment		ΔMxs	ΔMys	previous moment		Ly/Lx	cofficeints				ΔMxf	ΔMyf	Mxfd	Myfd
	Mxs	Mys	Mxs	Mys			Mxf	Myf		Cx1	Cy1	Cx2	Cy2				
p1	13.203	9.87	14.04	14.95	-0.837	-5.08	10.05	7.404	1.22	0.335	0.165	0.349	0.361	-2.05332	-1.97199	7.996685	2.265
p2	13.56	8.96	12.45	13.41	1.11	-4.45	9.19	6.66	1.158	0.346	0.192	0.331	0.368	-1.08889	-1.42448	2.871	1.08
p3	14.8	14.35	18.98	14.95	-4.18	-0.6	14.8	10.76	1.013	0.376	0.272	0.284	0.379	-1.74208	-1.36436	13.05792	9.39564
p4	11.26	7.35	18.98	13.35	-7.72	-6	8.27	5.51	1.397	0.315	0.111	0.397	0.332	-4.8138	-2.84892	3.4562	2.66108
p5	7.61	3.99	18.98	9.71	-11.37	-5.72	5.73	2.99	1.896	0.259	0.06	0.481	0.251	-5.69615	-2.11792	0.03385	0.87208
p7	14.53	14.35	6.35	6.35	8.18	8	10.76	10.76	1	0.38	0.28	0.28	0.38	5.3484	5.3304	16.1084	16.0904

Appendix 3Z**Reinforcement design for Roof solid slab**

Panel	Adjusted I	Φ	D	Km	Ks	Ascalc	Asmin	As require	as	Spacing	St provide	Spacing
P-1	Mxf=7.99	10	175	7.05	2.96	25.71	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=14.04	10	175	37.58	3.1	766.32	250.25	766.32	78.5	100	100	Φ10 c/c 100
	Myf=2.26	10	165	9.13	2.96	40.72	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=14.95	10	165	29.47	3.03	434.3	235.95	434.3	78.5	180	180	Φ10 c/c 180
P2	Mxf=2.87	10	175	9.68	2.96	48.54	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=12.41	10	175	37.58	3.1	766.32	250.25	766.32	78.5	100	100	Φ10 c/c 100
	Myf=1.08	10	165	6.29	2.96	19.37	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=13.41	10	165	23.04	2.98	260.98	235.95	260.98	78.5	300	300	Φ10 c/c 300
P3	Mxf=13.05	10	175	18.45	2.96	176.25	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=14.98	10	175	35.16	3.07	663.99	250.25	663.99	78.5	110	110	Φ10 c/c 110
	Myf=9.39	10	165	22.32	2.97	244.08	235.95	244.08	78.5	320	320	Φ10 c/c 320
	Mys=14.95	10	165	31.11	3.05	487.26	235.95	487.26	78.5	160	160	Φ10 c/c 160
P4	Mxf=3.45	10	175	16.44	2.96	140.05	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=18.98	10	175	35.16	3.07	663.99	250.25	663.99	78.5	110	110	Φ10 c/c 110
	Myf=2.66	10	165	14.39	2.96	101.18	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=13.35	10	165	25.58	2.99	322.74	235.95	322.74	78.5	240	240	Φ10 c/c 240
P5	Mxf=0.034	10	175	11.75	2.96	71.55	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=18.95	10	175	34.43	3.07	636.98	250.25	636.98	78.5	120	120	Φ10 c/c 120
	Myf=0.87	10	165	9.46	2.96	43.77	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=9.71	10	165	21.16	2.97	219.42	235.95	235.95	78.5	330	330	Φ10 c/c 330
p6	Mxf=1.89	10	175	8.46	2.96	37.04	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=6.16	10	175	34.43	3.07	636.98	250.25	636.98	78.5	120	120	Φ10 c/c 120
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=8.52	10	165	17.69	2.96	152.84	235.95	235.95	78.5	330	330	Φ10 c/c 330
p7	Mxf=16.10	10	175	4.19	2.96	9.13	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=18.35	10	175	37.84	3.1	776.95	250.25	776.95	78.5	100	100	Φ10 c/c 100
	Myf=16.09	10	165	4.57	2.96	10.23	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=6.35	10	165	39.1	3.12	787.19	235.95	787.19	78.5	99	99	Φ10 c/c 99
p8	Mxf=0.76	10	175	5.36	2.96	14.88	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=7.05	10	175	16.96	2.96	149.01	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=12.54	10	165	19.83	2.96	192.13	235.95	235.95	78.5	330	330	Φ10 c/c 330
C1	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=14.83	10	175	37.58	3.1	766.32	250.25	766.32	78.5	100	100	Φ10 c/c 100
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330

C3	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=14.95	10	165	31.11	3.05	487.26	235.95	487.26	78.5	160	160	Φ10 c/c 160
C4	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=37.85	10	175	35.16	3.07	663.99	250.25	663.99	78.5	110	110	Φ10 c/c 110
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C5	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=18.98	10	175	34.43	3.07	636.98	250.25	636.98	78.5	330	330	Φ10 c/c 330
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C6	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=18.98	10	165	39.1	3.08	777.09	235.95	777.09	78.5	100	100	Φ10 c/c 100
C7	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=29.66	10	175	37.84	3.1	776.95	250.25	776.95	78.5	100	100	Φ10 c/c 100
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
C8	Mxf=0.00	10	175	0	2.96	0	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Mxs=4.75	10	175	21.19	2.97	233.53	250.25	250.25	78.5	310	310	Φ10 c/c 310
	Myf=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330
	Mys=0.00	10	165	0	2.96	0	235.95	235.95	78.5	330	330	Φ10 c/c 330

Appendix 3AA Depth **check for serviceability for Roof solid slab**

Panel	D	b	St provide	As provide	po	ρ	Criteria	As require	F1	K	N	Lx	F2&F3	lx/d	d	depth check
P-1	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	4561	1	87.6001	52.06615	OK
	175	1000	100	1750	0.44721	1.06061	ρ>ρo	766.32	2.85455	1.5	12.444	4561	1	53.28306	85.59944	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	4561	1	86.10537	52.96998	OK
	165	1000	180	916.6667	0.44721	0.52381	ρ>ρo	434.3	2.63835	1.5	16.704	4561	1	66.10638	68.99486	OK
P2	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	3937	1	87.6001	44.94287	OK
	175	1000	100	1750	0.44721	1.06061	ρ>ρo	766.32	2.85455	1.5	12.444	3937	1	53.28306	73.8884	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	3937	1	86.10537	45.72305	OK
	165	1000	300	550	0.44721	0.31429	ρ<=ρo	260.98	2.6343	1.5	21.086	3937	1	83.32032	47.25138	OK
P3	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	5500	1	87.6001	62.78532	OK
	175	1000	110	1590.909	0.44721	0.96419	ρ>ρo	663.99	2.99498	1.5	13.007	5500	1	58.43355	94.12402	OK
	165	1000	320	515.625	0.44721	0.3125	ρ<=ρo	244.08	2.64066	1.5	22.433	5500	1	88.85674	61.89738	OK
	165	1000	160	1031.25	0.44721	0.58929	ρ>ρo	487.26	2.64553	1.5	15.991	5500	1	63.45708	86.67275	OK
P4	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	3937	1	87.6001	44.94287	OK
	175	1000	110	1590.909	0.44721	0.96419	ρ>ρo	663.99	2.99498	1.5	13.007	3937	1	58.43355	67.37568	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	3937	1	86.10537	45.72305	OK
	165	1000	240	687.5	0.44721	0.39286	ρ<=ρo	322.74	2.66275	1.5	18.654	3937	1	74.50633	52.84115	OK
P5	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	2900	1	87.6001	33.10498	OK
	175	1000	120	1458.333	0.44721	0.88384	ρ>ρo	636.98	2.86181	1.5	13.531	2900	1	58.08476	49.92704	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	2900	1	86.10537	33.67966	OK
	165	1000	330	500	0.44721	0.28571	ρ<=ρo	235.95	2.64887	1.5	22.793	2900	1	90.56341	32.02176	OK
p6	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	1751	1	87.6001	19.98856	OK
	175	1000	120	1458.333	0.44721	0.88384	ρ>ρo	636.98	2.86181	1.5	13.531	1751	1	58.08476	30.1456	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	1751	1	86.10537	20.33555	OK
	165	1000	330	500	0.44721	0.28571	ρ<=ρo	235.95	2.64887	1.5	22.793	1751	1	90.56341	19.33452	OK
p7	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	5500	1	87.6001	62.78532	OK
	175	1000	100	1750	0.44721	1.06061	ρ>ρo	776.95	2.8155	1.5	12.444	5500	1	52.55406	104.6541	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	5500	1	86.10537	63.87522	OK
	165	1000	99	1666.667	0.44721	0.95238	ρ>ρo	787.19	2.64654	1.5	13.082	5500	1	51.93314	105.9054	OK
p8	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	641	1	87.6001	7.317343	OK
	175	1000	310	564.5161	0.44721	0.34213	ρ<=ρo	250.25	2.81976	1.5	19.976	641	1	84.49132	7.586579	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	641	1	86.10537	7.444367	OK
	165	1000	330	500	0.44721	0.28571	ρ<=ρo	235.95	2.64887	1.5	22.793	641	1	90.56341	7.077913	OK
C1	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	2000	1	87.6001	22.83102	OK
	175	1000	100	1750	0.44721	1.06061	ρ>ρo	766.32	2.85455	1.5	12.444	2000	1	53.28306	37.53538	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	2000	1	86.10537	23.22735	OK
	165	1000	330	500	0.44721	0.28571	ρ<=ρo	235.95	2.64887	1.5	22.793	2000	1	90.56341	22.08397	OK

C2	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	1421	1	87.6001	16.22144	OK
	175	1000	310	564.5161	0.44721	0.34213	ρ<=ρo	250.25	2.81976	1.5	19.976	1421	1	84.49132	16.8183	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	1421	1	86.10537	16.50304	OK
	165	1000	180	916.6667	0.44721	0.52381	ρ>ρo	424.27	2.70072	1.5	16.704	1421	1	67.66917	20.99922	OK
C3	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	1421	1	87.6001	16.22144	OK
	175	1000	310	564.5161	0.44721	0.34213	ρ<=ρo	250.25	2.81976	1.5	19.976	1421	1	84.49132	16.8183	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	1421	1	86.10537	16.50304	OK
	165	1000	160	1031.25	0.44721	0.58929	ρ>ρo	487.26	2.64553	1.5	15.991	1421	1	63.45708	22.39309	OK
C4	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	2000	1	87.6001	22.83102	OK
	175	1000	110	1590.909	0.44721	0.96419	ρ>ρo	663.99	2.99498	1.5	13.007	2000	1	58.43355	34.22692	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	2000	1	86.10537	23.22735	OK
	165	1000	330	500	0.44721	0.28571	ρ<=ρo	235.95	2.64887	1.5	22.793	2000	1	90.56341	22.08397	OK
C5	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	2000	1	87.6001	22.83102	OK
	175	1000	330	530.303	0.44721	0.3214	ρ<=ρo	636.98	1.04066	1.5	20.763	2000	1	32.4108	61.70783	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	2000	1	86.10537	23.22735	OK
	165	1000	330	500	0.44721	0.28571	ρ<=ρo	235.95	2.64887	1.5	22.793	2000	1	90.56341	22.08397	OK
C6	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	1825	1	87.6001	20.83331	OK
	175	1000	310	564.5161	0.44721	0.34213	ρ<=ρo	250.25	2.81976	1.5	19.976	1825	1	84.49132	21.59985	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	1825	1	86.10537	21.19496	OK
	165	1000	100	1650	0.44721	0.94286	ρ>ρo	777.09	2.65413	1.5	13.142	1825	1	52.32092	34.88089	OK
C7	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	2000	1	87.6001	22.83102	OK
	175	1000	100	1750	0.44721	1.06061	ρ>ρo	776.95	2.8155	1.5	12.444	2000	1	52.55406	38.05605	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	2000	1	86.10537	23.22735	OK
	165	1000	330	500	0.44721	0.28571	ρ<=ρo	235.95	2.64887	1.5	22.793	2000	1	90.56341	22.08397	OK
C8	175	1000	310	564.5161	0.44721	0.32258	ρ<=ρo	250.25	2.81976	1.5	20.711	600	1	87.6001	6.849307	OK
	175	1000	310	564.5161	0.44721	0.34213	ρ<=ρo	250.25	2.81976	1.5	19.976	600	1	84.49132	7.101322	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	600	1	86.10537	6.968206	OK
	165	1000	330	500	0.44721	0.30303	ρ<=ρo	235.95	2.64887	1.5	21.671	600	1	86.10537	6.968206	OK

Appendix 3AB Load **transfer for Roof solid slab**

Panel	Lx	GK	QK	Coefficient		Shear due to dead		Shear due to live		Design shear
P-1	4.561	5.44	5	βvx,c=	0.39	Vx,c=	9.676618	Vx,c=	8.89395	26.40436
	4.561	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	4.561	5.44	5	βvy,c=	0.33	Vy,c=	8.187907	Vy,c=	7.52565	22.34215
	4.561	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
P2	3.937	5.44	5	βvx,c=	0.41	Vx,c=	8.781085	Vx,c=	8.07085	23.96074
	3.937	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	3.937	5.44	5	βvy,c=	0.36	Vy,c=	7.710221	Vy,c=	7.0866	21.0387
	3.937	5.44	5	βvy,d=	0.24	Vy,d=	5.140147	Vy,d=	4.7244	14.0258
P3	5.5	5.44	5	βvx,c=	0.33	Vx,c=	9.8736	Vx,c=	9.075	26.94186
	5.5	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	5.5	5.44	5	βvy,c=	0.33	Vy,c=	9.8736	Vy,c=	9.075	26.94186
	5.5	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
P4	3.937	5.44	5	βvx,c=	0.42	Vx,c=	8.995258	Vx,c=	8.2677	24.54515
	3.937	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	3.937	5.44	5	βvy,c=	0.33	Vy,c=	7.067702	Vy,c=	6.49605	19.28547
	3.937	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
P5	2.9	5.44	5	βvx,c=	0.49	Vx,c=	7.73024	Vx,c=	7.105	21.09332
	2.9	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	2.9	5.44	5	βvy,c=	0.33	Vy,c=	5.20608	Vy,c=	4.785	14.20571
	2.9	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
p6	1.751	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	1.751	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	1.751	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	1.751	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
p7	5.5	5.44	5	βvx,c=	0.33	Vx,c=	9.8736	Vx,c=	9.075	26.94186
	5.5	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	5.5	5.44	5	βvy,c=	0.33	Vy,c=	9.8736	Vy,c=	9.075	26.94186
	5.5	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
p8	0.8	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	0.8	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	0.8	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	0.8	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C1	2	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	2	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	2	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	2	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0

C2	1.421	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	1.421	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	1.421	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	1.421	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C3	1.421	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	1.421	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	1.421	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	1.421	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C4	2	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	2	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	2	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	2	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C5	2	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	2	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	2	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	2	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C6	1.825	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	1.825	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	1.825	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	1.825	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C7	2	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	2	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	2	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	2	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0
C8	0.6	5.44	5	βvx,c=	0	Vx,c=	0	Vx,c=	0	0
	0.6	5.44	5	βvx,d=	0	Vx,d=	0	Vx,d=	0	0
	0.6	5.44	5	βvy,c=	0	Vy,c=	0	Vy,c=	0	0
	0.6	5.44	5	βvy,d=	0	Vy,d=	0	Vy,d=	0	0

Appendix 3AC Check **Shear resistance of the section for Roof solid slab**

Panel	Design sh	D	K1	CRD,C	Vmin	σcp	fck	$\rho 1$	$VRd,c(m$	VRd,c	check for	Asprovide
P-1	26.404	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	766.32
	22.34	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	434.3
P2	23.96	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	766.32
	21.03	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	14.03	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	260.98
P3	26.94	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	663.99
	26.94	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	244.08
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	487.26
P4	24.54	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	663.99
	19.28	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	322.74
P5	21.09	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	636.98
	14.2	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
p6	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	636.98
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
p7	26.94	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	776.95
	26.94	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	787.19
p8	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
C1	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	766.32
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
C2	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	424.27

C3	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	487.26
C4	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	663.99
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
C5	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	636.98
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
C6	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	777.09
C7	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	776.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
C8	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	175	1.08	0.12	0.443	0	20	0.02	77.525	77.5656	SAFE	250.25
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95
	0	165	1.08	0.12	0.443	0	20	0.02	73.095	73.13328	SAFE	235.95

Appendix 7AReinforcement of a beam on axis A

Axis A	SRORY 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	22.09	250	352	0.036	340.55	Single	114.4	3520	186.49	186.49	0.5939	4Φ 20
Span	b/n1&2	16.09	250	352	0.026	343.74	Single	114.4	3520	134.57	134.57	0.4286	4Φ 20
Support	2	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	4Φ 20
Span	b/n2&3	8.27	250	352	0.013	347.8	Single	114.4	3520	68.36	114.4	0.3643	4Φ 20
Support	3	17.21	250	352	0.028	343.15	double	114.4	3520	144.19	144.19	0.4592	4Φ 20
Axis A	SRORY 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	27.186	250	352	0.044	337.8	Single	114.4	3520	231.38	231.38	0.7369	4Φ 20
Span	b/n1&2	16.86	250	352	0.027	343.33	Single	114.4	3520	141.18	141.18	0.4496	4Φ 20
Support	2	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	4Φ 20
Span	b/n2&3	10.54	250	352	0.017	346.63	Single	114.4	3520	87.419	114.4	0.3643	4Φ 20
Support	3	20.35	250	352	0.033	341.48	double	114.4	3520	171.33	171.33	0.5456	4Φ 20
Axis A	SRORY 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	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	4Φ 20
Span	b/n1&2	16.84	250	352	0.027	343.34	Single	114.4	3520	141.01	141.01	0.4491	4Φ 20
Support	2	26.42	250	352	0.043	338.21	Single	114.4	3520	224.58	224.58	0.7152	4Φ 20
Span	b/n2&3	10.54	250	352	0.017	346.63	Single	114.4	3520	87.419	114.4	0.3643	4Φ 20
Support	3	19.75	250	352	0.032	341.8	double	114.4	3520	166.12	166.12	0.529	4Φ 20
Axis A	SRORY 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	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	4Φ 20
Span	b/n1&2	16.69	250	352	0.027	343.42	Single	114.4	3520	139.72	139.72	0.445	4Φ 20
Support	2	34.47	250	352	0.056	333.77	Single	114.4	3520	296.91	296.91	0.9456	4Φ 20
Span	b/n2&3	10.54	250	352	0.017	346.63	Single	114.4	3520	87.419	114.4	0.3643	4Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20

Appendix 7B Reinforcement of a beam on axis B

Axis B	SRORY 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	23.3	250	352	0.038	339.9	Single	114.4	3520	197.08	197.08	0.6276	2Φ 20
Span	b/n1&2	16.23	250	352	0.026	343.66	Single	114.4	3520	135.77	135.77	0.4324	2Φ 20
Support	2	16.82	250	352	0.027	343.35	Single	114.4	3520	140.84	140.84	0.4485	2Φ 20
Span	b/n2&3	8.79	250	352	0.014	347.54	Single	114.4	3520	72.715	114.4	0.3643	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20

Axis B	SRORY 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	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Span	b/n1&2	19.47	250	352	0.031	341.95	Single	114.4	3520	163.7	163.7	0.5213	2Φ 20
Support	2	34.42	250	352	0.056	333.8	Single	114.4	3520	296.45	296.45	0.9441	2Φ 20
Span	b/n2&3	16.39	250	352	0.026	343.58	Single	114.4	3520	137.15	137.15	0.4368	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20

Axis B	SRORY 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	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Span	b/n1&2	19.44	250	352	0.031	341.97	Single	114.4	3520	163.44	163.44	0.5205	2Φ 20
Support	2	35.12	250	352	0.057	333.41	Single	114.4	3520	302.84	302.84	0.9645	2Φ 20
Span	b/n2&3	16.07	250	352	0.026	343.75	Single	114.4	3520	134.4	134.4	0.428	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20

Axis B	SRORY 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	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Span	b/n1&2	21.97	250	352	0.035	340.62	Single	114.4	3520	185.44	185.44	0.5906	2Φ 20
Support	2	53.1	250	352	0.086	322.98	Single	114.4	3520	472.66	472.66	1.5053	2Φ 20
Span	b/n2&3	16.07	250	352	0.026	343.75	Single	114.4	3520	134.4	134.4	0.428	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20

Appendix 7C Reinforcement of a beam on axis C

Axis C	SRORY 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	19.93	250	352	0.032	341.71	Single	114.4	3520	167.68	167.68	0.534	2Φ 20
Span	b/n1&2	13.26	250	352	0.021	345.22	Single	114.4	3520	110.43	114.4	0.3643	2Φ 20
Support	2	13.26	250	352	0.021	345.22	Single	114.4	3520	110.43	114.4	0.3643	2Φ 20
Span	b/n2&3	8.09	250	352	0.013	347.9	Single	114.4	3520	66.855	114.4	0.3643	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20

Axis C	SRORY 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	24.67	250	352	0.040	339.16	Single	114.4	3520	209.12	209.12	0.666	2Φ 20
Span	b/n1&2	15.06	250	352	0.024	344.28	Single	114.4	3520	125.76	125.76	0.4005	2Φ 20
Support	2	31.02	250	352	0.050	335.69	Single	114.4	3520	265.67	265.67	0.8461	2Φ 20
Span	b/n2&3	22.77	250	352	0.037	340.19	Single	114.4	3520	192.43	192.43	0.6128	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20

Axis C	SRORY 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	25.85	250	352	0.042	338.52	Single	114.4	3520	219.54	219.54	0.6992	2Φ 20
Span	b/n1&2	14.43	250	352	0.023	344.61	Single	114.4	3520	120.38	120.38	0.3834	2Φ 20
Support	2	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Span	b/n2&3	22.02	250	352	0.036	340.59	Single	114.4	3520	185.87	185.87	0.592	2Φ 20
Support	3	81.14	250	352	0.131	305.05	double	114.4	3520	764.7	764.7	2.4354	3Φ 20

Axis C	SRORY 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	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Span	b/n1&2	37.62	250	352	0.061	332	Single	114.4	3520	325.77	325.77	1.0375	2Φ 20
Support	2	77.52	250	352	0.125	307.51	Single	114.4	3520	724.76	724.76	2.3082	2Φ 20
Span	b/n2&3	22.3	250	352	0.036	340.44	Single	114.4	3520	188.32	188.32	0.5997	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20

Appendix 7DReinforcement of a beam on axis D

Axis D	SRORY 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	19.79	250	352	0.032	341.78	Single	114.4	3520	166.47	166.47	0.5302	2Φ 20
Span	b/n1&2	12.36	250	352	0.020	345.69	Single	114.4	3520	102.79	114.4	0.3643	2Φ 20
Support	2	11.85	250	352	0.019	345.95	Single	114.4	3520	98.476	114.4	0.3643	2Φ 20
Span	b/n2&3	10.14	250	352	0.016	346.84	Single	114.4	3520	84.051	114.4	0.3643	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20
Axis D	SRORY 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	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Span	b/n1&2	33.09	250	352	0.053	334.54	Single	114.4	3520	284.37	284.37	0.9056	2Φ 20
Support	2	45.1	250	352	0.073	327.71	Single	114.4	3520	395.66	395.66	1.2601	2Φ 20
Span	b/n2&3	22.63	250	352	0.037	340.26	Single	114.4	3520	191.21	191.21	0.6089	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20
Axis D	SRORY 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	45.9	250	352	0.074	327.24	Single	114.4	3520	403.25	403.25	1.2842	2Φ 20
Span	b/n1&2	31.68	250	352	0.051	335.33	Single	114.4	3520	271.61	271.61	0.865	2Φ 20
Support	2	31.83	250	352	0.051	335.24	Single	114.4	3520	272.97	272.97	0.8693	2Φ 20
Span	b/n2&3	22.25	250	352	0.036	340.47	Single	114.4	3520	187.88	187.88	0.5984	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20
Axis D	SRORY 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	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Span	b/n1&2	54.28	250	352	0.088	322.27	Single	114.4	3520	484.23	484.23	1.5421	2Φ 20
Support	2	101.84	250	352	0.164	290.02	Single	114.4	3520	1009.5	1009.5	3.2151	4Φ 20
Span	b/n2&3	22.3	250	352	0.036	340.44	Single	114.4	3520	188.32	188.32	0.5997	2Φ 20
Support	3	0	250	352	0.000	352	double	114.4	3520	0	114.4	0.3643	2Φ 20

Appendix 7EReinforcement of a beam on axis E

Axis E	SRORY 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	23.14	250	352	0.037	339.99	Single	114.4	3520	195.67	195.67	0.6232	2Φ 20
Span	b/n1&2	15.3	250	352	0.025	344.15	Single	114.4	3520	127.81	127.81	0.407	2Φ 20
Support	2	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Axis E	SRORY 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	52	250	352	0.084	323.64	Single	114.4	3520	461.93	461.93	1.4711	2Φ 20
Span	b/n1&2	49.94	250	352	0.081	324.87	Single	114.4	3520	441.95	441.95	1.4075	2Φ 20
Support	2	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Axis E	SRORY 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	55.72	250	352	0.090	321.4	Single	114.4	3520	498.42	498.42	1.5873	2Φ 20
Span	b/n1&2	39.76	250	352	0.064	330.78	Single	114.4	3520	345.57	345.57	1.1005	2Φ 20
Support	2	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Axis E	SRORY 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	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Span	b/n1&2	55.98	250	352	0.090	321.24	Single	114.4	3520	500.99	500.99	1.5955	2Φ 20
Support	2	67.71	250	352	0.109	313.93	Single	114.4	3520	620.08	620.08	1.9748	2Φ 20

Appendix 7F Reinforcement of a beam on axis F

Axis F	SRORY 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	21.76	250	352	0.035	340.73	Single	114.4	3520	183.6	183.6	0.5847	2Φ 20
Span	b/n1&2	13.69	250	352	0.022	345	Single	114.4	3520	114.08	114.4	0.3643	2Φ 20
Support	2	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Axis F	SRORY 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	66.15	250	352	0.107	314.93	Single	114.4	3520	603.88	603.88	1.9232	2Φ 20
Span	b/n1&2	51.79	250	352	0.084	323.77	Single	114.4	3520	459.88	459.88	1.4646	2Φ 20
Support	2	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Axis F	SRORY 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	68.49	250	352	0.111	313.43	Single	114.4	3520	628.23	628.23	2.0007	3Φ 20
Span	b/n1&2	50.05	250	352	0.081	324.8	Single	114.4	3520	443.01	443.01	1.4109	2Φ 20
Support	2	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Axis F	SRORY 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	250	352	0.000	352	Single	114.4	3520	0	114.4	0.3643	2Φ 20
Span	b/n1&2	52.61	250	352	0.085	323.28	Single	114.4	3520	467.87	467.87	1.49	2Φ 20
Support	2	65.26	250	352	0.105	315.49	Single	114.4	3520	594.69	594.69	1.8939	2Φ 20

Appendix 71Reinforcement of a beam on axis 1

Axis1	SRORY 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
Suppor	A	25.91	250	352	0.042	338.5	Single	114.4	3520	220.1	220.1	0.701	2Φ 20
Span	b/nA&I	18.51	250	352	0.030	342.5	Single	114.4	3520	155.4	155.4	0.495	2Φ 20
Suppor	B	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nB&(	9.91	250	352	0.016	347	Single	114.4	3520	82.12	114.4	0.364	2Φ 20
Suppor	C	17.25	250	352	0.028	343.1	Single	114.4	3520	144.5	144.5	0.46	2Φ 20
Span	b/nC&I	4.43	250	352	0.007	349.8	Single	114.4	3520	36.41	114.4	0.364	2Φ 20
Suppor	D	11.76	250	352	0.019	346	Single	114.4	3520	97.72	114.4	0.364	2Φ 20
Span	b/nD&I	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	E	25.89	250	352	0.042	338.5	Single	114.4	3520	219.9	219.9	0.7	2Φ 20
Span	b/nE&I	22.6	250	352	0.036	340.3	Single	114.4	3520	190.9	190.9	0.608	2Φ 20
Suppor	F	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20

Axis1	SRORY 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
Suppor	A	38.42	250	352	0.062	331.5	Single	114.4	3520	333.2	333.2	1.061	2Φ 20
Span	b/nA&I	24.23	250	352	0.039	339.4	Single	114.4	3520	205.2	205.2	0.654	2Φ 20
Suppor	B	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nB&(	8.61	250	352	0.014	347.6	Single	114.4	3520	71.21	114.4	0.364	2Φ 20
Suppor	C	17.01	250	352	0.027	343.3	Single	114.4	3520	142.5	142.5	0.454	2Φ 20
Span	b/nC&I	5.89	250	352	0.010	349	Single	114.4	3520	48.52	114.4	0.364	2Φ 20
Suppor	D	13.33	250	352	0.022	345.2	Single	114.4	3520	111	114.4	0.364	2Φ 20
Span	b/nD&I	0.8	250	352	0.001	351.6	Single	114.4	3520	6.541	114.4	0.364	2Φ 20
Suppor	E	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nE&I	21.8	250	352	0.035	340.7	Single	114.4	3520	184	184	0.586	2Φ 20
Suppor	F	14.45	250	352	0.023	344.6	Single	114.4	3520	120.6	120.6	0.384	2Φ 20

Axis1	SRORY 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
Suppor	A	16.93	250	352	0.027	343.3	Single	114.4	3520	141.8	141.8	0.452	2Φ 20
Span	b/nA&I	24.09	250	352	0.039	339.5	Single	114.4	3520	204	204	0.65	2Φ 20
Suppor	B	16.25	250	352	0.026	343.7	Single	114.4	3520	135.9	135.9	0.433	2Φ 20
Span	b/nB&(	8.57	250	352	0.014	347.6	Single	114.4	3520	70.87	114.4	0.364	2Φ 20
Suppor	C	11.8	250	352	0.019	346	Single	114.4	3520	98.05	114.4	0.364	2Φ 20
Span	b/nC&I	6.18	250	352	0.010	348.9	Single	114.4	3520	50.93	114.4	0.364	2Φ 20
Suppor	D	10.92	250	352	0.018	346.4	Single	114.4	3520	90.62	114.4	0.364	2Φ 20
Span	b/nD&I	0.2	250	352	3E-04	351.9	Single	114.4	3520	1.634	114.4	0.364	2Φ 20
Suppor	E	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nE&I	21.97	250	352	0.035	340.6	Single	114.4	3520	185.4	185.4	0.591	2Φ 20
Suppor	F	33.37	250	352	0.054	334.4	Single	114.4	3520	286.9	286.9	0.914	2Φ 20
Axis1	SRORY 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
Suppor	A	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nA&I	23.38	250	352	0.038	339.9	Single	114.4	3520	197.8	197.8	0.63	2Φ 20
Suppor	B	43.61	250	352	0.070	328.6	Single	114.4	3520	381.6	381.6	1.215	2Φ 20
Span	b/nB&(	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	C	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nC&I	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	D	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nD&I	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	E	56.9	250	352	0.092	320.7	Single	114.4	3520	510.1	510.1	1.625	2Φ 20
Span	b/nE&I	31.9	250	352	0.051	335.2	Single	114.4	3520	273.6	273.6	0.871	2Φ 20
Suppor	F	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20

Appendix 72 Reinforcement of a beam on axis 2

Axis2	SRORY 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
Suppor	A	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nA&I	15.07	250	352	0.024	344.3	Single	114.4	3520	125.8	125.8	0.401	2Φ 20
Suppor	B	19.33	250	352	0.031	342	Single	114.4	3520	162.5	162.5	0.517	2Φ 20
Span	b/nB&(	6.92	250	352	0.011	348.5	Single	114.4	3520	57.09	114.4	0.364	2Φ 20
Suppor	C	4.98	250	352	0.008	349.5	Single	114.4	3520	40.97	114.4	0.364	2Φ 20
Span	b/nC&I	4.05	250	352	0.007	350	Single	114.4	3520	33.27	114.4	0.364	2Φ 20
Suppor	D	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nD&I	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	E	20.41	250	352	0.033	341.4	Single	114.4	3520	171.8	171.8	0.547	2Φ 20
Span	b/nE&I	15.55	250	352	0.025	344	Single	114.4	3520	130	130	0.414	2Φ 20
Suppor	F	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Axis2	SRORY 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
Suppor	A	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nA&I	39.03	250	352	0.063	331.2	Single	114.4	3520	338.8	338.8	1.079	2Φ 20
Suppor	B	53.24	250	352	0.086	322.9	Single	114.4	3520	474	474	1.51	2Φ 20
Span	b/nB&(	16.34	250	352	0.026	343.6	Single	114.4	3520	136.7	136.7	0.435	2Φ 20
Suppor	C	11.79	250	352	0.019	346	Single	114.4	3520	97.97	114.4	0.364	2Φ 20
Span	b/nC&I	8.34	250	352	0.013	347.8	Single	114.4	3520	68.95	114.4	0.364	2Φ 20
Suppor	D	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nD&I	4.08	250	352	0.007	349.9	Single	114.4	3520	33.52	114.4	0.364	2Φ 20
Suppor	E	11.49	250	352	0.019	346.1	Single	114.4	3520	95.43	114.4	0.364	2Φ 20
Span	b/nE&I	15.68	250	352	0.025	344	Single	114.4	3520	131.1	131.1	0.417	2Φ 20
Suppor	F	24.3	250	352	0.039	339.4	Single	114.4	3520	205.9	205.9	0.656	2Φ 20

Axis2	SRORY 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
Suppor	A	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nA&	38.81	250	352	0.063	331.3	Single	114.4	3520	336.8	336.8	1.072	2Φ 20
Suppor	B	60.33	250	352	0.097	318.6	Single	114.4	3520	544.4	544.4	1.734	2Φ 20
Span	b/nB&(	16.46	250	352	0.027	343.5	Single	114.4	3520	137.7	137.7	0.439	2Φ 20
Suppor	C	11.86	250	352	0.019	345.9	Single	114.4	3520	98.56	114.4	0.364	2Φ 20
Span	b/nC&	8.48	250	352	0.014	347.7	Single	114.4	3520	70.12	114.4	0.364	2Φ 20
Suppor	D	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nD&	4.08	250	352	0.007	349.9	Single	114.4	3520	33.52	114.4	0.364	2Φ 20
Suppor	E	29.08	250	352	0.047	336.8	Single	114.4	3520	248.3	248.3	0.791	2Φ 20
Span	b/nE&	15.54	250	352	0.025	344	Single	114.4	3520	129.9	129.9	0.414	2Φ 20
Suppor	F	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20

Axis2	SRORY 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
Suppor	A	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nA&	41.78	250	352	0.067	329.6	Single	114.4	3520	364.4	364.4	1.161	2Φ 20
Suppor	B	90.8	250	352	0.147	298.3	Single	114.4	3520	875.2	875.2	2.787	3Φ 20
Span	b/nB&(	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	C	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nC&	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	D	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nD&	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	E	31.97	250	352	0.052	335.2	Single	114.4	3520	274.2	274.2	0.873	2Φ 20
Span	b/nE&	16.16	250	352	0.026	343.7	Single	114.4	3520	135.2	135.2	0.43	2Φ 20
Suppor	F	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20

Appendix 73Reinforcement of a beam on axis 3

Axis3	SRORY 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
Suppor	A	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nA&I	15.52	250	352	0.025	344	Single	114.4	3520	129.7	129.7	0.413	2Φ 20
Suppor	B	17.69	250	352	0.029	342.9	Single	114.4	3520	148.3	148.3	0.472	2Φ 20
Span	b/nB&(	8.42	250	352	0.014	347.7	Single	114.4	3520	69.62	114.4	0.364	2Φ 20
Suppor	C	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nC&I	1.68	250	352	0.003	351.2	Single	114.4	3520	13.75	114.4	0.364	2Φ 20
Suppor	D	9.21	250	352	0.015	347.3	Single	114.4	3520	76.24	114.4	0.364	2Φ 20
Axis3	SRORY 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
Suppor	A	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nA&I	27.06	250	352	0.044	337.9	Single	114.4	3520	230.3	230.3	0.733	2Φ 20
Suppor	B	41.85	250	352	0.068	329.6	Single	114.4	3520	365.1	365.1	1.163	2Φ 20
Span	b/nB&(	19.85	250	352	0.032	341.7	Single	114.4	3520	167	167	0.532	2Φ 20
Suppor	C	28.28	250	352	0.046	337.2	Single	114.4	3520	241.1	241.1	0.768	2Φ 20
Span	b/nC&I	9.34	250	352	0.015	347.3	Single	114.4	3520	77.33	114.4	0.364	2Φ 20
Suppor	D	15.65	250	352	0.025	344	Single	114.4	3520	130.8	130.8	0.417	2Φ 20

Axis3	SRORY 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
Suppor	A	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nA&I	27.06	250	352	0.044	337.9	Single	114.4	3520	230.3	230.3	0.733	2Φ 20
Suppor	B	42.19	250	352	0.068	329.4	Single	114.4	3520	368.2	368.2	1.173	2Φ 20
Span	b/nB&(	19.81	250	352	0.032	341.8	Single	114.4	3520	166.6	166.6	0.531	2Φ 20
Suppor	C	16.77	250	352	0.027	343.4	Single	114.4	3520	140.4	140.4	0.447	2Φ 20
Span	b/nC&I	8.98	250	352	0.014	347.4	Single	114.4	3520	74.31	114.4	0.364	2Φ 20
Suppor	D	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20

Axis3	SRORY 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
Suppor	A	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nA&I	26.52	250	352	0.043	338.2	Single	114.4	3520	225.5	225.5	0.718	2Φ 20
Suppor	B	53.66	250	352	0.087	322.6	Single	114.4	3520	478.1	478.1	1.523	2Φ 20
Span	b/nB&(	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	C	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Span	b/nC&I	0	250	352	0.000	352	Single	114.4	3520	0	114.4	0.364	2Φ 20
Suppor	D	0	250	352	0	352	Single	114.4	3520	0	114.4	0.364	2Φ 20

Appendix 7AA

	shear reinforcement		AXIS A						
			STORY 4						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	27.5	25.76	238.55m	191	543.08	226	226	Φ 8 c/c 220mm
	Near 2	37.5	35.07	238.55m	191	398.91	226	226	Φ 8 c/c 220mm
2--3	Near 2	18.3	16.91	238.55m	191	827.31	226	226	Φ 8 c/c 220mm
	Near 3	17.5	16.11	238.55m	191	868.39	226	226	Φ 8 cc 220mm
			STORY 3						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	29.2	27.35	238.55m	191	511.51	226	226	Φ 8 c/c 220mm
	Near 2	45.8	42.88	238.55m	191	326.26	226	226	Φ 8 c/c 220mm
2--3	Near 2	24.3	22.4	238.55m	191	624.55	226	226	Φ 8 c/c 220mm
	Near 3	21.9	20.22	238.55m	191	691.88	226	226	Φ 8 c/c 220mm
			STORY 2						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	26.7	24.95	238.55m	191	560.71	226	226	Φ 8 c/c 220mm
	Near 2	46	43.07	238.55m	191	324.82	226	226	Φ 8 c/c 220mm
2--3	Near 2	24.6	22.66	238.55m	191	617.38	226	226	Φ 8 c/c 220mm
	Near 3	21.2	19.59	238.55m	191	714.13	226	226	Φ 8 c/c 220mm
			STORY 1						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	27.8	26.05	238.55m	191	537.04	226	226	Φ 8 c/c 220mm
	Near 2	47.2	44.22	238.55m	191	316.37	226	226	Φ 8 c/c 220mm
2--3	Near 2	21.5	19.79	238.55m	191	706.91	226	226	Φ 8 c/c 220mm
	Near 3	25.7	23.69	238.55m	191	590.54	226	226	Φ 8 c/c 220mm

Appendix 7BB

	shear reinforcement		AXIS B						
			STORY 4						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	30.6	28.59	238.55m	190.85	489.33	226	225.79	Φ 8 c/c 220mm
	Near 2	29	27.16	238.55m	190.85	515.09	226	225.79	Φ 8 c/c 220mm
2--3	Near 2	21.9	20.19	238.55m	190.85	692.91	226	225.79	Φ 8 c/c 220mm
	Near 3	19.5	18.02	238.55m	190.85	776.35	226	225.79	Φ 8 c/c 220mm
			STORY 3						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	41	38.4	238.55m	190.85	364.32	226	225.79	Φ 8 c/c 220mm
	Near 2	48.3	45.23	238.55m	190.85	309.30	226	225.79	Φ 8 c/c 220mm
2--3	Near 2	44.8	41.31	238.55m	190.85	338.65	226	225.79	Φ 8 c/c 220mm
	Near 3	37.8	34.87	238.55m	190.85	401.20	226	225.79	Φ 8 c/c 220mm
			STORY 2						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	40.5	37.86	238.55m	190.85	369.51	226	225.79	Φ 8 c/c 220mm
	Near 2	48.7	45.56	238.55m	190.85	307.06	226	225.79	Φ 8 c/c 220mm
2--3	Near 2	45.4	41.9	238.55m	190.85	333.89	226	225.79	Φ 8 c/c 220mm
	Near 3	37.8	34.84	238.55m	190.85	401.55	226	225.79	Φ 8 c/c 220mm
			STORY 1						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	8.12	7.6	238.55m	190.85	1840.77	226	225.79	Φ 8 c/c 220mm
	Near 2	57	53.35	238.55m	190.85	262.23	226	225.79	Φ 8 c/c 220mm
2--3	Near 2	49.2	45.39	238.55m	190.85	308.21	226	225.79	Φ 8 c/c 220mm
	Near 3	6.98	6.44	238.55m	190.85	2172.33	226	225.79	Φ 8 c/c 220mm

Appendix 7CC

	shear reinforcement		AXIS C					
		STORY 4						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	24.9	23.3	238.55	191	601.20	225.79	226	Φ 8 c/c 220mm
	Near 2	21.6	20.2	238.55	191	691.54	225.79	226	Φ 8 c/c 220mm
2--3	Near 2	15.1	13.94	238.55	191	1003.57	225.79	226	Φ 8 c/c 220mm
	Near 3	17.7	16.4	238.55	191	854.60	225.79	226	Φ 8 c/c 220mm
		STORY 3						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	30.8	28.8	238.55	191	485.42	225.79	226	Φ 8 c/c 220mm
	Near 2	31	29	238.55	191	482.24	225.79	226	Φ 8 c/c 220mm
2--3	Near 2	47.6	43.9	238.55	191	318.67	225.79	226	Φ 8 c/c 220mm
	Near 3	56.7	52.3	238.55	191	267.39	225.79	226	Φ 8 c/c 220mm
		STORY 2						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	31.3	29.3	238.55	191	477.47	225.79	226	Φ 8 c/c 220mm
	Near 2	30.2	28.3	238.55	191	494.34	225.79	226	Φ 8 c/c 220mm
2--3	Near 2	47.5	43.8	238.55	191	319.40	225.79	226	Φ 8 c/c 220mm
	Near 3	56.1	51.8	238.55	191	270.02	225.79	226	Φ 8 c/c 220mm
		STORY 1						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	64.6	60.4	238.55	191	231.47	225.79	226	Φ 8 c/c 220mm
	Near 2	94.1	88	238.55	191	158.90	225.79	226	Φ 8 c/c 220mm
2--3	Near 2	111	102	238.55	191	136.81	225.79	226	Φ 8 c/c 220mm
	Near 3	89.7	82.7	238.55	191	169.10	225.79	226	Φ 8 c/c 220mm

Appendix 7DD

	shear reinforcement (		AXIS D						
			STORY 4						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	21.5	20.1	238.55mm	190.85	695.32	225.79	226	Φ 8 c/c 220mm
	Near 2	19.51	18.3	238.55mm	190.85	766.15	225.79	226	Φ 8 c/c 220mm
2--3	Near 2	8.39	7.14	238.55mm	190.85	1959.36	225.79	226	Φ 8 c/c 220mm
	Near 3	17.19	15.9	238.55mm	190.85	882.08	225.79	226	Φ 8 c/c 220mm
			STORY 3						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	65.88	61.7	238.55mm	190.85	226.89	225.79	226	Φ 8 c/c 220mm
	Near 2	67.46	63.1	238.55mm	190.85	221.57	225.79	226	Φ 8 c/c 220mm
2--3	Near 2	46.82	43.2	238.55mm	190.85	323.76	225.79	226	Φ 8 c/c 220mm
	Near 3	62.21	57.4	238.55mm	190.85	243.68	225.79	226	Φ 8 c/c 220mm
			STORY 2						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	66.86	62.6	238.55mm	190.85	223.55	225.79	226	Φ 8 c/c 220mm
	Near 2	65.84	61.6	238.55mm	190.85	227.00	225.79	226	Φ 8 c/c 220mm
2--3	Near 2	45.71	42.2	238.55mm	190.85	331.67	225.79	226	Φ 8 c/c 220mm
	Near 3	63.7	58.8	238.55mm	190.85	238.00	225.79	226	Φ 8 c/c 220mm
			STORY 1						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	114.3	107	238.55mm	190.85	130.72	225.79	226	Φ 8 c/c 220mm
	Near 2	131.8	123	238.55mm	190.85	113.41	225.79	226	Φ 8 c/c 220mm
2--3	Near 2	132.7	122	238.55mm	190.85	114.29	225.79	226	Φ 8 c/c 220mm
	Near 3	104.5	96.4	238.55mm	190.85	145.14	225.79	226	Φ 8 c/c 220mm

Appendix 7EE

	shear reinforcement			AXIS E					
			STORY 4						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	25.5	23.9	238.55	191	585.35	226	226	Φ 8 c/c 220mm
	Near 2	26.9	25.2	238.55	191	556.03	226	226	Φ 8 c/c 220mm
			STORY 3						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	80.7	75.6	238.55	191	185.12	226	226	Φ 8 c/c 220mm
	Near 2	81.4	76.2	238.55	191	183.67	226	226	Φ 8 c/c 220mm
			STORY 2						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	83	77.7	238.55	191	180.17	226	226	Φ 8 c/c 220mm
	Near 2	83.6	78.2	238.55	191	178.81	226	226	Φ 8 c/c 220mm
			STORY 1						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
1--2	Near 1	106	99.1	238.55	191	141.21	226	226	Φ 8 c/c 220mm
	Near 2	109	102	238.55	191	136.63	226	226	Φ 8 c/c 220mm

Appendix 7FF

	shear reinforcement		(AXIS F					
			STORY 4					
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	33.6	31.5	238.55	191	444.40	226	225.79	Φ 8 c/c 220mm
	Near 2	31.9	29.9	238.55	191	468.51	226	225.79	Φ 8 c/c 220mm
			STORY 3					
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	96.8	90.6	238.55	191	154.43	226	225.79	Φ 8 c/c 220mm
	Near 2	97.8	91.5	238.55	191	152.91	226	225.79	Φ 8 c/c 220mm
			STORY 2					
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	97.6	91.4	238.55	191	153.15	226	225.79	Φ 8 c/c 220mm
	Near 2	98.5	92.2	238.55	191	151.72	226	225.79	Φ 8 cc 220mm
			STORY 1					
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
1--2	Near 1	95.3	89.2	238.55	191	156.91	226	225.79	Φ 8 c/c 220mm
	Near 2	98.3	92	238.55	191	152.06	226	225.79	Φ 8 c/c 220mm

Appendix 711

	shear reinforcement		AXIS 1						
			STORY 4						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
A--B	Near A	35.5	33.3	239	191	420.37	226	226	Φ 8 c/c 220mm
	Near B	33.8	31.7	239	191	441.32	226	226	Φ 8 c/c 220mm
B--C	Near B	18.7	16.74	239	191	835.71	226	226	Φ 8 c/c 220mm
	Near C	21.6	19.4	239	191	722.62	226	226	Φ 8 c/c 220mm
C--D	Near C	14.3	12.6	239	191	1113.84	226	226	Φ 8 c/c 220mm
	Near D	13.1	11.6	239	191	1211.24	226	226	Φ 8 c/c 220mm
D--E	Near D	8.34	6.66	239	191	2100.58	226	226	Φ 8 c/c 220mm
	Near E	5.35	4.27	239	191	3276.31	226	226	Φ 8 c/c 220mm
E--F	Near E	30.6	28.7	239	191	488.301	226	226	Φ 8 c/c 220mm
	Near F	34.9	32.6	239	191	429.531	226	226	Φ 8 c/c 220mm
			STORY 3						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
A--B	Near A	55.8	52.3	239	191	267.59	226	226	Φ 8 c/c 220mm
	Near B	49.8	46.6	239	191	300.08	226	226	Φ 8 c/c 220mm
B--C	Near B	15.6	14.23	239	191	983.12	226	226	Φ 8 c/c 220mm
	Near C	21.9	19.9	239	191	702.30	226	226	Φ 8 c/c 220mm
C--D	Near C	13	11.5	239	191	1221.82	226	226	Φ 8 c/c 220mm
	Near D	18.1	15.9	239	191	879.311	226	226	Φ 8 c/c 220mm
D--E	Near D	15.1	12.1	239	191	1158.1	226	226	Φ 8 c/c 220mm
	Near E	6.83	5.46	239	191	2562.24	226	226	Φ 8 c/c 220mm
E--F	Near E	38.9	36.4	239	191	384.336	226	226	Φ 8 c/c 220mm
	Near F	45.9	43	239	191	325.269	226	226	Φ 8 c/c 220mm

			STORY 2						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
A--B	Near A	54.49	51.05	238.55	190.9	274.04	225.8	225.8	Φ 8 c/c 220mm
	Near B	49.83	46.68	238.55	190.9	299.70	225.8	225.8	Φ 8 c/c 220mm
B--C	Near B	16.36	14.90	238.55	190.9	938.91	225.8	225.8	Φ 8 c/c 220mm
	Near C	20.86	18.99	238.55	190.9	736.69	225.8	225.8	Φ 8 c/c 220mm
C--D	Near C	13.87	12.19	238.55	190.9	1147.6	225.8	225.8	Φ 8 c/c 220mm
	Near D	18.08	15.89	238.55	190.9	880.42	225.8	225.8	Φ 8 c/c 220mm
D--E	Near D	11.99	9.58	238.55	190.9	1460.3	225.8	225.8	Φ 8 c/c 220mm
	Near E	10.17	8.12	238.55	190.9	1722.9	225.8	225.8	Φ 8 c/c 220mm
E--F	Near E	38.58	36.13	238.55	190.9	387.21	225.8	225.8	Φ 8 c/c 220mm
	Near F	44.31	41.5	238.55	190.9	337.1	225.8	225.8	Φ 8 c/c 220mm

			STORY 1						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
A--B	Near A	55.97	52.43	238.55	190.9	266.83	225.8	225.8	Φ 8 c/c 220mm
	Near B	50.18	47.01	238.55	190.9	297.59	225.8	225.8	Φ 8 c/c 220mm
B--C	Near B	24.65	22.45	238.55	190.9	623.16	225.8	225.8	Φ 8 c/c 220mm
	Near C	34.78	31.67	238.55	190.9	441.74	225.8	225.8	Φ 8 c/c 220mm
C--D	Near C	13.75	12.08	238.55	190.9	1158.1	225.8	225.8	Φ 8 c/c 220mm
	Near D	16.98	14.92	238.55	190.9	937.66	225.8	225.8	Φ 8 c/c 220mm
D--E	Near D	3.56	1.4	238.55	190.9	9992.7	225.8	225.8	Φ 8 c/c 220mm
	Near E	11.64	9.3	238.55	190.9	1504.3	225.8	225.8	Φ 8 c/c 220mm
E--F	Near E	55.92	52.37	238.55	190.9	267.13	225.8	225.8	Φ 8 c/c 220mm
	Near F	55.45	51.93	238.55	190.9	269.4	225.8	225.8	Φ 8 c/c 220mm

Appendix 722

	shear reinforcement			AXIS 2					
			STORY 4						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
A--B	Near A	23.5	22	238.55	191	635.32	226	226	Φ 8 c/c 220mm
	Near B	26.8	25.1	238.55	191	557.81	226	226	Φ 8 c/c 220mm
B--C	Near B	18.4	16.79	238.55	191	833.22	226	226	Φ 8 c/c 220mm
	Near C	15.8	14.4	238.55	191	974.22	226	226	Φ 8 c/c 220mm
C--D	Near C	10.7	9.44	238.55	191	1481.97	226	226	Φ 8 c/c 220mm
	Near D	12.9	11.3	238.55	191	1235.85	226	226	Φ 8 c/c 220mm
D--E	Near D	1.85	1.48	238.55	191	9452.59	226	226	Φ 8 c/c 220mm
	Near E	6.62	5.29	238.55	191	2644.58	226	226	Φ 8 c/c 220mm
E--F	Near E	36.2	33.9	238.55	191	412.923	226	226	Φ 8 c/c 220mm
	Near F	30.4	28.5	238.55	191	491.734	226	226	Φ 8 c/c 220mm
			STORY 3						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
A--B	Near A	84.5	79.2	238.55	191	176.71	226	226	Φ 8 c/c 220mm
	Near B	89.7	84.1	238.55	191	166.43	226	226	Φ 8 c/c 220mm
B--C	Near B	52.9	48.19	238.55	191	290.31	226	226	Φ 8 c/c 220mm
	Near C	49.7	45.2	238.55	191	309.37	226	226	Φ 8 c/c 220mm
C--D	Near C	22.2	19.5	238.55	191	716.692	226	226	Φ 8 c/c 220mm
	Near D	26.8	23.6	238.55	191	593.292	226	226	Φ 8 c/c 220mm
D--E	Near D	10.3	8.25	238.55	191	1695.74	226	226	Φ 8 c/c 220mm
	Near E	16.7	13.4	238.55	191	1047.14	226	226	Φ 8 c/c 220mm
E--F	Near E	42.8	40.1	238.55	191	348.961	226	226	Φ 8 c/c 220mm
	Near F	37.9	35.5	238.55	191	393.969	226	226	Φ 8 c/c 220mm

			STORY 2						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
A--B	Near A	85.4	80	239	191	174.92	226	226	Φ 8 c/c 220mm
	Near B	90.2	84.5	239	191	165.56	226	226	Φ 8 c/c 220mm
B--C	Near B	53.6	48.79	239	191	286.74	226	226	Φ 8 c/c 220mm
	Near C	48.8	44.5	239	191	314.66	226	226	Φ 8 c/c 220mm
C--D	Near C	22.4	19.7	239	191	710.14	226	226	Φ 8 c/c 220mm
	Near D	25.7	22.6	239	191	618.75	226	226	Φ 8 c/c 220mm
D--E	Near D	7.77	6.21	239	191	2252.8	226	226	Φ 8 c/c 220mm
	Near E	19	15.2	239	191	923.42	226	226	Φ 8 c/c 220mm
E--F	Near E	42.5	39.8	239	191	351.41	226	226	Φ 8 c/c 220mm
	Near F	37.6	35.2	239	191	396.99	226	226	Φ 8 c/c 220mm
			STORY 1						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
A--B	Near A	88.5	82.9	239	191	168.82	226	226	Φ 8 c/c 220mm
	Near B	111	104	239	191	134.88	226	226	Φ 8 c/c 220mm
B--C	Near B	66.8	60.83	239	191	229.98	226	226	Φ 8 c/c 220mm
	Near C	43.9	40	239	191	350.18	226	226	Φ 8 c/c 220mm
C--D	Near C	23.9	21	239	191	666.5	226	226	Φ 8 c/c 220mm
	Near D	19	16.7	239	191	839.22	226	226	Φ 8 c/c 220mm
D--E	Near D	38	33.4	239	191	418.98	226	226	Φ 8 c/c 220mm
	Near E	26.9	23.6	239	191	592.54	226	226	Φ 8 c/c 220mm
E--F	Near E	45	39.5	239	191	353.9	226	226	Φ 8 c/c 220mm
	Near F	36.9	35.2	239	191	396.99	226	226	Φ 8 c/c 220mm

Appendix 733

	shear reinforcement of		AXIS 3						
			STORY 4						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
A--B	Near A	24.6	23	238.55	190.9	607.46	226	226	Φ 8 c/c 220mm
	Near B	47.3	44.3	238.55	190.9	316.01	226	226	Φ 8 c/c 220mm
B--C	Near B	18.8	17.12	238.55	190.9	817.16	226	226	Φ 8 c/c 220mm
	Near C	16	14.5	238.55	190.9	966.15	226	226	Φ 8 c/c 220mm
C--D	Near C	6.86	6.03	238.55	190.9	2320	226	226	Φ 8 c/c 220mm
	Near D	11.7	10.3	238.55	190.9	1356.9	226	226	Φ 8 c/c 220mm
			STORY 3						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
A--B	Near A	60.2	56.4	238.55	190.9	248.22	226	226	Φ 8 c/c 220mm
	Near B	56.7	53.1	238.55	190.9	263.56	226	226	Φ 8 c/c 220mm
B--C	Near B	52.2	47.51	238.55	190.9	294.46	226	226	Φ 8 c/c 220mm
	Near C	58	52.8	238.55	190.9	265.01	226	226	Φ 8 c/c 220mm
C--D	Near C	31.5	27.7	238.55	190.9	505.23	226	226	Φ 8 c/c 220mm
	Near D	36	31.6	238.55	190.9	442.72	226	226	Φ 8 c/c 220mm

			STORY 2						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
A--B	Near A	58.44	54.75	238.6	191	255.52	226	226	Φ 8 c/c 220mm
	Near B	56.89	53.3	238.6	191	262.47	226	226	Φ 8 c/c 220mm
B--C	Near B	52.65	47.94	238.6	191	291.82	226	226	Φ 8 c/c 220mm
	Near C	57.14	29.64	238.6	191	471.99	226	226	Φ 8 c/c 220mm
C--D	Near C	32.55	28.6	238.6	191	489.2	226	226	Φ 8 c/c 220mm
	Near D	34.27	30.11	238.6	191	464.6	226	226	Φ 8 c/c 220mm
			STORY 1						
span	Loc	Vmax	Ved	Sl,max	Sb,ma	Scalc	Smin	Sprov	Remark
A--B	Near A	39.25	36.77	238.6	191	380.47	226	226	Φ 8 c/c 220mm
	Near B	34.3	32.13	238.6	191	435.41	226	226	Φ 8 c/c 220mm
B--C	Near B	23.19	21.12	238.6	191	662.40	226	226	Φ 8 c/c 220mm
	Near C	29.21	26.6	238.6	191	525.93	226	226	Φ 8 c/c 220mm
C--D	Near C	14.72	12.93	238.6	191	1082	226	226	Φ 8 c/c 220mm
	Near D	15.96	14.02	238.6	191	997.8	226	226	Φ 8 c/c 220mm

Appendix 8A 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
Third	C2	400	400	250	400	300	0	5572	5500	0	29.72	29.72	29.33	29.33	297.76	297.73
Third	C3	400	400	250	400	300	5570	3937	4561	5500	12.30	12.30	13.30	13.30	294.71	295.09
Third	C4	400	400	250	400	300	0	5572	0	451	29.72	29.72	2.41	2.41	297.76	276.36
Third	C5	400	400	250	400	300	5572	3937	5500	0	12.30	12.30	29.33	29.33	294.71	297.73
Third	C6	400	400	250	400	300	3937	2900	5500	0	8.91	8.91	29.33	29.33	292.79	297.73
Third	C7	400	400	250	400	300	2900	1751	5500	0	5.82	5.82	29.33	29.33	289.24	297.73
Third	C8	400	400	250	400	300	2900	1751	4561	5500	5.82	5.82	13.30	13.30	289.24	295.09
Third	C9	400	400	250	400	300	3937	2900	4561	5500	8.91	8.91	13.30	13.30	292.79	295.09
Third	C10	400	400	250	400	300	1751	5549	4561	5500	7.10	7.10	13.30	13.30	291.06	295.09
Third	C11	400	400	250	400	300	1751	5549	0	4561	7.10	7.10	24.33	24.33	291.06	297.28
Third	C12	400	400	250	400	300	2900	1751	0	4561	5.82	5.82	24.33	24.33	289.24	297.28
Third	C13	400	400	250	400	300	3937	2900	0	4561	8.91	8.91	24.33	24.33	292.79	297.28
Third	C14	400	400	250	400	300	5572	3937	0	4561	12.30	12.30	24.33	24.33	294.71	297.28
Third	C15	400	400	250	400	300	5549	0	4561	5500	29.59	29.59	13.30	13.30	297.75	295.09
Third	C16	400	400	250	400	300	5549	0	0	4561	29.59	29.59	24.33	24.33	297.75	297.28
Third	C17	400	400	250	400	300	0	5572	4561	5500	29.72	29.72	13.30	13.30	297.76	295.09
Second	C2	400	400	250	400	300	0	4150	3450	0	22.13	22.13	18.40	18.40	297.01	296.42
Second	C3	400	400	250	400	300	5570	4150	5700	3450	12.68	12.68	11.46	11.46	294.86	294.33
Second	C4	400	400	250	400	300	0	5040	3450	0	26.88	26.88	18.40	18.40	297.53	296.42
Second	C5	400	400	250	400	300	5572	5040	5700	3450	14.11	14.11	11.46	11.46	295.37	294.33
Second	C6	400	400	250	400	300	3937	5040	0	5700	11.79	11.79	30.40	30.40	294.48	297.81
Second	C7	400	400	250	400	300	2900	4150	4150	4150	9.10	9.10	11.07	11.07	292.94	294.14
Second	C8	400	400	250	400	300	2900	5040	5700	4700	9.82	9.82	13.74	13.74	293.43	295.24
Second	C9	400	400	250	400	300	3937	5040	0	5700	11.79	11.79	30.40	30.40	294.48	297.81
Second	C10	400	400	250	400	300	1751	4250	5700	4700	6.61	6.61	13.74	13.74	290.44	295.24
Second	C11	400	400	250	400	300	1751	4250	0	5700	6.61	6.61	30.40	30.40	290.44	297.81
Second	C12	400	400	250	400	300	2900	5040	5700	4700	9.82	9.82	13.74	13.74	293.43	295.24
Second	C13	400	400	250	400	300	3937	0	0	5700	21.00	21.00	30.40	30.40	296.85	297.81
Second	C14	400	400	250	400	300	5572	4150	3450	0	12.69	12.69	18.40	18.40	294.86	296.42
Second	C15	400	400	250	400	300	5549	4150	4700	3450	12.66	12.66	10.61	10.61	294.85	293.90
Second	C16	400	400	250	400	300	5549	4150	4150	4150	12.66	12.66	11.07	11.07	294.85	294.14
Second	C17	400	400	250	400	300	0	4150	0	4700	22.13	22.13	25.07	25.07	297.01	297.35

First	C2	400	400	250	400	300	0	4150	3450	0	22.13	22.13	18.40	18.40	297.01	296.42
First	C3	400	400	250	400	300	5570	4150	5700	3450	12.68	12.68	11.46	11.46	294.86	294.33
First	C4	400	400	250	400	300	0	5040	3450	0	26.88	26.88	18.40	18.40	297.53	296.42
First	C5	400	400	250	400	300	5572	5040	5700	3450	14.11	14.11	11.46	11.46	295.37	294.33
First	C6	400	400	250	400	300	3937	5040	0	5700	11.79	11.79	30.40	30.40	294.48	297.81
First	C7	400	400	250	400	300	2900	4150	4150	4150	9.10	9.10	11.07	11.07	292.94	294.14
First	C8	400	400	250	400	300	2900	5040	5700	4700	9.82	9.82	13.74	13.74	293.43	295.24
First	C9	400	400	250	400	300	3937	5040	0	5700	11.79	11.79	30.40	30.40	294.48	297.81
First	C10	400	400	250	400	300	1751	4250	5700	4700	6.61	6.61	13.74	13.74	290.44	295.24
First	C11	400	400	250	400	300	1751	4250	0	5700	6.61	6.61	30.40	30.40	290.44	297.81
First	C12	400	400	250	400	300	2900	5040	5700	4700	9.82	9.82	13.74	13.74	293.43	295.24
First	C13	400	400	250	400	300	3937	0	0	5700	21.00	21.00	30.40	30.40	296.85	297.81
First	C14	400	400	250	400	300	5572	4150	3450	0	12.69	12.69	18.40	18.40	294.86	296.42
First	C15	400	400	250	400	300	5549	4150	4700	3450	12.66	12.66	10.61	10.61	294.85	293.90
First	C16	400	400	250	400	300	5549	4150	4150	4150	12.66	12.66	11.07	11.07	294.85	294.14
First	C17	400	400	250	400	300	0	4150	0	4700	22.13	22.13	25.07	25.07	297.01	297.35
Grond Flo	C2	400	400	250	400	300	0	4150	3450	0	22.13	22.13	18.40	18.40	297.01	296.42
Grond Flo	C3	400	400	250	400	300	5570	4150	5700	3450	12.68	12.68	11.46	11.46	294.86	294.33
Grond Flo	C4	400	400	250	400	300	0	5040	3450	0	26.88	26.88	18.40	18.40	297.53	296.42
Grond Flo	C5	400	400	250	400	300	5572	5040	5700	3450	14.11	14.11	11.46	11.46	295.37	294.33
Grond Flo	C6	400	400	250	400	300	3937	5040	0	5700	11.79	11.79	30.40	30.40	294.48	297.81
Grond Flo	C7	400	400	250	400	300	2900	4150	4150	4150	9.10	9.10	11.07	11.07	292.94	294.14
Grond Flo	C8	400	400	250	400	300	2900	5040	5700	4700	9.82	9.82	13.74	13.74	293.43	295.24
Grond Flo	C9	400	400	250	400	300	3937	5040	0	5700	11.79	11.79	30.40	30.40	294.48	297.81
Grond Flo	C10	400	400	250	400	300	1751	4250	5700	4700	6.61	6.61	13.74	13.74	290.44	295.24
Grond Flo	C11	400	400	250	400	300	1751	4250	0	5700	6.61	6.61	30.40	30.40	290.44	297.81
Grond Flo	C12	400	400	250	400	300	2900	5040	5700	4700	9.82	9.82	13.74	13.74	293.43	295.24
Grond Flo	C13	400	400	250	400	300	3937	0	0	5700	21.00	21.00	30.40	30.40	296.85	297.81
Grond Flo	C14	400	400	250	400	300	5572	4150	3450	0	12.69	12.69	18.40	18.40	294.86	296.42
Grond Flo	C15	400	400	250	400	300	5549	4150	4700	3450	12.66	12.66	10.61	10.61	294.85	293.90
Grond Flo	C16	400	400	250	400	300	5549	4150	4150	4150	12.66	12.66	11.07	11.07	294.85	294.14
Grond Flo	C17	400	400	250	400	300	0	4150	0	4700	22.13	22.13	25.07	25.07	297.01	297.35

Appendix 8B slenderness check for column

location	column	Mx top	Mx botm	My top	My bot	NED	M01x	M01y	M02x	M02y	λx	λy	λxlim	λylim	slenderness(x)	slenderness(y)
Third	C2	13.66	14.38	30.38.	39.82	154.23	16.74	42.90	17.46	42.90	2.58	2.58	39.13	36.96	short	short
Third	C3	8.69	7.34	13.32	23.31	24.43	7.83	13.81	9.18	23.80	2.55	2.56	112.37	148.55	short	short
Third	C4	14.27	14.79	27.49	35.74	154.23	17.35	30.57	17.87	38.82	2.58	2.39	38.49	48.18	short	short
Third	C5	6.69	7.34	11.42	12.63	206.48	10.82	15.55	11.47	16.76	2.55	2.58	34.53	35.24	short	short
Third	C6	6.19	5.34	16.83	20.77	1067.25	26.69	38.18	27.54	42.12	2.54	2.58	14.67	15.93	short	short
Third	C7	13.14	20.37	5.97	10.77	111.84	15.38	8.21	22.61	13.01	2.50	2.58	63.23	66.28	short	short
Third	C8	13.14	20.36	7.48	8.52	54.45	14.23	8.57	21.45	9.61	2.50	2.56	92.11	71.82	short	short
Third	C9	6.19	5.34	16.83	20.77	107.84	7.50	18.99	8.35	22.93	2.54	2.56	50.63	55.05	short	short
Third	C10	2o.83	44.23	5.90	7.47	164.00	47.51	9.18	47.51	10.75	2.52	2.56	35.84	43.32	short	short
Third	C11	17.99	43.06	30.55	35.47	164.64	21.28	33.84	46.35	38.76	2.52	2.57	63.41	42.26	short	short
Third	C12	18.15	37.95	6.48	8.84	144.25	21.04	9.37	40.84	11.73	2.50	2.57	64.69	49.20	short	short
Third	C13	20.62	25.12	3.20	2.30	101.23	22.64	4.32	27.14	5.22	2.54	2.57	56.42	56.84	short	short
Third	C14	23.57	27.40	11.41	16.74	142.07	26.41	14.25	30.24	19.58	2.55	2.57	45.47	53.48	short	short
Third	C15	21.44	49.66	24.19	24.24	190.65	25.25	28.00	53.47	28.05	2.58	2.56	58.30	33.33	short	short
Third	C16	25.25	52.59	37.99	38.96	193.65	29.12	41.86	56.46	42.83	2.58	2.57	55.80	34.05	short	short
Third	C17	13.66	14.38	29.63	50.17	147.84	16.62	32.59	17.34	53.13	2.58	2.56	39.99	58.60	short	short
Second	C2	7.87	11.23	27.92	38.64	337.61	14.62	34.67	17.98	45.39	2.57	2.57	31.65	33.41	short	short
Second	C3	2.56	4.14	20.69	27.43	688.18	16.32	34.45	17.90	41.19	2.55	2.55	19.70	21.59	short	short
Second	C4	9.14	13.18	24.20	34.10	337.61	15.89	30.95	19.93	40.85	2.58	2.57	32.21	33.63	short	short
Second	C5	2.56	4.14	27.93	38.64	530.80	13.18	38.55	14.76	49.26	2.56	2.55	22.97	26.11	short	short
Second	C6	6.57	8.56	14.41	19.57	856.86	23.71	31.55	25.70	36.71	2.55	2.58	17.41	18.83	short	short
Second	C7	14.87	18.36	9.21	11.15	208.75	19.05	13.39	22.54	15.33	2.54	2.55	38.80	37.51	short	short
Second	C8	14.87	18.36	6.97	8.29	376.90	22.41	14.51	25.90	15.83	2.54	2.56	28.19	26.46	short	short
Second	C9	6.54	8.56	9.15	11.57	463.64	15.81	18.42	17.83	20.84	2.55	2.58	24.77	24.85	short	short
Second	C10	6.57	8.56	4.12	5.81	439.72	15.36	12.91	17.35	14.60	2.52	2.56	25.47	25.51	short	short
Second	C11	39.54	46.53	20.68	28.30	392.47	47.39	28.53	54.38	36.15	2.52	2.58	27.42	30.14	short	short
Second	C12	32.88	37.70	4.40	5.48	292.10	38.72	10.24	43.54	11.32	2.54	2.56	31.10	30.51	short	short
Second	C13	15.63	17.26	2.02	2.33	284.16	21.31	7.70	22.94	8.01	2.57	2.58	29.99	28.73	short	short
Second	C14	17.77	23.49	14.32	18.81	420.92	26.19	22.74	31.91	27.23	2.55	2.57	28.10	27.64	short	short
Second	C15	45.18	56.72	14.70	20.71	506.62	55.31	24.83	66.85	30.84	2.55	2.55	25.42	26.07	short	short
Second	C16	47.15	58.49	23.66	33.08	414.98	55.45	31.96	66.79	41.38	2.55	2.55	28.00	29.86	short	short
Second	C17	7.87	11.23	41.39	52.56	499.36	17.86	51.38	21.22	62.55	2.57	2.58	25.19	25.78	short	short

First	C2	9.83	13.65	26.66	30.95	519.16	20.21	37.04	24.03	41.33	2.57	2.57	24.72	23.13	short
First	C3	4.29	6.39	18.28	21.95	1139.90	27.09	41.08	29.19	44.75	2.55	2.55	14.99	15.19	short
First	C4	8.35	10.28	23.38	27.69	519.16	18.73	33.76	20.66	38.07	2.58	2.57	22.83	23.40	short
First	C5	4.29	6.39	7.92	11.01	530.88	14.91	18.54	17.01	21.63	2.56	2.55	23.43	23.99	short
First	C6	6.39	2.97	12.17	12.74	668.50	16.34	25.54	19.76	26.11	2.55	2.58	22.14	18.31	short
First	C7	15.06	27.15	7.92	12.75	362.51	22.31	15.17	34.40	20.00	2.54	2.55	36.21	32.42	short
First	C8	15.06	27.15	6.53	10.53	635.91	27.78	19.25	39.87	23.25	2.54	2.56	26.09	22.67	short
First	C9	6.39	2.96	7.10	7.59	760.48	18.17	22.31	21.60	22.80	2.55	2.58	20.42	17.15	short
First	C10	39.68	74.13	4.66	6.66	720.67	54.09	19.07	88.54	21.07	2.52	2.56	26.60	19.42	short
First	C11	39.67	75.03	20.93	29.27	649.27	52.66	33.92	88.02	42.26	2.52	2.58	28.35	23.09	short
First	C12	34.65	71.53	2.65	1.75	476.37	44.18	11.28	81.06	12.18	2.54	2.56	34.70	23.25	short
First	C13	20.47	50.81	1.27	0.79	426.21	28.99	9.31	59.33	9.79	2.57	2.58	38.47	23.79	short
First	C14	18.16	29.31	13.22	17.11	652.70	31.21	26.27	42.36	30.16	2.55	2.57	24.72	21.28	short
First	C15	41.89	65.81	15.93	23.19	819.31	58.28	32.32	82.20	39.58	2.55	2.55	22.70	20.24	short
First	C16	43.93	67.81	26.27	39.24	681.26	57.56	39.90	81.44	52.87	2.55	2.55	24.95	23.75	short
First	C17	9.83	13.65	39.56	59.35	801.95	25.87	55.60	29.69	75.39	2.57	2.58	19.19	22.29	short
Grond Floor	C2	8.55	4.28	41.35	19.84	713.45	18.55	34.11	22.82	55.62	2.57	2.57	21.78	26.68	short
Grond Floor	C3	14.16	7.14	41.93	22.02	1644.70	40.03	54.91	47.05	74.82	2.55	2.55	13.73	15.62	short
Grond Floor	C4	11.92	9.49	34.55	19.88	713.45	23.76	34.15	26.19	48.82	2.58	2.57	19.46	24.56	short
Grond Floor	C5	14.16	7.13	6.43	5.34	206.48	11.26	9.47	18.29	10.56	2.56	2.55	49.48	36.65	short
Grond Floor	C6	4.12	3.39	5.67	4.91	916.49	21.72	23.24	22.45	24.00	2.55	2.58	15.87	15.85	short
Grond Floor	C7	21.07	15.52	8.15	6.39	558.25	26.69	17.56	32.24	19.32	2.54	2.55	24.20	21.95	short
Grond Floor	C8	21.07	11.52	9.22	5.25	1088.84	33.30	27.03	42.85	31.00	2.54	2.56	18.34	16.45	short
Grond Floor	C9	4.12	3.39	4.26	3.42	1272.70	28.84	28.87	29.57	29.71	2.55	2.58	13.32	13.39	short
Grond Floor	C10	47.68	30.41	2.84	0.82	149.89	33.41	3.82	50.68	5.84	2.52	2.56	55.74	56.02	short
Grond Floor	C11	50.95	28.83	14.78	8.77	980.36	48.44	28.38	70.56	34.39	2.52	2.58	21.22	18.32	short
Grond Floor	C12	50.19	28.83	1.25	0.85	734.59	43.52	15.54	64.88	15.94	2.54	2.56	24.90	17.54	short
Grond Floor	C13	37.82	21.77	1.44	12.44	672.42	35.22	14.89	51.27	25.89	2.57	2.58	25.62	28.44	short
Grond Floor	C14	17.71	10.46	10.31	6.31	886.46	28.19	24.04	35.44	28.04	2.55	2.57	19.92	18.56	short
Grond Floor	C15	37.41	25.25	14.12	9.35	1132.35	47.90	32.00	60.06	36.77	2.55	2.55	17.58	16.17	short
Grond Floor	C16	42.38	22.36	24.31	12.74	952.00	41.40	31.78	61.42	43.35	2.55	2.55	21.80	20.55	short
Grond Floor	C17	8.54	4.28	6.13	7.88	1118.93	26.66	28.51	30.92	30.26	2.57	2.58	16.42	14.86	short

Appendix 8C reinforcement of column

colmn	location	MEDx	MEDy	Vsd	μsdy	μsdx	ω	As	As, min	As, max	As, prov	NO	Prov
Third	C2	17.46	42.90	0.1	0.06	0.02	0.3	1839.54	320	6400	1839.54	4.069779	5ø24
Third	C3	9.18	23.80	0.0	0.03	0.01	0.4	2452.72	320	6400	2452.72	5.426372	6ø24
Third	C4	17.87	38.82	0.1	0.05	0.02	0.7	4292.26	320	6400	4292.26	9.49615	10ø24
Third	C5	11.47	16.76	0.1	0.02	0.02	0.5	3065.9	320	6400	3065.9	6.782965	7ø24
Third	C6	27.54	42.12	0.6	0.06	0.04	0.3	1839.54	320	6400	1839.54	4.069779	5ø24
Third	C7	22.61	13.01	0.1	0.02	0.03	0.2	1226.36	320	6400	1226.36	2.713186	4ø24
Third	C8	21.45	9.61	0.0	0.01	0.03	0.19	1165.042	320	6400	1165.042	2.577527	4ø24
Third	C9	8.35	22.93	0.1	0.03	0.01	0.4	2452.72	320	6400	2452.72	5.426372	6ø24
Third	C10	47.51	10.75	0.1	0.01	0.07	1.2	7358.16	320	6400	7358.16	16.27912	17ø24
Third	C11	46.35	38.76	0.1	0.05	0.06	0.7	4292.26	320	6400	4292.26	9.49615	10ø24
Third	C12	40.84	11.73	0.1	0.02	0.06	0.3	1839.54	320	6400	1839.54	4.069779	5ø24
Third	C13	27.14	5.22	0.1	0.01	0.04	0.2	1226.36	320	6400	1226.36	2.713186	4ø24
Third	C14	30.24	19.58	0.1	0.03	0.04	0.21	1287.678	320	6400	1287.678	2.848845	4ø24
Third	C15	53.47	28.05	0.1	0.04	0.07	1.2	7358.16	320	6400	7358.16	16.27912	17ø24
Third	C16	56.46	42.83	0.1	0.06	0.08	1.3	7971.34	320	6400	7971.34	17.63571	18ø24
Third	C17	17.34	53.13	0.1	0.07	0.02	1.1	6744.98	320	6400	6744.98	14.92252	15ø24
Second	C2	17.98	45.39	0.2	0.06	0.02	0.3	1839.54	320	6400	1839.54	4.069779	5ø24
Second	C3	17.90	41.19	0.4	0.06	0.02	0.3	1839.54	320	6400	1839.54	4.069779	5ø24
Second	C4	19.93	40.85	0.2	0.06	0.03	0.32	1962.176	320	6400	1962.176	4.341097	5ø24
Second	C5	14.76	49.26	0.3	0.07	0.02	1.1	6744.98	320	6400	6744.98	14.92252	15ø24
Second	C6	25.70	36.71	0.5	0.05	0.04	0.4	2452.72	320	6400	2452.72	5.426372	6ø24
Second	C7	22.54	15.33	0.1	0.02	0.03	0.23	1410.314	320	6400	1410.314	3.120164	4ø24
Second	C8	25.90	15.83	0.2	0.02	0.04	0.2	1226.36	320	6400	1226.36	2.713186	4ø24
Second	C9	17.83	20.84	0.3	0.03	0.02	0.12	735.816	320	6400	735.816	1.627912	4ø24
Second	C10	17.35	14.60	0.2	0.02	0.02	0.11	674.498	320	6400	674.498	1.492252	4ø24
Second	C11	54.38	36.15	0.2	0.05	0.07	0.61	3740.398	320	6400	3740.398	8.275217	9ø24
Second	C12	43.54	11.32	0.2	0.02	0.06	0.45	2759.31	320	6400	2759.31	6.104668	7ø24
Second	C13	22.94	8.01	0.2	0.01	0.03	0.26	1594.268	320	6400	1594.268	3.527142	4ø24
Second	C14	31.91	27.23	0.2	0.04	0.04	0.5	3065.9	320	6400	3065.9	6.782965	7ø24
Second	C15	66.85	30.84	0.3	0.04	0.09	0.7	4292.26	320	6400	4292.26	9.49615	10ø24
Second	C16	66.79	41.38	0.2	0.06	0.09	0.79	4844.122	320	6400	4844.122	10.71708	11ø24
Second	C17	21.22	62.55	0.3	0.09	0.03	0.6	3679.08	320	6400	3679.08	8.139558	9ø24

First	C2	24.03	41.33	0.3	0.06	0.03	0.5	3065.9	320	6400	3065.9	6.782965	7ø24
First	C3	29.19	44.75	0.6	0.06	0.04	0.4	2452.72	320	6400	2452.72	5.426372	6ø24
First	C4	20.66	38.07	0.3	0.05	0.03	0.31	1900.858	320	6400	1900.858	4.205438	5ø24
First	C5	17.01	21.63	0.3	0.03	0.02	0.15	919.77	320	6400	919.77	2.034889	4ø24
First	C6	19.76	26.11	0.4	0.04	0.03	0.2	1226.36	320	6400	1226.36	2.713186	4ø24
First	C7	34.40	20.00	0.2	0.03	0.05	0.32	1962.176	320	6400	1962.176	4.341097	5ø24
First	C8	39.87	23.25	0.4	0.03	0.05	0.32	1962.176	320	6400	1962.176	4.341097	5ø24
First	C9	21.60	22.80	0.4	0.03	0.03	0.3	1839.54	320	6400	1839.54	4.069779	5ø24
First	C10	88.54	21.07	0.4	0.03	0.12	0.9	5518.62	320	6400	5518.62	12.20934	13ø24
First	C11	88.02	42.26	0.4	0.06	0.12	0.9	5518.62	320	6400	5518.62	12.20934	413ø24
First	C12	81.06	12.18	0.3	0.02	0.11	0.79	4844.122	320	6400	4844.122	10.71708	11ø24
First	C13	59.33	9.79	0.2	0.01	0.08	0.6	3679.08	320	6400	3679.08	8.139558	9ø24
First	C14	42.36	30.16	0.4	0.04	0.06	0.3	1839.54	320	6400	1839.54	4.069779	5ø24
First	C15	82.20	39.58	0.5	0.05	0.11	0.57	3495.126	320	6400	3495.126	7.73258	8ø24
First	C16	81.44	52.87	0.4	0.07	0.11	0.6	3679.08	320	6400	3679.08	8.139558	9ø24
First	C17	29.69	75.39	0.4	0.10	0.04	0.25	1532.95	320	6400	1532.95	3.391482	4ø24
Grond Floor	C2	22.82	55.62	0.4	0.08	0.03	0.6	3679.08	320	6400	3679.08	8.139558	9ø24
Grond Floor	C3	47.05	74.82	0.9	0.10	0.06	0.4	2452.72	320	6400	2452.72	5.426372	6ø24
Grond Floor	C4	26.19	48.82	0.4	0.07	0.04	0.3	1839.54	320	6400	1839.54	4.069779	5ø24
Grond Floor	C5	18.29	10.56	0.1	0.01	0.03	0.21	1287.678	320	6400	1287.678	2.848845	4ø24
Grond Floor	C6	22.45	24.00	0.5	0.03	0.03	0.33	2023.494	320	6400	2023.494	4.476757	5ø24
Grond Floor	C7	32.24	19.32	0.3	0.03	0.04	0.36	2207.448	320	6400	2207.448	4.883735	5ø24
Grond Floor	C8	42.85	31.00	0.6	0.04	0.06	0.44	2697.992	320	6400	2697.992	5.969009	6ø24
Grond Floor	C9	29.57	29.71	0.7	0.04	0.04	0.3	1839.54	320	6400	1839.54	4.069779	5ø24
Grond Floor	C10	50.68	5.84	0.1	0.01	0.07	0.5	3065.9	320	6400	3065.9	6.782965	7ø24
Grond Floor	C11	70.56	34.39	0.5	0.05	0.10	0.9	5518.62	320	6400	5518.62	12.20934	13ø24
Grond Floor	C12	64.88	15.94	0.4	0.02	0.09	0.2	1226.36	320	6400	1226.36	2.713186	4ø24
Grond Floor	C13	51.27	25.89	0.4	0.04	0.07	0.5	3065.9	320	6400	3065.9	6.782965	7ø24
Grond Floor	C14	35.44	28.04	0.5	0.04	0.05	0.32	1962.176	320	6400	1962.176	4.341097	5ø24
Grond Floor	C15	60.06	36.77	0.6	0.05	0.08	0.6	3679.08	320	6400	3679.08	8.139558	9ø24
Grond Floor	C16	61.42	43.35	0.5	0.06	0.08	0.62	3801.716	320	6400	3801.716	8.410876	9ø24
Grond Floor	C17	30.92	30.26	0.6	0.04	0.04	0.3	1839.54	320	6400	1839.54	4.069779	5ø24