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STRUCTURAL DESIGN ON G+3 HOTEL BUILDING

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Summary

This project deals about the structural analysis and design of a G+3 building for hotel use considering all the internal and external effects according to ES EN 2015. It has 10 chapters and the contents and the duties accomplished in each chapter are explained below.

The first chapter deals about the general introduction of the basic structural components of a building, slab analysis & design approaches, and the main objective of the project including lay out of the report.

In the second chapter discuss the Roof analysis and design. The external wind pressure coming from different directions were collected and transferred to frames according to ES EN 2015

The third chapter deals about analysis and design of slab on the two method of slab analysis (i.e. coefficient & strip method). The slabs including two-way, one-way and special slabs which are treated as cantilever beam. The depths of all the slabs are made the same for construction simplicity and reinforcement of each is determined using ES EN 2015

The fourth chapter focuses on the analysis and design of Stair case,

Lateral load analysis in the fifth chapter. This chapter is about the calculation of lateral forces which are earthquakes loads and wind load on walls. The weight of the building was computed by considering all elements from small to large. The center of mass and center of stiffness were computed by assuming preliminary sections

The sixth chapter deals on the analysis of the frame of the structure. The building model was analyzed using ETABS V-9.6as space frame taking seven combinations for the existing Lateral and vertical loads.

The Seventh chapter deals with beam design for flexure and shear. All the beams of the building are designed for the envelope load combination with maximum negative moment at supports and maximum positive moment along span.

The 8th chapter deals with column design. All the columns have been checked whether they sway or non-sway and whether they are slender or short.

The chapter deals about analysis and design of foundation. We have calculated the bearing capacity of the soil profile on which our building is going to build.

Finally, 10th chapter contains Cost estimation of the building.

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Symbols

For the purposes of this standard, the following symbols apply.

A	Cross sectional area
A_c	Cross sectional area of concrete
A_p	Area of a prestressing tendon or tendons
A_s	Cross sectional area of reinforcement
$A_{s,min}$	minimum cross sectional area of reinforcement
A_{sw}	Cross sectional area of shear reinforcement
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
dg	Largest nominal maximum aggregate size
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
f_{tk}	Characteristic tensile strength of reinforcement
f_y	Yield strength of reinforcement
f_{yd}	Design yield strength of reinforcement
f_{yk}	Characteristic yield strength of reinforcement

1 Introduction

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

A structural design is executed in such a way that the building will remain fit with appropriate degrees of reliability and in an economic way. It should sustain all the actions and influences during execution and use. Therefore, structural design focuses on structural safety and serviceability with due durability. It must also optimize the cost expended in building the structure and maintenance.

This structural design is executed based on the Ethiopian Building Code of Practice (ES EN 2015)

This code follows the Limit State design approach. Limit state is a state beyond which the structure no longer satisfies the design performance requirements. 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.

Structural design is a very essential science in many civil engineering works for the safety of the design and the economic aspect of construction. In this building we design structural members for **G+3** building for Hotel. The drawing contains many divided slab panels for different uses and roof cover above floors. The design must check that the structure is capable of carrying the loads safely and that it will not deform excessively due to the applied loads. So we design and analyse as well as check deformation due to its weight and other external loads such as wind load and earth quick.

1.1 Design criteria for building

To analyse or design a structure, it is necessary to establish criteria for determining whether a given structure is acceptable for use in a specified circumstance or for use directly as a design objective that must be met. The most important criteria are; Serviceability. Efficiency, Construction, Costs, Other

The structure must be able to carry the design load safely without excessive material distress and with deformations with in an acceptable range. The ability of a structure to carry loads safely and without material distress is achieved by using safety factors in the design of the element. By altering the size, shape, and choice of material, stresses in a structure can be maintained at safe levels and such that material distress(e.g. cracking)does not occur. This is basically a strength criterion and is of fundamental importance.

1.2 Objectives

The main purpose of this project is to enable the group members to apply the knowledge gained in class and be able to analyse and design buildings.

In addition, the project will enable the group members to have the know-how of analysis and design of the building, soil investigation and selection of the best foundation for a given site and structure.

1.3 Scope of the project

The primary aim of all structural design is to insure that the structure will performs satisfactorily during its design life. Specifically, the design must check that the structure is capable of carrying the loads safely and that it will not deform excessively due to the applied loads. This requires the designer to make realistic estimates of the strengths of the materials composing the structure and the loading to which it may be subjected during its design life. Furthermore, the designer will need a basic understanding of structural behavior. The designer must make an assessment of the future likely level of loading, including self-weight, to which the structure may be subjected during its design life. Using computer methods or hand calculations the design loads acting on individual elements can be then be evaluated. The design loads are used to calculate the bending moments, shear forces and deflections at critical points along the elements. Finally, suitable dimensions for the element can be determined.

1.4 Material and Methodology

1.4.1 Location of structure

Building is located in southern Ethiopia Oromia region yabello Woreda 02 kebele

1.4.2 Analysis and design process

The overall structural analysis and design process typically consists of several steps

a) **Geometry definition: -** the basic geometry of structure is defined first, with particular attention paid to member hierarchies (which member supports which other members) and spanning directions.
b) **Load assessment:** - the loads acting on the structure are determined next. Typically, this involves determining loadings associated with the so called live loads on the structure resulting from its occupancy (e.g. loadings do to wind and earth quake forces) and the so called dead loads associated with the self waits of building elements.
c) **Modeling of the structure and boundary Conditions:-**the structure and its constituent elements are modeled. Typically, such modeling includes characterizing complex real world construction connections consisting of items such as anchor plates, bolts, etc as one or another an idealized set of support conditions (e.g. pins, rollers, or rigid joints)

❖ **We will follow the following steps under this project:**

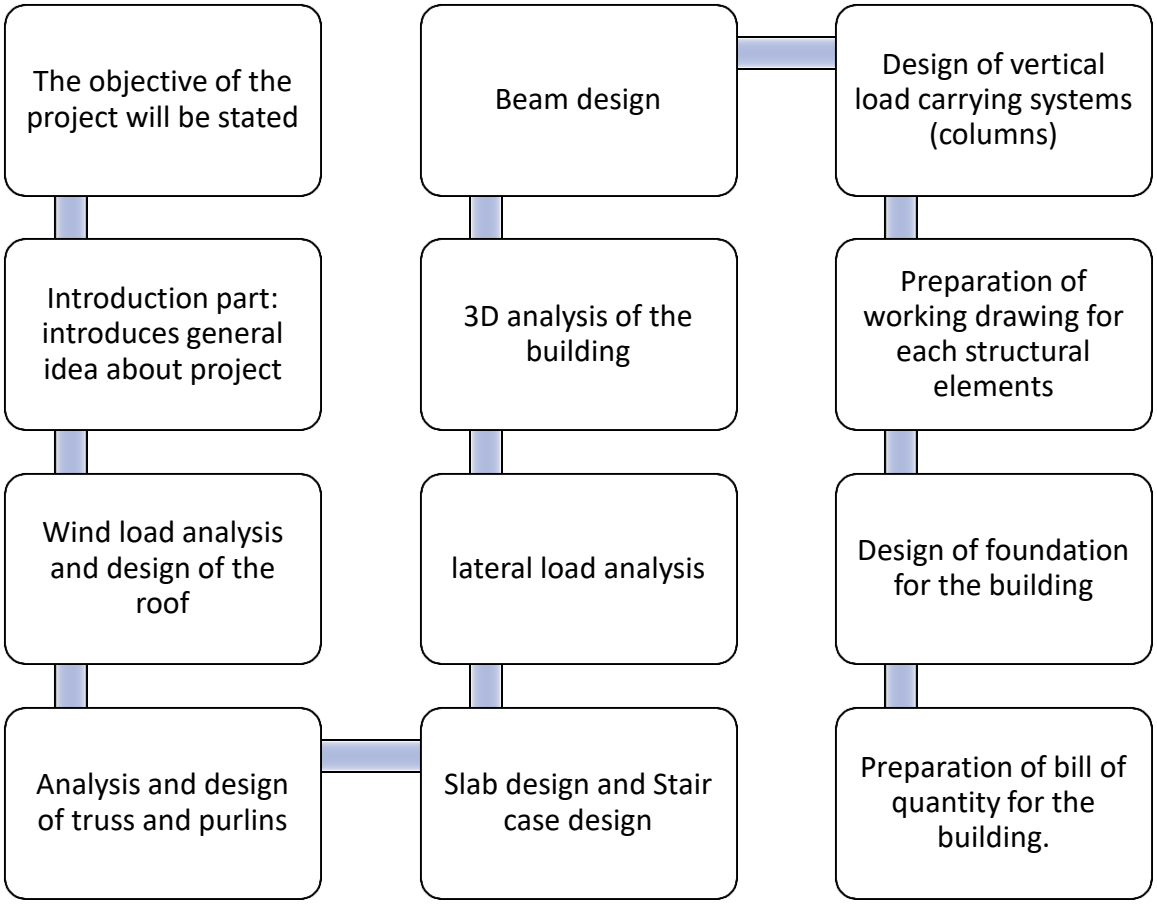


Figure 1 Steps to analysis

1.4.3 Design methods and analysis

The basic principles of analysis and design of structures will be employed in this project. During our task we will use different software including ETABS V9, AutoCAD 2007, Safe, Excel, Zotero and others to facilitate our task.

- Component of structural analysis and design

2 Roof design

Roof design which involves:

Generally, we should follow the following steps in analysing 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

ES EN 1991:2015, part 1-4 gives guidance on the determination of natural wind actions for structural design of buildings and civil engineering works for each of the loaded areas under consideration.

Determination of basic wind velocity (vb) [1]

$$Vb = Cdir * Csea * Vbo \dots\dots [2]$$

In which

$Cdir = direction\ factor$

$Csea = Seasonal\ factor$

$Vbo = fundamental\ basic\ wind\ velocity$

Determination of mean wind velocity, vm (z) [1]

External Pressure coefficients

The external pressure coefficients are divided into local coefficients and overall coefficients.

Local coefficients, Cpe1 give the pressure coefficients for loaded areas of 1m2.They may be used for the design of small elements and fixings.

Overall coefficients, Cpe 10 give the pressure coefficients for loaded areas of 10 m2.They may be used for loaded areas larger than 10 m2

The external pressure coefficients Cpe for buildings and parts of buildings depend on the size of the loaded area A, which is the area of the structure that produces the wind action in the section to be calculated.

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 A. The figure 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 Tables from ES-EN 1991,1-4:2014 SECTION 7.

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 due to 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 as illustrated in Fig.4.5

(a). Conversely, if the leeward face has more openings, then the interior is subjected to a negative (inward) pressure as ill ustrated in Fig. 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.

Wind load analysis

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

- Determine site design wind speed and basic velocity pressure
- Identify the roof type
- Determine wind pressure coefficients namely ext ernal 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 yabelo city. Therefore, according to ES-EN 1991,1 4:2014 TABLE 4.1 the area is categorized according to the following:

Terrain Category: According to ES-EN 1991,1-4:2014 TABLE 4.1 the location of the building in yabelo fall under Terrain Category- III.

Category III is characterized with

- Roughness length, z0 = 0.3m
- Minimum height, zmin = 5m

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

therefore,

Ct (z) = l according to terrain condition

Height of the building is 15.72m

Width of the roof 15.8m

Dynamic coefficient, Cd: According to Figure 3.7 of EBCS-1, 1995 dynamic coefficient is approximately 0.96 which is less than 1.2. Therefore we can apply static method of analysis of wind load.

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:

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

$Cr\ (z) = Cr(zmin)$, $for\ z < zmin$

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*$(Zo/Z0, II)^{0.07}$

 Bcalculation of mean windvelocity (Vm(zo))

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

Z= 15.72m

for terrian category III

Zmin= 5

Zo= 0.3

Zo,II= 0.05

roughness coefficient Cr(z)

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

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

= 0.19*(0.3/0.05)^0.07

= 0.215

Cr(z)= 0.215ln(15.72/0.3) =0.85

Orographic factor (Co(z))

Co(z)= 1 , Ø<0.05

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

$\text{Vm}(z) = (z) * (z) * vb$

Vm(z)= 0.85*1*22

= 18.7m/s

Altitude

Temperature and Barometer

But ES EN1991-1-4-2015 recommended as 1.25kg/m^3

 calculation of peak velocity pressure (qp (z))

$qp(z) = [1 + 7Iv(z)] * 1/2 * \rho air * vm^2(z)$

ρair=density of air, it is depend on :- Altitude Temperature and Barometer But ES EN1991-1-4-2015 recommended as 1.25kg/m^3

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(15.72/0.3)]

= 0.25

q(15.72) = [1+7*0.25]*1/2*1.25*18.7^2

= 0.6 Kpa

Wind load analysis for Roof 1 (Hipped roof) @ 15.72m

Determination of external pressure coefficients

External wind pressure coefficient cpe are derived from Appendix A. of ES-EN-2015

The roof analyzed in both direction of wind which is Θ= 0^0 &Θ= 90^0

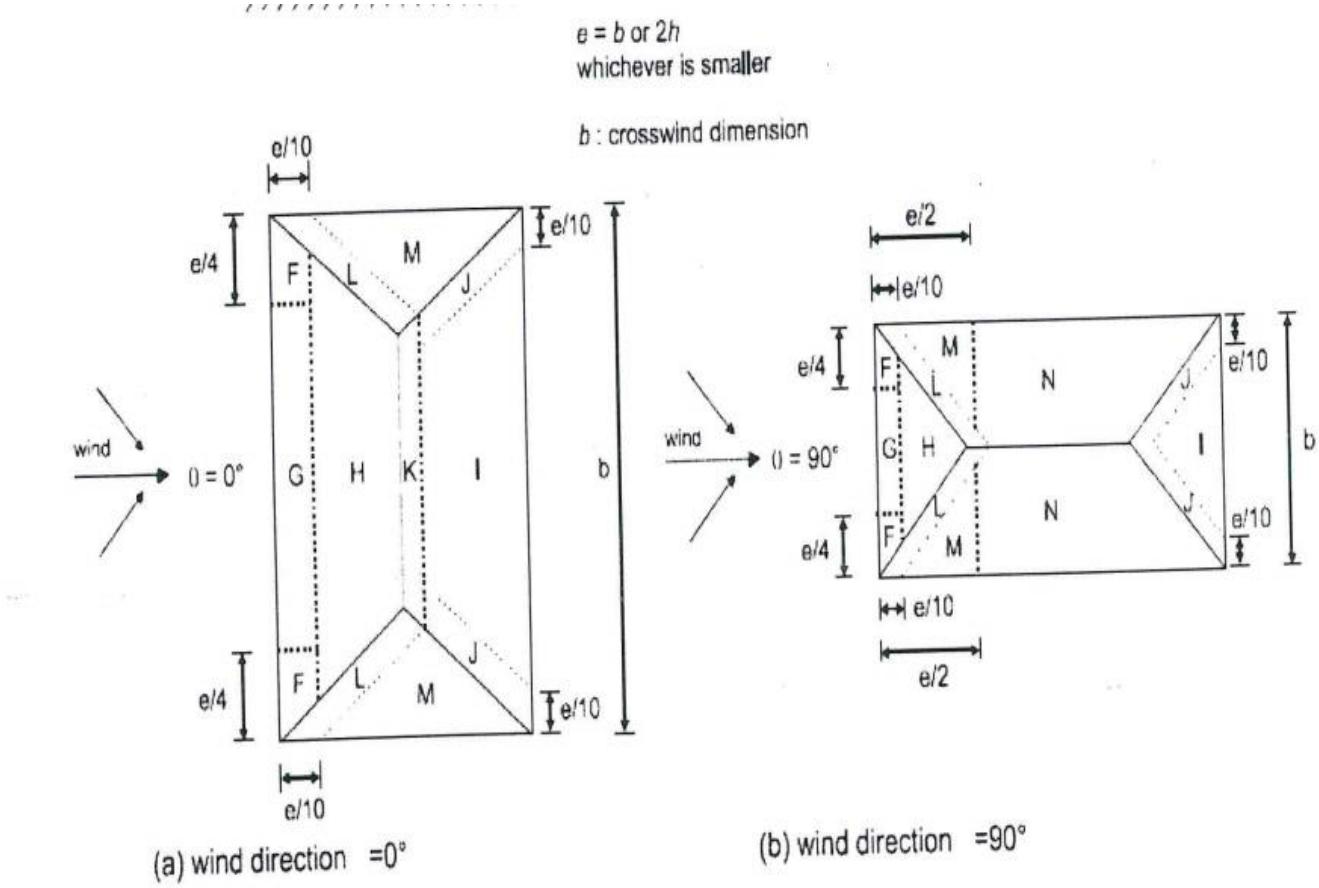


Figure 2 Key for hipped roof

Areas calculation

Table 1Areas of zones of the hipped roof for wind direction θ=0^0 in m^2

Zones	F	G	H	I	J	K	L	M
Areas	19.75	85	158.29	158.2	22.2	85	32.03	18.53

Table 2 Areas of zones of the hipped roof for wind direction θ=90^0 in m^2

Zones	F	G	H	I	J	L	M	N
Areas	19.75	0.25	15.52	15.52	17.52	32.03	29.94	119.93

Determination of pitch angle

Tanα=1.88/7.9, α=13.4^0

To calculate the external pressure coefficient

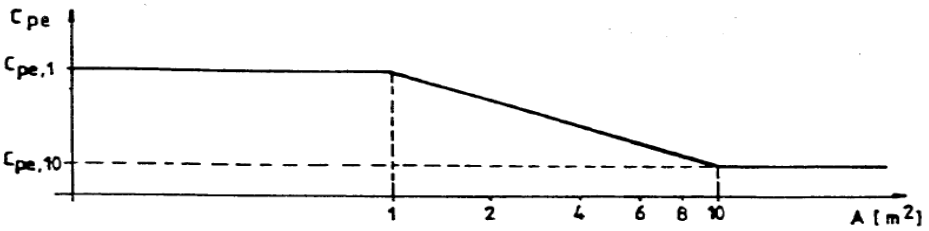


Figure A.1 Variation of External Pressure Coefficient for Buildings with Size of the Loaded Area *A*.

Note: The Figure is based on the following:

$c_{pe} = c_{pe,1}$ $\quad A \leq 1m^2$

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

$A \geq 10m^2$ $\quad c_{pe} = c_{pe,10}$

Activate Winc

Figure 3 Variation of external pressure coefficient

Table 3 External Pressure coefficients for hipped roof for wind direction θ=0^0

Zones	F	G	H	I	J	K	L	M
Area	19.75	85	158.29	158.2	22.2	85	32.03	18.53
CPe-ve	-0.5	-0.48	-0.35	-0.37	-0.7	-0.9	-0.98	-0.4
Cpe+ve	0.2	0.2	0.2					

Table 4 External Pressure coefficients for hipped roof for wind direction $\theta=90^0$

Zones	F	G	H	I	J	L	M	N
Area	19.75	0.25	15.52	15.52	17.52	32.03	29.94	119.93
CPe-ve	-0.5	-0.94	-0.35	-0.37	-0.7	-0.98	-0.4	-0.2
Cpe +ve	0.2	0.2	0.2					

Internal pressure coefficient

For closed buildings with internal partitions and opening windows the extreme values are

CPi=+0.2 or CPi= -0.3 on ESEN 1991-1-4:201

Table 5 Net External Pressure for hipped roof for wind direction $\theta=0^0$

Zones	F	G	H	I	J	K	L	M
Area	19.75	85	158.29	158.2	22.2	85	32.03	18.53
CPe-ve	-0.5	-0.48	-0.35	-0.37	-0.7	-0.9	-0.98	-0.4
Cpe+ve	0.2	0.2	0.2					
CPi -ve	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
Cpi+ve	+0.2	+0.2	+0.2	+0.2	+0.2	+0.2	+0.2	+0.2

CPe-Cpi	-0.7	-0.68	-0.55	-0.57	-0.9	-1.1	-1.18	-0.6
Cpe-Cpi	0.5	0.5	0.5	0.3	0.3	0.3	0.3	0.3

Hence maximum of cpe-cpi for suction and cpe-cpi for compression is -1.18 and 0.5 respectively

Wnet critical negative = 0.6*-1.18= -0.708 KN/m^2

Wnet critical positive = 0.6*0.5= 0.3 KN/m^2

Table 6 Net External Pressure for hipped roof for wind direction $\theta=90^0$

Zones	F	G	H	I	J	L	M	N
Area	19.75	0.25	15.52	15.52	17.52	32.03	29.94	119.93
CPe-ve	-0.5	-0.94	-0.35	-0.37	-0.7	-0.98	-0.4	-0.2
Cpe+ve	0.2	0.2	0.2					
CPi -ve	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
Cpi+ve	+0.2	+0.2	+0.2	+0.2	+0.2	+0.2	+0.2	+0.2
CPe-Cpi	-0.7	-1.14	-0.55	-0.57	-0.9	-1.18	-0.6	-0.4
Cpe-Cpi	0.5	0.5	0.5	0.3	0.3	0.3	0.3	0.3

Hence maximum of cpe-cpi for suction and cpe-cpi for compression is -1.18 and 0.5 respectively

Wnet critical negative = 0.6*-1.18= -0.708 KN/m^2

Wnet critical positive = 0.6*0.5= 0.3 KN/m^2

Wind load analysis for Mono pitched roof @ 3.84m

The building is located in yabelo city. Therefore, according to ES-EN 1991,1 4:2014 TABLE 4.1 the area is categorized according to the following:

Terrain Category: According to ES-EN 1991,1-4:2014 TABLE 4.1 the location of the building in yabelo fall under Terrain Category- III.

Category III is characterized with

- Roughness length, z0 = 0.3m
- Minimum height, zmin = 5m

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

therefore,

Ct (z) = l according to terrain condition

Height of the building is 5.44m

Width of the roof 8.9m

Dynamic coefficient, Cd: According to Figure 3.7 of EBCS-1, 1995 dynamic coefficient is approximately 0.96 which is less than 1.2. Therefore we can apply static method of analysis of wind load

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:

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

$Cr\ (z) = \ Cr(zmin),\ \ for\ z <\ zmin$

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*$(Zo/Z0, II)^{0.07}$

 Bcalculation of mean windvelocity (Vm(zo))

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

Z= 15.72m

for terrian category III

Zmin= 5

Zo= 0.3

Zo,II= 0.05

roughness coefficient Cr(z)

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

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

= 0.19*(0.3/0.05)^0.07

= 0.215

Cr(z)= 0.215ln(5.44/0.3) =0.6

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

$\text{Vm}\ (z) = (z) * (z) * vb$

Vm(z)= 0.6*1*22

= 13.2m/s

Altitude

Temperature and Barometer

But ES EN1991-1-4-2015 recommended as 1.25kg/m^3

 calculation of peak velocity pressure (qp (z))

qp(z) = [1 + 7Iv(z)] ∗1/2∗ ρair ∗ $vm^2(z)$

ρair=density of air, it is depend on :- Altitude Temperature and Barometer But ES EN1991-1-4-2015 recommended as 1.25kg/m^3

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(5.44/0.3)]

= 0.35

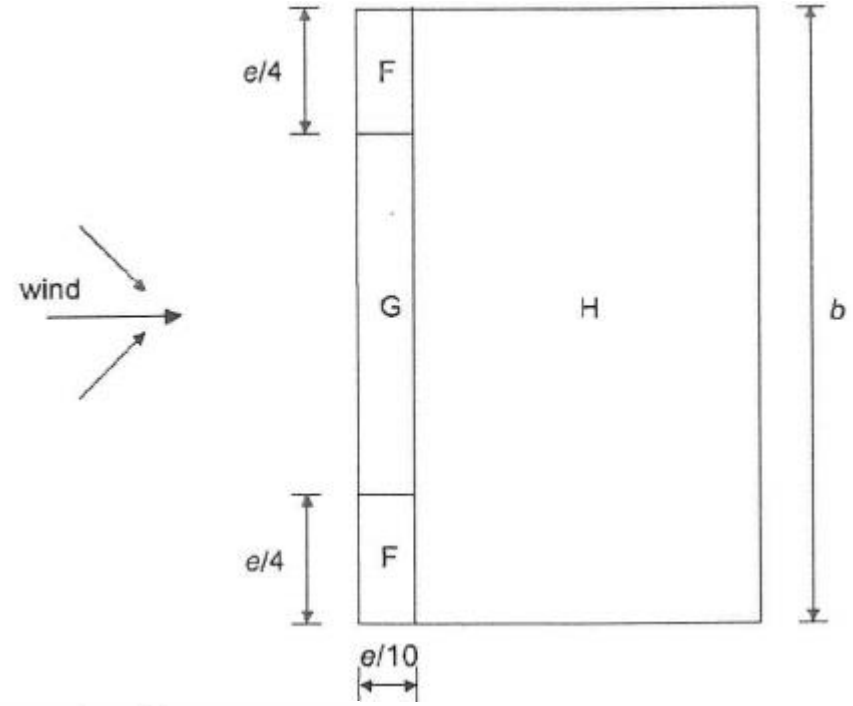
q(5.44) = [1+7*0.35]*1/2*1.25*13.2^2=375.7pa

= 0.4 Kpa

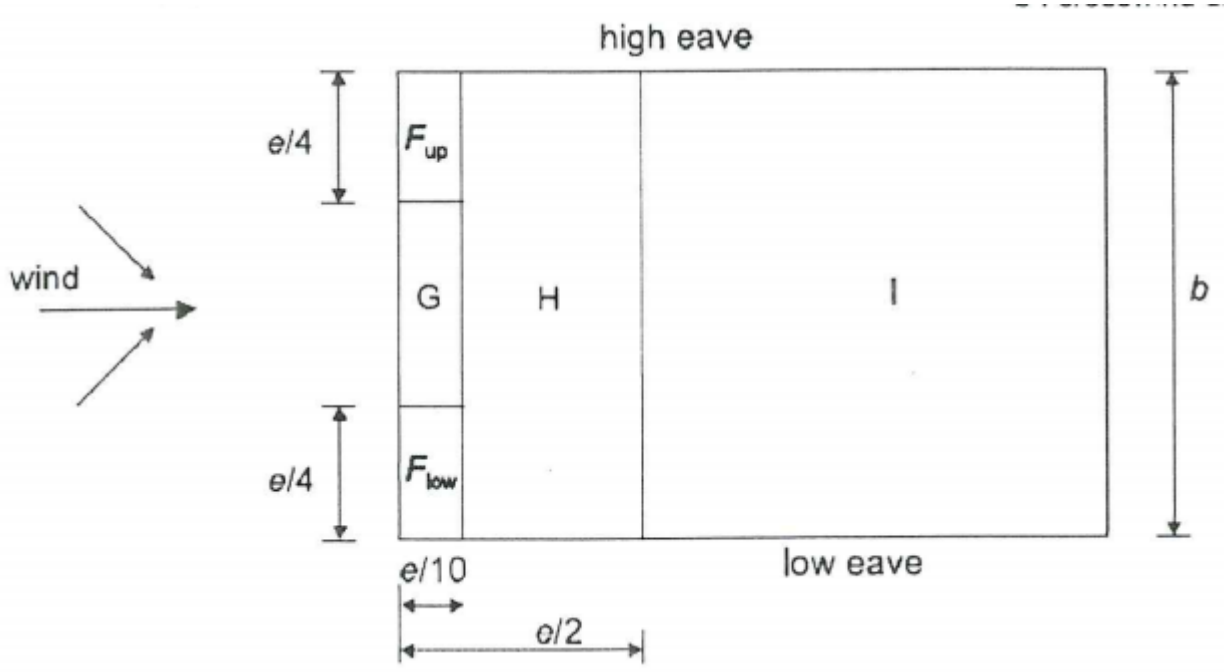
Determination of external pressure coefficients

External wind pressure coefficient cpe are derived from Appendix A. of ES-EN-2015

The roof analysed in both direction of wind which is Θ= 0^0 Θ= 180^0 Θ= 90^0



When Θ= 0^0 Θ= 180^0



When Θ= 90^0

Figure 4 Key for monopitch roof

Table 7 Areas calculation

Areas of zones of the mono pitched roof for wind direction ɵ=0^0 in m^2

Zones	F	G	H
Areas	2.96	8.46	103.1

Areas of zones of the mono pitched roof for wind direction ɵ=180^0 in m^2

Zones	F	G	H
Areas	2.96	8.46	103.1

Areas of zones of the mono pitched roof for wind direction ɵ=90^0 in m^2

Zones	F_{up}	F_{low}	G	H	I
Areas	2.96	2.96	8.46	57.42	71.81

Determination of pitch angle

Tanα=1.6/8.9, α=10.2^0

To calculate the external pressure coefficient

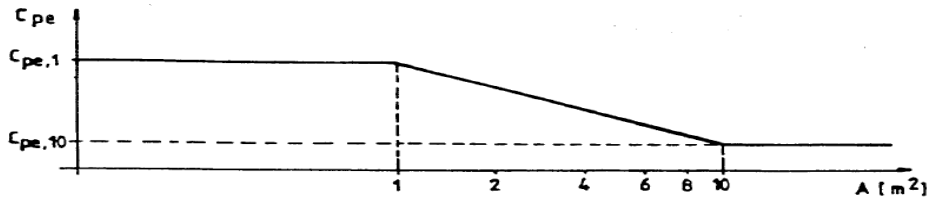


Figure A.1 Variation of External Pressure Coefficient for Buildings with Size of the Loaded Area *A*.

Note: The Figure is based on the following:

$c_{pe} = c_{pe,1}$ $\quad A \leq 1m^2$

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

$A \geq 10m^2 \quad c_{pe} = c_{pe,10}$

Activate Winc

Figure 5 Variation of external pressure coefficient

External Pressure coefficients for pitched roof for wind direction ɵ=0^0

Zones	F	G	H
Area	2.96	8.46	103.1
CPe-ve	-2.24	-1.74	-0.44
Cpe+ve	0.1	0.1	0.1

External Pressure coefficients for pitched roof for wind direction ɵ=180^0

Zones	F	G	H
Area	2.96	8.46	103.1
CPe-ve	-2.66	-2	-0.85

External Pressure coefficients for pitched roof for wind direction ɵ=90^0

Zones	F_{up}	F_{low}	G	H	I
Area	2.96	2.96	8.46	57.42	71.81
CPe-ve	-2.76	-2.4	-2.26	-0.7	-0.6

Internal pressure coefficient

For closed buildings with internal partitions and opening windows the extreme values are

CPi=+0.2 or CPi= -0.3 on ESEN 1991-1-4:2015

Table 8 Net External Pressure for pitched roof for wind direction $\theta=0^0$

Zones	F	G	H
Area	2.96	8.46	103.1
CPe-ve	-2.24	-1.74	-0.44
Cpe+ve	0.1	0.1	0.1
CPi -ve	-0.3	-0.3	-0.3
Cpi+ve	+0.2	+0.2	+0.2
CPe-Cpi	-2.44	-1.94	-0.64
Cpe-Cpi	0.4	0.4	0.4

Hence maximum of cpe-cpi for suction and cpe-cpi for compression is -2.44and 0.4 respectively

Wnet critical negative = 0.4*-2.44= -0.98 KN/m^2

Wnet critical positive = 0.4*0.4 = 0.16 KN/m^2

Net External Pressure for pitched roof for wind direction ɵ=180^0

Zones	F	G	H
Area	2.96	8.46	103.1
CPe-ve	-2.66	-2	-0.85
Cpe+ve	-	-	-
CPi -ve	-0.3	-0.3	-0.3
Cpi+ve	+0.2	+0.2	+0.2
CPe-Cpi	-2.86	-2.2	-1.05
Cpe-Cpi	0.3	0.3	0.3

Hence maximum of cpe-cpi for suction and cpe-cpi for compression is -2.86and 0.3respectively

Wnet critical negative = 0.4*-2.86= -1.14 KN/m^2

Wnet critical positive = 0.4*0.3= 0.12 KN/m^2

Net External Pressure for pitched roof for wind direction ɵ=90^0

Zones	F_{up}	F_{low}	G	H	I
Area	2.96	2.96	8.46	57.42	71.81
CPe-ve	-2.76	-2.4	-2.26	-0.7	-0.6
Cpe+ve	-	-	-	-	-
CPi -ve	-0.3	-0.3	-0.3	-0.3	-0.3
Cpi+ve	+0.2	+0.2	+0.2	+0.2	+0.2
CPe-Cpi	-2.96	-2.6	-2.46	-0.9	-0.8
Cpe-Cpi	0.3	0.3	0.3	0.3	0.3

Hence maximum of cpe-cpi for suction and cpe-cpi for compression is -2.96 and 0.3 respectively

Wnet critical negative = 0.4*-2.96 = -1.18 KN/m^2

Wnet critical positive = 0.4*0.3= 0.12 KN/m^2

Load combination and purlin design

Load combination

The design load combinations are:

$Comb_1$ =1.35DL+1.5LL

$Comb_2$ =1.35DL+1.5LL+0.9WL_S

$Comb_3 = 1.35DL+1.35LL+0.9WL_C$

Dead load of EGA sheet from kality steel manufacturing industry

EGA- 500, thickness of 0.5mm weight =3.92kg/m=3.92*9.81=38.46N/m=0.04KN/m

Self-weight of EGA sheet applying additional 50% of its self-weight for overlapping and fasting of EGA sheet: Total self-weight of EGA sheet=0.04 +0.5*0.04 =0.06 KN/m

Live load Q_K and q_k are given for roof which is not accessible for nominal maintenance repairs Implies Category H pr EN 1991-1-1-2001are:

q_k=0.5KN/M^2 Q_K =1KN (point load) table 6.10 page 29

Purlin design using the aid of etabs

Purlin spacing is select from kality specification manual according to the load and the thickness of the EGA sheet, so use purlin spacing 1.25m

DL of EGA sheet (KN\m) =0.06 KN\m

Distributed live load on purlin (q_k) in KN\m=0.5*1.25=0.625KN\m

Concentrated live load on purlin (Q_K) = 1.0KN

Wind load suction in (KN\m) = -1.18*1.25=-1.48KN\m

Wind load compression in (KN\m)=0.16*1.25=0.2 KN\m

Assign the loads using the above combinations

Use 1^{st} trial steel section of ST80 (80*80mm) with 4mm flange and web thickness after analysis and design this section of purlin it becomes uneconomical

The stress to capacity ratio becomes below 0.5 it implies the capacity of the section is more than two times of the applied stress, so it is uneconomical then revise the section

2^{nd} trial of section uses ST40 (40mm*40mm) tube section with 3.5mm flange and web thickness

The stress to capacity ratio becomes in the range of 0.7-1.0 it is safe and economical the specific stress to capacity ratio is 0.8

Joint loads from the analysis of purlin to assign on the joint of the truss:

Dead load=0.11 KN
Live load=1.2 KN

Wind load compression=0.25 KN
Wind load suction= -1.57KN

Truss analysis and design

External truss analysis and design

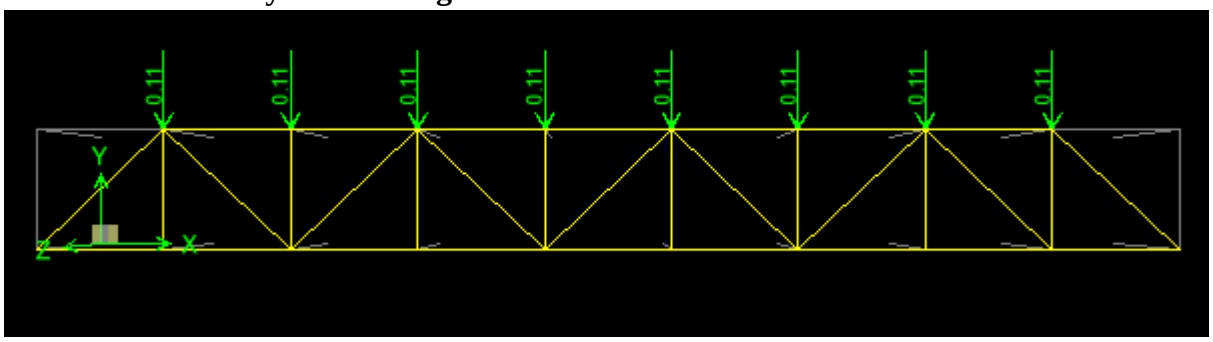


Figure 6 Assigned dead load on external truss

Assigned dead load on external truss (0.11 KN on each joints)

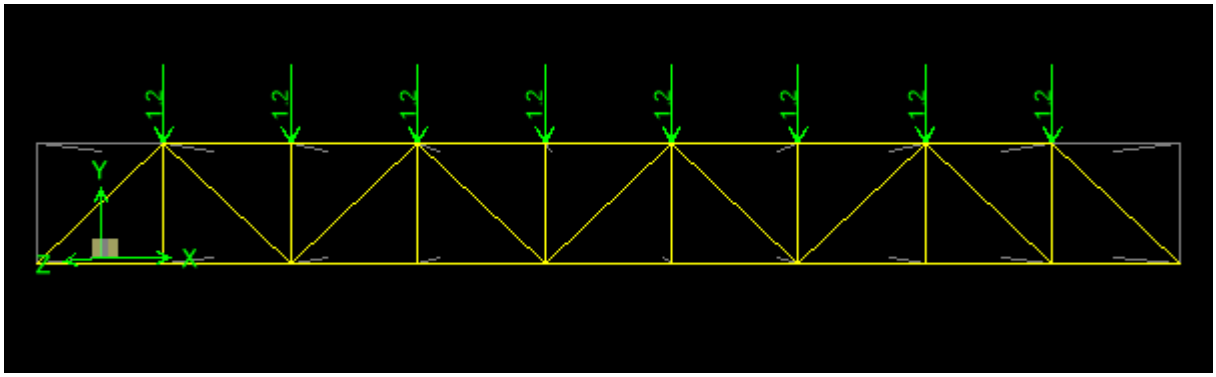


Figure 7 Assigned live load on external truss

Assigned live load on external truss (1.2 KN on each joints)

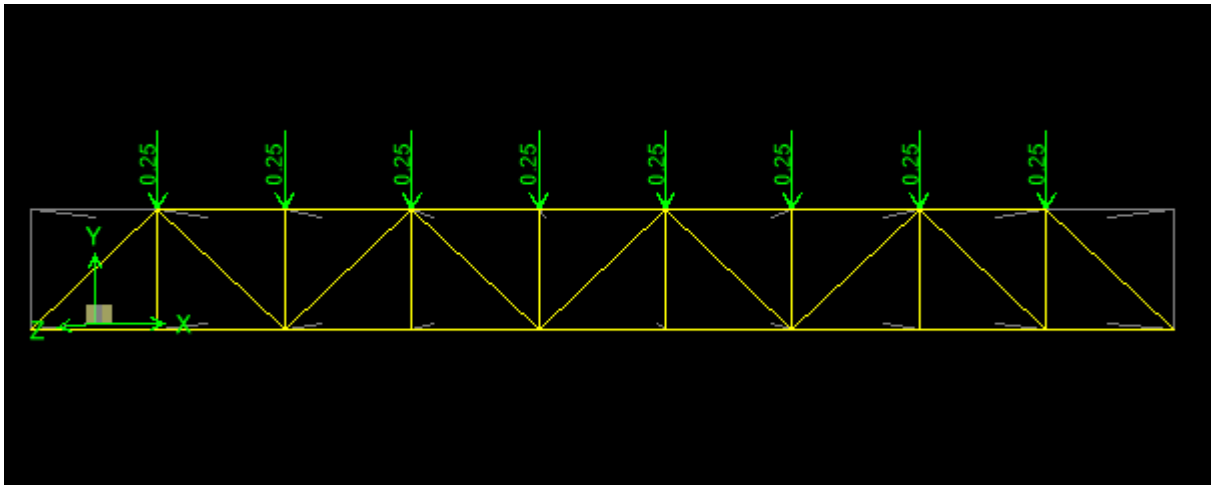


Figure 8 Assigned Wind load compression on external truss

Assigned Wind load compression on external truss (**0.25** KN on each joints)

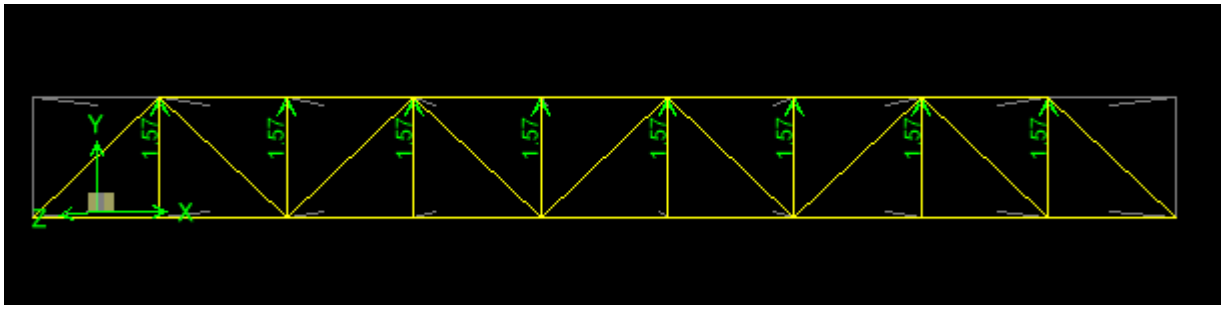


Figure 9 Assigned Wind load suction on external truss

Assigned Wind load suction on external truss (-1.57KN on each joints)

Analysis and design results of truss system of external truss

For dead load

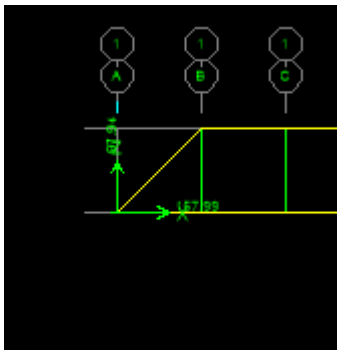


Figure 10 Reaction force due to dead load

=3.4 KN

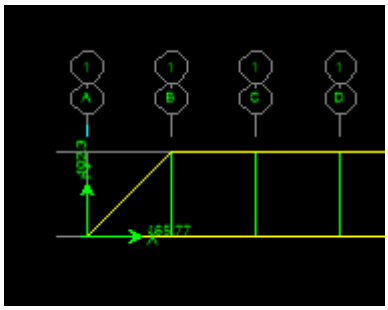


Figure 11 Reaction force due to live load

Reaction force due to live load=7.05

Reaction due to wind load compression=1.46

Reaction due to wind load suction= -15.13

To resist the applied load on the truss the section of the truss should be adequate, so use steel section for the first trial

For top chord: ST 40*40 with a thickness of 2.5mm

For bottom chord: ST 40*40 with a thickness of 2.0mm

For king, queen and princess post: ST 40*40 with a thickness of 2.5mm

For braces ST 12*12 with a thickness of 1.0mm

These cross sections are become safe and economical

Internal truss analysis and design

Internal trusses support load in two sides, so assign the joint loads by multiplying by 2

Assigned dead load on internal truss (0.11*2=0.22KN)

Assigned live load on internal truss (1.2*2=2.4KN)

Assigned wind load compression on internal truss (0.25 *2=0.5KN)

Assigned wind load suction on internal truss (-1.57*2=-3.14KN)

Analysis and design results of truss system of internal truss

Reaction force due to dead load=3.45 KN

Reaction force due to live load=7.05

Reaction due to wind load compression=1.46

Reaction due to wind load suction= -15.13

To resist the applied load on the truss the section of the truss should be adequate, so use steel section for the first trial for chords use ST 40*40 with 4.0mm thickness and for braces 30*30 with thickness of 3.0mm then run the design process with the maximum effect of load combination

Therefore, the top and bottom chord section become red this implies that the section is not safe, so increase the section and also the braces are uneconomical, so minimize the section through a certain process of trial

Second trial

For the top and bottom chord use ST 50*50 with a thickness of 3.5mm

For braces use ST 20*20 with a thickness of 1.0mm

For king, queen and princess post use 50*50 with a thickness of 2.5mm

These cross sections are become safe and economica

3 Slab design

Solid Slab Design

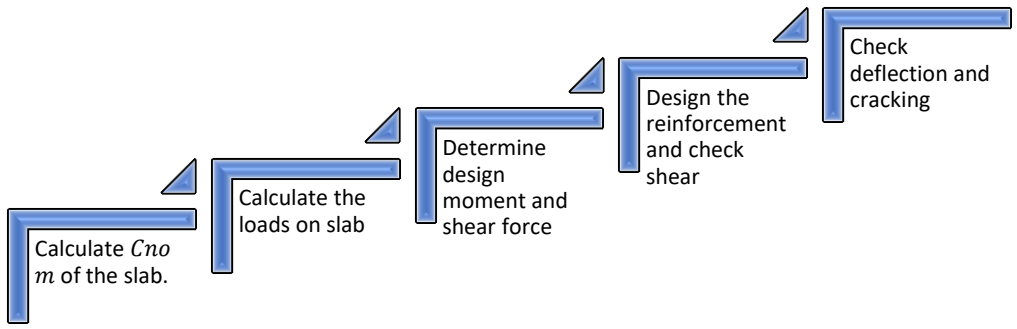
Types of RCC solid slabs

Based on their load transfer mechanism solid slabs are classified as:

One-way slabs	Two-way slabs	Cantilever slabs
• They are those either supported on the two out of four opposite sides or the longer span to short span ratio is 2	• They are those supported on all four sides and the longer span to short span ratio is less than equal to 2.	• They are those with a fixed support on only one out of four sides

:

3.1 Procedures for design of slab



3.2 Design of 1st floor slab

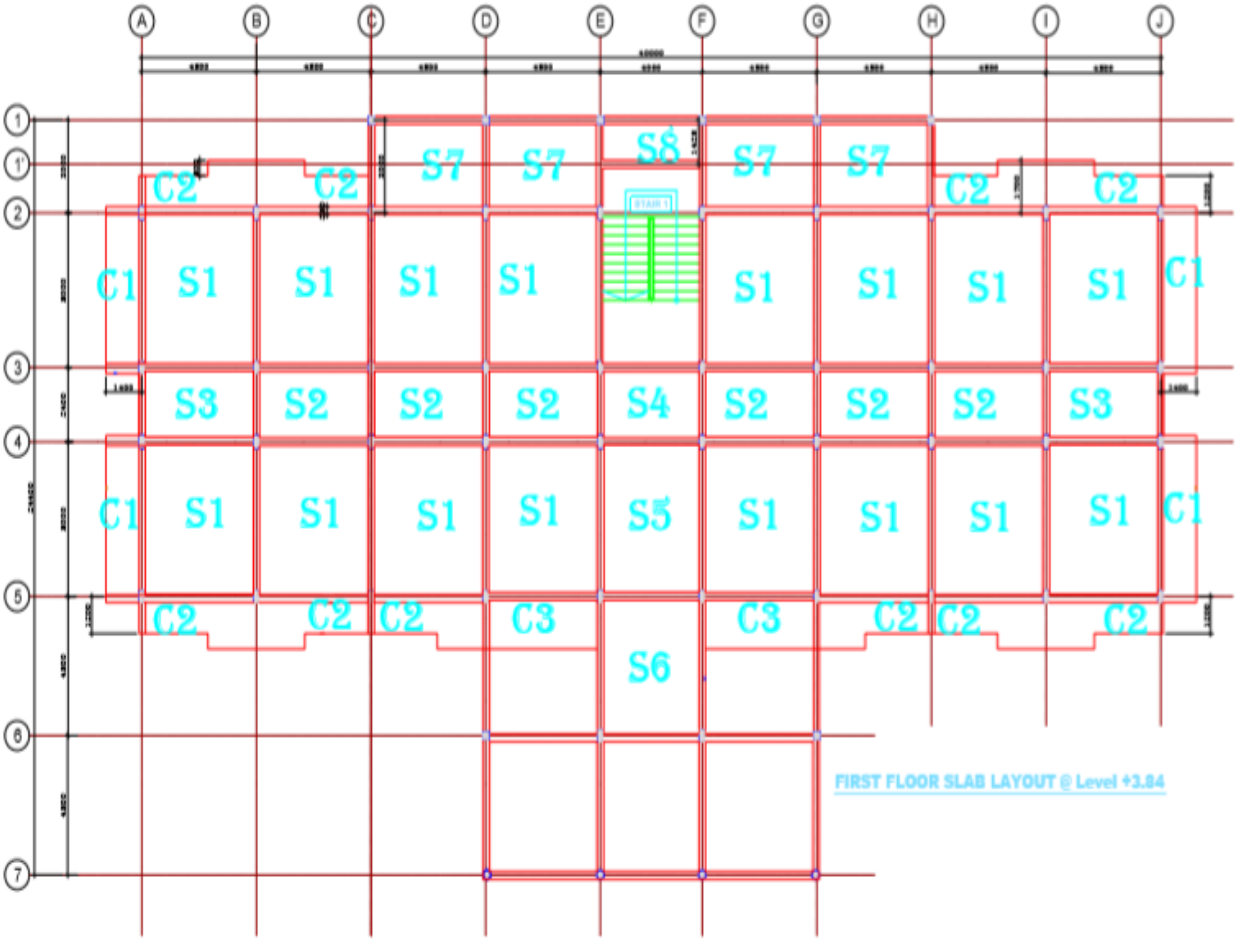


Figure 12 floor slab layout

Table 9 Classifying the slab whether two way/one way

panel	support condition	longer span(Ly)	shorter span(Lx)	span ratio Ly/Lx	slab type
S-1	interior	5000	4500	1.11	Two way
S-2	interior	4500	2400	1.88	Two way
S-3	end span	4500	2400	1.88	Two way
S-4	interior	4000	2400	1.67	Two way
S-5	interior	5000	4000	1.25	Two way
S-6	end span	4500	4000	1.13	Two way
S-7	end span	4500	3000	1.50	Two way
S-8	end span	4000	1425	2.81	one way
C-1	cantilever	5000	4500	1.11	cantilever
C-2	cantilever	4500	1700	2.65	cantilever

3.2.1 Concrete cover determination

The concrete cover is the distance from the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface. The nominal cover shall be specified on the drawings.it is defined as Cmin

$$Cnom=Cmin+\Delta Cdev \ldots\ldots\ldots\ldots [3] \quad \text{Art. 4.4.1.2}$$

Where, Cmin – minimum cover and

ΔCdev is allowance in design for deviation

Minimum cover, shall provide in order to ensure

Safe transmission of bond force

Corrosion resistance/ Durability

Fire resistance

c_{min} = max (4.2)

Where:

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

Cover Design for Corrosion/Durability

Our building is found on dry or permanently wet so the condition of exposure is given to be XC1...ESEN 1992

Member with slab geometry and XC1

From table E.1N - EC2 we see that, in order to obtain an adequate concrete durability, the reference (min.) concrete strength class for exposure class XC1 is C20/25; the resistance class adopted (C25/30) is suitable as it is higher than the reference strength class.

The recommended Structural Class (design working life of 50 years)

The structural class is S3.

Therefore the value of minimum cover required for durability of reinforcement steel is determined using ES EN 1992:2015 table 4.4N.

C-25/30

S-300

-Slab reinforcement φ12

Table 10 Exposure classes.

environmental requirement				
exposure class according to table 4.4				
structural class	X0	xc1	Xc2/xc3	xc4
s1	10	10	10	15
s2	10	10	15	20
s3	10	10	20	25
s4	10	15	25	30
s5	15	20	30	35
s6	20	25	35	40

With: Cmin,b = 12 mm

We obtain from table 4.4N

cmin,dur = 10 mm

Moreover:

Δcdur,γ = 0 ;

Δcdur,st = 0 ;

Δcdur,add = 0 .

From relation (3.2):

cmin = max (cmin,b; cmin,dur + Δcdur,γ - Δcdur,st - Δcdur,add; 10 mm)

= max (12; 10 + 0 – 0 – 0; 10 mm) = 12mm

Δc_{dev}=10mm (recommended)

$Cnom=Cmin+\Delta Cdev=12mm+10mm=22mm$

3.2.2 Effective Depth Determination:

Serviceability requirement [3]

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

Thickness of slab is given in the architectural drawing as 150mm. Material used

C-25/30

S-300

Slab reinforcement φ10

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o \qquad (7.16a)$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \text{if } \rho > \rho_o \qquad (7.16b)$$

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

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

Assumptions

Initially we can't know ρ and ṕ. So let's assume ρ=ρo and equation 7.16a. AS, req= As, provided

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

Where:

Refer Appendix

l/d	•the limit span/depth
K	•the factor to take into account the different structural systems
ρo	•the reference reinforcement ratio= ⟦10⟧^(-∛fck)
ρ	•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	•=500/ σs =500/(fyk*As,req/As,pro) •As,prov =the area of steel provided at this section •As,req = the area of steel required at this section for ultimate limit state •σs=tensile steel stress at mid-span
F2&F3	•=1, (because span of slab ≤ 7m)

Where; $N = 11+1.5\sqrt{fck}\frac{\rho o}{\rho} + 3.2\sqrt{fck}(\frac{\rho o}{\rho} - 1)^{\frac{3}{2}}$ [3]

but ρo=ρ (assumption)

ρo=$\sqrt{fck} * 10^{-3}$ = *=0.00447=0.0045%

N=11+1.5*20^0.5=17.708

F1==1.67

F2=1

F3=1

After determining the value of effective depth

D = d + d',

d'= clear cover + ø/2

d'=22+6=28mm

The value of K for interior span is 1.5, end span is 1.3, cantilever is 0.4 and flat slab is 1.2.*using table* 7.4 *NofESEN*1992:2015

Table 11 Depth calculation

panel	support condition	shorter span(Lx)	fck	N	K	l/d	d	d'	D=d+d'
S-1	interior	4500	20	17.7082	1.5	44.3591	101.445	28	129.445
S-2	interior	2400	20	17.7082	1.5	44.3591	54.104	28	82.104
S-3	end span	2400	20	17.7082	1.3	38.4445	62.4276	28	90.4276
S-4	interior	2400	20	17.7082	1.5	44.3591	54.104	28	82.104
S-5	interior	4000	20	17.7082	1.5	44.3591	90.1733	28	118.173
S-6	end span	4000	20	17.7082	1.3	38.4445	104.046	28	132.046
S-7	end span	3000	20	17.7082	1.3	38.4445	78.0345	28	106.035
S-8	end span	1425	20	17.7082	1.3	38.4445	37.0664	28	65.0664
C-1	cantilever	1400	20	17.7082	0.4	11.8291	118.352	28	146.352
C-2	cantilever	1700	20	17.7082	0.4	11.8291	143.714	28	171.714

Therefore, governing depth is 171.714mm (use D=180mm)

3.2.3 Load calculation

Dead load(DL)

The dead load is composed of the self-weight of the slab itself, weights of the partition walls, weight of the finishing and other considerable permanent loads. Self-weight of the slab is equal to the overall depth times unit weight of concrete. Partition loads are distributed over

Taking 1m width strip

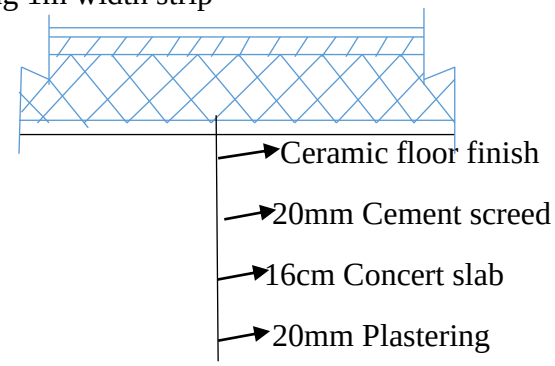


Table 12 Room function and their finishing materials

NO.	Functions	Finishing	Thickness (mm)	Unit weight (KN/m2)
1	Bed room	Ceramic tile	20	21
2	Bed room	Ceramic tile	20	21
3	Balcony	Ceramic tile	20	21
4	Corridor	Ceramic tile	20	21
5	toilet	Ceramic tile	20	21
6	Lobby	Ceramic tile	20	21
7	sitting	Ceramic tile	20	21
		Unit weight of other materials		
1	Rc slab concrete			25
2	Cement screed			23
3	plastering			23

Dead load calculation

Table 13 Calculation of dead load

S-1	depth(m)	unit weight(KN/m3)	dead load(KN/m2)
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-2			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-3			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-4			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84

S-5	depth(m)	unit weight(KN/m3)	dead load(KN/m2)
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-6			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-7			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-8			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84

For Cantilever panel slab

C-1	depth(m)	unit weight(KN/m3)	dead load(KN/m2)
self weight	0.16	25	4
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.34
C-2			
self weight	0.16	25	4
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.34

DEAD LOAD DUE TO PARTITION WALL

Partition wall dead load computation

Material data

Thickness of HCB=20cm=0.2m

Thickness of plastering on two side=20mm=0.02m

Unit weight of HCB=12 KN/m3

Unit weight of plastering =23 KN/m3

Height of wall=3.2m

Unit weight of Glass =25KN/m3

Thickness of glass=0.004m

Unit weight of HCB for light weight aggregate range from 10-14 KN/m3 & lower values are used for smaller size thickness (100mm to 200mm) ,so we use HCB=12 KN/m3

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

D.L pw=D.L pl + D.L HCB,

D.L pw (kN) = (Hpl * Lpl * tpl * ϒpl) + (HHCB * LHCB * tHCB * ϒHCB)

D.L pw (kN) = (3.2m* Lpl*0.02m*23 KN/m3) + (3.2m* LHCB*0.2m*12 KN/m3)

But Lpl= LHCB= Lpw

Therefore D.L pw (kN/m2) =

Therefore D.L pw (kN/m2) =

Where, D.L pw=Dead load of Partition Wall

Hpl, Lpl , tpl & pl =Height, Length, Thickness, Unit Weight of Plastering Respectively.

H(HCB) , L(HCB) , t(HCB) & HCB= Height, Length, Thickness, Unit Weight of HCB Respectively

Table 14 Dead loads and partition wall load

pannel	Lpl(m)	3.2*0.02*Lpl*23	L(HCB)	3.2*0.2*L(HCB)*12	D.L pw	Area of panel	$\frac{D.L\ pw(kN)}{Area\ of\ Panel}$
S1	2.7	3.9744	2.7	20.736	24.7104	22.5	1.09824
S2		0		0	0		
S3		0		0	0		
S4		0		0	0		
S5	9.1	13.3952	9.1	69.888	83.2832	20	4.16416
S6	3.8	5.5936	3.8	29.184	34.7776	18	1.932088889
S7	1.5	2.208	1.5	11.52	13.728	13	1.056
S8		0		0	0		
C1	7.6	11.1872	7.6	58.368	69.5552	6.5	10.7008
C2	1.5	2.208	1.5	11.52	13.728	4.5	3.050666667

Total dead load for floors

the slab if they are not large enough to cause localized effects [4]

Live load (LL)

Since the building is multifunction building the live loads are different depending on the function of the building. (see appendix C)

Pd = 1.35Gk+1.5Qk [4]

Pd=design load

Gk=total dead load on slab

Qk=total live load on slab

Table 15 Factored loads

pannel	dead of slabs	dead loads by partition wall	total dead loads	Live loads	Pd
S1	5.84	1.1	6.9	3	13.9
S2	5.84		5.8	3	12.4
S3	5.84		5.8	3	12.4
S4	5.84		5.8	3	12.4
S5	5.84	4.2	10.0	3	18.0
S6	5.84	1.9	7.8	3	15.0
S7	5.84	1.1	6.9	3	13.8
S8	5.84		5.8	3	12.4
C1	5.34	10.7	16.0	3	26.2
C2	5.34	3.1	8.4	3	15.8

3.2.4 Moment calculation

❑Maximum design moments per unit width are given by the following equations. [5]

The moment is design by using the coefficient method see the following table

Calculation

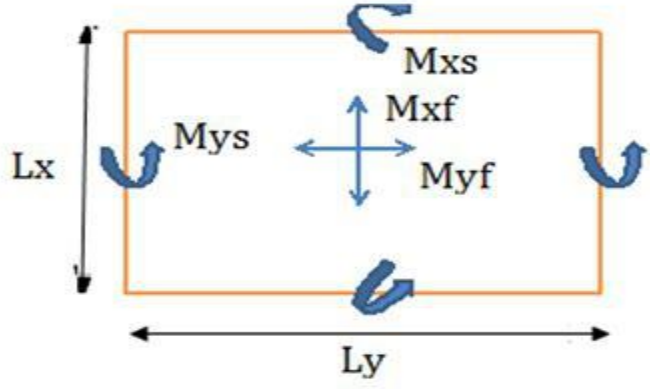
For panel

Ly



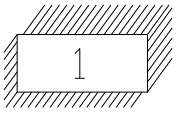
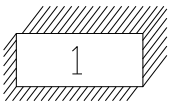
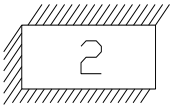
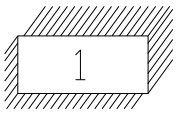
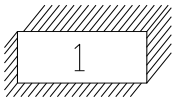
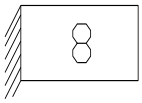
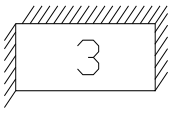
Figure 19 shows the type of slab and its support condition

Where Pd is the design load $\beta sx, and\ \beta sy\ are\ moment\ coefficients$ (see appendix D



$Mxs=\beta xs*Pd*Lx^2$
$Mxf=\beta xf*Pd*Lx^2$
$Mys=\beta ys*Pd*Lx^2$
$Myf=\beta yf*Pd*Lx^2$

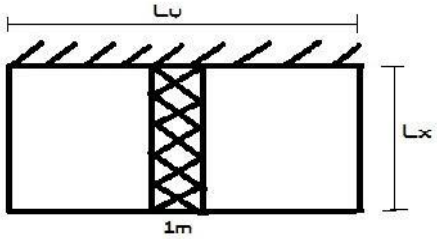
Table 16 Moment calculation for 1st floor slabs

Span	Lx	Ly	Support Condition	Ly/Lx	Pd	ßxs	ßys	ßxf	ßyf	Mxs	Mys	Mxf	Myf
S1	4.5	5		1.11	13.87	0.0375	0.032	0.0284	0.024	10.53	8.99	7.98	6.74
S2	2.4	4.5		1.88	12.38	0.061	0.032	0.046	0.024	4.35	2.28	3.28	1.71
S3	2.4	4.5		1.88	12.38	0.065	0.039	0.048	0.029	4.64	2.78	3.42	2.07
S4	2.4	4		1.51	12.38	0.053	0.032	0.4	0.024	3.78	2.28	28.52	1.71
S5	4	5		1.25	18.00	0.044	0.032	0.033	0.024	12.67	9.22	9.50	6.91
S6	4	4.5		1.13	14.99		0.058	0.057	0.044	0.00	13.91	13.67	10.55
S7	3	4.5		1.50	13.81	0.073	0.039	0.055	0.03	9.07	4.85	6.84	3.73

Case II- Moment Calculation for one way and cantilever slab

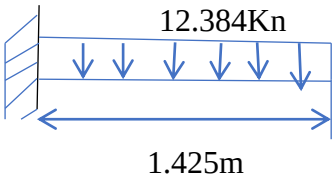
Taking 1m strip of the slab in the shorter direction

For panel 8



Lx=1.425

DL=12.384

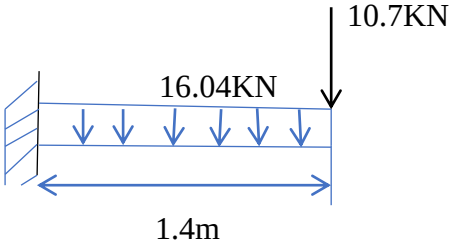


$M_{max}=DL*L^2/2=12.384*=12.54KN$

For C1

load=16.04kN

Lx=1.4m

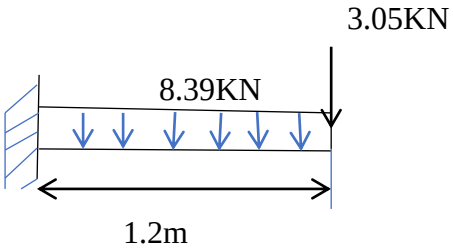


$M_{span}=M_{max}=DL*L^2/2+WL =16.04*=26.42KN$

For C2

Pd=16.04kN

Lx=1.4m



$M_{span}=M_{max}=DL*L^2/2+WL =8.39*=9.82KN$

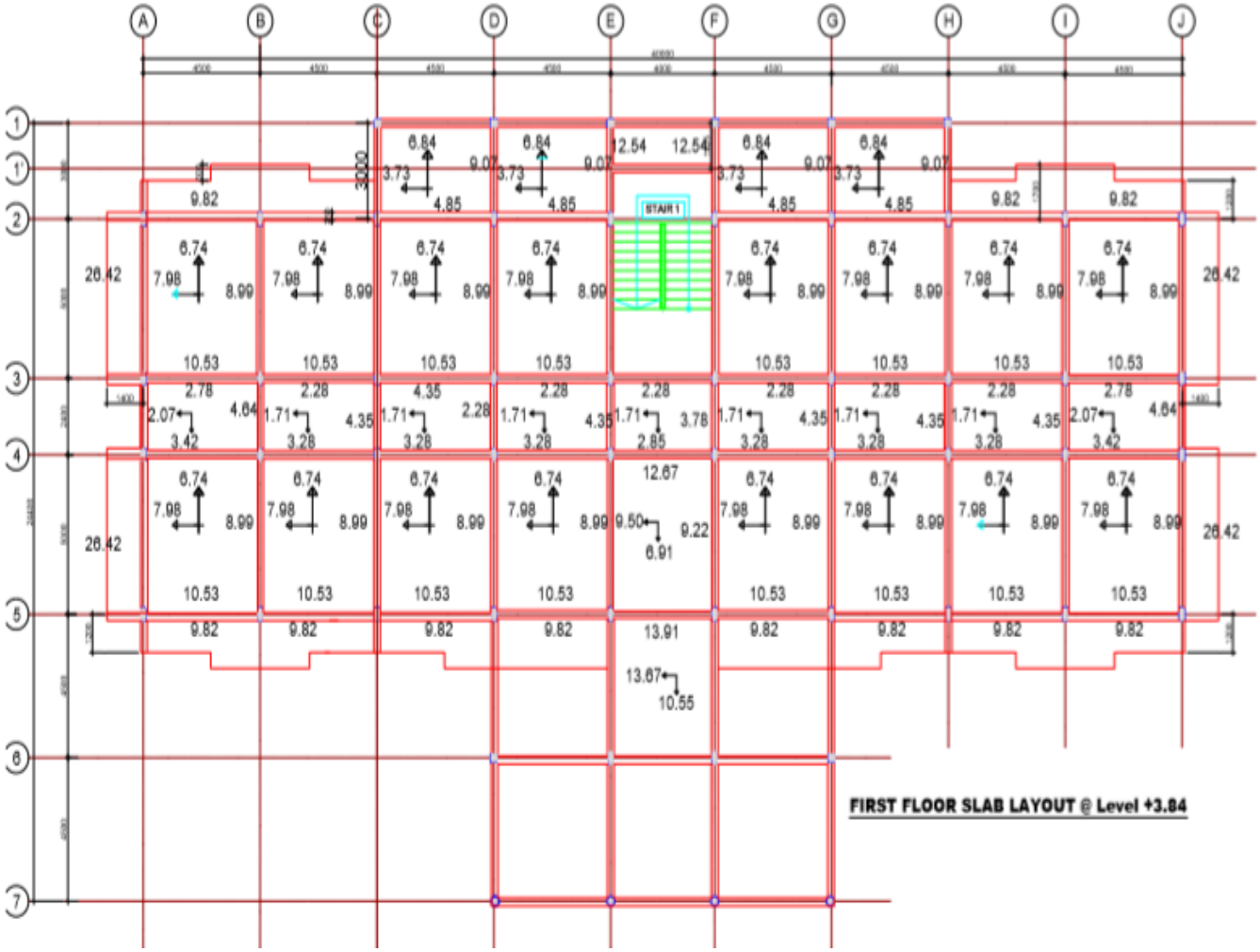


Figure 13 Unadjusted moment

3.2.5 Moment adjustment

If Moments from the two panels is not the same

In these circumstances, the slab may be reinforced throughout for the worst case span and support moments. [4]

Support moment adjustment (EBCS-2,1995 section A.3.3)

Two methods of moment adjusments are used.

V. Method-I
   - ✓ If $\Delta M_{sup} < 20\% M_{large}$, use simple arthimatic averaging for the design moment of the supports with out further field moment adjustment.

VI. Method-II
   - ✓ $\Delta M_{sup} > 20\% M_{large}$, moments are balanced based on the stiffness of the slabs based on the effective length.
     - If M_s is decreased , then field moments are increased to allow equilibrium compatibility.
     - If M_s is increased , no field moment adjustment.

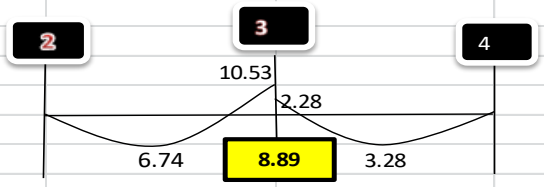
Field moment adjustment

Note: If $M_{s.}$ is decreased, then field moments are increased to allow equilibrium compatibility. If $M_{s.}$ is increased, no field moment adjustment is required.

Adjustments are required between **(S1 & S2), (S2 & S3), (S1 & S3), (S1 & S5), (S5 & S6), (S1 & S7)**

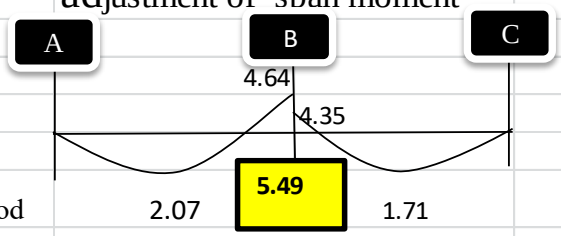
Table 17 Moment adjustment

FOR(S1 & S2)				
Mlarge	10.53			adjustment of field moment
Msmall	2.28			
ΔMs	8.25			
0.2*Mlarge		2.106		
ΔMs>0.2*Mlarge therefore we use stiffnes method				
		3		
	10.53	6.18	4.35	
2	5		2.4	4
DF	0.265		0.735	
	1.63588		4.544	(10.53+6.74)-8.89=8.38
	8.89412		8.89	
$K_{23}=\frac{3I}{4*5}=$		0.15	I	
$K_{34}=\frac{I}{2.4}=$		0.42	I	
DF23= $\frac{K23}{K34+K34}=$			0.265	
DF34= $\frac{K34}{K23+K34}=$			0.735	



FOR (S2 & S3)

adjustment of span moment



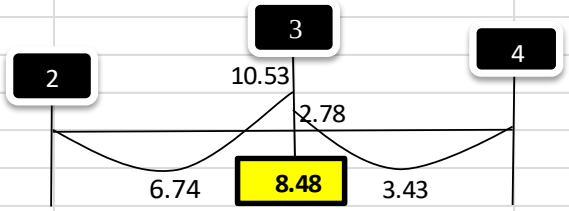
Mlarge	4.64	
Msmall	4.35	
ΔMs	0.29	
0.2*Mlarge		0.928

ΔMs<0.2*Mlarge therefore we use simply averaging method

$\frac{4.64+4.35}{2}=5.49$

(4.64+2.07)-5.49=1.22

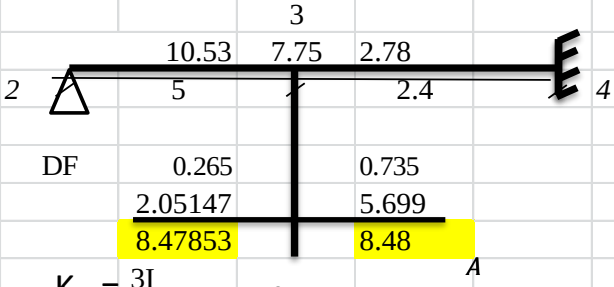
FOR (S1 &S3)



Mlarge	10.53	
Msmall	2.78	
ΔMs	7.75	
0.2*Mlarge		2.106

ΔMs>0.2*Mlarge Therefore we use stiffnes method

(10.53+6.74)-8.48=8.79



DF	0.265		0.735
	2.05147		5.699
	8.47853		8.48

A

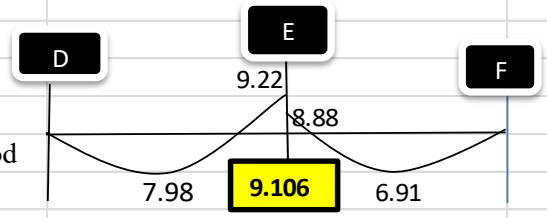
$K_{23}=\frac{3I}{4*5}=\frac{0.15\ I}{}$

$K_{34}=\frac{I}{2.4}=\frac{0.42\ I}{}$

$DF23=\frac{K23}{K23+K34}=0.265$

$DF34=\frac{K34}{K23+K34}=0.735$

FOR (S1 &S5)



Mlarge	9.22	
Msmall	8.99	
ΔMs	0.23	
0.2*Mlarge		1.844

ΔMs<0.2*Mlarge therefore we use simply averaging method

$\frac{9.22+8.99}{2}=9.105$

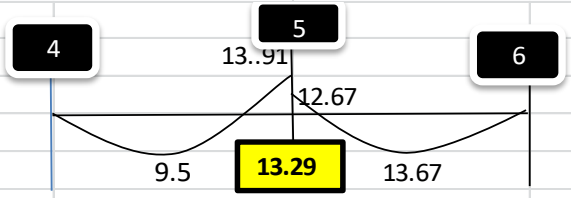
(9.22+7.98)-9.106=8.094

FOR (S5 &S6)

Mlarge	13.91	
Msmall	12.67	
ΔMs	1.24	
0.2*Mlarge		2.782

ΔMs<0.2*Mlarge therefore we use simply averaging method

$\frac{13.91+12.67}{2}=13.29$

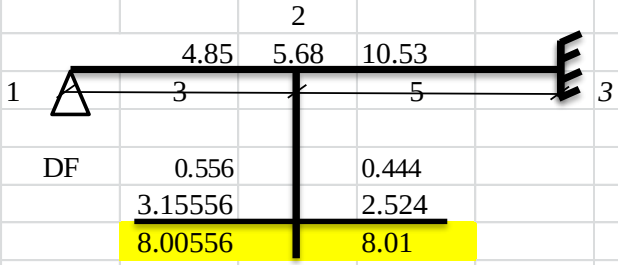


(13.91+9.5)-13.29=10.12

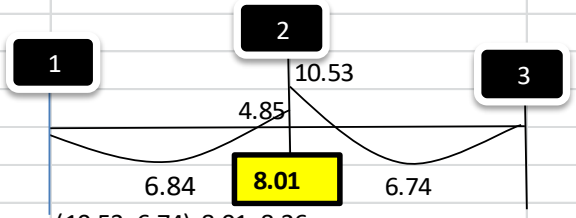
FOR (S1 &S7)

Mlarge	10.53	
Msmall	4.85	
ΔMs	5.68	
0.2*Mlarge		2.106

ΔMs>0.2*Mlarge Therefore we use stiffnes method



DF	0.556		0.444
	3.15556		2.524
	8.00556		8.01



(10.53+6.74)-8.01=8.26

$K_{12}=\frac{3I}{4*3}=\frac{0.25\ I}{}$

$K_{23}=\frac{I}{5}=\frac{0.20\ I}{}$

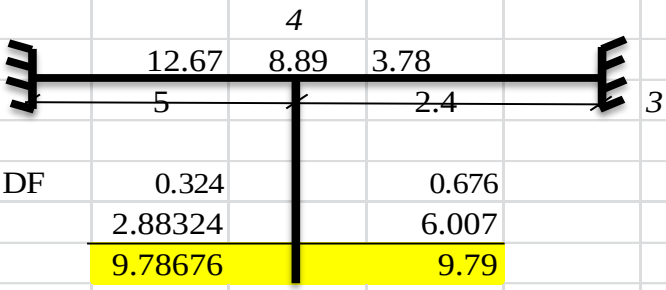
$DF12=\frac{K12}{K12+K23}=$ 0.556

$DF23=\frac{K23}{K12+K23}=$ 0.444

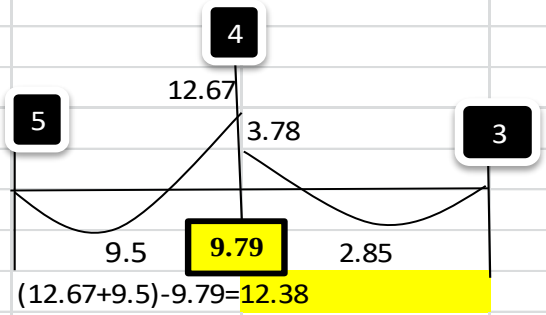
FOR (S4 &S5)

Mlarge	12.67
Msmall	3.78
ΔMs	8.89
0.2*Mlarge	2.534

ΔMs>0.2*Mlarge Therefore we use stiffnes method



DF	0.324		0.676
	2.88324		6.007
	9.78676		9.79



(12.67+9.5)-9.79=12.38

$K_{34}=\frac{I}{L}=\frac{I}{2.4}=0.2\ I$

$K_{45}=\frac{I}{L}=\frac{I}{5}=0.42\ I$

$DF34=\frac{K34}{K34+K45}=0.324$

$DF45=\frac{K45}{K45+K34}=0.676$

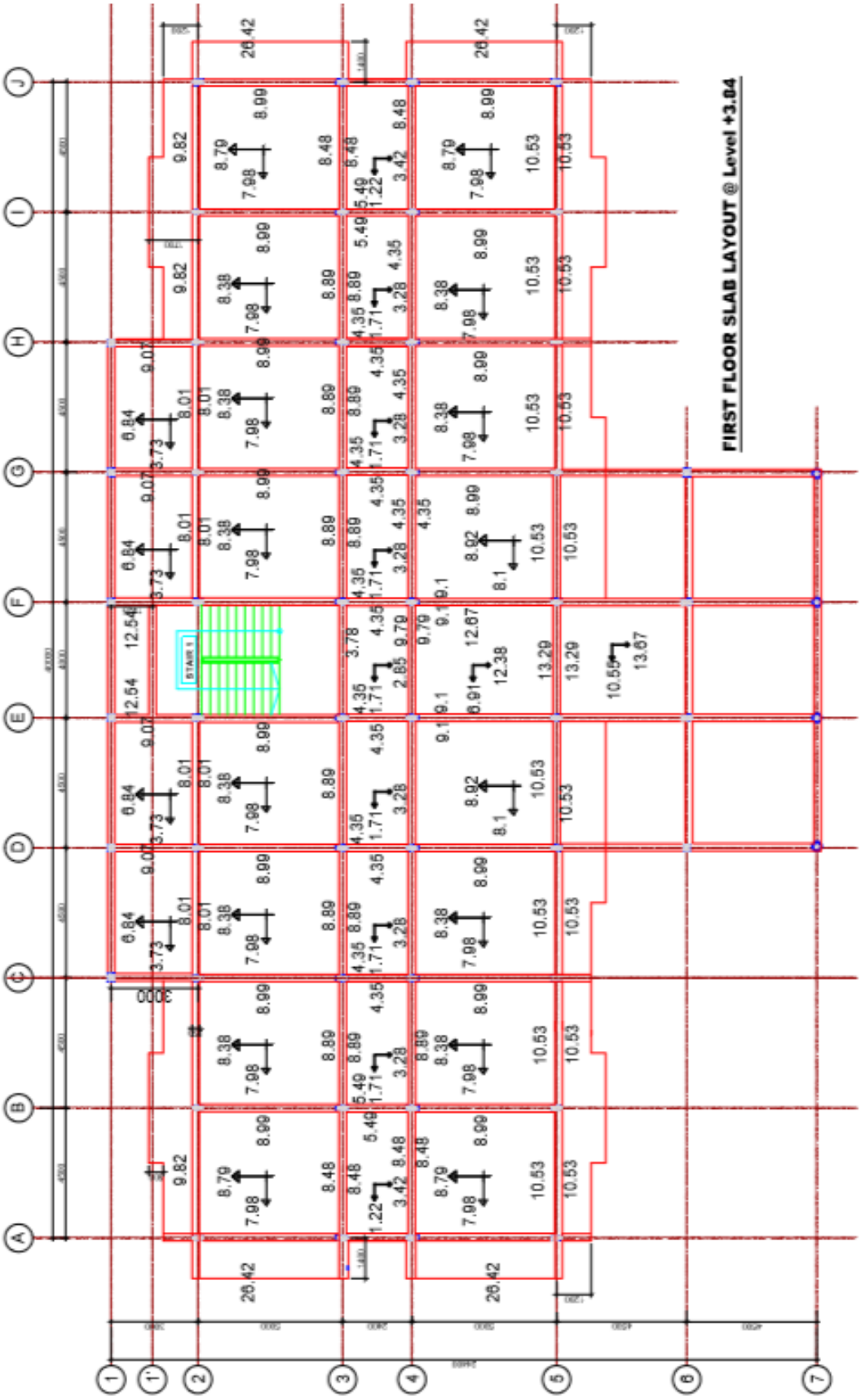
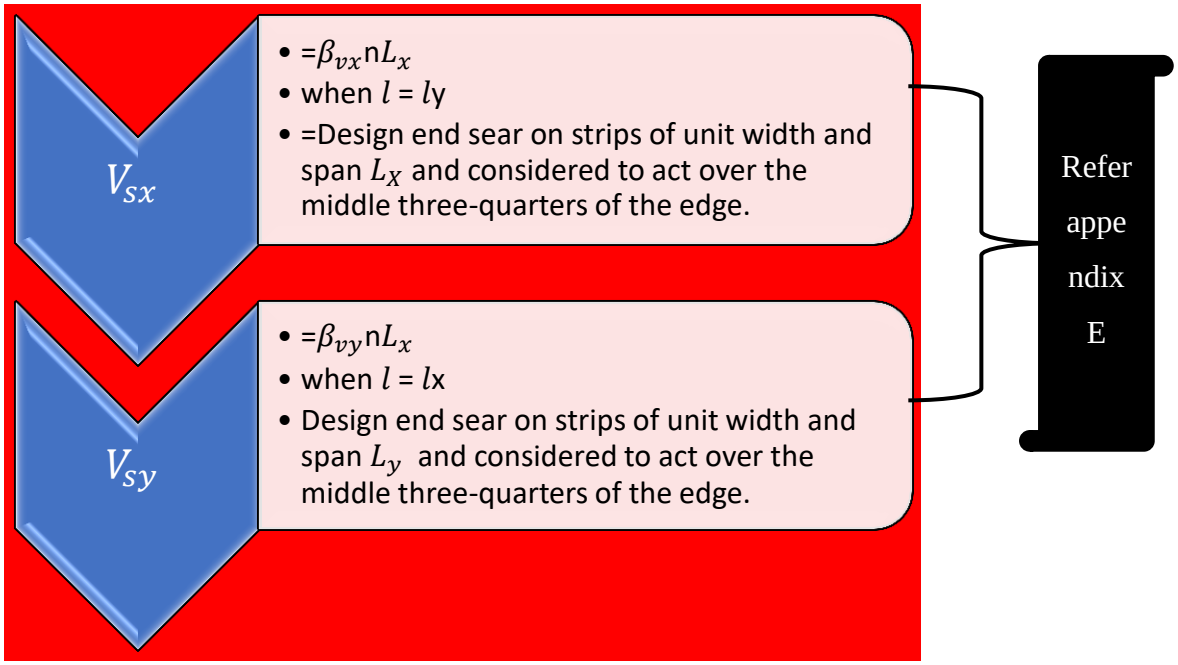


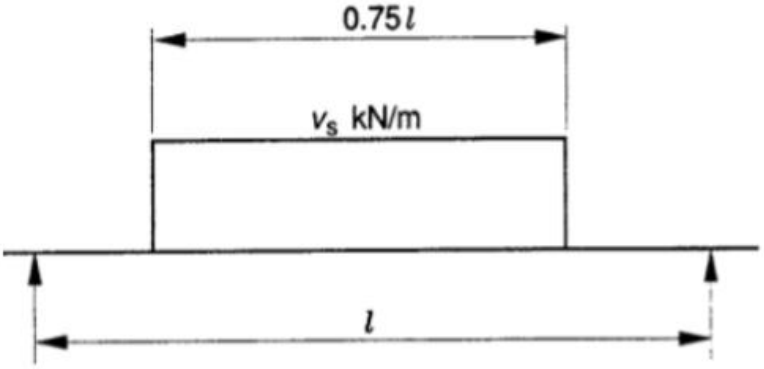
Figure 14 Adjusted moment

3.2.6 Load Transferring Mechanism

Since the design of the slab is done by coefficient method, loads are transferred to beam by load transferring mechanism. Dead 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

Design load on beams supporting solid slab spanning into directions at right angles:





[4]

Figure 15 Distribution of load on a beam supporting a two-way spanning slab

For frame analysis using ETABS, the input of loads should be unfactored loads; we will compute load transfer from panels to beams separately for dead, live and wind loads.

Dead load transfer

$VDx = \beta \quad x * DL * Lx$

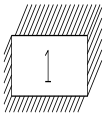
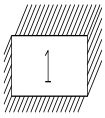
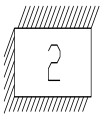
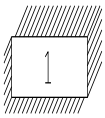
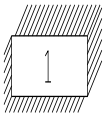
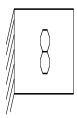
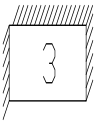
$VDy = \beta vy * DL * Lx$

Live load transfer

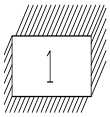
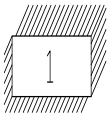
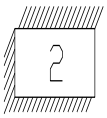
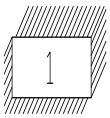
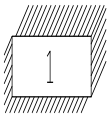
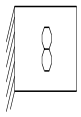
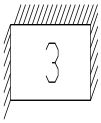
$VLx = \beta vx * LL * Lx$

$VLy = \beta vy * LL * Lx$

Table 18 Un-factored Dead shear & Live shear transferring to beam

panel	support condition	Ly	Lx	Ly/Lx	DL	LL	coefficient		shear forces due to DL		shear forces due to LL	
S-1		5	4.6	1.09	6.80	3	β_{vxc} =	0.34	V_{xc} =	10.64	V_{xc} =	4.69
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.33	V_{yc} =	10.32	V_{yc} =	4.55
							β_{vyd} =	0	V_{yd} =	-	V_{yd} =	-
S-2		4.5	2.4	1.875	5.8	3	β_{vxc} =	0.49	V_{xc} =	6.82	V_{xc} =	3.53
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.33	V_{yc} =	4.59	V_{yc} =	2.38
							β_{vyd} =	0	V_{yd} =	-	V_{yd} =	-
S-3		4.5	2.4	1.88	5.80	3	β_{vxc} =	0.51	V_{xc} =	7.10	V_{xc} =	3.67
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.36	V_{yc} =	5.01	V_{yc} =	2.59
							β_{vyd} =	0.24	V_{yd} =	3.34	V_{yd} =	1.73
S-4		4	2.4	1.67	5.80	3	β_{vxc} =	0.39	V_{xc} =	5.43	V_{xc} =	2.81
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.33	V_{yc} =	4.59	V_{yc} =	2.38
							β_{vyd} =	0	V_{yd} =	-	V_{yd} =	-
S-5		5	4	1.25	5.80	3	β_{vxc} =	0.4	V_{xc} =	9.28	V_{xc} =	4.80
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.33	V_{yc} =	7.66	V_{yc} =	3.96
							β_{vyd} =	0	V_{yd} =	-	V_{yd} =	-
S-6		4.5	4	1.13	7.80	3	β_{vxc} =	0	V_{xc} =	-	V_{xc} =	-
							β_{vxd} =	0.34	V_{xd} =	10.61	V_{xd} =	4.08
							β_{vyc} =	0.45	V_{yc} =	14.04	V_{yc} =	5.40
							β_{vyd} =	0.3	V_{yd} =	9.36	V_{yd} =	3.60
S-7		4.5	3	1.50	6.90	3	β_{vxc} =	0.49	V_{xc} =	10.14	V_{xc} =	4.41
							β_{vxd} =	0.32	V_{xd} =	6.62	V_{xd} =	2.88
							β_{vyc} =	0.36	V_{yc} =	7.45	V_{yc} =	3.24
							β_{vyd} =	0	V_{yd} =	-	V_{yd} =	-

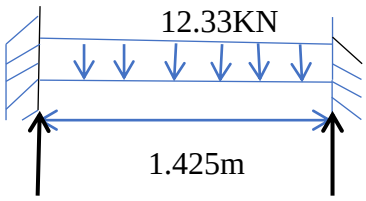
Design shear

panel	support condition	Ly	Lx	Ly/Lx	DL	LL	Pd	coefficient		shear forces		90%
S-1		5	4.6	1.09	6.80	3	13.7	β_{vxc} =	0.34	V_{xc} =	21.40	19.26
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.33	V_{yc} =	20.77	18.69
								β_{vyd} =	0	V_{yd} =	-	-
S-2		4.5	2.4	1.875	5.8	3	12.3	β_{vxc} =	0.49	V_{xc} =	14.50	13.05
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.33	V_{yc} =	9.77	8.79
								β_{vyd} =	0	V_{yd} =	-	-
S-3		4.5	2.4	1.88	5.80	3	12.3	β_{vxc} =	0.51	V_{xc} =	15.09	13.58
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.36	V_{yc} =	10.65	9.59
								β_{vyd} =	0.24	V_{yd} =	7.10	6.39
S-4		4	2.4	1.67	5.80	3	12.3	β_{vxc} =	0.39	V_{xc} =	11.54	10.39
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.33	V_{yc} =	9.77	8.79
								β_{vyd} =	0	V_{yd} =	-	-
S-5		5	4	1.25	5.80	3	12.3	β_{vxc} =	0.4	V_{xc} =	19.73	17.76
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.33	V_{yc} =	16.28	14.65
								β_{vyd} =	0	V_{yd} =	-	-
S-6		4.5	4	1.13	7.80	3	15	β_{vxc} =	0	V_{xc} =	-	-
								β_{vxd} =	0.34	V_{xd} =	20.44	18.40
								β_{vyc} =	0.45	V_{yc} =	27.05	24.35
								β_{vyd} =	0.3	V_{yd} =	18.04	16.23
S-7		4.5	3	1.50	6.90	3	13.8	β_{vxc} =	0.49	V_{xc} =	20.31	18.28
								β_{vxd} =	0.32	V_{xd} =	13.26	11.94
								β_{vyc} =	0.36	V_{yc} =	14.92	13.43
								β_{vyd} =	0	V_{yd} =	-	-

Load transfer one way and cantilever panels

For S8

Design shear

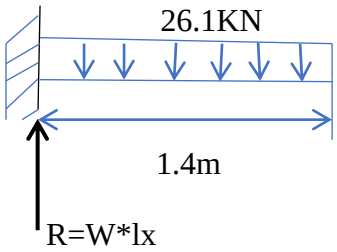


R=Wlx/2

R= Wlx/2=*1.425/2=4.1

For C1

Design shear

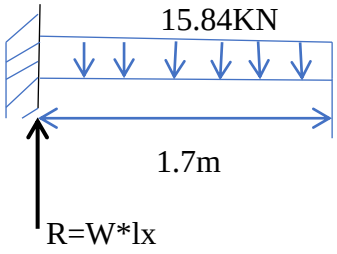


R= W*lx=26.1*1.4=36.54KN

90%=32.88

For C2

Design shear



R= W*lx=15.84*1.7=26.9KN

90%=24.2

3.2.7 Reinforcement design

Designing the slab for flexure

Reinforcement design is done using one of the three methods of calculation, those are: -

Using Design Table
Using Design Chart

We use the first method (Design table method) to calculate the area of reinforcement required for the design moment.

The spacing of bars should not exceed Smax,slab From ECBCS EN 1992 - 1 -1 : 2014

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**

Provision of principal reinforcement

d = D –cover –Φbar/2 = 180-22 -12/2 = 152 mm

X-direction=180-(22+6) =152mm

Y-direction=180-(22+12+6) =140mm

Width, b=1000mm

Material data C=25

S=300

C=25, f_{ck}=$\frac{fcu}{1.25}$=$\frac{25}{1.25}$=20Mpa

Using S-300, f_{yk}=300Mpa

f_{yd}=$\frac{f_{yk}}{1.15}$=300/1.15=260.86Mpa

Using general design table no.1 from ES EN1992-1-1:2015

$$K_m = \sqrt{\frac{M_{sd,s}}{bd^2}}$$

$$A_{S=} = \frac{K_s M_{sd,s}}{d}$$

Using corresponding value of Km take Ks value from the table 1a

Design moment, M=8.48KNm

$$K_m = \sqrt{\frac{M_{sd,s}}{bd^2}} = \sqrt{\frac{8.48}{1*0.152^2}} = 19.16$$

$K_{s=}$3.96

$$A_{Scalc=} = \frac{K_s M_{sd,s}}{d} = \frac{3.96 * 8.48kNm}{0.152} = 220.9mm^2$$

Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slabs.

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

$fctm$ = 0.3($Fck^{2/3}$) =$0.3 * 20^{2/3} = 2.2$

$$A_{s,min}=Max\begin{cases}\frac{0.26fctmbtd}{fyk} = \frac{0.26*2.2*1000*152}{300} = 289.8mm^2 \\ 0.00013btd = 0.00013 * 1000 * 152 = 19.76\end{cases}$$ .....[3] $Art.$9.2.1.1 (1)

$$Asreq=Max\begin{cases}A_{s,calc} = 220.9mm^2 \\ A_{s,min} = 289.8mm^2\end{cases}$$

Ast, max=0.04*Ac……...[3] Art.9.2.1.1 (3)

$$a_{s=} \frac{\pi d^2}{4} = \frac{3.14 * 12^2}{4} = 113.09mm^2$$

$$Spacing = \frac{b * as}{Asreq} = \frac{1000*113.04}{289.8} = 390.1mm$$

For slab maximum spacing is defined in section 9.3.1.1(3) of ES EN 1992-1-1

$$Sprovide = min\begin{cases} 3D = 3 * 180 = 540 \\ 400mm \\ Spacing = 390.1mm \end{cases} \quad \text{.....[3]}$$

Provide, Ø12 C/C 390.1mm

Table 19 Reinforcement calculation for 1st floor slab

panel	M	values	d(m)	Km	Ks	As(calc)	As(min)	AS(req)mm	spacing	S(prov)	remarks
S1	Mxs	8.48	0.152	19.16	3.96	220.93	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mys	8.99	0.14	21.42	3.966	254.67	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mxf	7.98	0.152	18.58	3.94	206.85	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Myf	8.79	0.14	21.18	3.96	248.63	289.8	289.8	390.23	390.235	Φ12C/C390.235
S2	Mxs	4.35	0.152	13.72	3.94	112.76	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mys	8.89	0.14	21.3	3.953	251.02	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mxf	3.28	0.152	11.91	3.934	84.89	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Myf	1.71	0.14	9.34	3.933	48.04	289.8	289.8	390.23	390.235	Φ12C/C390.235
S3	Mxs	5.49	0.152	15.41	3.944	142.45	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mys	8.48	0.14	20.8	3.953	239.44	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mxf	3.42	0.152	12.17	3.938	88.61	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Myf	1.22	0.14	7.89	3.931	34.26	289.8	289.8	390.23	390.235	Φ12C/C390.235
S4	Mxs	4.35	0.152	13.72	3.941	112.79	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mys	9.79	0.14	22.35	3.955	276.57	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mxf	2.85	0.152	11.11	3.936	73.80	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Myf	1.71	0.14	9.34	3.933	48.04	289.8	289.8	390.23	390.235	Φ12C/C390.235
S5	Mxs	13.29	0.152	23.98	3.958	346.06	289.8	346.06	326.79	326.789	Φ12C/C326.789
	Mys	12.67	0.14	25.42	3.96	358.4	289.8	358.38	315.56	315.559	Φ12C/C315.559
	Mxf	6.91	0.152	17.29	3.947	179.4	289.8	289.80	390.23	390.235	Φ12C/C390.235
	Myf	12.38	0.14	25.13	3.96	350.2	289.8	350.18	322.95	322.951	Φ12C/C322.951
S6	Mxs	13.29	0.152	23.98	3.958	346.1	289.8	346.06	326.79	326.789	Φ12C/C326.789
	Mxf	10.55	0.152	21.37	3.958	274.7	289.8	289.80	390.23	390.235	Φ12C/C390.235
	Myf	13.67	0.14	26.41	3.962	386.9	289.8	386.86	292.33	292.327	Φ12C/C392.327
S7	Mxs	9.07	0.152	19.81	3.921	234.0	289.8	289.80	390.23	390.235	Φ12C/C390.235
	Mys	8.01	0.14	20.22	3.953	226.2	289.8	289.80	390.23	390.235	Φ12C/C390.235
	Mxf	6.84	0.152	17.21	3.946	177.6	289.8	289.80	390.23	390.235	Φ12C/C390.235
	Myf	3.73	0.14	13.8	3.94	105.0	289.8	289.80	390.23	390.235	Φ12C/C390.235
S8	Mxf	12.54	0.152	23.3	3.957	326.5	289.8	326.45	346.42	346.421	Φ12C/C3346.421
C1	Mxf	26.42	0.152	33.82	4.09	710.9	289.8	710.91	159.08	159.079	Φ12C/C159.08
C2	Mxf	9.82	0.152	20.62	3.953	255.4	289.8	289.80	390.23	390.235	Φ12C/C390.235

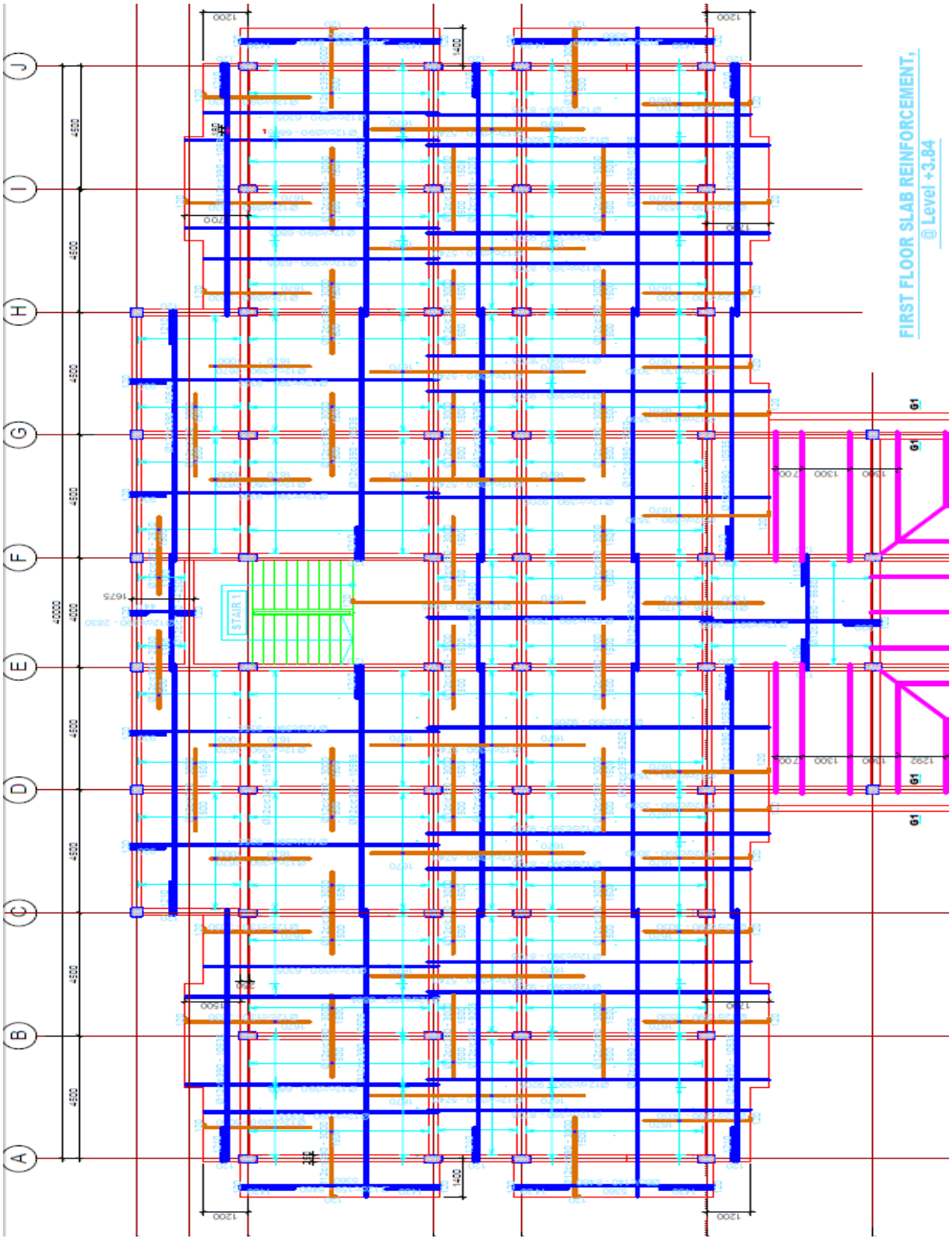

FIRST FLOOR SLAB REINFORCEMENT,
@ Level +3.84
STAIR 1
J
I
H
G
F
E
D
C
B
A
G1

Figure 16 Reinforcement detail for 1st floor slab

3.2.8 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

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

$\rho = \frac{A_{st,req}}{bd} = \frac{289.8}{1000*152} = 0.19\%$ ……..*N*1992: 2017 . 7.4.2(7.16*a*)

$\rho o = \frac{\sqrt{fck}}{1000} = \sqrt{20}/1000 = 0.4472\%$

Now, $\rho = 0.19\% < \rho o = 0.44721\%$

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

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o \quad (7.16a)$$

$$N = \left[11 + 1.5\sqrt{20}\frac{0.4472}{0.19} + 3.2\sqrt{20}\left(\frac{0.4472}{0.19} - 1\right)^{3/2}\right] = 49.33$$

Where

K…… from appendix F

F1=500 /($f_{yk}A_{s,req}$ / $A_{s,prov}$)……[3]…. (7.17)

F2&F3==1, (because span of slab ≤ 7m)…….[3] … section 5.3.2.2 (1).

Then we work similarly other slabs by the excel below

Table 20 Check Depth for Deflection

panel	M	values	Lx	d(m)	Ks	As(calc)	As(min)	AS(req)mm	ρ	ρo	F1	K	Lx/d	d	check
S1	Mxs	8.48	4.5	0.152	3.96	220.93	289.8	289.80	0.191	0.44721	1.67	1.3	106.3	42.323	OK
	Mys	8.99		0.14	3.966	254.67	289.8	289.8	0.207	0.44721	1.67	1.3	94	47.875	OK
	Mxf	7.98		0.152	3.94	206.85	289.8	289.8	0.191	0.44721	1.67	1.3	106.3	42.323	OK
	Myf	8.79		0.14	3.96	248.63	289.8	289.8	0.207	0.44721	1.67	1.3	94	47.875	OK
S2	Mxs	4.35	2.4	0.152	3.94	112.76	289.8	289.8	0.191	0.44721	1.67	1.5	122.7	19.562	OK
	Mys	8.89		0.14	3.953	251.02	289.8	289.8	0.207	0.44721	1.67	1.5	108.5	22.129	OK
	Mxf	3.28		0.152	3.934	84.89	289.8	289.8	0.191	0.44721	1.67	1.5	122.7	19.562	OK
	Myf	1.71		0.14	3.933	48.04	289.8	289.8	0.207	0.44721	1.67	1.5	108.5	22.129	OK
S3	Mxs	5.49	2.4	0.152	3.944	142.45	289.8	289.8	0.191	0.44721	1.67	1.3	106.3	22.572	OK
	Mys	8.48		0.14	3.953	239.44	289.8	289.8	0.207	0.44721	1.67	1.3	94	25.533	OK
	Mxf	3.42		0.152	3.938	88.61	289.8	289.8	0.191	0.44721	1.67	1.3	106.3	22.572	OK
	Myf	1.22		0.14	3.931	34.26	289.8	289.8	0.207	0.44721	1.67	1.3	94	25.533	OK
S4	Mxs	4.35	2.4	0.152	3.941	112.79	289.8	289.8	0.191	0.44721	1.67	1.5	122.7	19.562	OK
	Mys	9.79		0.14	3.955	276.57	289.8	289.8	0.207	0.44721	1.67	1.5	108.5	22.129	OK
	Mxf	2.85		0.152	3.936	73.80	289.8	289.8	0.191	0.44721	1.67	1.5	122.7	19.562	OK
	Myf	1.71		0.14	3.933	48.04	289.8	289.8	0.207	0.44721	1.67	1.5	108.5	22.129	OK
S5	Mxs	13.29	4	0.152	3.958	346.06	289.8	346.06	0.228	0.44721	1.67	1.5	94.32	42.409	OK
	Mys	12.67		0.14	3.96	358.4	289.8	358.38	0.256	0.44721	1.67	1.5	79.9	50.064	OK
	Mxf	6.91		0.152	3.947	179.4	289.8	289.80	0.191	0.44721	1.67	1.5	122.7	32.604	OK
	Myf	12.38		0.14	3.96	350.2	289.8	350.18	0.25	0.44721	1.67	1.5	82.51	48.48	OK
S6	Mxs	13.29	4	0.152	3.958	346.1	289.8	346.06	0.228	0.44721	1.67	1.3	81.74	48.934	OK
	Mxf	10.55		0.152	3.958	274.7	289.8	289.80	0.191	0.44721	1.67	1.3	106.3	37.62	OK
	Myf	13.67		0.14	3.962	386.9	289.8	386.86	0.276	0.44721	1.67	1.3	62.43	64.067	OK
S7	Mxs	9.07	3	0.152	3.921	234.0	289.8	289.80	0.191	0.44721	1.67	1.3	106.3	28.215	OK
	Mys	8.01		0.14	3.953	226.2	289.8	289.80	0.207	0.44721	1.67	1.3	94	31.916	OK
	Mxf	6.84		0.152	3.946	177.6	289.8	289.80	0.191	0.44721	1.67	1.3	106.3	28.215	OK
	Myf	3.73		0.14	3.94	105.0	289.8	289.80	0.207	0.44721	1.67	1.3	94	31.916	OK
S8	Mxf	12.54	1.4	0.152	3.957	326.5	289.8	326.45	0.215	0.44721	1.67	1.3	89.01	16.01	OK
C1	Mxf	26.42	1.4	0.152	4.09	710.9	289.8	710.91	0.468	0.44721	1.67	0.4	####	#####	#NUM!
C2	Mxf	9.82	1.7	0.152	3.953	255.4	289.8	289.80	0.191	0.44721	1.67	0.4	32.72	51.963	OK

3.2.9 Check Shear resistance of the section

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

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

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

with a minimum of

$V_{Rd,c}= (v_{min}+ k_1\sigma_{cp})\ b_w d$ (6.2b)

where:

f_{ck} is in MPa

$k = 1+\sqrt{\frac{200}{d}} \le 2.0$ with d in mm…….[3]

$\rho_1 = \frac{A_{sl}}{b_w d} \le 0.02$ [3]

…….

A_{sl} is the area of the tensile reinforcement, which extends ≥ (l_{bd} + d) beyond the section considered (see Figure 6.3)

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

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

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

A_C is the area of concrete cross section [mm^2]

$V_{Rd,c}$ is [N]

3.3 2nd and 3rd typical floor slab design

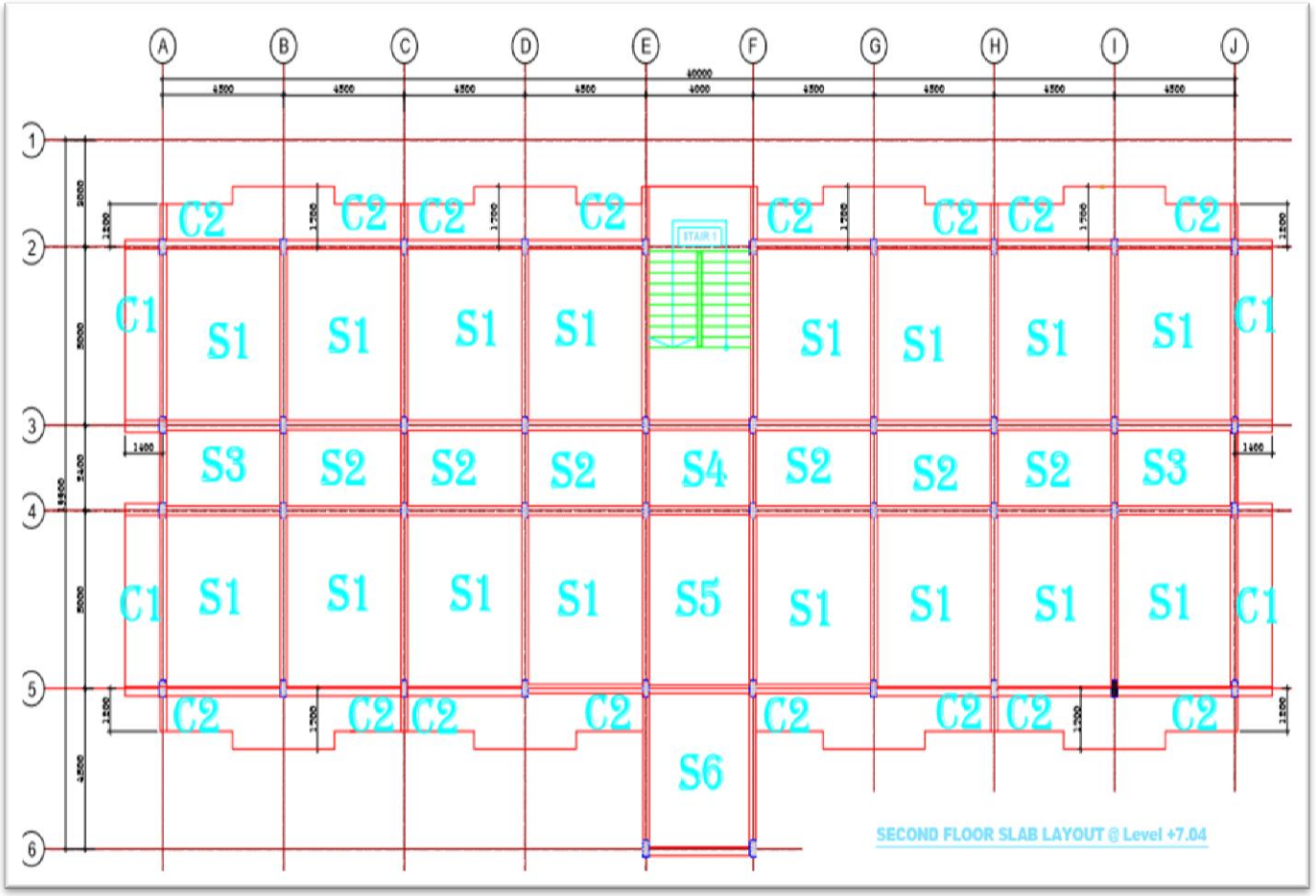


Figure 17 typical2nd and 3rd floor slab

Note:- the 2nd and 3rd floors are the same with 1st floor but the difference is only S7 and S8, which means there is no S8and S8 on 2nd and 3rd floor. Therefore we can take the parameters from 1st floor

Table 21 Depth calculation

panel	support condition	shorter span(Lx)	fck	N	K	l/d	d	d'	D=d+d'
S-1	interior	4500	20	17.7082	1.5	44.3591	101.445	28	129.445
S-2	interior	2400	20	17.7082	1.5	44.3591	54.104	28	82.104
S-3	end span	2400	20	17.7082	1.3	38.4445	62.4276	28	90.4276
S-4	interior	2400	20	17.7082	1.5	44.3591	54.104	28	82.104
S-5	interior	4000	20	17.7082	1.5	44.3591	90.1733	28	118.173
S-6	end span	4000	20	17.7082	1.3	38.4445	104.046	28	132.046
C-1	cantilever	1400	20	17.7082	0.4	11.8291	118.352	28	146.352
C-2	cantilever	1700	20	17.7082	0.4	11.8291	143.714	28	171.714

Therefore, governing depth is 171.714mm (use D=180mm)

3.3.1 Load calculation

Dead load(DL)

The dead load is composed of the self-weight of the slab itself, weights of the partition walls, weight of the finishing and other considerable permanent loads. Self-weight of the slab is equal to the overall depth times unit weight of concrete. Partition loads are distributed over

Taking 1m width strip

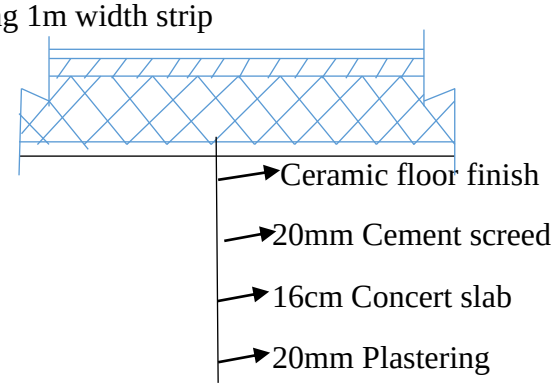


Table 22 Dead load calculation

S-1	depth(m)	unit weight(KN/m3)	dead load(KN/m2)
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-2			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-3			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-4			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84

S-5	depth(m)	unit weight(KN/m3)	dead load(KN/m2)
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84
S-6			
self weight	0.18	25	4.5
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.84

For Cantilever slab

C-1	depth(m)	unit weight(KN/m3)	dead load(KN/m2)
self weight	0.16	25	4
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.34
C-2			
self weight	0.16	25	4
ceramics	0.02	21	0.42
cement screed	0.02	23	0.46
plastering	0.02	23	0.46
		Total	5.34

DEAD LOAD DUE TO PARTITION WALL

Partition wall dead load computation

Material data

Thickness of HCB=20cm=0.2m

Thickness of plastering on two side=20mm=0.02m

Unit weight of HCB=12 KN/m3

Unit weight of plastering =23 KN/m3

Height of wall=3.2m

Unit weight of Glass =25KN/m3

Thickness of glass=0.004m

Unit weight of HCB for light weight aggregate range from 10-14 KN/m3 & lower values are used for smaller size thickness (100mm to 200mm) ,so we use HCB=12 KN/m3

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

D.L pw=D.L pl + D.L HCB,

D.L pw (kN) = (Hpl * Lpl * tpl * ϒpl) + (HHCB * LHCB * tHCB * ϒHCB)

D.L pw (kN) = (3.2m* Lpl*0.02m*23 KN/m3) + (3.2m* LHCB*0.2m*12 KN/m3)

But Lpl= LHCB= Lpw

Therefore D.L pw (kN/m2) =

Therefore D.L pw (kN/m2) =

Where, D.L pw=Dead load of Partition Wall

Hpl, Lpl, tpl & pl =Height, Length, Thickness, Unit Weight of Plastering Respectively.

H(HCB) , L(HCB) , t(HCB) & HCB= Height, Length, Thickness, Unit Weight of HCB Respectively

Table 23 Dead load and partition wall loads

pannel	Lpl(m)	3.2*0.02*Lpl*23	L(HCB)	3.2*0.2*L(HCB)*12	D.L pw	Area of panel	$\frac{D.L\ pw(kN)}{Area\ of\ Panel}$
S1	2.7	3.9744	2.7	20.736	24.7104	22.5	1.09824
S2		0		0	0		
S3		0		0	0		
S4		0		0	0		
S5	9.1	13.3952	9.1	69.888	83.2832	20	4.16416
S6	3.8	5.5936	3.8	29.184	34.7776	18	1.932088889
C1	7.6	11.1872	7.6	58.368	69.5552	6.5	10.7008
C2	1.5	2.208	1.5	11.52	13.728	4.5	3.050666667

Total dead load for floors

the slab if they are not large enough to cause localized effects [4]

Live load (LL)

Since the building is multifunction building the live loads are different depending on the function of the building. (See appendix C)

Pd = 1.35Gk+1.5Qk [4]

Pd=design load

Gk=total dead load on slab

Qk=total live load on slab

Table 24 Factored loads for 2nd and 3rd floor

pannel	Function	dead of slabs	dead loads by partition wall	total dead loads	Live loads	Pd
S1		5.84	1.09824	6.93824	3	13.866624
S2		5.84		5.84	3	12.384
S3		5.84		5.84	3	12.384
S4		5.84		5.84	3	12.384
S5		5.84	4.16416	10.00416	3	18.005616
S6		5.84	1.932088889	7.772088889	3	14.99232
C1		5.34	10.7008	16.0408	3	26.15508
C2		5.34	3.050666667	8.390666667	3	15.8274

3.3.2 Moment calculation

❑Maximum design moments per unit width are given by the following equations. [5]

The moment is design by using the coefficient method see the following table

Calculation

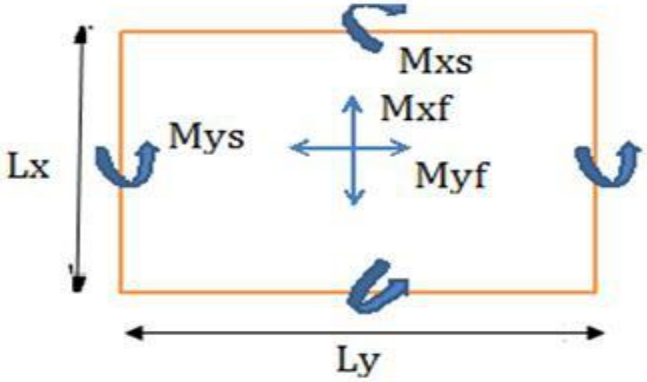
For panel

Ly



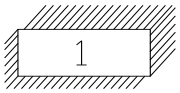
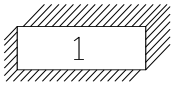
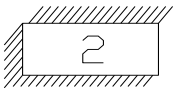
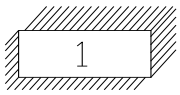
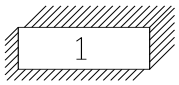
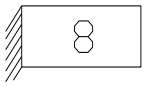
Figure 19 shows the type of slab and its support condition

Where Pd is the design load $\beta sx, and\ \beta sy\ are\ moment\ coefficients$ (see appendix D)



$Mxs=\beta xs*Pd*Lx^2$
$Mxf=\beta xf*Pd*Lx^2$
$Mys=\beta ys*Pd*Lx^2$
$Myf=\beta yf*Pd*Lx^2$

Table 25 Moment calculation for 2nd and 3rd floor slab

Span	Lx	Ly	Support Condition	Ly/Lx	Pd	ßxs	ßys	ßxf	ßyf	Mxs	Mys	Mxf	Myf
S1	4.5	5		1.11	13.87	0.0375	0.032	0.0284	0.024	10.53	8.99	7.98	6.74
S2	2.4	4.5		1.88	12.38	0.061	0.032	0.046	0.024	4.35	2.28	3.28	1.71
S3	2.4	4.5		1.88	12.38	0.065	0.039	0.048	0.029	4.64	2.78	3.42	2.07
S4	2.4	4		1.51	12.38	0.053	0.032	0.04	0.024	3.78	2.28	2.85	1.71
S5	4	5		1.25	18.00	0.044	0.032	0.033	0.024	12.67	9.22	9.50	6.91
S6	4	4.5		1.13	14.99		0.058	0.057	0.044	0.00	13.91	13.67	10.55

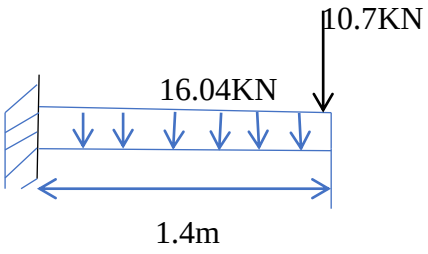
Case II- Moment Calculation for one way and cantilever slab

Taking 1m strip of the slab in the shorter direction

For C1

load=16.04kN

Lx=1.4m

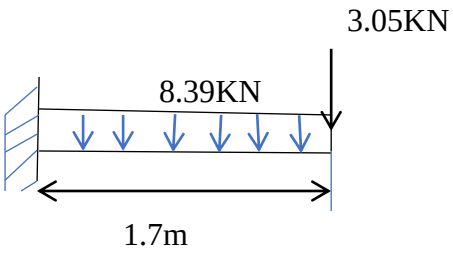


$M_{span}=M_{max}=DL*L^2/2+WL$ =16.04*=26.42KN

For C2

Pd=16.04kN

Lx=1.7m



$M_{span}=M_{max}=DL*L^2/2+WL$ =8.39*=9.82KN

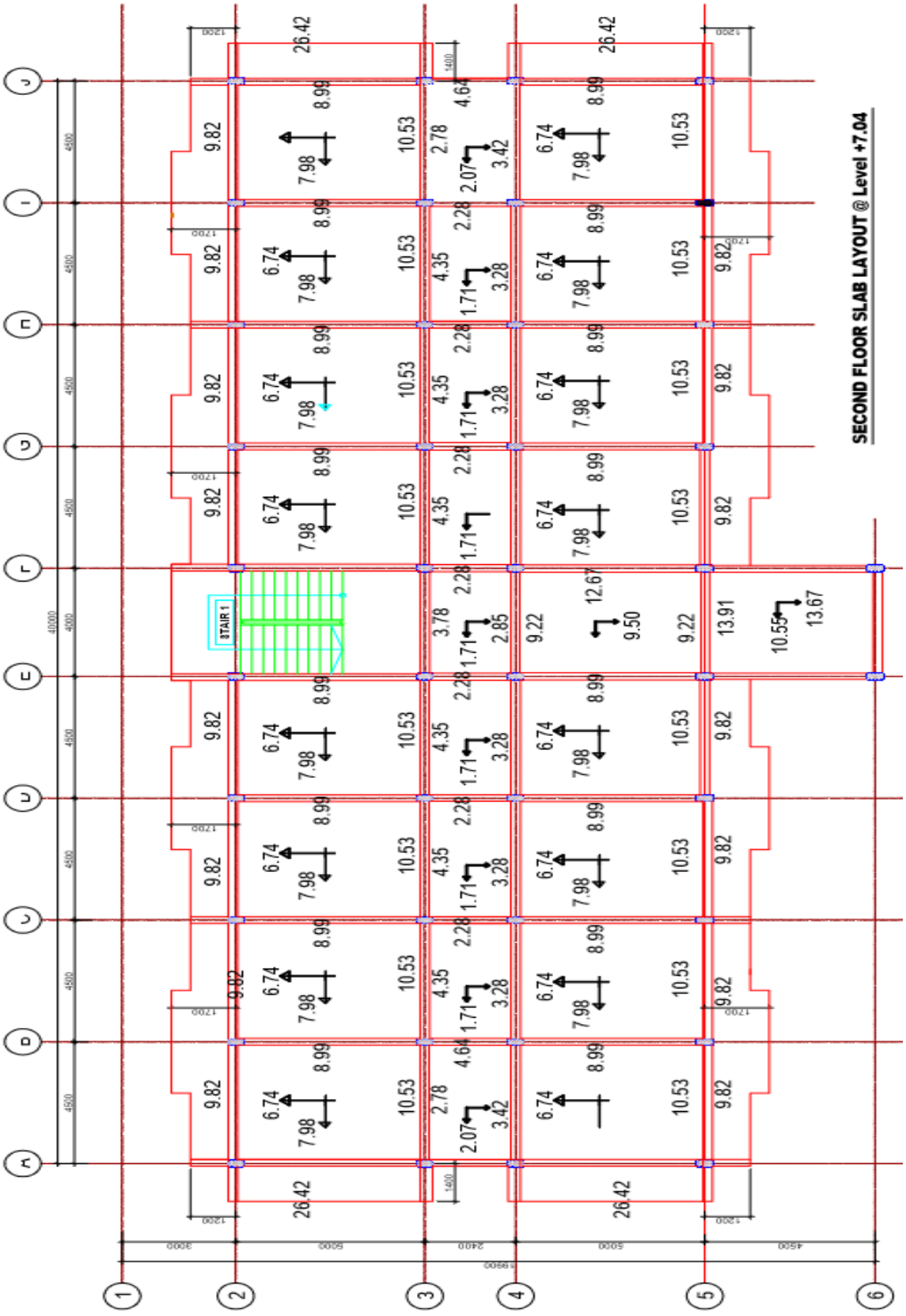


Figure 18 Unadjusted moment for 2nd and 3rd floor slab

3.3.3 Moment adjustment

If Moments from the two panels is not the same

In these circumstances, the slab may be reinforced throughout for the worst case span and support moments. [4]

Support moment adjustment (EBCS-2,1995 section A.3.3)

Two methods of moment adjusments are used.

V. Method-I
   - ✓ If $\Delta M_{sup} < 20\% M_{large}$, use simple arthimatic averaging for the design moment of the supports with out further field moment adjustment.

VI. Method-II
   - ✓ $\Delta M_{sup} > 20\% M_{large}$, moments are balanced based on the stiffness of the slabs based on the effective length.
     - If M_s is decreased , then field moments are increased to allow equilibrium compatibility.
     - If M_s is increased , no field moment adjustment.

Field moment adjustment

Note: If $M_{s.}$ is decreased, then field moments are increased to allow equilibrium compatibility. If $M_{s.}$ is increased, no field moment adjustment is required.

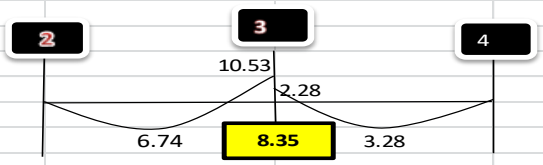
Adjustments are required between **(S1 & S2), (S2 & S3), (S1 & S3), (S1 & S5), (S5 & S6),**

Table 26 Span and support moment adjustment of 2nd and 3rd floor slab

FOR(S1 & S2)

Mlarge	10.53		
Msmall	2.28		
ΔMs	8.25		
0.2*Mlarge		2.106	
ΔMs>0.2*Mlarge therefore we use stiffnes method			
		3	
	10.53	8.25	2.28
2	5		2.4 4
DF	0.265		0.735
	2.18382		6.066
	8.34618		8.35

adjustment of span moment



(10.53+6.74)-8.35=8.92

$K_{23}=\frac{3I}{4*5}=$ 0.15 I

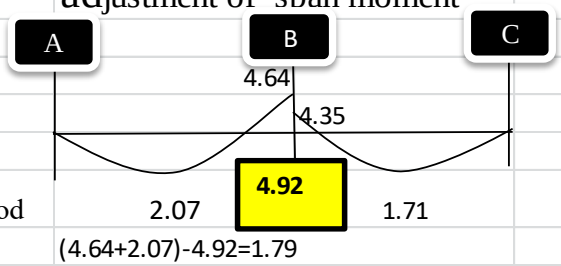
$K_{34}=\frac{I}{2.4}$ 0.42 I

$DF23=\frac{K23}{K34+K34}=$ 0.265

$DF34=\frac{K34}{K23+K34}=$ 0.735

FOR (S2 & S3)

adjustment of span moment



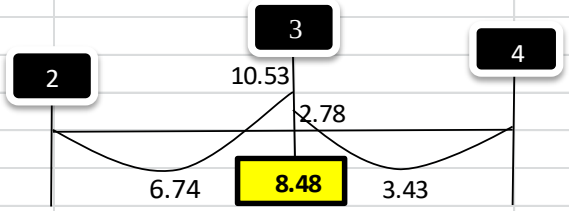
(4.64+2.07)-4.92=1.79

Mlarge	4.64	
Msmall	2.28	
ΔMs	2.36	
0.2*Mlarge		0.928

ΔMs<0.2*Mlarge therefore we use simply averaging method

$\frac{4.64+2.28}{2}=4.92$

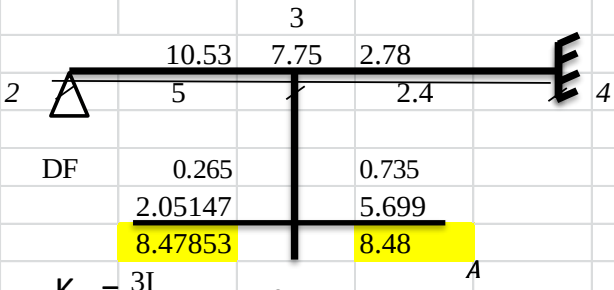
FOR (S1 &S3)



(10.53+6.74)-8.48=8.79

Mlarge	10.53	
Msmall	2.78	
ΔMs	7.75	
0.2*Mlarge		2.106

ΔMs>0.2*Mlarge Therefore we use stiffnes method



DF	0.265		0.735
	2.05147		5.699
	8.47853		8.48

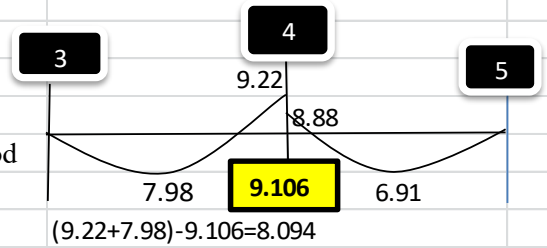
$K_{23}=\frac{3I}{4*5}=\frac{0.15\ I}{}$

$K_{34}=\frac{I}{2.4}=\frac{0.42\ I}{}$

$DF23=\frac{K23}{K23+K34}=0.265$

$DF34=\frac{K34}{K23+K34}=0.735$

FOR (S1 &S5) $\frac{4.35+3.78}{2}=4.065$



(9.22+7.98)-9.106=8.094

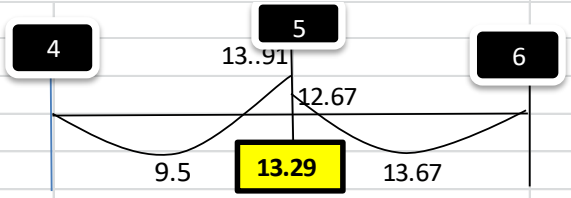
Mlarge	9.22	
Msmall	8.99	
ΔMs	0.23	
0.2*Mlarge		1.844

ΔMs<0.2*Mlarge therefore we use simply averaging method

$\frac{9.22+8.99}{2}=9.105$

FOR (S5 &S6)

Mlarge	13.91	
Msmall	12.67	
ΔMs	1.24	
0.2*Mlarge		2.782



(13.91+9.5)-13.29=10.12

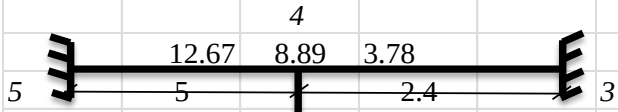
ΔMs<0.2*Mlarge therefore we use simply averaging method

$\frac{13.91+12.67}{2}$=13.29

FOR (S4 &S5)

Mlarge	12.67	
Msmall	3.78	
ΔMs	8.89	
0.2*Mlarge		2.534

ΔMs>0.2*Mlarge Therefore we use stiffnes method



DF	0.324	0.676
	2.88324	6.007
	9.78676	9.79



(12.67+9.5)-9.79=12.38

$K_{34}=\frac{I}{L}=\frac{I}{2.4}=0.2\ I$

$K_{45}=\frac{I}{L}=\frac{I}{5}=0.42\ I$

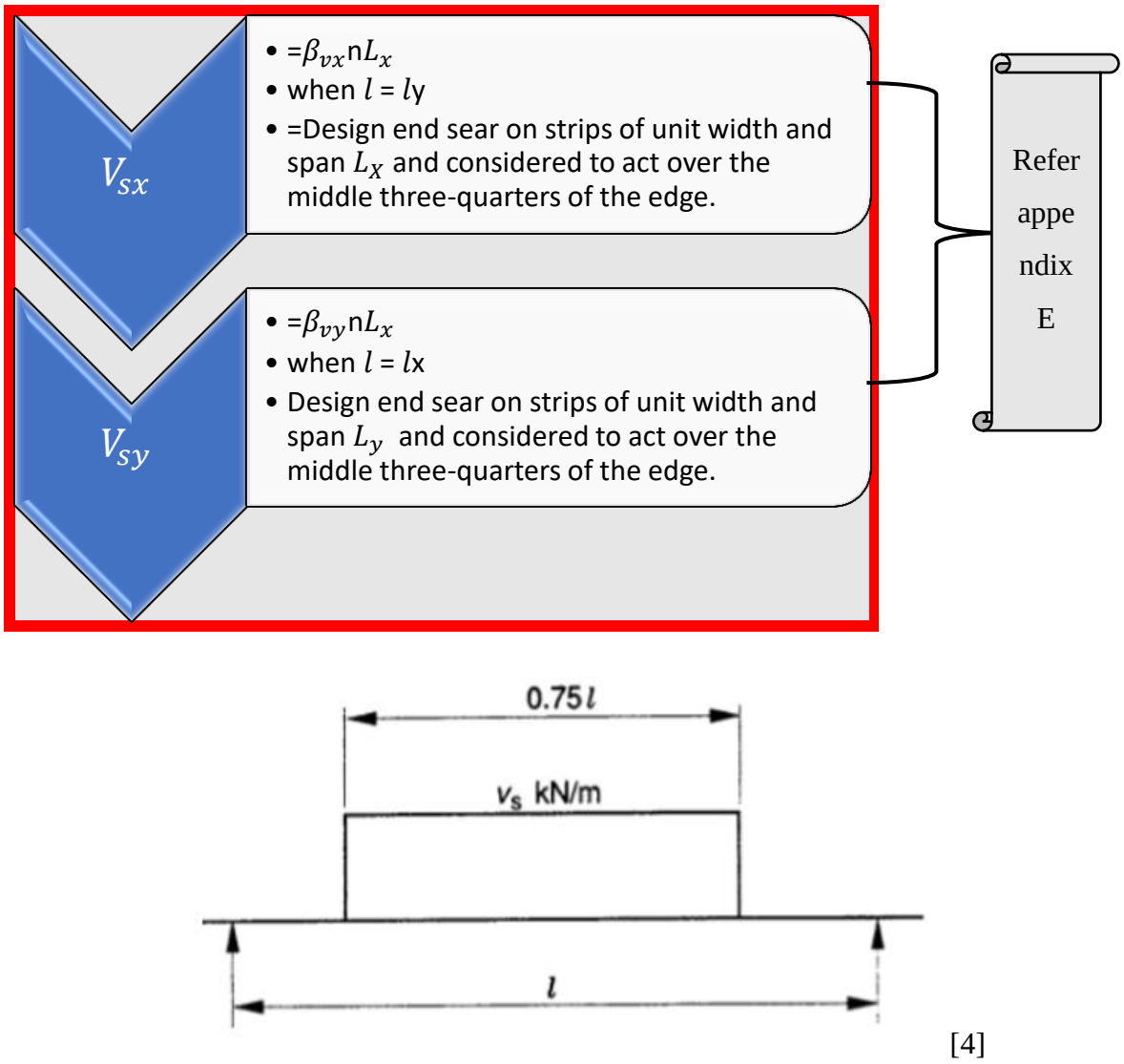
$DF34=\frac{K34}{K34+K45}=0.324$

$DF45=\frac{K45}{K45+K34}=0.676$

3.3.4 Load Transferring Mechanism

Since the design of the slab is done by coefficient method, loads are transferred to beam by load transferring mechanism. Dead 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

Design load on beams supporting solid slab spanning into directions at right angles:



[4]

Figure 19 Distribution of load on a beam supporting a two-way spanning slab

For frame analysis using ETABS, the input of loads should be unfactored loads; we will compute load transfer from panels to beams separately for dead, live and wind loads.

Dead load transfer

$VDx = \beta vx * DL * Lx$

$VDy = \beta vy * DL * Lx$

Live load transfer

$VLx = \beta vx * LL * Lx$

$VLy = \beta vy * LL * Lx$

Table 27 Un-factored Dead shear & Live shear transferring to beam

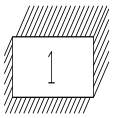
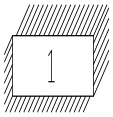
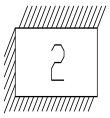
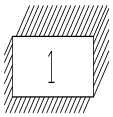
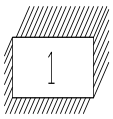
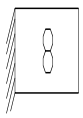
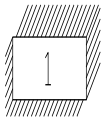
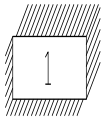
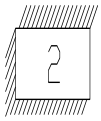
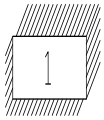
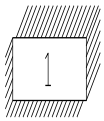
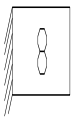
panel	support condition	Ly	Lx	Ly/Lx	DL	LL	coefficient		shear forces due to DL		shear forces due to LL	
S-1		5	4.6	1.09	6.80	3	β_{vxc} =	0.34	V_{xc} =	10.64	V_{xc} =	4.69
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.33	V_{yc} =	10.32	V_{yc} =	4.55
							β_{vyd} =	0	V_{yd} =	-	V_{yd} =	-
S-2		4.5	2.4	1.875	5.8	3	β_{vxc} =	0.49	V_{xc} =	6.82	V_{xc} =	3.53
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.33	V_{yc} =	4.59	V_{yc} =	2.38
							β_{vyd} =	0	V_{yd} =	-	V_{yd} =	-
S-3		4.5	2.4	1.88	5.80	3	β_{vxc} =	0.51	V_{xc} =	7.10	V_{xc} =	3.67
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.36	V_{yc} =	5.01	V_{yc} =	2.59
							β_{vyd} =	0.24	V_{yd} =	3.34	V_{yd} =	1.73
S-4		4	2.4	1.67	5.80	3	β_{vxc} =	0.39	V_{xc} =	5.43	V_{xc} =	2.81
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.33	V_{yc} =	4.59	V_{yc} =	2.38
							β_{vyd} =	0	V_{yd} =	-	V_{yd} =	-
S-5		5	4	1.25	10.0	3	β_{vxc} =	0.4	V_{xc} =	16.00	V_{xc} =	4.80
							β_{vxd} =	0	V_{xd} =	-	V_{xd} =	-
							β_{vyc} =	0.33	V_{yc} =	13.20	V_{yc} =	3.96
							β_{vyd} =	0	V_{yd} =	-	V_{yd} =	-
S-6		4.5	4	1.13	7.80	3	β_{vxc} =	0	V_{xc} =	-	V_{xc} =	-
							β_{vxd} =	0.34	V_{xd} =	10.61	V_{xd} =	4.08
							β_{vyc} =	0.45	V_{yc} =	14.04	V_{yc} =	5.40
							β_{vyd} =	0.3	V_{yd} =	9.36	V_{yd} =	3.60

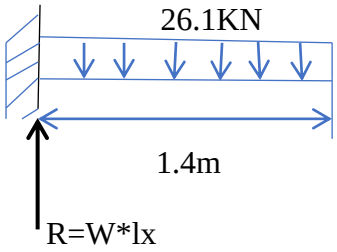
Table 28 Design shear for 2nd and 3rd floor slab

panel	support condition	Ly	Lx	Ly/Lx	DL	LL	Pd	coefficient		shear forces		90%
S-1		5	4.6	1.09	6.80	3	13.7	β_{vxc} =	0.34	V_{xc} =	21.40	19.26
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.33	V_{yc} =	20.77	18.69
								β_{vyd} =	0	V_{yd} =	-	-
S-2		4.5	2.4	1.875	5.8	3	12.3	β_{vxc} =	0.49	V_{xc} =	14.50	13.05
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.33	V_{yc} =	9.77	8.79
								β_{vyd} =	0	V_{yd} =	-	-
S-3		4.5	2.4	1.88	5.80	3	12.3	β_{vxc} =	0.51	V_{xc} =	15.09	13.58
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.36	V_{yc} =	10.65	9.59
								β_{vyd} =	0.24	V_{yd} =	7.10	6.39
S-4		4	2.4	1.67	5.80	3	12.3	β_{vxc} =	0.39	V_{xc} =	11.54	10.39
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.33	V_{yc} =	9.77	8.79
								β_{vyd} =	0	V_{yd} =	-	-
S-5		5	4	1.25	5.80	3	12.3	β_{vxc} =	0.4	V_{xc} =	19.73	17.76
								β_{vxd} =	0	V_{xd} =	-	-
								β_{vyc} =	0.33	V_{yc} =	16.28	14.65
								β_{vyd} =	0	V_{yd} =	-	-
S-6		4.5	4	1.13	7.80	3	15	β_{vxc} =	0	V_{xc} =	-	-
								β_{vxd} =	0.34	V_{xd} =	20.44	18.40
								β_{vyc} =	0.45	V_{yc} =	27.05	24.35
								β_{vyd} =	0.3	V_{yd} =	18.04	16.23

Load transfer cantilever panels

For C1

Design shear

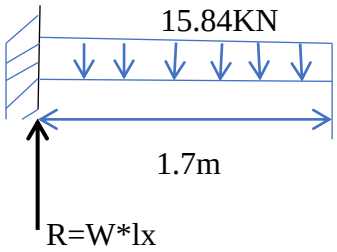


R= W*lx=26.1*1.4=36.54KN

90%=32.88

For C2

Design shear



R= W*lx=15.84*1.7=26.9KN

90%=24.2

3.3.5 Reinforcement design

Designing the slab for flexure

Reinforcement design is done using one of the three methods of calculation, those are: -

Using Design Table
Using Design Chart

We use the first method (Design table method) to calculate the area of reinforcement required for the design moment.

The spacing of bars should not exceed Smax,slab From ECBCS EN 1992 - 1 -1 : 2014

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**

Provision of principal reinforcement

d = D –cover –Φbar/2 = 180- 22 -12/2 = 152 mm

X-direction=180-(22+6) =152mm

Y-direction=180-(22+12+6) =140mm

Width, b=1000mm

Material data C=25

S=300

C=25, f_{ck}=$\frac{fcu}{1.25}$=$\frac{25}{1.25}$=20Mpa

Using S-300, f_{yk}=300Mpa

f_{yd}=$\frac{f_{yk}}{1.15}$=300/1.15=260.86Mpa

Using general design table no.1 from ES EN1992-1-1:2015

$$K_m = \sqrt{\frac{M_{sd,s}}{bd^2}}$$

$$A_{S=} = \frac{K_s M_{sd,s}}{d}$$

Using corresponding value of Km take Ks value from the table 1a

Design moment, M=8.48KNm

$$K_m = \sqrt{\frac{M_{sd,s}}{bd^2}} = \sqrt{\frac{8.48}{1*0.152^2}} = 19.16$$

$K_{s=}3.96$

$$A_{Scalc=} = \frac{K_s M_{sd,s}}{d} = \frac{3.96 * 8.48kNm}{0.152} = 220.9mm^2$$

Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slabs.

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

$fctm = 0.3(Fck^{2/3}) = 0.3 * 20^{2/3} = 2.2$

$$A_{s,min} = Max\begin{cases} \frac{0.26fctmbtd}{fyk} = \frac{0.26*2.2*1000*152}{300} = 289.8mm^2 \\ 0.00013btd = 0.00013 * 1000 * 152 = 19.76 \end{cases}$$ .....[3] $Art.9.2.1.1$ (1)

$$Asreq = Max\begin{cases} A_{s,calc} = 220.9mm^2 \\ A_{s,min} = 289.8mm^2 \end{cases}$$

Ast, max=0.04*Ac……...[3] Art.9.2.1.1 (3)

$$a_{s=} \frac{\pi d^2}{4} = \frac{3.14 * 12^2}{4} = 113.09mm^2$$

$$Spacing = \frac{\text{b} * \text{as}}{\text{Asreq}} = \frac{1000*113.04}{289.8} = 390.1mm$$

For slab maximum spacing is defined in section 9.3.1.1(3) of ES EN 1992-1-1

$$Sprovide = min\begin{cases} 3D = 3 * 180 = 540 \\ 400mm \\ Spacing = 390.1mm \end{cases}$$ .....[3]

Provide, Ø12 C/C 390.1mm

Table 29 Reinforcement calculation for 2nd and 3rd floors

panel	M	values	d(m)	Km	Ks	As(calc)	As(min)	AS(req)mm	spacing	S(prov)	remarks
S1	Mxs	8.48	0.152	19.16	3.96	220.93	289.8	289.8	271.91	271.912	Φ12C/C390.235
	Mys	8.99	0.14	21.42	3.966	254.67	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mxf	7.98	0.152	18.58	3.94	206.85	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Myf	8.79	0.14	21.18	3.96	248.63	289.8	289.8	390.23	390.235	Φ12C/C390.235
S2	Mxs	4.35	0.152	13.72	3.94	112.76	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mys	8.89	0.14	21.3	3.953	251.02	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mxf	3.28	0.152	11.91	3.934	84.89	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Myf	1.71	0.14	9.34	3.933	48.04	289.8	289.8	390.23	390.235	Φ12C/C390.235
S3	Mxs	5.49	0.152	15.41	3.944	142.45	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mys	8.48	0.14	20.8	3.953	239.44	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mxf	3.42	0.152	12.17	3.938	88.61	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Myf	1.22	0.14	7.89	3.931	34.26	289.8	289.8	390.23	390.235	Φ12C/C390.235
S4	Mxs	4.35	0.152	13.72	3.941	112.79	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mys	9.79	0.14	22.35	3.955	276.57	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Mxf	2.85	0.152	11.11	3.936	73.80	289.8	289.8	390.23	390.235	Φ12C/C390.235
	Myf	1.71	0.14	9.34	3.933	48.04	289.8	289.8	390.23	390.235	Φ12C/C390.235
S5	Mxs	13.29	0.152	23.98	3.958	346.06	289.8	346.06	326.79	326.789	Φ12C/C326.789
	Mys	12.67	0.14	25.42	3.96	358.4	289.8	358.38	315.56	315.559	Φ12C/C315.559
	Mxf	6.91	0.152	17.29	3.947	179.4	289.8	289.80	390.23	390.235	Φ12C/C390.235
	Myf	12.38	0.14	25.13	3.96	350.2	289.8	350.18	322.95	322.951	Φ12C/C322.951
S6	Mxs	13.29	0.152	23.98	3.958	346.1	289.8	346.06	326.79	326.789	Φ12C/C326.789
	Mxf	10.55	0.152	21.37	3.958	274.7	289.8	289.80	390.23	390.235	Φ12C/C390.235
	Myf	13.67	0.14	26.41	3.962	386.9	289.8	386.86	292.33	292.327	Φ12C/C392.327
C1	Mxf	26.42	0.152	33.82	4.09	710.9	289.8	710.91	159.08	159.079	Φ12C/C159.08
C2	Mxf	9.82	0.152	20.62	3.953	255.4	289.8	289.80	390.23	390.235	Φ12C/C390.235

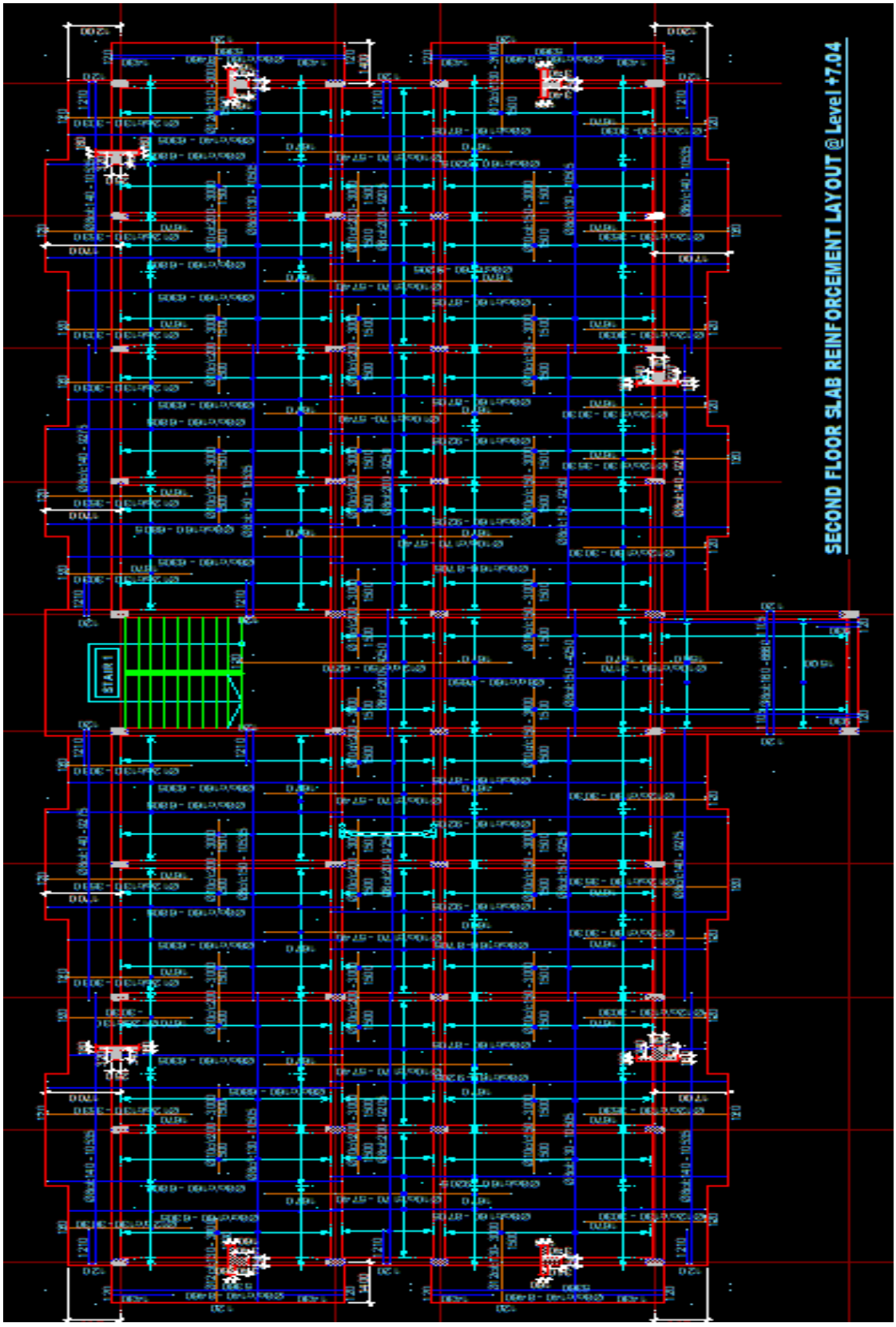


Figure 20 Reinforcement detail for 2nd and 3rd floor slab

3.3.6 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

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

$\rho = \frac{A_{st,req}}{bd} = \frac{289.8}{1000*152} = 0.19\%$ ……..[3]

$\rho o = \frac{\sqrt{fck}}{1000} = \sqrt{20}/1000 = 0.4472\%$

Now, $\rho = 0.19\% < \rho o = 0.44721\%$

$l/d = N * K$*F1*F2*F3 ……[3]

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \text{if } \rho \le \rho_o \quad (7.16a)$$

$$N = \left[11 + 1.5\sqrt{20}\frac{0.4472}{0.19} + 3.2\sqrt{20}\left(\frac{0.4472}{0.19} - 1\right)^{3/2}\right] = 49.33$$

Where

K……. from appendix F

F1=500 /($f_{yk}A_{s,req}$ / $A_{s,prov}$)……[3]…. (7.17)

F2&F3==1, (because span of slab ≤ 7m)…….[3] … section 5.3.2.2 (1).

Then we work similarly other slabs by the excel below

Table 30 Check Depth for Deflection 2nd and 3rd floor slab

panel	M	values	Lx	d(m)	Ks	As(calc)	As(min)	AS(req)mm	ρ	ρo	F1	K	Lx/d	d	check
S1	Mxs	8.48	4.5	0.152	3.96	220.93	289.8	289.80	0.191	0.44721	1.67	1.3	106.3	42.323	OK
	Mys	8.99		0.14	3.966	254.67	289.8	289.8	0.207	0.44721	1.67	1.3	94	47.875	OK
	Mxf	7.98		0.152	3.94	206.85	289.8	289.8	0.191	0.44721	1.67	1.3	106.3	42.323	OK
	Myf	8.79		0.14	3.96	248.63	289.8	289.8	0.207	0.44721	1.67	1.3	94	47.875	OK
S2	Mxs	4.35	2.4	0.152	3.94	112.76	289.8	289.8	0.191	0.44721	1.67	1.5	122.7	19.562	OK
	Mys	8.89		0.14	3.953	251.02	289.8	289.8	0.207	0.44721	1.67	1.5	108.5	22.129	OK
	Mxf	3.28		0.152	3.934	84.89	289.8	289.8	0.191	0.44721	1.67	1.5	122.7	19.562	OK
	Myf	1.71		0.14	3.933	48.04	289.8	289.8	0.207	0.44721	1.67	1.5	108.5	22.129	OK
S3	Mxs	5.49	2.4	0.152	3.944	142.45	289.8	289.8	0.191	0.44721	1.67	1.3	106.3	22.572	OK
	Mys	8.48		0.14	3.953	239.44	289.8	289.8	0.207	0.44721	1.67	1.3	94	25.533	OK
	Mxf	3.42		0.152	3.938	88.61	289.8	289.8	0.191	0.44721	1.67	1.3	106.3	22.572	OK
	Myf	1.22		0.14	3.931	34.26	289.8	289.8	0.207	0.44721	1.67	1.3	94	25.533	OK
S4	Mxs	4.35	2.4	0.152	3.941	112.79	289.8	289.8	0.191	0.44721	1.67	1.5	122.7	19.562	OK
	Mys	9.79		0.14	3.955	276.57	289.8	289.8	0.207	0.44721	1.67	1.5	108.5	22.129	OK
	Mxf	2.85		0.152	3.936	73.80	289.8	289.8	0.191	0.44721	1.67	1.5	122.7	19.562	OK
	Myf	1.71		0.14	3.933	48.04	289.8	289.8	0.207	0.44721	1.67	1.5	108.5	22.129	OK
S5	Mxs	13.29	4	0.152	3.958	346.06	289.8	346.06	0.228	0.44721	1.67	1.5	94.32	42.409	OK
	Mys	12.67		0.14	3.96	358.4	289.8	358.38	0.256	0.44721	1.67	1.5	79.9	50.064	OK
	Mxf	6.91		0.152	3.947	179.4	289.8	289.80	0.191	0.44721	1.67	1.5	122.7	32.604	OK
	Myf	12.38		0.14	3.96	350.2	289.8	350.18	0.25	0.44721	1.67	1.5	82.51	48.48	OK
S6	Mxs	13.29	4	0.152	3.958	346.1	289.8	346.06	0.228	0.44721	1.67	1.3	81.74	48.934	OK
	Mxf	10.55		0.152	3.958	274.7	289.8	289.80	0.191	0.44721	1.67	1.3	106.3	37.62	OK
	Myf	13.67		0.14	3.962	386.9	289.8	386.86	0.276	0.44721	1.67	1.3	62.43	64.067	OK
C1	Mxf	26.42	1.4	0.152	4.09	710.9	289.8	710.91	0.468	0.44721	1.67	0.4	####	#####	#NUM!
C2	Mxf	9.82	1.7	0.152	3.953	255.4	289.8	289.80	0.191	0.44721	1.67	0.4	32.72	51.963	OK

4 Stair case design

STAIR,:- are structures which provide access passage to different floor levels one of the structural elements constructed with steps rising without a break from floor to floor or with steps rising to landing between floors with a serious of steps rising further from the landing to floor above.

4.1 Design procedures

4.1.1 Concrete cover design

Determination of depth for deflection: which is a function of design tensile strength of steel, effective span length of the shortest span in which more load is expected to transfer and support condition

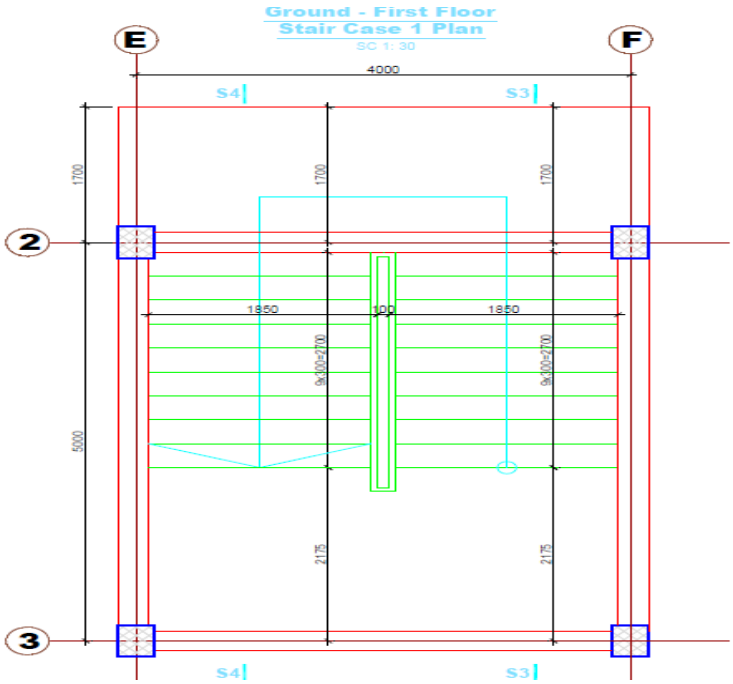
Loading: which determines the total load in the stair and landing

Analysis: determines moment and shear forces based on the analyzed moment

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

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

Detailing: the arrangement of reinforcement



Stair No. 2

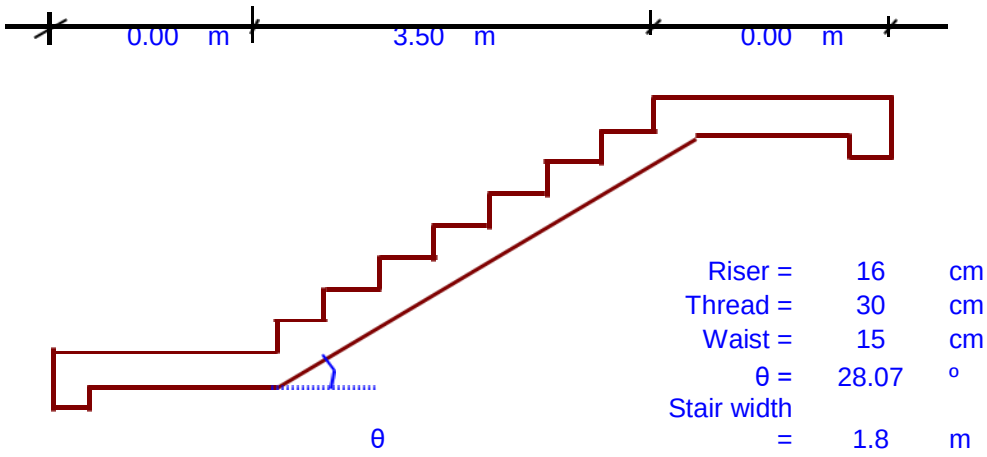


Figure 21 Stair case

Degree of weist=$\tan^{-1}\frac{0.16}{0.3}$=28

Clear height b/n landing and floor=1.6m

<u>Step 1</u> Concrete cover design

The concrete cover is the distance from the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface. The nominal cover shall be specified on the drawings.it is defined as Cmin

Cnom=Cmin+ΔCdev……[3] Art. 4.4.1.2

Where, Cmin – minimum cover and

ΔCdev is allowance in design for deviation

Minimum cover, shall provide in order to ensure

Safe transmission of bond force

Corrosion resistance/ Durability

Fire resistance

Cmin=max

Where;

Cmin, b -minimum cover due to bond requirement, see ES EN1992:2015 Art. 4.4.1.2 (3).

Cmin,dur - minimum cover due to environmental conditions, see [3] Art 4.4.1.2 (5)

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

ΔCdur,st -reduction of minimum cover for use of stainless steel, see [3] Art 4.4.1.2 (7) ΔCdur,add -reduction of minimum cover for use of additional protection, [3] Art 4.4.1.2 (8) But; the recommended value of ΔCdur,γ , ΔCdur,st , and ΔCdur,add is zero [3] Art. 4.4.1.2 (6, 7, and 8).

Cover Design for Bond

Assume Φ10 longitudinal bar and Φ25 nominal maximum aggregate size;

Therefore; Cmin, b=10mm

Note: The recommended Structural Class (design working life of 50 years) is S3 for the

Indicative concrete strengths given in Annex E and the recommended modifications to the

Structural class is given in Table 4.3N. But based on the above table the exposure class is reducing by 1 and the structural class would be S3.

The value of minimum cover required for durability of reinforcement steel is determined by using ES EN 1992:2015 tables 4.4N.

Using structural class, S3 and XC1:

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+ΔCdev=10+10=20mm

The live loads for design of stair case are to be taken from table 2.10 ES EN 1-1 on loading standards. The value used is category A (see Appendix C)

4.1.2 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 section 7.4.3, with a limit value

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

Where

K……. from appendix F

$$N = \left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \text{if } \rho > \rho_o$$ …(7.16a) refer Appendix F

$F1 = 500/(f_{yk}A_{s,req}/A_{s,prov})$……[3]…. (7.17)

F2&F3==1, (because span of slab ≤ 7m)…….[3] … section 5.3.2.2 (1).

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.

Where; $N = 11+1.5\sqrt{fck}\frac{\rho o}{\rho} + 3.2\sqrt{fck}(\frac{\rho o}{\rho} - 1)^{\frac{3}{2}}$ but ρo=ρ (assumption)

$\rho o = \sqrt{fck} * 10^{-3}$ = *=0.00447=0.0045%

N=11+1.5*20^0.5=17.708

F1=1.67

F2=1

F3=1

K=1.3

L=1.85m=1850mm

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

$\frac{l}{d}$=1.3*17.708*1.67=38.44mm

d= 48.12mm

After determining the value of effective depth

D = d + d',

d'= clear cover + ø/2

d'=20+6=26mm

D=48.12+26=74.12mm take D=136mm

Material used

C-25

S-300

4.1.3 Load computation

Material Data

Unit weight of ceramic finishing=21 KN/m^3

Unit weight of cement screed=23 KN/m^3

Unit weight of concrete=25 KN/m^3

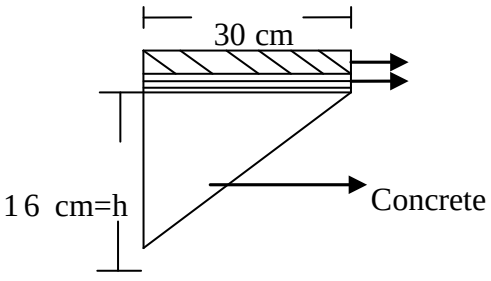
Unit weight of plastering=23 KN/m^3

Thickness of cement screed=3cm

Thickness of plastering=2cm

Step dead load

Take 1m width strip



❖ Loading

Dead load

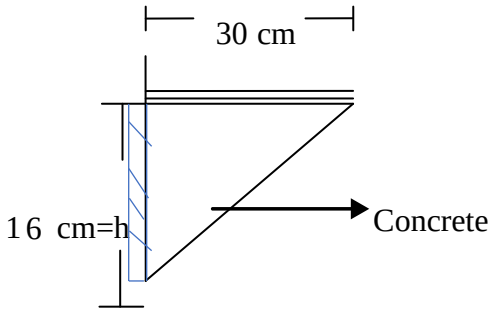
Cement Screed = tsc*rsc = 0.03*23 = 0.69KN/m

DL of ceramics finishing = tfn*rfn = 0.03*21 = 0.63 KN/m

DL of concrete 1/2*h*rc = 0.5*0.16*25 = 2KN/m

Total DL of step = 3.33KN/m

Dead Load of Riser



DL of cement screed = $\frac{no\ of\ riser(hcs*tsc*rsc)}{projected\ length} = \frac{9(0.16*0.03*23)}{2.7} = 0.42KN/m$

DLof ceramics finish = $\frac{no\ of\ riser(hcs*tsc*rsc)}{projected\ length} = \frac{9(0.16*0.03*21)}{2.7} = 0.494KN/m$

Total DL = 0.42+0.494 = 0.914KN/m

Waist Dead Load

$Tan^{-1}(1.6/2.7) = 30°$

Sinθ = 1.6/Linc, LInclined = 1.6/**Sin**θ= 1.6/sin30 = 3.2m

DL Of concrete = $\frac{DL\ inc*\gamma conc}{projected\ length} = \frac{0.15*3.2*25}{2.7} = 4.42KNm$

DL Plaster = $\frac{DL\ tpl*Linc*\gamma conc}{projected\ length} = \frac{0.02*3.2*23}{2.7} = 0.565KNm$

Total DL Waist =4.42+0.565 =5.966KN/M

Landing Dead load

DL landing =DL f/f+DLCeS+DL of conc +DL OF Plaster

=tceram*γparci+tcs*γcs+tc*γc+tpl*γpl

=0.03*21+0.03*23+0.15*25+0.02*23=5.76KN/M

Total dead load and design laod

For inclined slab

Total DL= DL of step +DLof riser+ DLwaist

=4.42+0.739+4.98

=9.139KN/M

Live load(LL)=3KN/M^2*1m (EBCS-2)

PD=1.35*9.138+1.5*3 =16.84KN/M

FOR landing

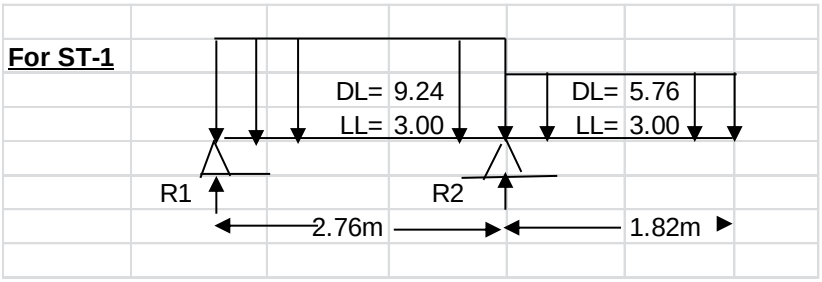
Live load (LL)=3KN/m^2*1m (EBCS-2)

DL=5.76KN/m

LL = 3KN/m^2 * 1 m = 2KN/m

Pd = 1.35*5.76 + 1.5*3=

= 12.23KN/m



Etabs out put for stair case

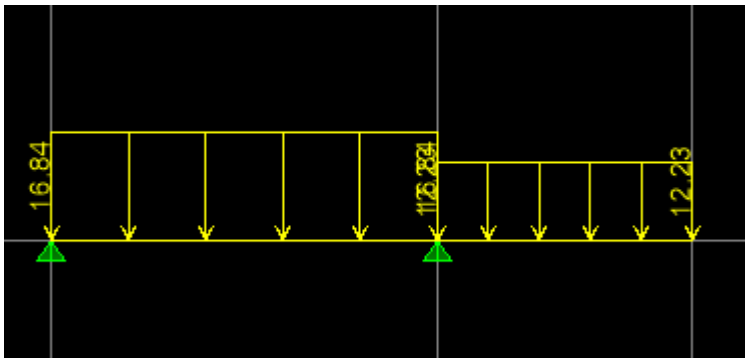


Figure 22Loadings on stair

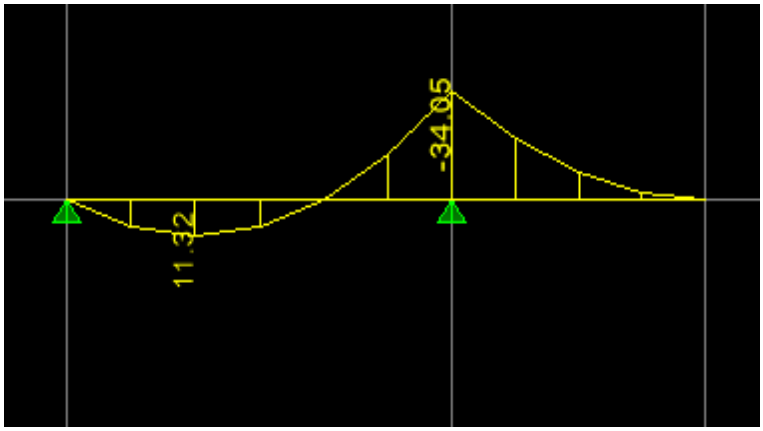


Figure 23 Bending moment diagram for stair

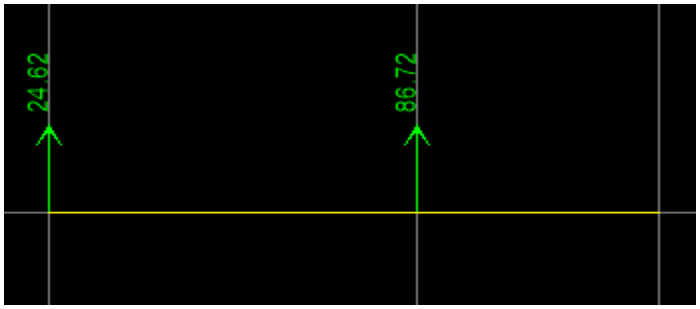


Figure 24 support reaction

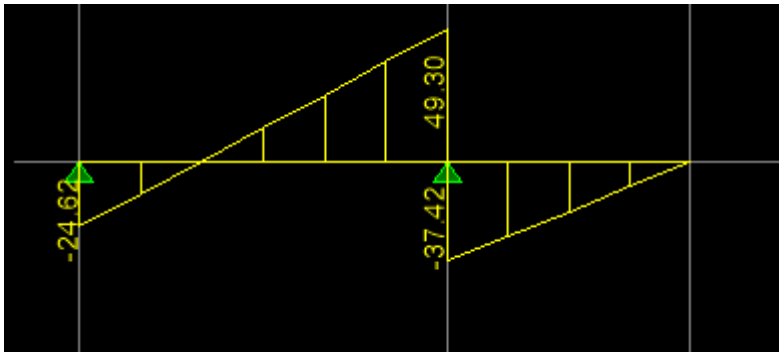


Figure 25 Shear force diagram

Moment and shear forces analyzed by ETABS v 9.6

4.1.4 Reinforcement calculation

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. $As, \geq 20\%\ As,$

Max design moment at support= 34.05KNm

Max design moment in the span= 11.32KNm

Considering maximum moment: M =34.05 KN.m

Material Data, C-25, S-300

$using\ \emptyset 12$

Effective depth=74.12mm=80mm

d = D –cover –Φbar/2 = 136-2-12/2 =110 mm

Width, b=1000mm

Moments calculated for each panels

Using design charts

For span moment

Using Mu = 34.05KN/m, calculate reinforcement,

Using general design **table no.1** from ES EN1992-1-1:2015

$K_m = \sqrt{\frac{M_{sd,s}}{bd^2}}$……[6]

$$A_{S=} = \frac{K_s M_{sd,s}}{d} \dots\dots\dots\dots\dots\dots [6]$$

Using corresponding value of Km take Ks value from the table 1a

Design moment, M=34.05KNm

$$K_m = \sqrt{\frac{M_{sd,s}}{bd^2}} = \sqrt{\frac{34.05}{1*0.11^2}} = 53.05 < K_m * = 57.83$$

Compression reinforcement is not required.

$$K_s = 4.514$$

$$A_{Scalc=} = \frac{K_s M_{sd,s}}{d} = \frac{4.512 * 34.05kNm}{0.11} = 1396mm^2$$

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

$fctm$ = 0.3($Fck^{2/3}$) =$0.3 * 20^{2/3} = 2.2$

$$A_{s,min}=Max\begin{cases}\frac{0.26fctmbtd}{fyk}=\frac{0.26*2.2*1000*110}{300}=209.7mm^2\\0.00013btd=0.00013*1000*110=14.3\end{cases}$$ .....[3] $Art.9.2.1.1$ (1)

$$Asreq=Max\begin{cases}A_{s,calc}=1396mm^2\\A_{s,min}=209.7mm^2\end{cases}$$

Ast, max=0.04*Ac……...[3] Art.9.2.1.1 (3)

$$a_{s=}\frac{\pi d^2}{4}=\frac{3.14*12^2}{4}=113.09mm^2$$

$$Spacing=\frac{\text{b}*\text{as}}{\text{Asreq}}=\frac{1000*113.04}{1396mm^2}=81mm=85$$

For slab maximum spacing is defined in section 9.3.1.1(3) of ES EN 1992-1-1

$$Sprovide=min\begin{cases}3D=3*136=408\\400mm\\Spacing=81mm\end{cases}$$ .....[3]

Provide, Ø12 C/C 85mm

B) Transverse reinforcement

According to Euro code 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.

- $As, \geq 20\%\ As,$

$A_{s,} \geq 0.2*$ 1396 =279mm2

Therefore take the minimum reinforcement

d=136-20-8-4=104mm, for Ø8 main transverse rebar

$A_{s=}279mm^2$

$$A_{s,min}=\frac{0.26fctmbtd}{fyk}=\frac{0.26*2.2*1000*104}{300}=198.3mm^2<A_{s,=}\ 279\text{mm2}$$

$$a_{s=}\frac{\pi d^2}{4}=\frac{3.14*8^2}{4}=50.24mm^2$$

$$Spacing=\frac{\text{b}*\text{as}}{\text{Asreq}}=\frac{1000*50.24}{279}=180mm$$

Provide, Ø8 C/C 180mm

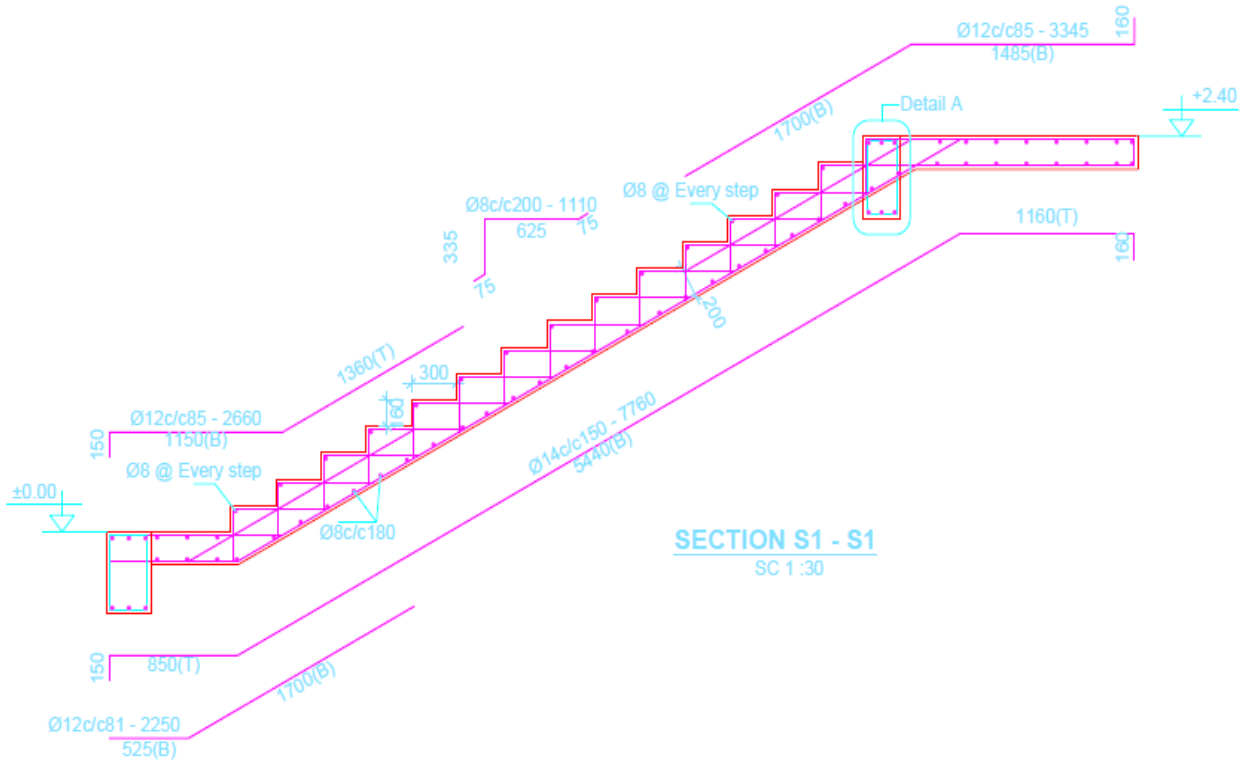


Figure 26 Detail f stair reinforcement

5 Lateral load analysis

5.1 Introduction

Building structure 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.

5.2 Earth quake analysis

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 $T1 \leq \begin{cases} 4TC \\ 2Sec \end{cases}$ …..[7]

5.2.1 Center of mass

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.

$$X_m = \frac{\sum W_i X_i}{\sum W_i}, \quad Y_m = \frac{\sum W_i Y_i}{\sum W_i} \quad \text{…….[7]}$$

Where X_m, and Y_m are the coordinate of the point of application of Fi when the seismic action is parallel to the Y-direction and X-direction respectively.

Table5.1: center of mass calculation for foundation

i. Column on	Width/Diameter	Depth	Height	Weight	Moment Arm		Moment	
					X	Y	Mx=Wi*Yi	My=Wi*Xi
	(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis - A1	0.25	0.4	1.5	3.75	0	0	0	0
Axis - A2	0.25	0.4	1.5	3.75	0	3	11.25	0
Axis - A3	0.25	0.4	1.5	3.75	0	5	18.75	0
Axis - A4	0.25	0.4	1.5	3.75	0	2.4	9	0
Axis - A5	0.25	0.4	1.5	3.75	0	5	18.75	0
Axis - A6	0.25	0.4	1.5	3.75	0	4.5	16.875	0
Axis - A7	0.25	0.4	1.5	3.75	0	4.5	16.875	0
Axis - B1	0.25	0.4	1.5	3.75	0	0	0	0
Axis - B2	0.25	0.4	1.5	3.75	4.5	3	11.25	16.875
Axis - B3	0.25	0.4	1.5	3.75	4.5	5	18.75	16.875
Axis - B4	0.25	0.4	1.5	3.75	4.5	2.4	9	16.875
Axis - B5	0.25	0.4	1.5	3.75	4.5	5	18.75	16.875
Axis - B6	0.25	0.4	1.5	3.75	4.5	4.5	16.875	16.875
Axis - B7	0.25	0.4	1.5	3.75	4.5	4.5	16.875	16.875
Axis - C1	0.25	0.4	1.5	3.75	9	0	0	33.75
Axis - C2	0.25	0.4	1.5	3.75	9	3	11.25	33.75
Axis - C3	0.25	0.4	1.5	3.75	9	5	18.75	33.75
Axis - C4	0.25	0.4	1.5	3.75	9	2.4	9	33.75
Axis - C5	0.25	0.4	1.5	3.75	9	5	18.75	33.75
Axis - C6	0.25	0.4	1.5	3.75	9	4.5	16.875	33.75
Axis - C7	0.25	0.4	1.5	3.75	9	4.5	16.875	33.75
Axis - D1	0.25	0.4	1.5	3.75	13.5	0	0	50.625
Axis - D2	0.25	0.4	1.5	3.75	13.5	3	11.25	50.625
Axis - D3	0.25	0.4	1.5	3.75	13.5	5	18.75	50.625
Axis - D4	0.25	0.4	1.5	3.75	13.5	2.4	9	50.625
Axis - D5	0.25	0.4	1.5	3.75	13.5	5	18.75	50.625
Axis - D6	0.25	0.4	1.5	3.75	13.5	4.5	16.875	50.625
Axis - D7	0.25	0.4	1.5	3.75	13.5	4.5	16.875	50.625
Axis -E1	0.25	0.4	1.5	3.75	18	0	0	67.5
Axis -E2	0.25	0.4	1.5	3.75	18	3	11.25	67.5
Axis -E3	0.25	0.4	1.5	3.75	18	5	18.75	67.5
Axis -E4	0.25	0.4	1.5	3.75	18	2.4	9	67.5
Axis -E5	0.25	0.4	1.5	3.75	18	5	18.75	67.5
Axis -E6	0.25	0.4	1.5	3.75	18	4.5	16.875	67.5
Axis -E7	0.25	0.4	1.5	3.75	18	4.5	16.875	67.5
Axis-F1	0.25	0.4	1.5	3.75	22	0	0	82.5
Axis-F2	0.25	0.4	1.5	3.75	22	3	11.25	82.5
Axis-F3	0.25	0.4	1.5	3.75	22	5	18.75	82.5
Axis-F4	0.25	0.4	1.5	3.75	22	2.4	9	82.5
Axis-F5	0.25	0.4	1.5	3.75	22	5	18.75	82.5
Axis-F6	0.25	0.4	1.5	3.75	22	4.5	16.875	82.5

Axis-F7	0.25	0.4	1.5	3.75	22	4.5	16.875	82.5
Axis-G1	0.25	0.4	1.5	3.75	26.5	0	0	99.375
Axis-G2	0.25	0.4	1.5	3.75	26.5	3	11.25	99.375
Axis-G3	0.25	0.4	1.5	3.75	26.5	5	18.75	99.375
Axis-G4	0.25	0.4	1.5	3.75	26.5	2.4	9	99.375
Axis-G5	0.25	0.4	1.5	3.75	26.5	5	18.75	99.375
Axis-G6	0.25	0.4	1.5	3.75	26.5	4.5	16.875	99.375
Axis-G7	0.25	0.4	1.5	3.75	26.5	4.5	16.875	99.375
Axis-H1	0.25	0.4	1.5	3.75	31	0	0	116.25
Axis-H2	0.25	0.4	1.5	3.75	31	3	11.25	116.25
Axis-H3	0.25	0.4	1.5	3.75	31	5	18.75	116.25
Axis-H4	0.25	0.4	1.5	3.75	31	2.4	9	116.25
Axis-H5	0.25	0.4	1.5	3.75	31	5	18.75	116.25
Axis-H6	0.25	0.4	1.5	3.75	31	4.5	16.875	116.25
Axis-H7	0.25	0.4	1.5	3.75	31	4.5	16.875	116.25
Axis-I1	0.25	0.4	1.5	3.75	35.5	0	0	133.125
Axis-I2	0.25	0.4	1.5	3.75	35.5	3	11.25	133.125
Axis-I3	0.25	0.4	1.5	3.75	35.5	5	18.75	133.125
Axis-I4	0.25	0.4	1.5	3.75	35.5	2.4	9	133.125
Axis-I5	0.25	0.4	1.5	3.75	35.5	5	18.75	133.125
Axis-I6	0.25	0.4	1.5	3.75	35.5	4.5	16.875	133.125
Axis-I7	0.25	0.4	1.5	3.75	35.5	4.5	16.875	133.125
Axis-J1	0.25	0.4	1.5	3.75	40	0	0	150
Axis-J2	0.25	0.4	1.5	3.75	40	3	11.25	150
Axis-J3	0.25	0.4	1.5	3.75	40	5	18.75	150
Axis-J4	0.25	0.4	1.5	3.75	40	2.4	9	150
Axis-J5	0.25	0.4	1.5	3.75	40	5	18.75	150
Axis-J6	0.25	0.4	1.5	3.75	40	4.5	16.875	150
Axis-J7	0.25	0.4	1.5	3.75	40	4.5	16.875	150
	Total Weight of Footing column			262.5			915	4535.625

$$X_m = \frac{\sum (W_i * X_i)}{\sum W_i} = \frac{4535.625}{262.5} = 17.27 \qquad Y_m = \frac{\sum (W_i * Y_i)}{\sum W_i} = \frac{915}{262.5} = 3.48$$

Table 31 center of mass calculation for Ground

			Ground mass	**Center**			
i. Column							
Column on	Width/Diameter	Depth	Height	Weight	Moment Arm		Moment	
					X	Y	Mx=Wi*Yi	My=Wi*Xi
	(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis - A1	0.25	0.4	3.84	9.6	0	0	0	0

Axis - A2	0.25	0.4	3.84	9.6	0	3	28.8	0
Axis - A3	0.25	0.4	3.84	9.6	0	5	48	0
Axis - A4	0.25	0.4	3.84	9.6	0	2.4	23.04	0
Axis - A5	0.25	0.4	3.84	9.6	0	5	48	0
Axis - A6	0.25	0.4	3.84	9.6	0	4.5	43.2	0
Axis - A7	0.25	0.4	3.84	9.6	0	4.5	43.2	0
Axis - B1	0.25	0.4	3.84	9.6	0	0	0	0
Axis - B2	0.25	0.4	3.84	9.6	4.5	3	28.8	43.2
Axis - B3	0.25	0.4	3.84	9.6	4.5	5	48	43.2
Axis - B4	0.25	0.4	3.84	9.6	4.5	2.4	23.04	43.2
Axis - B5	0.25	0.4	3.84	9.6	4.5	5	48	43.2
Axis - B6	0.25	0.4	3.84	9.6	4.5	4.5	43.2	43.2
Axis - B7	0.25	0.4	3.84	9.6	4.5	4.5	43.2	43.2
Axis - C1	0.25	0.4	3.84	9.6	9	0	0	86.4
Axis - C2	0.25	0.4	3.84	9.6	9	3	28.8	86.4
Axis - C3	0.25	0.4	3.84	9.6	9	5	48	86.4
Axis - C4	0.25	0.4	3.84	9.6	9	2.4	23.04	86.4
Axis - C5	0.25	0.4	3.84	9.6	9	5	48	86.4
Axis - C6	0.25	0.4	3.84	9.6	9	4.5	43.2	86.4
Axis - C7	0.25	0.4	3.84	9.6	9	4.5	43.2	86.4
Axis - D1	0.25	0.4	3.84	9.6	13.5	0	0	129.6
Axis - D2	0.25	0.4	3.84	9.6	13.5	3	28.8	129.6
Axis - D3	0.25	0.4	3.84	9.6	13.5	5	48	129.6
Axis - D4	0.25	0.4	3.84	9.6	13.5	2.4	23.04	129.6
Axis - D5	0.25	0.4	3.84	9.6	13.5	5	48	129.6
Axis - D6	0.25	0.4	3.84	9.6	13.5	4.5	43.2	129.6
Axis - D7	0.25	0.4	3.84	9.6	13.5	4.5	43.2	129.6
Axis -E1	0.25	0.4	3.84	9.6	18	0	0	172.8
Axis -E2	0.25	0.4	3.84	9.6	18	3	28.8	172.8
Axis -E3	0.25	0.4	3.84	9.6	18	5	48	172.8
Axis -E4	0.25	0.4	3.84	9.6	18	2.4	23.04	172.8
Axis -E5	0.25	0.4	3.84	9.6	18	5	48	172.8
Axis -E6	0.25	0.4	3.84	9.6	18	4.5	43.2	172.8
Axis -E7	0.25	0.4	3.84	9.6	18	4.5	43.2	172.8
Axis-F1	0.25	0.4	3.84	9.6	22	0	0	211.2
Axis-F2	0.25	0.4	3.84	9.6	22	3	28.8	211.2
Axis-F3	0.25	0.4	3.84	9.6	22	5	48	211.2
Axis-F4	0.25	0.4	3.84	9.6	22	2.4	23.04	211.2
Axis-F5	0.25	0.4	3.84	9.6	22	5	48	211.2
Axis-F6	0.25	0.4	3.84	9.6	22	4.5	43.2	211.2
Axis-F7	0.25	0.4	3.84	9.6	22	4.5	43.2	211.2
Axis-G1	0.25	0.4	3.84	9.6	26.5	0	0	254.4
Axis-G2	0.25	0.4	3.84	9.6	26.5	3	28.8	254.4
Axis-G3	0.25	0.4	3.84	9.6	26.5	5	48	254.4
Axis-G4	0.25	0.4	3.84	9.6	26.5	2.4	23.04	254.4
Axis-G5	0.25	0.4	3.84	9.6	26.5	5	48	254.4
Axis-G6	0.25	0.4	3.84	9.6	26.5	4.5	43.2	254.4
Axis-G7	0.25	0.4	3.84	9.6	26.5	4.5	43.2	254.4
Axis-H1	0.25	0.4	3.84	9.6	31	0	0	297.6
Axis-H2	0.25	0.4	3.84	9.6	31	3	28.8	297.6

Axis-H3	0.25	0.4	3.84	9.6	31	5	48	297.6
Axis-H4	0.25	0.4	3.84	9.6	31	2.4	23.04	297.6
Axis-H5	0.25	0.4	3.84	9.6	31	5	48	297.6
Axis-H6	0.25	0.4	3.84	9.6	31	4.5	43.2	297.6
Axis-H7	0.25	0.4	3.84	9.6	31	4.5	43.2	297.6
Axis-I1	0.25	0.4	3.84	9.6	35.5	0	0	340.8
Axis-I2	0.25	0.4	3.84	9.6	35.5	3	28.8	340.8
Axis-I3	0.25	0.4	3.84	9.6	35.5	5	48	340.8
Axis-I4	0.25	0.4	3.84	9.6	35.5	2.4	23.04	340.8
Axis-I5	0.25	0.4	3.84	9.6	35.5	5	48	340.8
Axis-I6	0.25	0.4	3.84	9.6	35.5	4.5	43.2	340.8
Axis-I7	0.25	0.4	3.84	9.6	35.5	4.5	43.2	340.8
Axis-J1	0.25	0.4	3.84	9.6	40	0	0	384
Axis-J2	0.25	0.4	3.84	9.6	40	3	28.8	384
Axis-J3	0.25	0.4	3.84	9.6	40	5	48	384
Axis-J4	0.25	0.4	3.84	9.6	40	2.4	23.04	384
Axis-J5	0.25	0.4	3.84	9.6	40	5	48	384
Axis-J6	0.25	0.4	3.84	9.6	40	4.5	43.2	384
Axis-J7	0.25	0.4	3.84	9.6	40	4.5	43.2	384
	Total Weight of Footing Column			672			2342.4	13396.8

ii.Beam

Beam on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-A	1&5	0.25	0.4	15.4	38.5	0	17.6	677.6	0
Axis-B	1&6	0.25	0.4	19.9	49.75	6.8	22.1	1099.475	338.3
Axis-C	1&6	0.25	0.4	19.9	49.75	11.3	22.1	1099.475	562.175
Axis-D	1&7	0.25	0.4	24.4	61	15.8	26.6	1622.6	963.8
Axis-E	1&7	0.25	0.4	24.4	61	20.3	26.6	1622.6	1238.3
Axis-F	1&7	0.25	0.4	24.4	61	24.3	26.6	1622.6	1482.3
Axis-G	1&7	0.25	0.4	24.4	61	28.8	26.6	1622.6	1756.8
Axis-H	1&6	0.25	0.4	19.9	49.75	33.3	22.1	1099.475	1656.675
Axis-I	1&6	0.25	0.4	19.9	49.75	37.8	22.1	1099.475	1880.55
Axis-J	1&5	0.25	0.4	15.4	38.5	42.3	17.6	677.6	1628.55
Axis-1	C&H	0.25	0.4	22	55	26.5	0	0	1457.5
Axis-2	A&J	0.25	0.4	40	100	42.3	5.2	520	4230
Axis-3	A&J	0.25	0.4	40	100	42.3	10.2	1020	4230
Axis-4	A&J	0.25	0.4	40	100	42.3	12.6	1260	4230
Axis-5	A&J	0.25	0.4	40	100	42.3	17.6	1760	4230
Axis-6	B&I	0.25	0.4	31	77.5	35.5	22.1	1712.75	2751.25
Axis-7	D&G	0.25	0.4	13	32.5	17.5	26.6	864.5	568.75
					1085				

Weight due to beam

Weight of the stair

iii. Stair									
Stair Type	Name	Width/Diameter	Length	DL	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(kN/m2)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Stair 1	Flight 1	1.85	2.7	9.13	45.60435	3.8	4.6	209.78001	173.29653
Stair 1	Flight 2	1.85	2.7	9.13	45.60435	3.8	4.75	216.620663	216.6206625
			Weight due to stair		91.2087				

Weight due to partition wall

iv. Partition Wall									
Wall on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-C&E	1&2	0.15	1.6	3	10.08	9	0	0	90.72
Axis-F&H	1&2	0.2	1.35	3	11.34	9	5.2	58.968	102.06
Axis-A&C	2&3	0.2	1.35	3	11.34	11.3	8	90.72	128.142
Axis H&J	2&3	0.2	3.2	3	26.88	9	8	215.04	241.92
Axis-A&C	4&5	0.2	1.35	3	11.34	11.3	7.4	83.916	128.142
Axis-F&H	4&5	0.2	1.35	3	11.34	9	7.4	83.916	102.06
Axis-H&J	4&5	0.2	3.2	3	26.88	9	7.4	198.912	241.92
Axis-D&F	5&6	0.15	1.6	3	10.08	8.5	9.5	95.76	85.68
Axis-F&G	5&6	0.2	3.2	3	26.88	4.5	9.5	255.36	120.96
	canti 1	0.2	1.3	5.4	19.656	2	5	98.28	39.312
	canti 2	0.15	1.7	5.4	19.278	2.2	7.4	142.6572	42.4116
			Weight due to partition wall		185.094				

Total Weight of Ground Floor 2033.30

Table 32 center of mass calculation for First floor

First Floor Center of Mass								
i. Column								
Column on	Width/Diameter	Depth	Height	Weight	Moment Arm		Moment	
					X	Y	Mx=Wi*Yi	My=Wi*Xi
	(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis - A1	0.25	0.4	3.84	9.6	0	0	0	0
Axis - A2	0.25	0.4	3.84	9.6	0	3	28.8	0
Axis - A3	0.25	0.4	3.84	9.6	0	5	48	0
Axis - A4	0.25	0.4	3.84	9.6	0	2.4	23.04	0
Axis - A5	0.25	0.4	3.84	9.6	0	5	48	0
Axis - A6	0.25	0.4	3.84	9.6	0	4.5	43.2	0
Axis - A7	0.25	0.4	3.84	9.6	0	4.5	43.2	0
Axis - B1	0.25	0.4	3.84	9.6	0	0	0	0
Axis - B2	0.25	0.4	3.84	9.6	4.5	3	28.8	43.2
Axis - B3	0.25	0.4	3.84	9.6	4.5	5	48	43.2
Axis - B4	0.25	0.4	3.84	9.6	4.5	2.4	23.04	43.2
Axis - B5	0.25	0.4	3.84	9.6	4.5	5	48	43.2
Axis - B6	0.25	0.4	3.84	9.6	4.5	4.5	43.2	43.2
Axis - B7	0.25	0.4	3.84	9.6	4.5	4.5	43.2	43.2
Axis - C1	0.25	0.4	3.84	9.6	9	0	0	86.4

Axis - C2	0.25	0.4	3.84	9.6	9	3	28.8	86.4
Axis - C3	0.25	0.4	3.84	9.6	9	5	48	86.4
Axis - C4	0.25	0.4	3.84	9.6	9	2.4	23.04	86.4
Axis - C5	0.25	0.4	3.84	9.6	9	5	48	86.4
Axis - C6	0.25	0.4	3.84	9.6	9	4.5	43.2	86.4
Axis - C7	0.25	0.4	3.84	9.6	9	4.5	43.2	86.4
Axis - D1	0.25	0.4	3.84	9.6	13.5	0	0	129.6
Axis - D2	0.25	0.4	3.84	9.6	13.5	3	28.8	129.6
Axis - D3	0.25	0.4	3.84	9.6	13.5	5	48	129.6
Axis - D4	0.25	0.4	3.84	9.6	13.5	2.4	23.04	129.6
Axis - D5	0.25	0.4	3.84	9.6	13.5	5	48	129.6
Axis - D6	0.25	0.4	3.84	9.6	13.5	4.5	43.2	129.6
Axis - D7	0.25	0.4	3.84	9.6	13.5	4.5	43.2	129.6
Axis -E1	0.25	0.4	3.84	9.6	18	0	0	172.8
Axis -E2	0.25	0.4	3.84	9.6	18	3	28.8	172.8
Axis -E3	0.25	0.4	3.84	9.6	18	5	48	172.8
Axis -E4	0.25	0.4	3.84	9.6	18	2.4	23.04	172.8
Axis -E5	0.25	0.4	3.84	9.6	18	5	48	172.8
Axis -E6	0.25	0.4	3.84	9.6	18	4.5	43.2	172.8
Axis -E7	0.25	0.4	3.84	9.6	18	4.5	43.2	172.8
Axis-F1	0.25	0.4	3.84	9.6	22	0	0	211.2
Axis-F2	0.25	0.4	3.84	9.6	22	3	28.8	211.2
Axis-F3	0.25	0.4	3.84	9.6	22	5	48	211.2
Axis-F4	0.25	0.4	3.84	9.6	22	2.4	23.04	211.2
Axis-F5	0.25	0.4	3.84	9.6	22	5	48	211.2
Axis-F6	0.25	0.4	3.84	9.6	22	4.5	43.2	211.2
Axis-F7	0.25	0.4	3.84	9.6	22	4.5	43.2	211.2
Axis-G1	0.25	0.4	3.84	9.6	26.5	0	0	254.4
Axis-G2	0.25	0.4	3.84	9.6	26.5	3	28.8	254.4
Axis-G3	0.25	0.4	3.84	9.6	26.5	5	48	254.4
Axis-G4	0.25	0.4	3.84	9.6	26.5	2.4	23.04	254.4
Axis-G5	0.25	0.4	3.84	9.6	26.5	5	48	254.4
Axis-G6	0.25	0.4	3.84	9.6	26.5	4.5	43.2	254.4
Axis-G7	0.25	0.4	3.84	9.6	26.5	4.5	43.2	254.4
Axis-H1	0.25	0.4	3.84	9.6	31	0	0	297.6

Axis-H2	0.25	0.4	3.84	9.6	31	3	28.8	297.6
Axis-H3	0.25	0.4	3.84	9.6	31	5	48	297.6
Axis-H4	0.25	0.4	3.84	9.6	31	2.4	23.04	297.6
Axis-H5	0.25	0.4	3.84	9.6	31	5	48	297.6
Axis-H6	0.25	0.4	3.84	9.6	31	4.5	43.2	297.6
Axis-H7	0.25	0.4	3.84	9.6	31	4.5	43.2	297.6
Axis-I1	0.25	0.4	3.84	9.6	35.5	0	0	340.8
Axis-I2	0.25	0.4	3.84	9.6	35.5	3	28.8	340.8
Axis-I3	0.25	0.4	3.84	9.6	35.5	5	48	340.8
Axis-I4	0.25	0.4	3.84	9.6	35.5	2.4	23.04	340.8
Axis-I5	0.25	0.4	3.84	9.6	35.5	5	48	340.8
Axis-I6	0.25	0.4	3.84	9.6	35.5	4.5	43.2	340.8
Axis-I7	0.25	0.4	3.84	9.6	35.5	4.5	43.2	340.8
Axis-J1	0.25	0.4	3.84	9.6	40	0	0	384
Axis-J2	0.25	0.4	3.84	9.6	40	3	28.8	384
Axis-J3	0.25	0.4	3.84	9.6	40	5	48	384
Axis-J4	0.25	0.4	3.84	9.6	40	2.4	23.04	384
Axis-J5	0.25	0.4	3.84	9.6	40	5	48	384
Axis-J6	0.25	0.4	3.84	9.6	40	4.5	43.2	384
Axis-J7	0.25	0.4	3.84	9.6	40	4.5		
	Total Weight of Footing Column			672			2342.4	13396.8

ii.Beam Weight of beam

Beam on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-A	1&5	0.3	0.4	15.4	46.2	0	17.6	813.12	0
Axis-B	1&6	0.3	0.4	19.9	59.7	6.8	22.1	1319.37	405.96
Axis-C	1&6	0.3	0.4	19.9	59.7	11.3	22.1	1319.37	674.61
Axis-D	1&7	0.3	0.4	24.4	73.2	15.8	26.6	1947.12	1156.56
Axis-E	1&7	0.3	0.4	24.4	73.2	20.3	26.6	1947.12	1485.96
Axis-F	1&7	0.3	0.4	24.4	73.2	24.3	26.6	1947.12	1778.76
Axis-G	1&7	0.3	0.4	24.4	73.2	28.8	26.6	1947.12	2108.16
Axis-H	1&6	0.3	0.4	19.9	59.7	33.3	22.1	1319.37	1988.01
Axis-I	1&6	0.3	0.4	19.9	59.7	37.8	22.1	1319.37	2256.66
Axis-J	1&5	0.3	0.4	15.4	46.2	42.3	17.6	813.12	1954.26
Axis-1	C&H	0.3	0.4	22	66	26.5	0	0	1749
Axis-2	A&J	0.3	0.4	40	120	42.3	5.2	624	5076
Axis-3	A&J	0.3	0.4	40	120	42.3	10.2	1224	5076
Axis-4	A&J	0.3	0.4	40	120	42.3	12.6	1512	5076
Axis-5	A&J	0.3	0.4	40	120	42.3	17.6	2112	5076
Axis-6	B&I	0.3	0.4	31	93	35.5	22.1	2055.3	3301.5
Axis-7	D&G	0.3	0.4	13	39	17.5	26.6	1037.4	682.5

	Weight due to beam	1302

iii. Stair

Stair Type	Name	Width/Diameter	Length	DL	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(kN/m2)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Stair 1	Flight 1	1.85	2.7	9.13	45.60435	3.8	4.6	209.78001	173.29653
Stair 1	Flight 2	1.85	2.7	9.13	45.60435	3.8	4.75	216.620663	216.6206625
			Weight due to stair		91.2087				

iv. Partition Wall

Wall on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN
Axis-A&B	2&3	0.15	1.6	3	10.08	5.8	0	0	58.
Axis-C&D	2&3	0.2	1.35	3	11.34	5.2	5	56.7	58.
Axis-E&F	2&3	0.2	1.35	3	11.34	4	5	56.7	45
Axis-G&H	2&3	0.2	3.2	3	26.88	4.5	5	134.4	120
Axis-I&J	2&3	0.2	1.35	3	11.34	9.7	5	56.7	109
Axis-A&B	4&5	0.2	1.35	3	11.34	0	5	56.7	
Axis-C&D	4&7	0.2	3.2	3	26.88	5.2	14	376.32	139
Axis-E&F	4&6	0.15	1.6	3	10.08	4	9.5	95.76	40
Axis-G&H	4&5	0.2	3.2	3	26.88	4.5	5	134.4	120
Axis-I&J	4&5	0.2	3.2	3	26.88	9.7	5	134.4	260
	canti 1	0.2	1.3	5.4	19.656	2	5	98.28	39.
	canti 2	0.15	1.7	5.4	19.278	2.2	7.4	142.6572	42.
		Weight due to partition wall			211.974				

Weight of first floor slab

v. Slab

Panel Name	Lx	Ly	DL	Weight	Moment Arm		Moment	
	(m)	(m)	(kN/m2)	(kN)	X (m)	Y (m)	Mx=Wi*Yi (kN.m)	My=Wi*Xi (kN.m)
S-1	4.5	5	6.9	155.25	4.5	5	776.25	698.625
S-2	2.4	4.5	5.8	62.64	4.5	2.4	150.336	281.88
S-3	2.4	4.5	5.8	62.64	4.5	2.4	150.336	281.88
S-4	2.4	4	10	96	4	2.4	230.4	384
S-5	4	5	7.8	156	4	5	780	624
S-6	4	4.5	6.9	124.2	4	5	621	496.8
S-7	3	4.5	6.9	93.15	4.5	3	279.45	419.175
S-8	1.425	4	5.8	33.06	4	1.4	46.284	132.24
C-1	4.5	5	16	360	1.4	5.4	1944	504
C-2	1.7	4.5	8.4	64.26	1.7	1.7	109.242	109.242
	weight due to slab			1207.2			5087.298	3931.842
Total Weight of First Floor				3484.38				

$$X_m = \frac{\sum (W_i * X_i)}{\sum W_i} = \frac{3931.842}{3484.38} = 1.128$$

$$Y_m = \frac{\sum (W_i * Y_i)}{\sum W_i} = \frac{5087.298}{3484.38} = 1.460$$

Table 33 center of mass calculation for Second floor

i. Column						
Column on	Width/Diameter	Depth	Height	Weight	Moment Arm	Moment

					X		Mx =Wi*Yi	My =Wi*Xi
	(m)	(m)	(m)	(kN)	(m)	m)	(kN.m)	(kN.m)
Axis - A1	0.25	0.4	3.84	9.6	0		0	0
Axis - A2	0.25	0.4	3.84	9.6	0		28.8	0
Axis - A3	0.25	0.4	3.84	9.6	0		48	0
Axis - A4	0.25	0.4	3.84	9.6	0	.4	23.04	0
Axis - A5	0.25	0.4	3.84	9.6	0		48	0
Axis - A6	0.25	0.4	3.84	9.6	0	.5	43.2	0
Axis - A7	0.25	0.4	3.84	9.6	0	.5	43.2	0
Axis - B1	0.25	0.4	3.84	9.6	0		0	0
Axis - B2	0.25	0.4	3.84	9.6	4.5		28.8	43.2
Axis - B3	0.25	0.4	3.84	9.6	4.5		48	43.2
Axis - B4	0.25	0.4	3.84	9.6	4.5	.4	23.04	43.2
Axis - B5	0.25	0.4	3.84	9.6	4.5		48	43.2
Axis - B6	0.25	0.4	3.84	9.6	4.5	.5	43.2	43.2
Axis - B7	0.25	0.4	3.84	9.6	4.5	.5	43.2	43.2
Axis - C1	0.25	0.4	3.84	9.6	9		0	86.4
Axis - C2	0.25	0.4	3.84	9.6	9		28.8	86.4
Axis - C3	0.25	0.4	3.84	9.6	9		48	86.4
Axis - C4	0.25	0.4	3.84	9.6	9	.4	23.04	86.4
Axis - C5	0.25	0.4	3.84	9.6	9		48	86.4
Axis - C6	0.25	0.4	3.84	9.6	9	.5	43.2	86.4
Axis - C7	0.25	0.4	3.84	9.6	9	.5	43.2	86.4
Axis - D1	0.25	0.4	3.84	9.6	13.5		0	129.6
Axis - D2	0.25	0.4	3.84	9.6	13.5		28.8	129.6

Axis - D3	0.25	0.4	3.84	9.6	13.5		48	129.6
Axis - D4	0.25	0.4	3.84	9.6	13.5	.4	23.04	129.6
Axis - D5	0.25	0.4	3.84	9.6	13.5		48	129.6
Axis - D6	0.25	0.4	3.84	9.6	13.5	.5	43.2	129.6
Axis - D7	0.25	0.4	3.84	9.6	13.5	.5	43.2	129.6
Axis - E1	0.25	0.4	3.84	9.6	18		0	172.8
Axis - E2	0.25	0.4	3.84	9.6	18		28.8	172.8
Axis - E3	0.25	0.4	3.84	9.6	18		48	172.8
Axis - E4	0.25	0.4	3.84	9.6	18	.4	23.04	172.8
Axis - E5	0.25	0.4	3.84	9.6	18		48	172.8
Axis - E6	0.25	0.4	3.84	9.6	18	.5	43.2	172.8
Axis - E7	0.25	0.4	3.84	9.6	18	.5	43.2	172.8
Axis-F1	0.25	0.4	3.84	9.6	22		0	211.2
Axis-F2	0.25	0.4	3.84	9.6	22		28.8	211.2
Axis-F3	0.25	0.4	3.84	9.6	22		48	211.2
Axis-F4	0.25	0.4	3.84	9.6	22	.4	23.04	211.2
Axis-F5	0.25	0.4	3.84	9.6	22		48	211.2
Axis-F6	0.25	0.4	3.84	9.6	22	.5	43.2	211.2
Axis-F7	0.25	0.4	3.84	9.6	22	.5	43.2	211.2
Axis-G1	0.25	0.4	3.84	9.6	26.5		0	254.4
Axis-G2	0.25	0.4	3.84	9.6	26.5		28.8	254.4
Axis-G3	0.25	0.4	3.84	9.6	26.5		48	254.4
Axis-G4	0.25	0.4	3.84	9.6	26.5	.4	23.04	254.4
Axis-G5	0.25	0.4	3.84	9.6	26.5		48	254.4
Axis-G6	0.25	0.4	3.84	9.6	26.5	.5	43.2	254.4

Axis-G7	0.25	0.4	3.84	9.6	26.5	.5	43.2	254.4
Axis-H1	0.25	0.4	3.84	9.6	31		0	297.6
Axis-H2	0.25	0.4	3.84	9.6	31		28.8	297.6
Axis-H3	0.25	0.4	3.84	9.6	31		48	297.6
Axis-H4	0.25	0.4	3.84	9.6	31	.4	23.04	297.6
Axis-H5	0.25	0.4	3.84	9.6	31		48	297.6
Axis-H6	0.25	0.4	3.84	9.6	31	.5	43.2	297.6
Axis-H7	0.25	0.4	3.84	9.6	31	.5	43.2	297.6
Axis-I1	0.25	0.4	3.84	9.6	35.5		0	340.8
Axis-I2	0.25	0.4	3.84	9.6	35.5		28.8	340.8
Axis-I3	0.25	0.4	3.84	9.6	35.5		48	340.8
Axis-I4	0.25	0.4	3.84	9.6	35.5	.4	23.04	340.8
Axis-I5	0.25	0.4	3.84	9.6	35.5		48	340.8
Axis-I6	0.25	0.4	3.84	9.6	35.5	.5	43.2	340.8
Axis-I7	0.25	0.4	3.84	9.6	35.5	.5	43.2	340.8
Axis-J1	0.25	0.4	3.84	9.6	40		0	384
Axis-J2	0.25	0.4	3.84	9.6	40		28.8	384
Axis-J3	0.25	0.4	3.84	9.6	40		48	384
Axis-J4	0.25	0.4	3.84	9.6	40	.4	23.04	384
Axis-J5	0.25	0.4	3.84	9.6	40		48	384
Axis-J6	0.25	0.4	3.84	9.6	40	.5	43.2	384
Axis-J7	0.25	0.4	3.84	9.6	40	.5	43.2	384
	Total Weight of Footing Column			672			2342.4	13396.8

ii.Beam

Beam on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-A	1&5	0.3	0.4	15.4	46.2	0	17.6	813.12	0
Axis-B	1&6	0.3	0.4	19.9	59.7	6.8	22.1	1319.37	405.96
Axis-C	1&6	0.3	0.4	19.9	59.7	11.3	22.1	1319.37	674.61
Axis-D	1&7	0.3	0.4	24.4	73.2	15.8	26.6	1947.12	1156.56
Axis-E	1&7	0.3	0.4	24.4	73.2	20.3	26.6	1947.12	1485.96
Axis-F	1&7	0.3	0.4	24.4	73.2	24.3	26.6	1947.12	1778.76
Axis-G	1&7	0.3	0.4	24.4	73.2	28.8	26.6	1947.12	2108.16
Axis-H	1&6	0.3	0.4	19.9	59.7	33.3	22.1	1319.37	1988.01
Axis-I	1&6	0.3	0.4	19.9	59.7	37.8	22.1	1319.37	2256.66
Axis-J	1&5	0.3	0.4	15.4	46.2	42.3	17.6	813.12	1954.26
Axis-1	C&H	0.3	0.4	22	66	26.5	0	0	1749
Axis-2	A&J	0.3	0.4	40	120	42.3	5.2	624	5076
Axis-3	A&J	0.3	0.4	40	120	42.3	10.2	1224	5076
Axis-4	A&J	0.3	0.4	40	120	42.3	12.6	1512	5076
Axis-5	A&J	0.3	0.4	40	120	42.3	17.6	2112	5076
Axis-6	B&I	0.3	0.4	31	93	35.5	22.1	2055.3	3301.5
Axis-7	D&G	0.3	0.4	13	39	17.5	26.6	1037.4	682.5
		Weight due to beam			1302				

iii. Stair

Stair Type	Name	Width/Diameter	Length	DL	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(kN/m2)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Stair 1	Flight 1	1.85	2.7	9.13	45.60435	3.8	4.6	209.78001	173.29653
Stair 1	Flight 2	1.85	2.7	9.13	45.60435	3.8	4.75	216.620663	216.6206625
			Weight due to stair		91.2087				

iv. partion wall

Wall on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-A&B	2&3	0.15	1.35	3.96	11.22	4.5	0	0	50.49
Axis-C&D	2&3	0.2	1.35	4.5	17.01	13.5	10.4	176.904	229.635
Axis-E&F	2&3	0.2	1.3	4.4	16.01	22	10.4	166.504	352.22
Axis-G&H	2&3	0.2	1.3	4.5	16.38	31	10.4	170.352	507.78
Axis-I&J	2&3	0.2	1.3	4.7	17.1	40	10.4	177.84	684
Axis-A&B	4&5	0.2	1.3	6.5	23.66	0	15.4	364.364	0
Axis-C&D	4&5	0.2	1.3	4.5	16.38	13.5	15.4	252.252	221.13
Axis-E&F	4&6	0.15	1.3	4.2	11.46	22	19.9	228.054	252.12
Axis-G&H	4&5	0.2	1.3	4.6	16.74	31	15.4	257.796	518.94
Axis-I&J	4&5	0.2	1.3	4.7	19.65	40	15.4	302.61	786
	canti 1	0.2	1.3	5.4	19.656	2	5	98.28	39.312
	canti 2	0.15	1.7	5.4	19.278	2.2	7.4	142.6572	42.4116
					204.544				

Weight due to partition wall

v. Slab

Panel Name	Lx	Ly	DL	Weight	Moment Arm		Moment	
	(m)	(m)	(kN/m2)	(kN)	X (m)	Y (m)	Mx=Wi*Yi (kN.m)	My=Wi*Xi (kN.m)
S-1	4.5	5	6.9	155.25	4.5	5	776.25	698.625
S-2	2.4	4.5	5.8	62.64	4.5	2.4	150.336	281.88
S-3	2.4	4.5	5.8	62.64	4.5	2.4	150.336	281.88
S-4	2.4	4	10	96	4	2.4	230.4	384
S-5	4	5	7.8	156	4	5	780	624
S-6	4	4.5	6.9	124.2	4	5	621	496.8
C-1	4.5	5	16	360	1.4	5.4	1944	504
C-2	1.7	4.5	8.4	64.26	1.7	1.7	109.242	109.242
weight due to slab				1080.99			4761.564	3380.427
Total Weight of second Floor				3350.742				

$$X_m = \frac{\sum (W_i * X_i)}{\sum W_i} = \frac{3380.427}{3350.742} = 1.008$$

$$Y_m = \frac{\sum (W_i * Y_i)}{\sum W_i} = \frac{4761.564}{3350.742} = 1.421$$

Table5.5: center of mass calculation for 3rd floor

i. Column

Column on	Width/Diameter	Depth	Height	Weight	Moment Arm	Moment	
					X	Y	Mx=Wi*Yi
	(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)
Axis - A1	0.25	0.4	3.84	9.6	0	0	0
Axis - A2	0.25	0.4	3.84	9.6	0	3	28.8
Axis - A3	0.25	0.4	3.84	9.6	0	5	48
Axis - A4	0.25	0.4	3.84	9.6	0	2.4	23.04
Axis - A5	0.25	0.4	3.84	9.6	0	5	48
Axis - A6	0.25	0.4	3.84	9.6	0	4.5	43.2
Axis - A7	0.25	0.4	3.84	9.6	0	4.5	43.2
Axis - B1	0.25	0.4	3.84	9.6	0	0	0

Axis - B2	0.25	0.4	3.84	9.6	4.5	3	28.8
Axis - B3	0.25	0.4	3.84	9.6	4.5	5	48
Axis - B4	0.25	0.4	3.84	9.6	4.5	2.4	23.04
Axis - B5	0.25	0.4	3.84	9.6	4.5	5	48
Axis - B6	0.25	0.4	3.84	9.6	4.5	4.5	43.2
Axis - B7	0.25	0.4	3.84	9.6	4.5	4.5	43.2
Axis - C1	0.25	0.4	3.84	9.6	9	0	0
Axis - C2	0.25	0.4	3.84	9.6	9	3	28.8
Axis - C3	0.25	0.4	3.84	9.6	9	5	48
Axis - C4	0.25	0.4	3.84	9.6	9	2.4	23.04
Axis - C5	0.25	0.4	3.84	9.6	9	5	48
Axis - C6	0.25	0.4	3.84	9.6	9	4.5	43.2
Axis - C7	0.25	0.4	3.84	9.6	9	4.5	43.2
Axis - D1	0.25	0.4	3.84	9.6	13.5	0	0
Axis - D2	0.25	0.4	3.84	9.6	13.5	3	28.8
Axis - D3	0.25	0.4	3.84	9.6	13.5	5	48
Axis - D4	0.25	0.4	3.84	9.6	13.5	2.4	23.04
Axis - D5	0.25	0.4	3.84	9.6	13.5	5	48
Axis - D6	0.25	0.4	3.84	9.6	13.5	4.5	43.2
Axis - D7	0.25	0.4	3.84	9.6	13.5	4.5	43.2
Axis -E1	0.25	0.4	3.84	9.6	18	0	0
Axis -E2	0.25	0.4	3.84	9.6	18	3	28.8
Axis -E3	0.25	0.4	3.84	9.6	18	5	48
Axis -E4	0.25	0.4	3.84	9.6	18	2.4	23.04
Axis -E5	0.25	0.4	3.84	9.6	18	5	48
Axis -E6	0.25	0.4	3.84	9.6	18	4.5	43.2
Axis -E7	0.25	0.4	3.84	9.6	18	4.5	43.2
Axis-F1	0.25	0.4	3.84	9.6	22	0	0
Axis-F2	0.25	0.4	3.84	9.6	22	3	28.8
Axis-F3	0.25	0.4	3.84	9.6	22	5	48
Axis-F4	0.25	0.4	3.84	9.6	22	2.4	23.04
Axis-F5	0.25	0.4	3.84	9.6	22	5	48
Axis-F6	0.25	0.4	3.84	9.6	22	4.5	43.2
Axis-F7	0.25	0.4	3.84	9.6	22	4.5	43.2
Axis-G1	0.25	0.4	3.84	9.6	26.5	0	0
Axis-G2	0.25	0.4	3.84	9.6	26.5	3	28.8
Axis-G3	0.25	0.4	3.84	9.6	26.5	5	48
Axis-G4	0.25	0.4	3.84	9.6	26.5	2.4	23.04
Axis-G5	0.25	0.4	3.84	9.6	26.5	5	48
Axis-G6	0.25	0.4	3.84	9.6	26.5	4.5	43.2
Axis-G7	0.25	0.4	3.84	9.6	26.5	4.5	43.2
Axis-H1	0.25	0.4	3.84	9.6	31	0	0
Axis-H2	0.25	0.4	3.84	9.6	31	3	28.8
Axis-H3	0.25	0.4	3.84	9.6	31	5	48
Axis-H4	0.25	0.4	3.84	9.6	31	2.4	23.04

Axis-H5	0.25	0.4	3.84	9.6	31	5	48
Axis-H6	0.25	0.4	3.84	9.6	31	4.5	43.2
Axis-H7	0.25	0.4	3.84	9.6	31	4.5	43.2
Axis-I1	0.25	0.4	3.84	9.6	35.5	0	0
Axis-I2	0.25	0.4	3.84	9.6	35.5	3	28.8
Axis-I3	0.25	0.4	3.84	9.6	35.5	5	48
Axis-I4	0.25	0.4	3.84	9.6	35.5	2.4	23.04
Axis-I5	0.25	0.4	3.84	9.6	35.5	5	48
Axis-I6	0.25	0.4	3.84	9.6	35.5	4.5	43.2
Axis-I7	0.25	0.4	3.84	9.6	35.5	4.5	43.2
Axis-J1	0.25	0.4	3.84	9.6	40	0	0
Axis-J2	0.25	0.4	3.84	9.6	40	3	28.8
Axis-J3	0.25	0.4	3.84	9.6	40	5	48
Axis-J4	0.25	0.4	3.84	9.6	40	2.4	23.04
Axis-J5	0.25	0.4	3.84	9.6	40	5	48
Axis-J6	0.25	0.4	3.84	9.6	40	4.5	43.2
Axis-J7	0.25	0.4	3.84	9.6	40	4.5	43.2
	Total Weight of Footing Column			672			2342.4

ii.Beam

Beam on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-A	1&5	0.3	0.4	15.4	46.2	0	17.6	813.12	0
Axis-B	1&6	0.3	0.4	19.9	59.7	6.8	22.1	1319.37	405.96
Axis-C	1&6	0.3	0.4	19.9	59.7	11.3	22.1	1319.37	674.61
Axis-D	1&7	0.3	0.4	24.4	73.2	15.8	26.6	1947.12	1156.56
Axis-E	1&7	0.3	0.4	24.4	73.2	20.3	26.6	1947.12	1485.96
Axis-F	1&7	0.3	0.4	24.4	73.2	24.3	26.6	1947.12	1778.76
Axis-G	1&7	0.3	0.4	24.4	73.2	28.8	26.6	1947.12	2108.16
Axis-H	1&6	0.3	0.4	19.9	59.7	33.3	22.1	1319.37	1988.01
Axis-I	1&6	0.3	0.4	19.9	59.7	37.8	22.1	1319.37	2256.66
Axis-J	1&5	0.3	0.4	15.4	46.2	42.3	17.6	813.12	1954.26
Axis-1	C&H	0.3	0.4	22	66	26.5	0	0	1749
Axis-2	A&J	0.3	0.4	40	120	42.3	5.2	624	5076
Axis-3	A&J	0.3	0.4	40	120	42.3	10.2	1224	5076
Axis-4	A&J	0.3	0.4	40	120	42.3	12.6	1512	5076
Axis-5	A&J	0.3	0.4	40	120	42.3	17.6	2112	5076
Axis-6	B&I	0.3	0.4	31	93	35.5	22.1	2055.3	3301.5
Axis-7	D&G	0.3	0.4	13	39	17.5	26.6	1037.4	682.5
		Weight due to beam			1302				

iii.

Stair

Stair Type	Name	Width/Diameter	Length	DL	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(kN/m2)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Stair 1	Flight 1	1.85	2.7	9.13	45.60435	3.8	4.6	209.78001	173.29653
Stair 1	Flight 2	1.85	2.7	9.13	45.60435	3.8	4.75	216.620663	216.620662
			Weight due to stair		91.2087				

iv. partion wall

Wall on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-A&B	2&3	0.15	1.35	3.96	11.22	4.5	0	0	50.49
Axis-C&D	2&3	0.2	1.35	4.5	17.01	13.5	10.4	176.904	229.635
Axis-E&F	2&3	0.2	1.3	4.4	16.01	22	10.4	166.504	352.22
Axis-G&H	2&3	0.2	1.3	4.5	16.38	31	10.4	170.352	507.78
Axis-I&J	2&3	0.2	1.3	4.7	17.1	40	10.4	177.84	684
Axis-A&B	4&5	0.2	1.3	6.5	23.66	0	15.4	364.364	0
Axis-C&D	4&5	0.2	1.3	4.5	16.38	13.5	15.4	252.252	221.13
Axis-E&F	4&6	0.15	1.3	4.2	11.46	22	19.9	228.054	252.12
Axis-G&H	4&5	0.2	1.3	4.6	16.74	31	15.4	257.796	518.94
Axis-I&J	4&5	0.2	1.3	4.7	19.65	40	15.4	302.61	786
	canti 1	0.2	1.3	5.4	19.656	2	5	98.28	39.312
	canti 2	0.15	1.7	5.4	19.278	2.2	7.4	142.6572	42.4116
					204.544				
		Weight due to partition wall							

v. Slab

Panel Name	Lx	Ly	DL	Weight	Moment Arm		Moment	
	(m)	(m)	(kN/m2)	(kN)	X (m)	Y (m)	Mx=Wi*Yi (kN.m)	My=Wi*Xi (kN.m)
S-1	4.5	5	6.9	155.25	4.5	5	776.25	698.625
S-2	2.4	4.5	5.8	62.64	4.5	2.4	150.336	281.88

S-3	2.4	4.5	5.8	62.64	4.5	2.4	150.336	281.88
S-4	2.4	4	10	96	4	2.4	230.4	384
S-5	4	5	7.8	156	4	5	780	624
S-6	4	4.5	6.9	124.2	4	5	621	496.8
C-1	4.5	5	16	360	1.4	5.4	1944	504
C-2	1.7	4.5	8.4	64.26	1.7	1.7	109.242	109.242
	weight due to slab			1080.99			4761.564	3380.427
	Total Weight of Third Floor			3350.742				

$$X_m = \frac{\sum (W_i * X_i)}{\sum W_i}$$ **= 3380.427 = 1.008**

3350.742

$$Y_m = \frac{\sum (W_i * Y_i)}{\sum W_i}$$ **= 4761.564 = 1.421**

3350.742

Table 34 center of mass calculation for Roof Floor

i. Column								
Column on	Width/Diameter	Depth	Height	Weight	Moment Arm		Moment	
					X	Y	Mx=Wi*Yi	My=Wi*Xi
	(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis - A1	0.25	0.3	3.5	6.56	0	0	0	0

Axis - A2	0.25	0.3	3.5	6.56	0	3	19.68	0
Axis - A3	0.25	0.3	3.5	6.56	0	5	32.8	0
Axis - A4	0.25	0.3	3.5	6.56	0	2.4	15.744	0
Axis - A5	0.25	0.3	3.5	6.56	0	5	32.8	0
Axis - A6	0.25	0.3	3.5	6.56	0	4.5	29.52	0
Axis - A7	0.25	0.3	3.5	6.56	0	4.5	29.52	0
Axis - B1	0.25	0.3	3.5	6.56	0	0	0	0
Axis - B2	0.25	0.3	3.5	6.56	4.5	3	19.68	29.52
Axis - B3	0.25	0.3	3.5	6.56	4.5	5	32.8	29.52
Axis - B4	0.25	0.3	3.5	6.56	4.5	2.4	15.744	29.52
Axis - B5	0.25	0.3	3.5	6.56	4.5	5	32.8	29.52
Axis - B6	0.25	0.3	3.5	6.56	4.5	4.5	29.52	29.52
Axis - B7	0.25	0.3	3.5	6.56	4.5	4.5	29.52	29.52
Axis - C1	0.25	0.3	3.5	6.56	9	0	0	59.04
Axis - C2	0.25	0.3	3.5	6.56	9	3	19.68	59.04
Axis - C3	0.25	0.3	3.5	6.56	9	5	32.8	59.04
Axis - C4	0.25	0.3	3.5	6.56	9	2.4	15.744	59.04
Axis - C5	0.25	0.3	3.5	6.56	9	5	32.8	59.04
Axis - C6	0.25	0.3	3.5	6.56	9	4.5	29.52	59.04
Axis - C7	0.25	0.3	3.5	6.56	9	4.5	29.52	59.04
Axis - D1	0.25	0.3	3.5	6.56	13.5	0	0	88.56
Axis - D2	0.25	0.3	3.5	6.56	13.5	3	19.68	88.56
Axis - D3	0.25	0.3	3.5	6.56	13.5	5	32.8	88.56
Axis - D4	0.25	0.3	3.5	6.56	13.5	2.4	15.744	88.56
Axis - D5	0.25	0.3	3.5	6.56	13.5	5	32.8	88.56
Axis - D6	0.25	0.3	3.5	6.56	13.5	4.5	29.52	88.56
Axis - D7	0.25	0.3	3.5	6.56	13.5	4.5	29.52	88.56
Axis -E1	0.25	0.3	3.5	6.56	18	0	0	118.08
Axis -E2	0.25	0.3	3.5	6.56	18	3	19.68	118.08
Axis -E3	0.25	0.3	3.5	6.56	18	5	32.8	118.08
Axis -E4	0.25	0.3	3.5	6.56	18	2.4	15.744	118.08
Axis -E5	0.25	0.3	3.5	6.56	18	5	32.8	118.08
Axis -E6	0.25	0.3	3.5	6.56	18	4.5	29.52	118.08
Axis -E7	0.25	0.3	3.5	6.56	18	4.5	29.52	118.08
Axis-F1	0.25	0.3	3.5	6.56	22	0	0	144.32
Axis-F2	0.25	0.3	3.5	6.56	22	3	19.68	144.32
Axis-F3	0.25	0.3	3.5	6.56	22	5	32.8	144.32
Axis-F4	0.25	0.3	3.5	6.56	22	2.4	15.744	144.32
Axis-F5	0.25	0.3	3.5	6.56	22	5	32.8	144.32
Axis-F6	0.25	0.3	3.5	6.56	22	4.5	29.52	144.32
Axis-F7	0.25	0.3	3.5	6.56	22	4.5	29.52	144.32
Axis-G1	0.25	0.3	3.5	6.56	26.5	0	0	173.84
Axis-G2	0.25	0.3	3.5	6.56	26.5	3	19.68	173.84
Axis-G3	0.25	0.3	3.5	6.56	26.5	5	32.8	173.84
Axis-G4	0.25	0.3	3.5	6.56	26.5	2.4	15.744	173.84

Axis-G5	0.25	0.3	3.5	6.56	26.5	5	32.8	173.84
Axis-G6	0.25	0.3	3.5	6.56	26.5	4.5	29.52	173.84
Axis-G7	0.25	0.3	3.5	6.56	26.5	4.5	29.52	173.84
Axis-H1	0.25	0.3	3.5	6.56	31	0	0	203.36
Axis-H2	0.25	0.3	3.5	6.56	31	3	19.68	203.36
Axis-H3	0.25	0.3	3.5	6.56	31	5	32.8	203.36
Axis-H4	0.25	0.3	3.5	6.56	31	2.4	15.744	203.36
Axis-H5	0.25	0.3	3.5	6.56	31	5	32.8	203.36
Axis-H6	0.25	0.3	3.5	6.56	31	4.5	29.52	203.36
Axis-H7	0.25	0.3	3.5	6.56	31	4.5	29.52	203.36
Axis-I1	0.25	0.3	3.5	6.56	35.5	0	0	232.88
Axis-I2	0.25	0.3	3.5	6.56	35.5	3	19.68	232.88
Axis-I3	0.25	0.3	3.5	6.56	35.5	5	32.8	232.88
Axis-I4	0.25	0.3	3.5	6.56	35.5	2.4	15.744	232.88
Axis-I5	0.25	0.3	3.5	6.56	35.5	5	32.8	232.88
Axis-I6	0.25	0.3	3.5	6.56	35.5	4.5	29.52	232.88
Axis-I7	0.25	0.3	3.5	6.56	35.5	4.5	29.52	232.88
Axis-J1	0.25	0.3	3.5	6.56	40	0	0	262.4
Axis-J2	0.25	0.3	3.5	6.56	40	3	19.68	262.4
Axis-J3	0.25	0.3	3.5	6.56	40	5	32.8	262.4
Axis-J4	0.25	0.3	3.5	6.56	40	2.4	15.744	262.4
Axis-J5	0.25	0.3	3.5	6.56	40	5	32.8	262.4
Axis-J6	0.25	0.3	3.5	6.56	40	4.5	29.52	262.4
Axis-J7	0.25	0.3	3.5	6.56	40	4.5	29.52	262.4
	Total Weight of Footing Column			459.2			1600.64	9154.48

ii.Beam

Beam on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-A	1&5	0.25	0.3	15.4	28.875	0	17.6	508.2	0
Axis-B	1&6	0.25	0.3	19.9	37.312	6.8	22.1	824.5952	253.7216
Axis-C	1&6	0.25	0.3	19.9	37.312	11.3	22.1	824.5952	421.6256
Axis-D	1&7	0.25	0.3	24.4	45.75	15.8	26.6	1216.95	722.85
Axis-E	1&7	0.25	0.3	24.4	45.75	20.3	26.6	1216.95	928.725
Axis-F	1&7	0.25	0.3	24.4	45.75	24.3	26.6	1216.95	1111.725
Axis-G	1&7	0.25	0.3	24.4	45.75	28.8	26.6	1216.95	1317.6
Axis-H	1&6	0.25	0.3	19.9	37.312	33.3	22.1	824.5952	1242.4896
Axis-I	1&6	0.25	0.3	19.9	37.312	37.8	22.1	824.5952	1410.3936
Axis-J	1&5	0.25	0.3	15.4	28.875	42.3	17.6	508.2	1221.4125
Axis-1	C&H	0.25	0.3	22	41.25	26.5	0	0	1093.125
Axis-2	A&J	0.25	0.3	40	75	42.3	5.2	390	3172.5
Axis-3	A&J	0.25	0.3	40	75	42.3	10.2	765	3172.5
Axis-4	A&J	0.25	0.3	40	75	42.3	12.6	945	3172.5
Axis-5	A&J	0.25	0.3	40	75	42.3	17.6	1320	3172.5
Axis-6	B&I	0.25	0.3	31	58.125	35.5	22.1	1284.5625	2063.4375
Axis-7	D&G	0.25	0.3	13	24.375	17.5	26.6	648.375	426.5625
					813.748				

Weight due to beam

iii. Stair

Stair Type	Name	Width/Diameter	Length	DL	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(kN/m2)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Stair 1	Flight 1	1.85	2.7	9.13	45.60435	3.8	4.6	209.78001	173.29653
Stair 1	Flight 2	1.85	2.7	9.13	45.60435	3.8	4.75	216.620663	216.6206625
			Weight due to stair		91.2087				

iv. Partion Wall

Wall on	Between Axis	Width/Diameter	Depth	Length	Weight	Moment Arm		Moment	
						X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)	(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-A&B	1&2	0.15	1.35	3.96	11.22	4.5	0	0	50.49
Axis-C&D	1&2	0.2	1.35	4.5	17.01	13.5	3	51.03	229.635
Axis-E&F	1&2	0.2	1.3	4.4	16.01	22	3	48.03	352.22
Axis-G&H	1&2	0.2	1.3	4.5	16.38	31	3	49.14	507.78
Axis-I&J	1&2	0.2	1.3	4.7	17.1	40	3	51.3	684
Axis-A&B	4&5	0.2	1.3	6.5	23.66	0	18.793	444.64238	0
Axis-C&D	4&5	0.2	1.3	4.5	16.38	13.5	18.793	307.82934	221.13
Axis-E&F	4&5	0.15	1.3	4.2	11.46	22	18.793	215.36778	252.12
Axis-G&H	4&5	0.2	1.3	4.6	16.74	31	18.793	314.59482	518.94
Axis-I&J	4&5	0.2	1.3	4.7	19.65	40	18.793	369.28245	786
	canti 1	0.2	1.3	5.4	19.656	2	5	98.28	39.312
	canti 2	0.15	1.7	5.4	19.278	2.2	7.4	142.6572	42.4116
					204.544				
	Weight due to partition wall								

Total Weight of Roof	1568.70

Table 35: center of mass calculation for Water tanker

i. Partition Wall										
Wall on	Between Axis	Width/Diameter		Depth	Length	Weight	Moment Arm		Moment	
							X	Y	Mx=Wi*Yi	My=Wi*Xi
		(m)		(m)	(m)	(kN)	(m)	(m)	(kN.m)	(kN.m)
Axis-E&F	2&4		0.15	1.6	3	10.08	4	5	50.4	40.32
		Weight of Water tank				10.08				

ii. Slab								
Panel Name	Lx	Ly	DL	Weight	Moment Arm		Moment	
	(m)	(m)	(kN/m2)	(kN)	X (m)	Y (m)	Mx=Wi*Yi (kN.m)	My=Wi*Xi (kN.m)
S-3	4	2.25	5.8	52.2	4	5	261	208.8
	weight due to slab			52.2				
	Total Weight of Water tank			62.28				

$$X_m = \frac{\sum (W_i * X_i)}{\sum W_i} = \frac{208.8}{62.28} = 3.35 \qquad Y_m = \frac{\sum (W_i * Y_i)}{\sum W_i} = \frac{261}{62.28} = 4.19$$

5.2.2 Base shear determination

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

where

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

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

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

λ is the correction factor, the value of which is equal to: $\lambda = 0{,}85$ 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.

(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:

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

where

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

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

For concrete moment resisting frame Ct=0.075, H=34.94m
T1=0.075*19.08 $^{0.75}$= 0.684sec

T1≤ min { 4Tc(s) =4*0.25=1sec ; 2sec

T1=0.684sec < 1sec….. ok
Take T1=0.684sec

Table 3.3: Values of the parameters describing the recommended Type 2 elastic response spectra

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 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 3.4: Recommended values of 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

Design spectrum (Sd(T)),

Story	Remark	Floors	Weight (kN)
GR	w1	Foundation	2033.3
1st	W1	Ground Floor Wt.	3484.38
2nd	W2	Wt. of First Floor	3350.742
3rd	W3	Wt. of Second Floor	3350.742
4th	RF	Wt. of Rf Floor	1568.7
5th	WT	Wt. of Water T	62.28
		Total Weight	**13850.144**

For the horizontal components of the seismic action the design spectrum, Sd(T), shall be defined by the following expressions.

For frame system and DCM, $q_o = 3\ \alpha_u/\alpha_1$

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

$q_o = 3\ \alpha_u/\alpha_1 - 0.2*3\ \alpha_u/\alpha_1 = 0.8 * 3\ \alpha_u/\alpha_1$

 α1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in a 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 sec the development of overall structural instability, while all other design actions remain constant. The factor αu may be obtaine static (pushover) global analysis.

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

(4)P For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

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

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

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

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

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.

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

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

The factor k_w reflecting the prevailing failure mode in structural systems with walls shall be 1.0 for frame and fram

$$q = q_o k_w \geq 1{,}5 \quad \text{.....[7]}$$

Kw =1
q=0.8*3*1.3*1=3.12

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

Sd(T) = Max{**0.26380.1962**
Sd(T)=0.2638

$F_b = S_d(T_1) * W_{Tot} = 2499.10$

$F_t = 0.07 T_1 F_b = 119.65$

Weight Summary According to Storey level								
Computation of Storey Shear Force(Fi)								
Level	Remark	Wi	hi	Wihi	Fb	Ft	Fb -Ft	Fi
Ground Floor	w1	2033.3	3.84	7807.872	2499.1	119.65	2379.45	384.56
First Floor	W1	3484.38	3.84	13380.0192	2499.1	119.65	2379.45	659.01
Second Floor	W2	3350.742	3.2	10722.3744	2499.1	119.65	2379.45	528.11
Third Floor	W3	3350.742	3.2	10722.3744	2499.1	119.65	2379.45	528.11
Roof Floor	W4	1568.7	3.5	5490.45	2499.1	119.65	2379.45	270.42
Water Tank	W5	62.28	3	186.84	2499.1	119.65	2379.45	9.2
				48309.93				2379.41

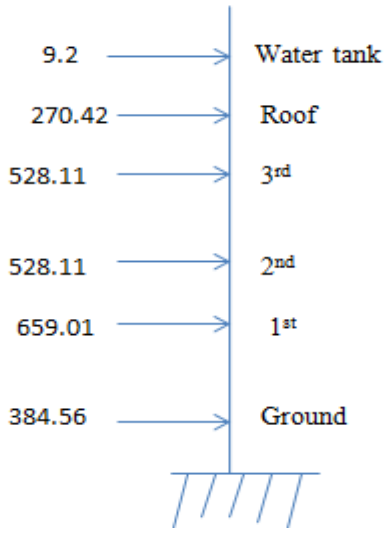


Figure 27 Total lateral load on the building

6 Frame Analyses

Buildings are designed to satisfy the basic structural and functional requirements. Thus, the design of a structure should be strong enough to withstand all lateral loads without excessive deformation or deflection. Normally in the design of building the lateral load due to earth quake and wind load shall be calculated and compared so that the maximum of the two is taken for design

- We analyze lateral load using ETABS V 9.6

6.1 Steps to analyze

Step 1

- Plot grid Coordinates that represent the given structural design

Step 2 Define Material

- We define two types of material those are C-25 Concrete and S-300 Rebar with their material

Properties.

I.e. ***C-25 Material type: concrete***

- Symmetry type: Isotropic
- Modulus of Elasticity: 29Gpa
- Poisson‘s ratio: 0.2
- Shear Modulus (G):10356491
- Coefficient of Thermal Expansion: 10E-06
- Unit Weight: 25 KN/m3

S300 : Material type: Rebar

- Unit Weight: 7.849 KN/m3.
- Modulus of Elasticity: 2.1E+08
- Poisson‘s ratio: 0.3
- Shear Modulus (G):76903069

- Coefficient Thermal Expansion: 1.17E-05
- Minimum Yield Stress, Fy: 260.870 Mpa
- Minimum Tensile Stress, Fu: 300.000 Mpa

Step 3 Define Frame Section

We define three types of Frame Section those are

- Rectangular column (40x30 cm),
- Floor Beam (40x30 cm)
- Grade beam(40x25)
- Roof beam(30x20)

Step 4 Draw the different Structural Members

Using the grid System Draw the structural Members with their Defined Frame Section

Properties.

It includes assignment of Restraints (fixed Joint).

Step 5 Assignment of Loads

First, Introduce Live Load on Definition of load Pattern then

We use Load Combination:

First, Introduce Live Load and load transferred from slab to the beam as dead load on Definition of load Pattern then,

: Here the total shear force assumed to be distributed over the entire interior to interior length of the beam. But 90% of the shear may also be taken to be distributed over the entire length.

Table 36Load combination

Combination	Factored Loading Combination	
Comb 1	1.35*DL+1.5*LL	-
Comb 2	0.75(1.35*DL+1.5*LL)	+EQX1
Comb 3	0.75(1.35*DL+1.5*LL)	- EQX1
Comb 4	0.75(1.35*DL+1.5*LL)	+EQx2
Comb 5	0.75(1.35*DL+1.5*LL)	-EQx2
Comb 6	0.75(1.35*DL+1.5*LL)	+EQY1
Comb 7	0.75(1.35*DL+1.5*LL)	-EQY1
Comb 8	0.75(1.35*DL+1.5*LL)	+EQY2
Comb 9	0.75(1.35*DL + 1.5 LL)	-EQY2
Comb 10	DL + LL(serviceability)	
Comb 11	ENV(comb2+comb3+comb4+comb5)	
Comb 12	ENV(comb6+comb7+comb8+comb+9)	

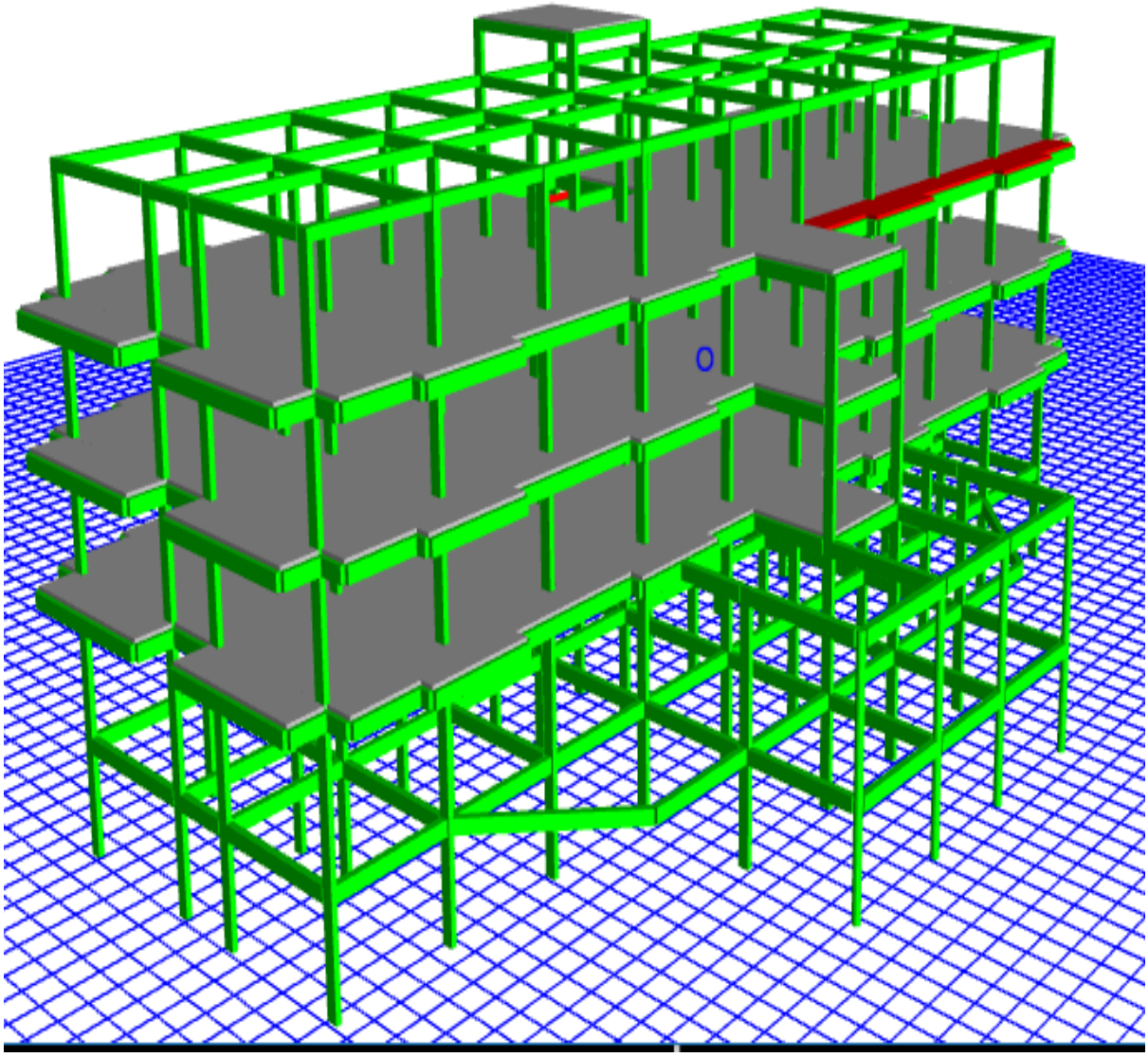


Figure 28 3Dimensional view of the building

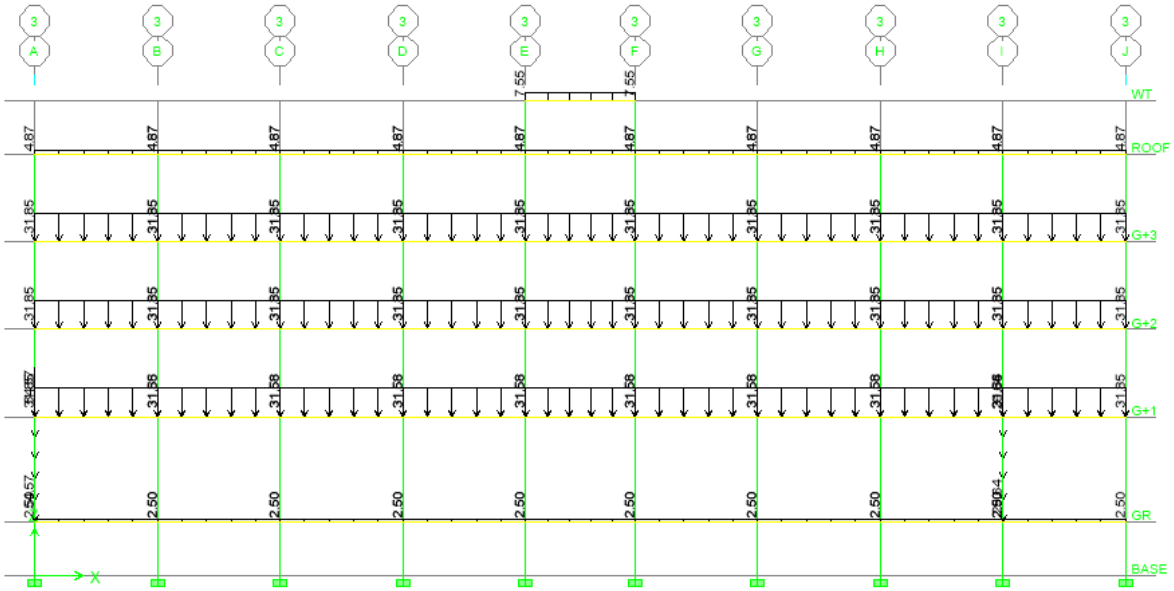


Figure 29 Transferred load to the beam which means 90% of the shear

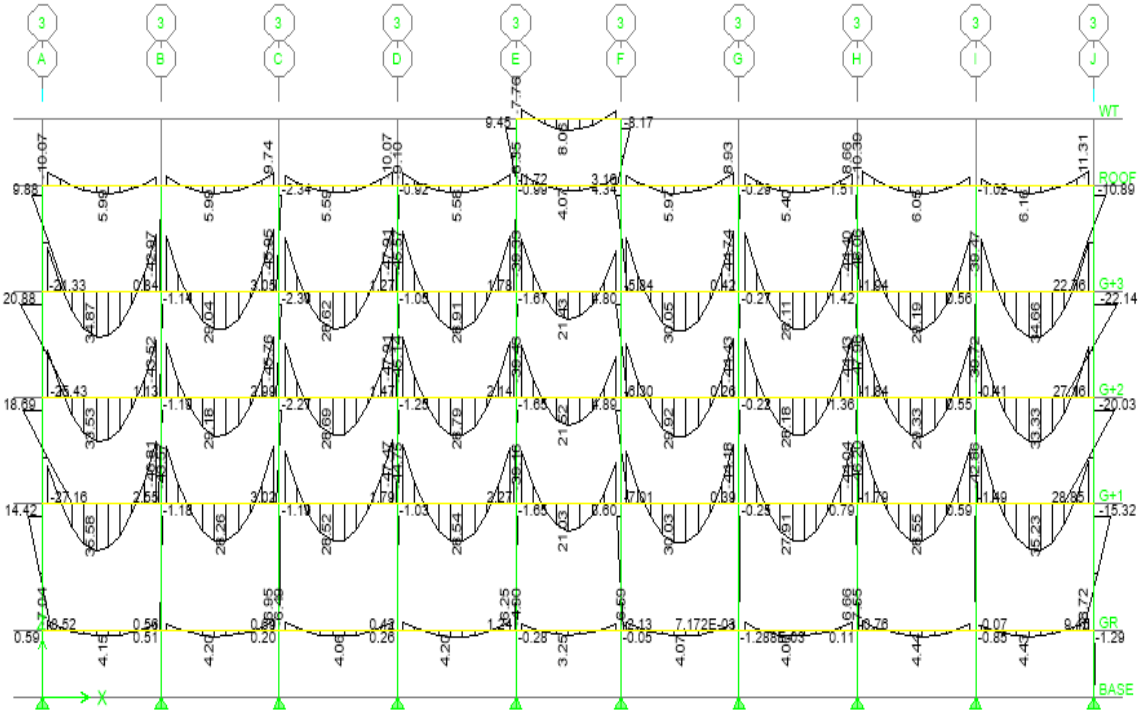


Figure 30 bending moment diagram

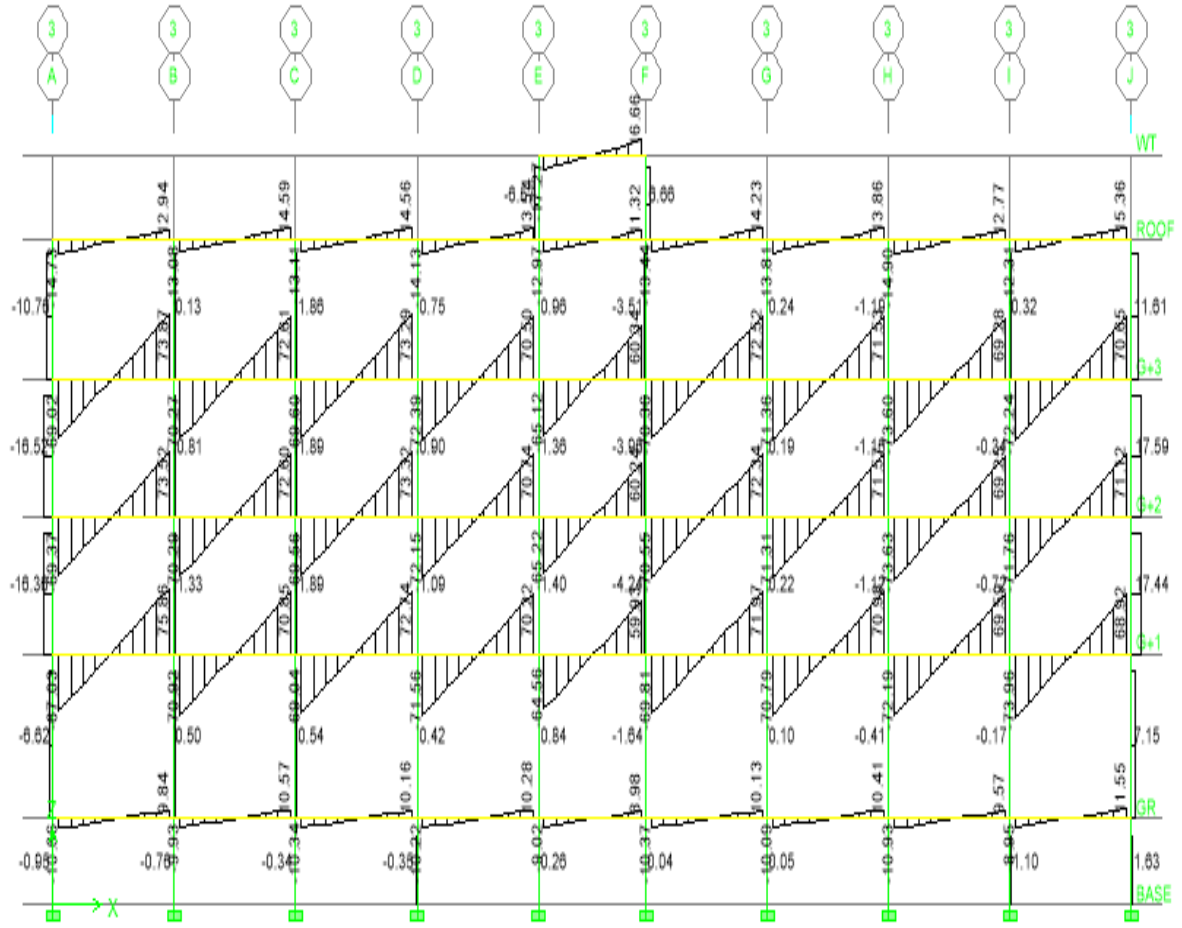


Figure 31 Shear force diagram

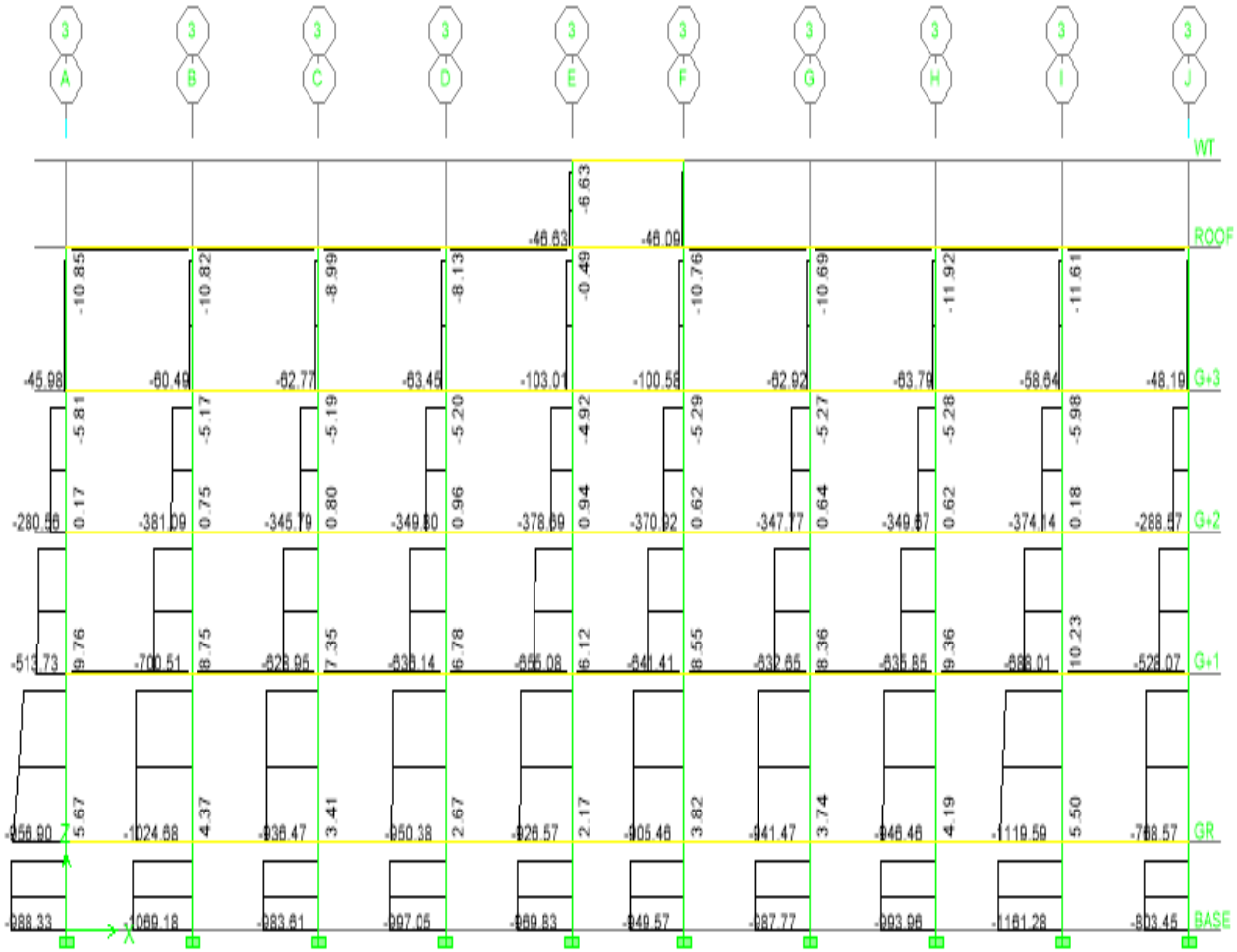


Figure 32 Axial force diagram

7 Beam design

Beams are flexural members which are used to transfer the loads from slab to columns. Basically beams should be designed for flexure (moment). Furthermore, it is essential to check and design the beam sections for torsion and shear. Beams may be designed for flexural moment depending on the magnitude of the moment and the X- sectional dimensions. On the other hand, the beam can be singly reinforced, doubly reinforced section. The objective of reinforced beam design in accordance with ESEN-2015 is to ensure that the member will behave satisfactory under working load. To build a beam carries loads coming from slabs and capable of resisting the forces acting on it.

We have used the following data:

Grade beam=40x25

Intermediate Beam - 40cm x 25cm

Roof beam =30x30cm

Material Data

C-25

S-300

Using Φ16mm

Class-I work

Constants

- *f*cd= 0.85fck/ c=0.85*(25/1.25)Mpa/1.5=11.33 Mpa
- *f*yd =fyk/ s=300 Mpa/1.15=260.87 Mpa

Procedures to design

7.1 Concrete Cover Determination

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

$c_{nom} = c_{min} + \Delta c_{dev}$ ..... [3]

Minimum concrete cover, c_{min} shall be provided in order to ensure:

- the safe transmission of bond forces (see also Sections 7 and 8)
- the protection of the steel against corrosion (durability)
- an adequate fire resistance (see EN 1992-1-2)

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 $C_{min,b}$ given in Table 4.2. [3]

-Slab reinforcement φ16

With: Cmin,b = 16 mm

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

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

Therefore the value of minimum cover required for durability of reinforcement steel is determined using ES EN 1992:2015 table 4.4N.(refer appendix B)

cmin = max (cmin,b; cmin,dur + Δcdur,γ - Δcdur,st - Δcdur,add; 10 mm

Table 37 exposure class to find depth of the slab.

environmental requirement				
exposure class according to table 4.4				
structural class	X0	xc1	Xc2/xc3	xc4
s1	10	10	10	15
s2	10	10	15	20
s3	10	10	20	25
s4	10	15	25	30
s5	15	20	30	35
s6	20	25	35	40

We obtain from table 4.4N

- ✓ cmin,dur = 10 mm

Moreover:

- ✓ Δcdur,γ = 0 ;
- ✓ Δcdur,st = 0 ;
- ✓ Δcdur,add = 0 .

From relation (3.2):

cmin = max (cmin,b; cmin,dur + Δcdur,γ - Δcdur,st - Δcdur,add; 10 mm)

= max (16; 10 + 0 – 0 – 0; 10 mm) = 16mm

Δc_{dev}=10mm (recommended)

$Cnom=Cmin+\Delta Cdev$=16mm+10mm=26mm

7.2 Effective Depth Determination: Serviceability requirement

Serviceability requirement [3]

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

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

$$N = \left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right] \text{if } \rho > \rho_o \quad [3]$$

……..

Initially we can't know ρ and ρ'. So let's assume ρ= ρo and use equation 7.16a. As, req = As, provided(Refer appendix F)

Assumptions

Initially we can't know ρ and ṕ. So let's assume ρ=ρo and equation 7.16a. AS, req= A_s, provided

Where; N = $11+1.5\sqrt{fck}\frac{\rho o}{\rho} + 3.2\sqrt{fck}(\frac{\rho o}{\rho} - 1)^{\frac{3}{2}}$ [3] but ρo=ρ (assumption)

ρo=$\sqrt{fck} * 10^{-3}$ = *=0.00447=0.0045%

N=11+1.5*20^0.5=17.708

- F1==1.67
- F2=1
- F3=1

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

l/d=17.708*1.3*1.67*1*1=38.444

L=5000mm

d=5000/38.444=130.05mm=130mm

After determining the value of effective depth

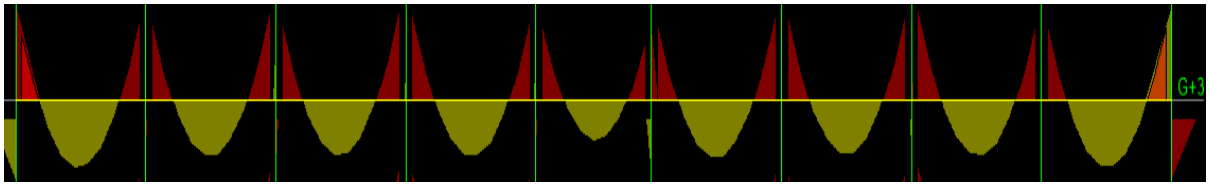
D = d + d',

d'= Cc+$\varnothing_{stirrup}+\varnothing/2$ using Φ8 stirrup

d'=26+16/2+8 =26+8+8=42mmd

D=130mm+42mm=172mm=200mm

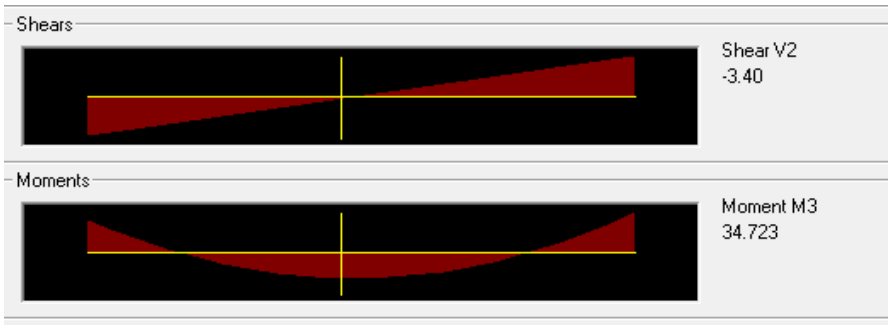
Figure 33 Sample of moment of the beam on axis 3-3 A



Design for flexure (using design chart)

For axis 3 span AB

Sample moment and shear



Mmax=34.723kNm

D=300mm

d=300mm-26mm-8-8=258mm

$$\mu_{sd,s} = \frac{M_{sd,s}}{f_{cd} * bd^2} = \frac{34.72 * 10^6}{11.33 * 250 * 258^2} = 0.184$$

$$d \geq \sqrt{\frac{M_{sd,s}}{\mu_{sd,s} f_{cd} * b}} = \sqrt{\frac{34.723 * 10^6}{0.184 * 11.33 * 250}} = 258.2\text{mm}$$ Not ok!!!!!!!!!!

Use D=400mm

d=400mm-42mm=358mm

$$\mu_{sd,s} = \frac{M_{sd,s}}{f_{cd} * bd^2} = \frac{34.72 * 10^6}{11.33 * 250 * 358^2} = 0.1$$

$$d \geq \sqrt{\frac{M_{sd,s}}{\mu_{sd,s} f_{cd} * b}} = \sqrt{\frac{34.723 * 10^6}{1 * 11.33 * 250}} = 110\text{mm} < 358\text{mm}$$ ok!!!!!!!!!!

D=110+26+8+8=152<400mm

d=400-42=358mm

kz from the general design chart No 1A [3]

Z_k=k*d ……..EN ES-2 2015

Then $\mu_{sd,s} = 0.1$
K=0.94

Z=0.94*358mm=331.82mm^2

f_{yd}=300 Mpa/1.15=260.87 Mpa

$$A_{S=} = \frac{M_{sd,s}}{Z * f_{yd}} = \frac{34.723 * 10^6}{331.82 * 260.87} = 401.2mm^2$$

7.3 Check the minimum and maximum limit for area of reinforcement

$$A_{s,min}=Max\begin{cases}\frac{0.26fctmbtd}{fyk} = \frac{0.26*2.2*250*358}{300} = 170.6mm^2 < 401.2mm^2 \\ 0.00013btd = 0.00013 * 250 * 358 = 11.63mm\end{cases}$$ ...[3] $Art.9.2.1.1$

$170.6mm^2 < 401.2mm^2$ …… ok!!!

A_{st}, max=0.04*Ac=0.04*250*400=4000$mm^2 > A_S$…OK!!!!……...[3] Art.9.2.1.1 (3)

1) Calculate the number of reinforcement

Use Ø16 bars and Ø8 stirrups

$$a_{s=} \frac{\pi d^2}{4} = \frac{3.14 * 16^2}{4} = 200.96mm^2$$

$$N=\frac{A_S}{as} = \frac{401.2mm^2}{200.96mm^2} = 1.99 = 2$$

2) Spacing of bars according to EN 1992-1-1-2004

The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should be not less than

$$S_{min} = \max\begin{cases}k_1 \bullet bar\ diameter \\ d_g + k_2\ mm \\ 20\ mm\end{cases}$$

Where dg is the maximum size of aggregate

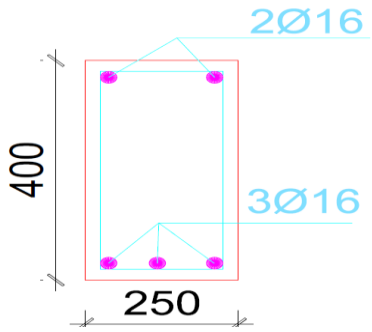
The recommended values of k_1 and k_2 are 1 and 5 mm respectively

b=2Cc+2$\emptyset st$+No $\emptyset$+no of spacing*S

S=$\frac{2Cc+2\emptyset st+\text{No }\emptyset}{\text{no of spacing}}=\frac{2*26+2*8+2}{1}$=70mm

$$S_{min=}max\begin{cases}1*16=16mm\\20+5=25mm\\20mm\end{cases}$$

$S_{min=}25mm$<70mm **OK!!!!!!!**



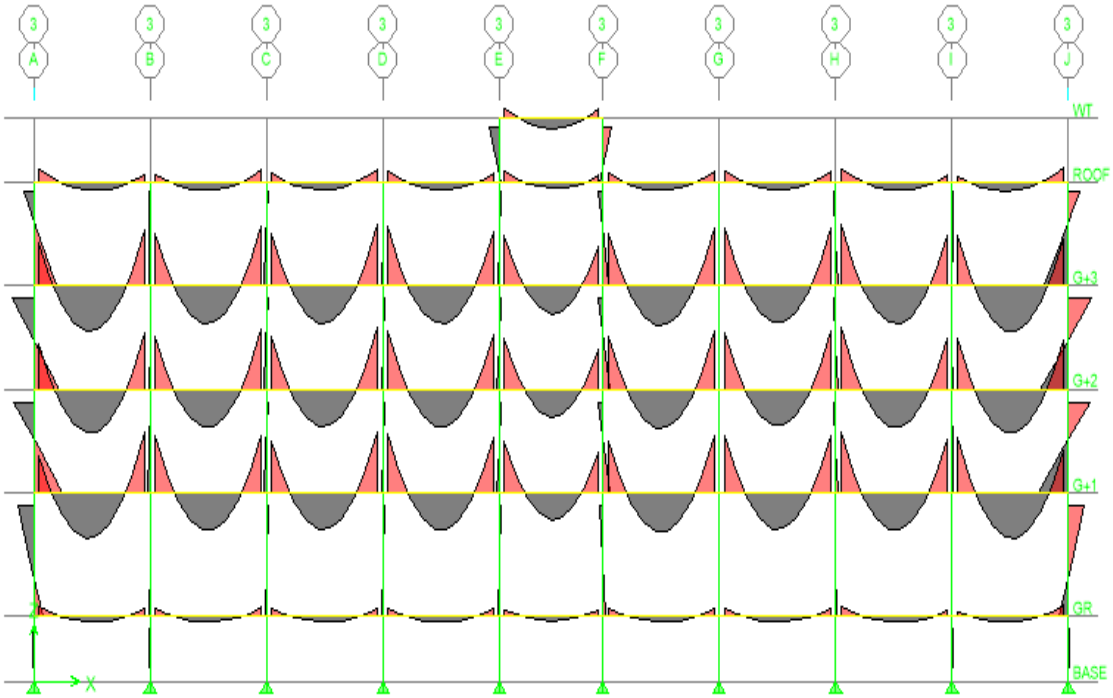


Figure 34 Moment of building

By the same methods the following beams are

Table 38 Reinforcement

		Floor beam on axis 3-3								
type	location	moment	$\mu_{sd,s}$	K	Z	Ascalc	N	Use N	S	REMARK
support	A	24.33	0.067	0.97	347.26	268.57	1.34	2	150	2Ø16C/C150
span	AB	34.87	0.096	0.96	343.68	388.93	1.94	2	150	2Ø16C/C1510
support	B	42.97	0.118	0.94	336.52	489.47	2.44	3	67	3Ø16C/C67
span	BC	29.04	0.080	0.94	336.52	330.80	1.65	2	150	2Ø16C/C150
support	C	45.95	0.127	0.93	332.94	529.05	2.63	3	67	3Ø16C/C67
span	CD	28.66	0.079	0.96	343.68	319.67	1.59	2	150	2Ø16C/C150
support	D	47.91	0.132	0.92	329.36	557.61	2.77	3	67	3Ø16C/C67
span	DE	28.91	0.080	0.94	336.52	329.32	1.64	2	150	2Ø16C/C150
support	E	33.33	0.092	0.95	340.1	375.67	1.87	2	150	2Ø16C/C150
span	EF	21.45	0.059	0.95	340.1	241.77	1.20	2	150	2Ø16C/C150
support	F	14.8	0.041	0.97	347.26	163.37	0.81	2	150	2Ø16C/C150
span	FG	30.86	0.085	0.98	350.84	337.18	1.68	2	150	2Ø16C/C150
support	G	44.74	0.123	0.93	332.94	515.12	2.56	3	67	3Ø16C/C67
span	GH	28.11	0.077	0.96	343.68	313.53	1.56	2	150	2Ø16C/C150
support	H	44.4	0.122	0.93	332.94	511.20	2.54	3	67	3Ø16C/C67
span	HI	29.19	0.080	0.94	336.52	332.51	1.65	2	150	2Ø16C/C150
support	I	39.47	0.109	0.92	329.36	459.38	2.29	3	67	3Ø16C/C67
span	IJ	34.66	0.095	0.93	332.94	399.06	1.99	2	150	2Ø16C/C150
support	J	22.16	0.061	0.95	340.1	249.77	1.24	2	150	2Ø16C/C150

Reinforcement detail for axis 3-3 beam

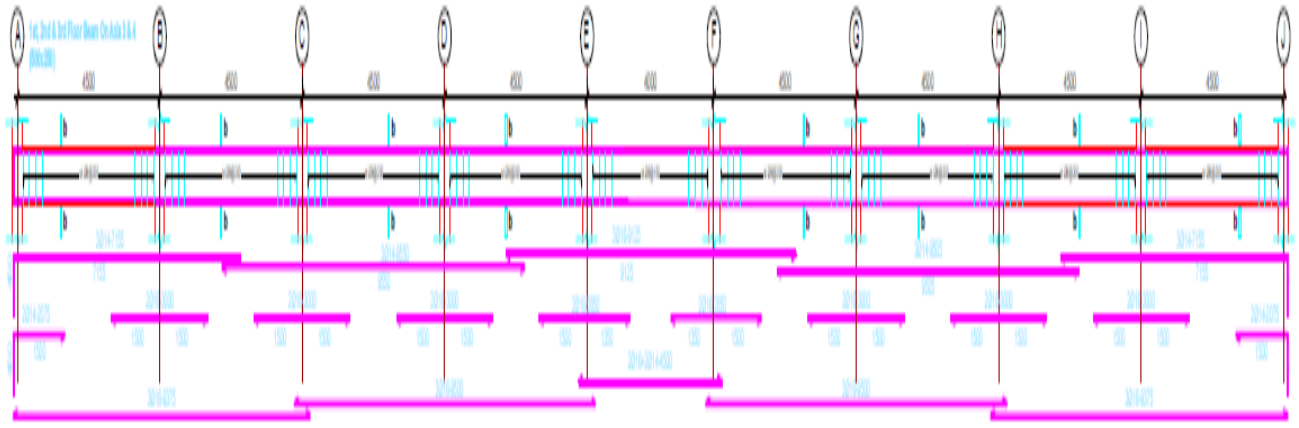


Figure 35 Reinforcement detail for beam

Reinforcement calculation

7.4 Design beam for shear

The design shear is located d distance from the face of the column. By similarity of triangle, VEd can obtain from shear force diagram from etabs output:

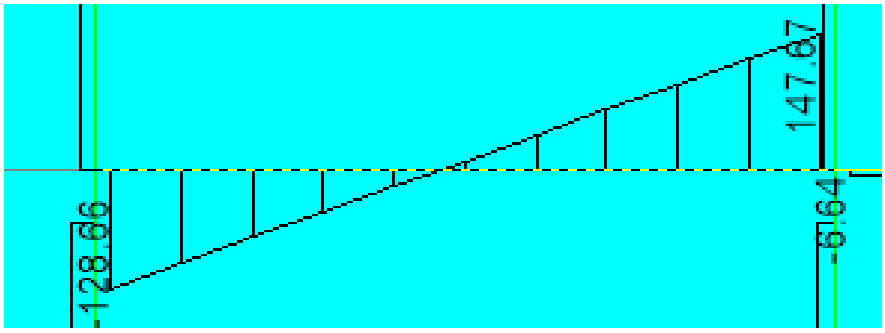


Figure 36 Sample maximum shear on Axis A-A

Concrete capacity

- Width of column = 400cm, beam width =200mm
- Depth of beam =400mm effective depth d=358mm
- From all beam select the maximum shear Vsdmax=147.67KN
- Check the crushing strength Vrd max

Shear forces from etabs

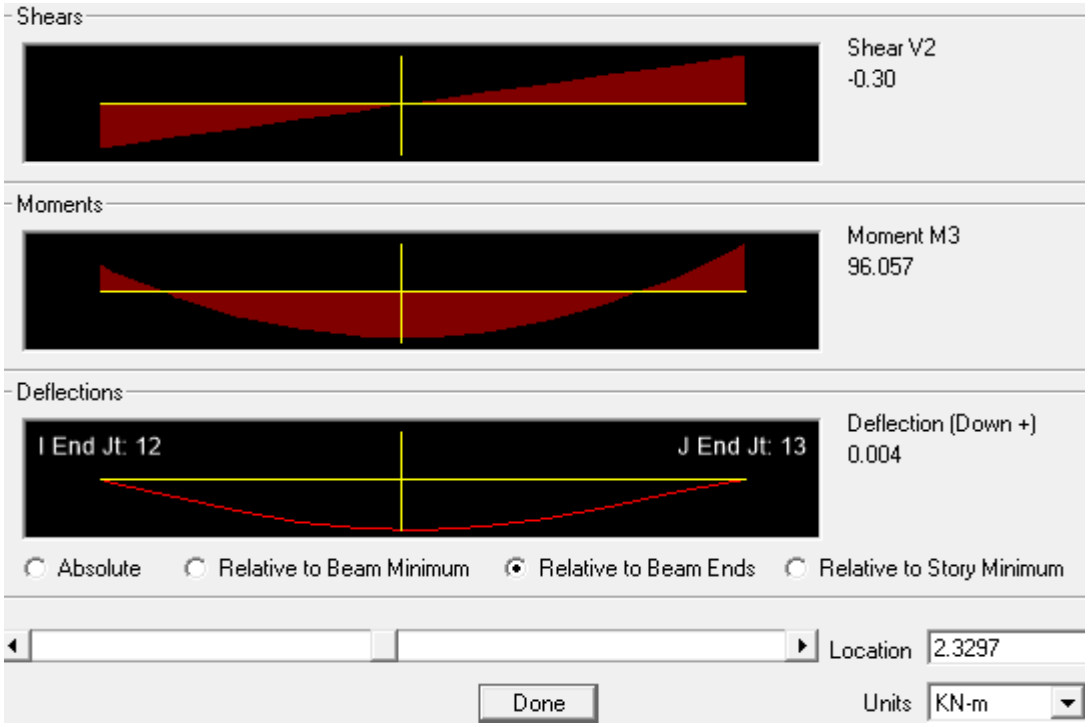
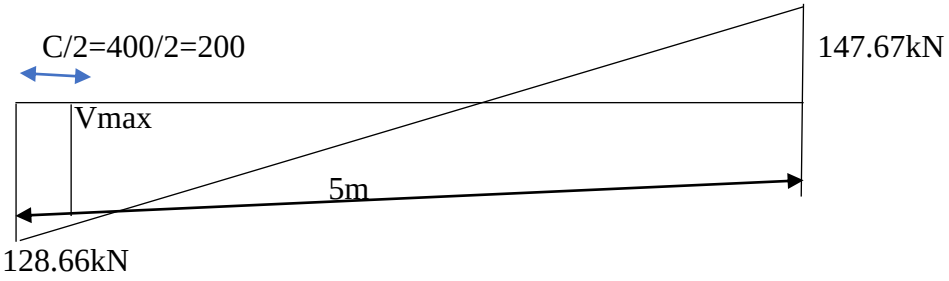


Figure 37 Shear force and bending on axis 33

We get the distance in which the shear is zero (=2.33m) from above diagram

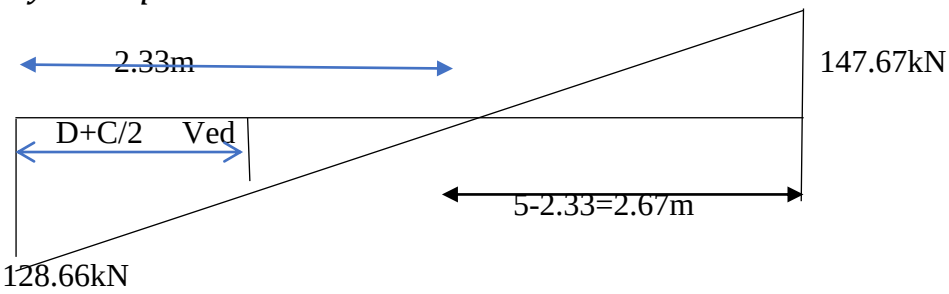
Sample example for shear from above output of Etabs



Shear calculation

So $D + C/2 = 400 + 400/2 = 600mm = 0.6M$

$$By\ interpolation \rightarrow VEDmax = 117.62KN$$



$$\frac{147.67}{2.67} = \frac{V_{ED}}{2.33 - 0.6} \rightarrow V_{ED} = 95.53KN < V_{EDmax=117.62} \ldots\ldots ok!!!$$

Diagonal compression capacity of concrete ($V_{Rd,max}$)

$V_{Rd,c}= [C_{Rd,c}k(100\ \rho_l\ f_{ck})^{1/3} + k_1\sigma_{cp}]\ b_w d$……*ES EN 1992-2015*

$V_{Rd,c}= (v_{min}+ k_1\sigma_{cp})\ b_w d$

Determining $VRd_{,max}$, $\cot\theta = 2.5$

$$VRd, max = \frac{v * \alpha_c b_w * z * f_{cd}}{cot\theta + tan\theta} \ldots\ldots\ldots\ldots\ldots\ldots ES\ EN\ \ 1992 - 2015$$

Then $\alpha_c = 1$

$b_w = 250$

$z = 0.9 * d = 0.9 * 358 = 322mm$

$v = 0.6$

$f_{cd} = 11.33$

$$VRd,max = \frac{0.6 * 1 * 250 * 322 * 11.33}{2.9} = 188.7 > V_{EDmax=117.62} \ldots\ldots ok!!!$$

Check concrete capacity of section for shear

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

$V_{Rd,c}$= (v_{min}+ $k_1\sigma_{cp}$) $b_w d$

Where f_{ck} is in MPa

k $= 1+\sqrt{\frac{200}{d}} \le 2.0$ with d in mm=$1+\sqrt{\frac{200}{358}} = 1.74 < 2$ ok

ρ_1 $= \frac{A_{sl}}{b_w d} \le 0.02$

$= \frac{558}{250*358} = 0.0062$

$K1=0.15$

σ_{cp}=0

$$CRd,c = \frac{0.18}{1.5} = 0.12$$

$$V_{min} = 0.035 * k^{3/2} f_{ck}^{1/2} = 0.035 * 2.29 * 4.48 = 0.036$$

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

$$VRd,c = max\begin{cases}(0.12 * 1.74\sqrt[3]{100 * 0.0062 * 20} + 1.74 * 0) * 250 * 358 = 43.25 \\ (0.036 + 0.15 * 0) * 250 * 358 = 32.22\end{cases}$$

$VRd,c = 43.25 < V_{ED=95,53.} \ldots\ldots\ldots web\ reinforcement\ should\ be\ provided$

Calculate the necessary amount of shear reinforcement

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta \rightarrow s = \frac{A_{sw} z f_{ywd} \cot\theta}{V_{Rd,s}}$$

Take a single rectangular loop of diameter 8 stirrup,

$V_{Rd,\ max}$= $\alpha_{cw}\ b_w z\ \nu_1 f_{cd}/(\cot\theta + \tan\theta)$

$$VRd,max = \frac{0.6 * 1 * 250 * 322 * 11.33}{2.9} = 188.7$$

$$S = \frac{2 * 3.14 * 4^2 * 0.9 * 358 * 260.86 * 2.5}{95.53 * 1000} = 221mm$$

Use ϕ 8 c/c 220 mm

Check minimum web reinforcement ratio and maximum spacing limit

$$\rho_{min} = \frac{0.8\sqrt{f_{ck}}}{f_{yk}} = \frac{0.8\sqrt{f_{20}}}{300} = 0.0119$$

$$\rho_{provoded} = \frac{A_{sw}}{sb_w \sin\alpha}$$

[6]

$$\rho_{provided} = \frac{2 * 3.14 * 16}{220 * 250 * sin90°} = 0.0183 > \rho_{min} \dots\dots\dots OK!!!!$$

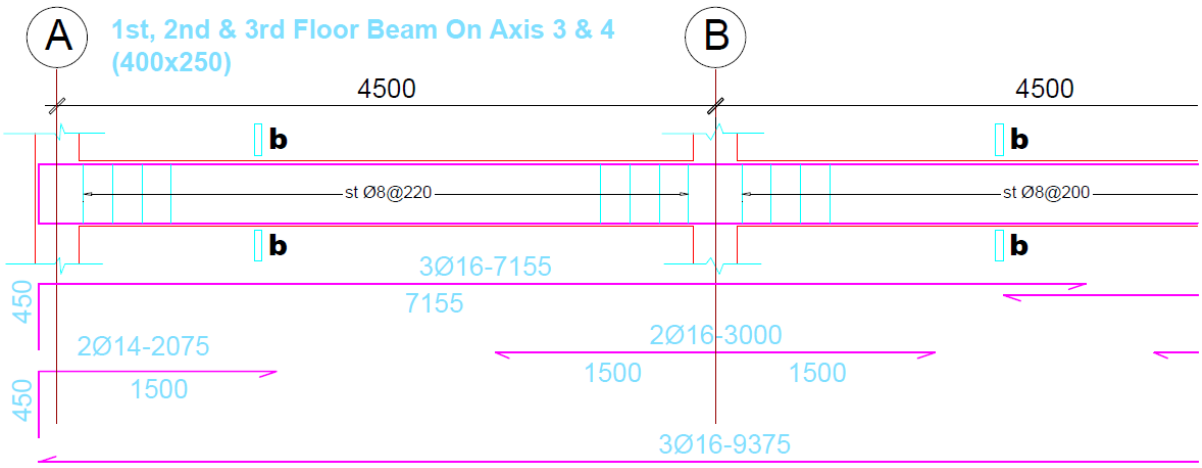


Figure 38 Beam reinforcement detail

8 Design and analysis 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, L-shaped, T-shaped, etc. depending on the structural or architectural requirements.
- Classification on the basis of degree of slenderness; short column, slender 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

Location	Footing	ground	1st	2nd	3rd	gruop	Use axial load (col. Design)
c1	127.41	87.92				1	296.15
c2	1149.14	1110.81	747.23	423.23	99.83	3	1161.28
c3	1072.78	1027.69	697.94	381.76	66.31	3	1161.28
c4	1120.57	1076.26	725.77	391.97	66.94	3	1161.28
c5	296.15	269.88	155.31	95.81	36.06	1	296.15
c6	988.33	956.9	546.71	293.01	49.41	2	988.33
c7	1107.71	1070.56	727.13	389.42	60.49	3	1161.28
c8	1129.04	1086.79	717.42	384.41	62.77	3	1161.28
c9	1144.49	1103.18	731.53	392.75	63.45	3	1161.28
c10	1149.14	1110.81	747.23	423.28	103.01	3	1161.28
c11	1092.65	1084.31	731.67	414.04	100.56	3	1161.28
c12	1133.91	1092.47	727.02	390	62.92	3	1161.28
c13	1137.5	1096.68	723.09	387.61	63.79	3	1161.28
c14	1161.28	1119.59	728.93	390.17	67.71	3	1161.28
c15	823.04	793.85	544.35	291.04	48.19	2	988.33

Rectangular column sample calculation for 3rd - ground floor from group 1&2

Given data

Top beam	depth(m)	width(m)	Length(m)
b1(x,x)	0.3	0.25	5
b2(x,x)	0.3	0.25	2.4
b3(y,y)	0.3	0.25	4.5
b4(y,y)	0.3	0.25	1.4

bottom beam	depth(m)	width(m)	length(m)
b1(x,x)	0.4	0.3	5
b2(x,x)	0.4	0.3	2.4
b3(y,y)	0.4	0.3	4.5
b4(y,y)	0.4	0.3	1.4

Column	0.4	0.3	3.2

M_{topx-x}=9.88KNm

M_{topy-y}=9.88KNm

M_{botx-x}= -21.33KNm

M_{boty-y}= -21.33KNm

NED=45.98KN

Concrete cover design

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

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,
From table 4.2 bond requirement, arrangement of bars for separated minimum cover
Cmin, b= diameter of bar
Assume Φ16 longitudinal bar and Φ25 nominal maximum aggregate size;
Therefore; Cmin, b= 16mm.
If nominal maximum aggregate size is greater than 32mm Cmin, b should be increased by 5mm

Cover Design for Corrosion/Durability

The condition of exposure is assumed to be XC1
Exposure Class to determine Cmin, dur
ForXC1, member with slab geometry (position of reinforcement not affect by construction process the exposure class is reduced class by one
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 Table 4.3N. But based on the above table the exposure class is reducing by 1 and the structural class would be S3.

The value of minimum cover required for durability of reinforcement steel is determined by using ES EN 1992:2015 tables 4.4N.

Using structural class, S3 and XC1

First, the concrete cover

With:

$c_{min,b}$ = 16 mm

$c_{min,dur}$ = 10 mm

^{c}min = max (cmin,b; cmin,dur ; 10 mm) =max (16; 10; 10 mm) = 16 mm

Δc_{dev} = 16 mm.

ΔCdev (allowance in Design for Variation)

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

We obtain from relation

$c_{nom} = c_{min} + \Delta c_{dev}$ = 16 + 10 = 26 mm

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 Table 5.8 0f EN 1992-1-2

Cfire = 20mm

Hs = 120mm

Governing Cover for Design

Governing cover accounting for

- Corrosion
- Bond/Durability
- Fire

Cover = 26mm

Concrete C 20/25

Rebar S400

Fcd =0.85*fck/rc=0.85*20/1.5=11.33M

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

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√((1+ $\boldsymbol{K1}$ **0.45**+$\boldsymbol{k1}$) (1+ $\boldsymbol{K2}$ **0.45**+$\boldsymbol{k2}$))

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/Σbeam stiffness

$$\mathbf{K1}\ \text{x} - \text{x} = \frac{\left(\frac{EI}{I}\right)c}{\left(2 * \frac{EI}{I}\right)b} > 0.1$$

Table 39 Effective length calculation

h	b	Length	toptie beam	imbbeam	column	beam 1T	beam2T	beam3i	beam4 int	beam1 b	beem2b	beam3t	beam4t	inertia
300	250	5000	562500000			112500	234375	355555.556	1142857.14	320000	666666.667	125000	401785.714	
400	300	2400		1600000000	1600000000									
400	300	4500												
400	300	1400												

ky1	kx1	ky2	kx2	2(ei/i)b inx1	(ei/i)b inx2	(ei/i)b iny1	(ei/i)b iny2
1518.644068	2306.306306	533.8983051	810.8108108	693750	1973333.333	1053571.429	1973333.333

k1x-x	k2x-x	k1y-y	k2y-y	0.45+k1x-x	0.45+k2x-x	0.45+k1y-y	0.45+ky2-2	axx	bxx	cyy
2306.306306	810.8108108	1518.644068	533.8983051	2306.756306	811.2608	1519.094	534.3483051	1.999804921	1.999445308	1.999703771

dyy	product in xx	product in in yy	radicalin xx	radicalinyy	loxx	loyy
1.999157853	3.998500566	3.997723496	2.0	2.0	3.00	3.00

Step 3 maximum and minimum reinforcement calculation

		column size		**height**	**effective length**	
	column	**H**	**b**	**L**	**L0x**	**L0y**
	Foundation	**0.4**	**0.3**	**1.5**	**3.00**	**3.00**
c1	**ground**	**0.4**	**0.3**	**3.84**	**3.00**	**3.00**
	1st	**0.4**	**0.3**	**3.2**	**3.00**	**3.00**
	2nd	**0.4**	**0.3**	**3.2**	**3.00**	**3.00**
	3rd	**0.4**	**0.3**	**3.2**	**3.00**	**3.00**

		axial load	m2-2miner axises				m3-3 major axises		
		P	Mx-xT	MX-XB	MX-XT/MX-XB	MY-YT	MY-YB	MY-YT/MY-YB	m01xcompare
	Foundation	988.29	0.59	0.5	1.18	0.5	0.5	1	0.5
	ground	956.86	14.42	8.52	1.692488263	14.44	8.5	1.698823529	8.52
c1	1st	513.73	18.69	27.16	0.68814433	18.69	27.15	0.68839779	18.69
	2nd	280.56	20.88	25.43	0.821077468	20.88	25.43	0.821077468	20.88
	3rd	45.98	9.88	21.33	0.463197375	9.88	21.33	0.463197375	9.88

First order moment calculation including geometric imperfaction										
M01, Xmim	M02,Xmax compare	M0e, Xmax	m0ex	ME Dx-x	M01,ymincompare	M01,y-ymin	M02,ymaxcompare	M01max, y	m0ey	ME Dx-x
20.2658	0.59	20.3558	20.3198	8.14232	0.5	20.2658	0.5	20.2658	20.2658	8.10632
27.6572	14.42	33.5572	31.1972	13.42288	8.5	27.6372	14.44	33.5772	31.2012	13.43088
28.9646	27.16	37.4346	34.0466	14.97384	18.69	28.9646	27.15	37.4246	34.0406	14.96984
26.4912	25.43	31.0412	29.2212	12.41648	20.88	26.4912	25.43	31.0412	29.2212	12.41648
10.7996	21.33	22.2496	17.6696	8.89984	9.88	10.7996	21.33	22.2496	17.6696	8.89984

		axial load	m2-2miner axises			m3-3 major axises
		P	Mx-xT	MX-XB	MY-YT	MY-YB
	Foundation	988.29	0.59	0.5	0.5	0.5
	ground	956.86	14.42	8.52	14.44	8.5
c1	1st	513.73	18.69	27.16	18.69	27.15
	2nd	280.56	20.88	25.43	20.88	25.43
	3rd	45.98	9.88	21.33	9.88	21.33

First order moment calculation including geometric imperfaction						check slenderness			

M01,X	M02, X	M0e, X	M01, y	M02, y	M0e, y	λ	λlim	2nd order effect	
20.27	20.36	20.32	20.27	20.27	20.27	0.14	16.32	Neglected	
27.66	33.56	31.20	27.64	33.58	31.20	0.14	15.30	Neglected	
28.96	37.43	34.05	28.96	37.42	34.04	0.14	32.73	Neglected	
26.49	31.04	29.22	26.49	31.04	29.22	0.14	38.47	Neglected	
10.80	22.25	17.67	10.80	22.25	17.67	0.14	133.71	Neglected	

root n	l0x	loy	ix	iy	λx	λy	λmax	λlimx-x	λlimy-y	λlim
0.660407882	3	3	28.28	21.2	0.106082037	0.141509434	0.141509434	12.12583953	16.32324553	16.32324553
0.649821756	3	3	28.28	21.2	0.106082037	0.141509434	0.141509434	0.178019203	0.027880949	0.178019203
0.476143117	3	3	28.28	21.2	0.106082037	0.141509434	0.141509434	32.72666715	32.71846943	32.72666715
0.351870553	3	3	28.28	21.2	0.106082037	0.141509434	0.141509434	38.46700692	38.46700692	38.46700692
0.142447401	3	3	28.28	21.2	0.106082037	0.141509434	0.141509434	133.7108312	133.7108312	133.7108312

				Longtudnal Reinforcement calculation									
	Column b\n elevation	NED	Msd, x	Msd, y	Vsd	μsd, X	μsd, y	ω	ø	As,req	As,min	As,max	as	Provide
	Foundation	988.29	20.36	20.27	0.44	0.02	0.02	0.24	16	1563.66	400	16000	20.96	8
c1	ground	956.86	33.56	33.58	0.42	0.03	0.03	0.24	16	1563.66	400	16000	20.96	8
	1st	51	37.	37.4	0.2	0.03	0.03	0.2	16	162	40	16	20	6

		3.73	43	2	3			5		8.81	0	000	0.96	
	2nd	280.56	31.04	31.04	0.12	0.03	0.03	0.26	16	1693.96	400	16000	200.96	6
	3rd	45.98	22.25	22.25	0.02	0.02	0.02	0.39	16	2540.94	400	16000	200.96	4

shear reinforcement calculation

	Under normal Length					
	Column	ø	Spacing	Provide	ø	Spacing
	Foundation	8	300	Provide Ø8 C/C300mm	8	300
	ground	8	300	Provide Ø8 C/C300mm	8	300
c1	1st	8	300	Provide Ø8 C/C300mm	8	300
	2nd	8	300	Provide Ø8 C/C300mm	8	300
	3rd	8	300	Provide Ø8 C/C300mm	8	300

Rectangular column sample calculation for 2nd floor from group 1

Given data

Top beam	depth(m)	width(m)	Length(m)
b1(x,x)	0.4	0.3	5
b2(x,x)	0.4	0.3	2.4
b3(y,y)	0.4	0.3	4.5
b4(y,y)	0.4	0.3	1.4

bottom	depth(m)	width(m)	length(m)

beam			
b1(x,x)	0.4	0.3	5
b2(x,x)	0.4	0.3	2.4
b3(y,y)	0.4	0.3	4.5
b4(y,y)	0.4	0.3	1.4
Column	0.4	0.3	3.2

M_{topx-x}=20.88KNm

M_{topy-y}=20.88KNm

M_{botx-x}= -25.43KNm

M_{boty-y}= -25.43KNm

NED=45.98KN

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,

From table 4.2 bond requirement, arrangement of bars for separated minimum cover Cmin, b= diameter of bar

Assume Φ16 longitudinal bar and Φ25 nominal maximum aggregate size;

Therefore; Cmin, b= 16mm.

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

Cover Design for Corrosion/Durability

The condition of exposure is assumed to be XC1

Exposure Class to determine Cmin, dur

ForXC1, member with slab geometry (position of reinforcement not affect by construction process the exposure class is reduced class by one

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 Table 4.3N. But based on the above table the exposure class is reducing by 1 and the structural class would be S3.

The value of minimum cover required for durability of reinforcement steel is determined by using ES EN 1992:2015 tables 4.4N.

Using structural class, S3 and XC1

First, the concrete cover

With:

$\mathbf{c_{min,b}}$ = 16 mm

$\mathbf{c_{min,dur}}$ = 10 mm

^{c}min = max (cmin,b; cmin,dur ; 10 mm) =max (16; 10; 10 mm) = 16 mm

$\mathbf{\Delta c_{dev}}$ = 16 mm.

ΔCdev (allowance in Design for Variation)

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

We obtain from relation

$\mathbf{c_{nom} = c_{min} + \Delta c_{dev}}$ = 16 + 10 = 26 mm

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 Table 5.8 0f EN 1992-1-2

Cfire = 20mm

Hs = 120mm

Governing Cover for Design

Governing cover accounting for

- Corrosion
- Bond/Durability
- Fire

Cover = 26mm

Concrete C 20/25

Rebar S400

Fcd =0.85*fck/rc=0.85*20/1.5=11.33M

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

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√((1+ $\boldsymbol{K1\ 0.45+k1}$) (1+ $\boldsymbol{K2\ 0.45+k2}$))

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/Σbeam stiffness

$$\mathbf{K1}\ \mathrm{x}-\mathrm{x}=\frac{\left(\frac{EI}{l}\right)c}{\left(2*\frac{EI}{l}\right)b}>0.1$$

K1x-x=$(\boldsymbol{(EI/l)c/(2*EI/l)b})$= 2306.3

K2x-x=$(\boldsymbol{(EI/l)c/(2*EI/l)b})$= 810.81

K1y-y= $(\boldsymbol{(EI/l)/(2*EI/l)b})$= 1518.6

K2y-y=$(\boldsymbol{(EI/l)c/(2*EI/l)b})$= 533.89

Lox-x=0.5 *l√((1+2306.3$\boldsymbol{/0.45}$+2306.3) (1+810.81$\boldsymbol{/0.45}$+810.81))

Lox-x= 1.50

Loy-y=0.5 *l√((1+1518.6/0.45+1518.6) (1+533.89/0.45+533.89))
Loy-y= 1.50

Step 3 maximum and minimum reinforcement calculation

As,min=max $\begin{cases}(0.1*NED/fyd)=0.1*280.56/347.8=18\text{m2}\\ 0.002*Ac=0.002*400*300=240\text{mm2}\end{cases}$

As,min=240mm2

As,max=$0.08*Ac=0.08*400*300=9600$mm2

Step 4 design action

The effect of imperfection taken from According to ES EN 1992: Art 5.2.7

eix-x= max $\begin{cases} lo/400=1500/400=3.75mm\\ h/30=320/30=10.6mm\\ 20\text{mm}\end{cases}$

eix-x = 20mm

eiy-y= max $\begin{cases} lo/400=1500/400=3.75mm\\ h/30=320/30=10.6mm\\ 20\text{mm}\end{cases}$

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 {|20.88|,|25.43|+0.02*280.56}

M01x-x=26.49KNm

M02x-x=max {|Mtopx-x|,|Mbottomx-x|+ei*NED}

M02x-x=max {|20.88|,|25.43|+0.02*280.56}

M02x-x=31.04KNm

Moex-x=0.4*M01x-x+0.6*M02x-x≥0.4*M02x-x

Moex-x=0.4*26.49+0.6*31.04≥0.4*31.04

Moex-x=29.22KNm

MEDx-x=12.4KNm

In the Y-dirn

M01y-y=min {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M01y-y=min {|20.88|,||,|25.43|+0.02*280.56}

M01y-y=26.49KNm

M02y-y=max {|Mtopy-y|,|Mbottomy-y|+ei*NED}

M02y-y=max {|20.88|,|25.43|+0.02*280.56}

M02y-y=31.04KNm

Moey-y=0.4*M01y-y+0.6*M02y-y≥0.4*M02y-y

Moey-y=0.4*26.49+0.6*31.04≥0.4*31.04

Moey-y=29.2KNm

MEDy-y=12.42KNm

Check slenderness

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

i-is radius of gyration of the uncracked concrete

The minimum limiting value of slenderness is

$\lambda lim = 20*A*B*C/\sqrt{n}$

$\lambda limx\text{-}x$

$=20*A*B*C/\sqrt{n}$

Where, A – $1/(1 + 0.2\varphi ef)$, φef-effective creep ratio

B - $\sqrt{1 + 2\omega}$,-Asfyd /Acfcd, mechanical reinforcement ratio

C = 1.7-rm

n=NEd/Acfcd, relative normal force

rm,moment ratio=Mo1x-x/Mo2x-x=26.49/31.04=0.85

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-r_m

X- dxn y- dxn

r_m =mo1/mo2=26.49/31.04=0.85

r_m=mo1/mo2=26.49/31.04=0.85

Ac=$b * h$ = 400 ∗ 300 = 120000mm^2

for rectangular column

n=NEd/Acfcd=(280.56*1000/(120000*11.33)=0.21

Therefore, slenderness limit (ƛlimx-x) =20*A*B*C/√n

=20*0.7*1.1*0.85/√0.21=28.56

A=0.7

B=1.1

C=1.7-(0.85)=0.85

n=NEd/Acfcd=(280.56*1000/(120000*11.33)=0.21

λlimy-y =20*A*B*C/√n==20*0.7*1.1*0.85/√0.21=28.56

$\lambda x = \frac{lox}{ix}$ where i= √ Ic-x/Ac=0.12m

$$\lambda x = \frac{3}{0.12} = 25$$

$\lambda y = \frac{loy}{iy}$where iy =√ Ic-y/Ac=0.11 m

$\lambda y = \frac{3}{0.11} = 27.27$

Check slenderness

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=25< 28.56, short column

λy- y< λ limy-y= 27.27 <28.56, short column

vsd=Nsd/(Ac*fcd,) =280.56*1000/ (400*300*11.33) =0.206

μsdx-x=MEDx-x/(fcd*Ac*h)= 12.4*1000000/ (11.33*400*300*400) =0.02

μsdy-y= MEDy-y/(fcd*Ac*h) =12.42*1000000/ (11.33*400*300*400) =0.02

Using biaxial rectangular chart number

ω=0.2

A. Step reinforcement calculation

 Longitudinal reinforcement

As,tot = ω*Ac*fcd/fyd

As,tot=0.2*120000*11.33/347.8

As,tot=781.83mm²

As min=240mm2 and As max=9600mm2As min <As<As max……OK

Take Ø16mm, as=3.14*(16)²/4=200.96mm²

Number of bar=As/as=781.83mm²/200.96mm²=3.89≈4bar

Provide 4 Ø16mm

 for shear reinforcement

Diameter of bar = max
- ¼*longitudinal bar diameter=16/**4** =4
- 6mm
- Diameter of stirrup = 8mm

From above take Ø8mm

Spacing of shear reinforcement= min
- 20*Ø$\boldsymbol{long}$, =20*16mm=320mm
- lesser dimension of column =300mm
- 400mm

Finally, provideØ8 c /c 300mm

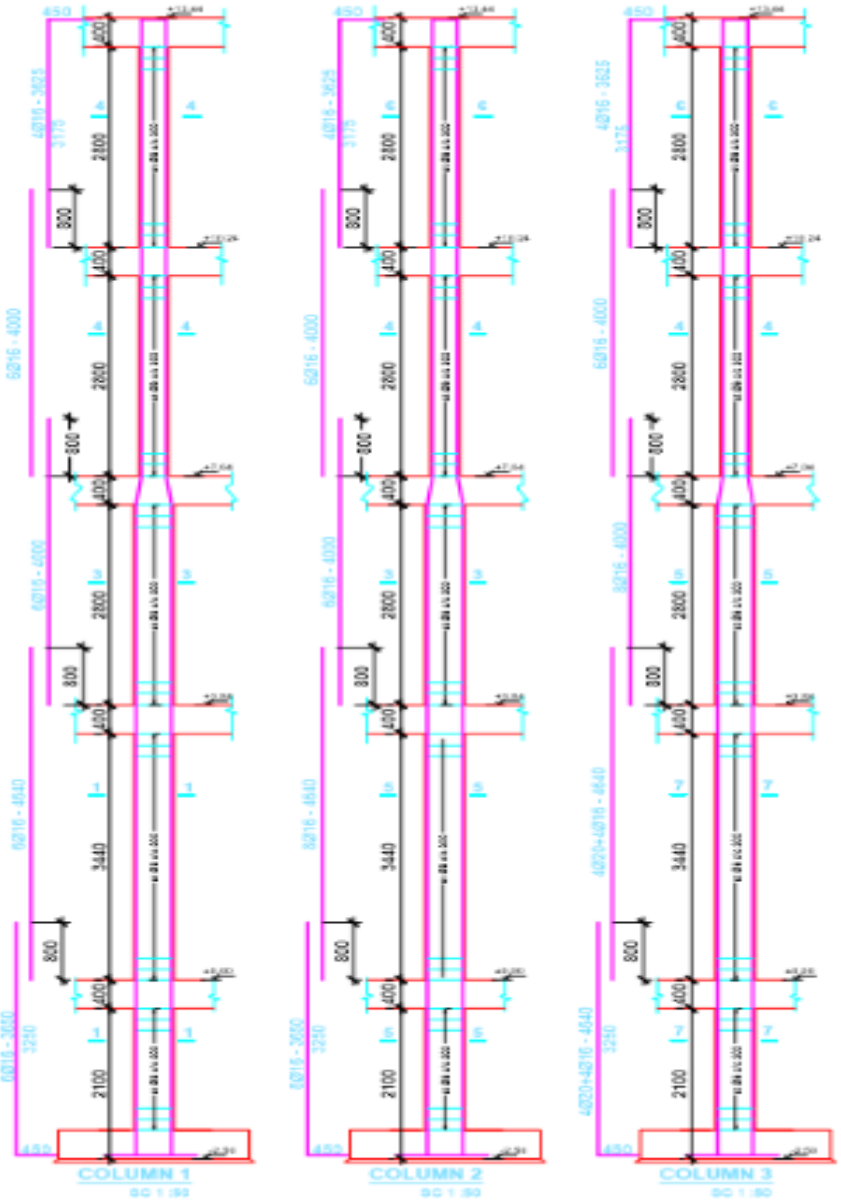


Figure 39 Column reinforcement layout

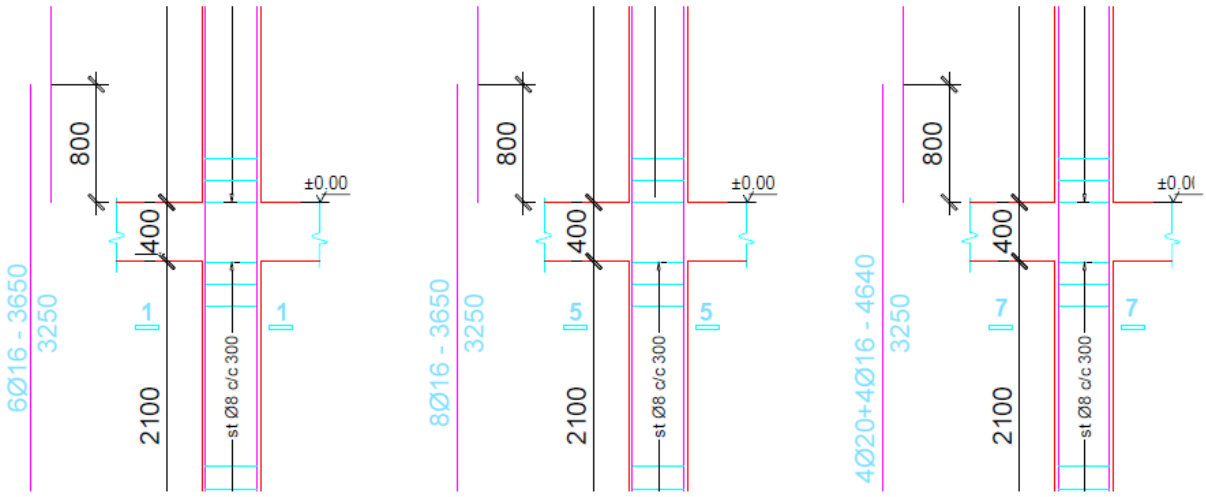


Figure 40 Column reinforcement detail

9 Foundation design

9.1 Procedures to design

Material Data

C-30

S-400

Using Φ12mm

Class-I work

Constants

- fcd= 0.85fck/ c=0.85*(30/1.25)Mpa/1.5=13.6Mpa
- fyd =fyk/ s=400Mpa/1.15=347.8Mpa

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

Therefore the value of minimum cover required for durability of reinforcement steel is determined using ES EN 1992:2015 table 4.4N.(refer appendix B)

We obtain from table 4.4N

- ✓ cmin,dur =25 mm

Moreover:

- ✓ Δcdur,γ = 0 ;
- ✓ Δcdur,st = 0 ;
- ✓ Δcdur,add = 0 .

From relation (3.2):

cmin = max (cmin,b; cmin,dur + Δcdur,γ - Δcdur,st - Δcdur,add; 10 mm) [3]

= max (12; 25 + 0 – 0 – 0; 10 mm) = 25mm

Δc_{dev}=10mm (recommended)

$Cnom=Cmin+\Delta Cdev=25mm+10mm=35mm$

$Cnom=35mm$

Maximum factored load from ETABS result

Ultimate limit state

C-1 (column on axis 2 and axis E)

P=1149.4KN Mx=6.84KNm My=11.4KNm

From factored load we select the maximum super structure load

9.2 Area proportioning

Factor of safety (F.S) **for C-1**

F.S –we can use 2-3….then we take

F.S = 1.5

Allowable bearing capacity of soil is 600Kpa

σ_{all} = 600kpa

σult=1.5*σall

=1.5*600KPa

=900KPa

$\gamma_{SOIL} = 150\ {}^{N}/_{mm^2}$

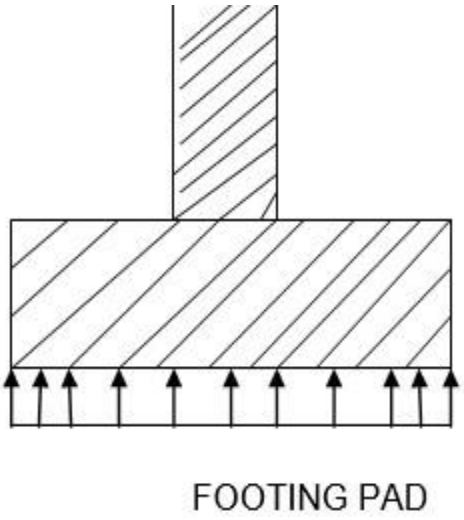


Figure 41 Footing pad

$$Service\ axial\ = \frac{1149.4}{1.5} = 766.26KN$$

$$Service\ moment = \frac{11.4}{1.5} = 7.6knm$$

Assumed self-weight is 10% of service load=76.62KN

$$Area\ of\ footing\ required = \frac{N+W}{\gamma_{soil}} = \frac{766.6+76.6}{150} = 5.63m^2$$

$$Tri\ footing = B * H * d = 1.7m * 1.7m * 0.65m = 2.88m^2$$

Self-weight=2.88*0.65*25=46.8KN

Inertia $I_{XX} = \frac{BH^3}{12} = \frac{1.71.7^3}{12} = 0.69m^4 =$

$$y = \frac{H}{2} = 0.85$$

$$Maximum\ soil\ pressure = P = \frac{N+W}{A} + \frac{My}{I}$$

$$= \frac{766.67+46.8}{5.63} + \frac{11.4*0.85}{0.69} = 145\ {}^{KN}/_{m^2} < 150\ {}^{KN}/_{m^2} \dots\dots OK!!!$$

$$Ultimate\ soil\ pressure\ P = \frac{N}{A} \pm \frac{MY}{I} = \frac{1149.4}{5.63} \pm \frac{11.4*0.85}{0.69} \rightarrow$$

$$Pmax = 219\ {}^{KN}/_{m^2}\ and \quad Pmin = 190\ {}^{KN}/_{m^2}$$

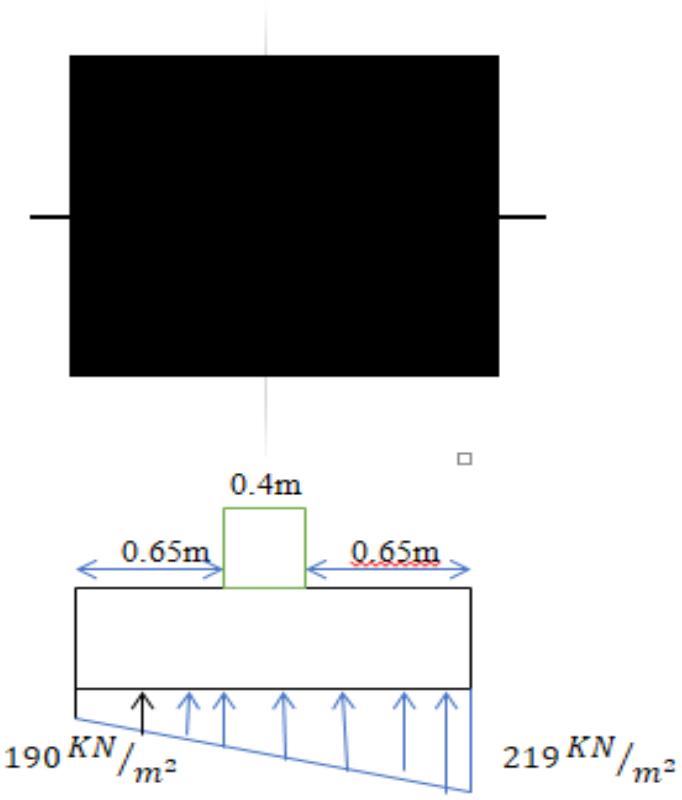


Figure 42 Max and minimum pressure

$$M_{xx} = \left(204 * \frac{0.65}{2}\right) + (219 - 204) * \left(\frac{0.65}{2}\right) * \left(0.65 * \frac{2}{3}\right)$$

$$M_{xx} = 45.3KNm * 1.7 = 77KNm$$

$$M_{yy} = \left(\frac{219 + 190}{2}\right) * \left(\frac{0.65^2}{2}\right) = 43.2KNm * 1.7 = 73KNm$$

9.3 Calculating Effective depth

$$d_x = h - Cc - 0.5\emptyset bar = 650 - 35 - 0.5 * 12 = 609mm$$

$$d_x = h - c - 1.5\emptyset bar = 650 - 35 - 1.5 * 12 = 597mm$$

- **Longitudinal bar calculation**

$$\boldsymbol{K} = \frac{M_{xx}}{fck * b * d^2} = \frac{77 * 10^6}{30 * 1.7 * 609^2} = 0.004 < Kbcl = 0.167$$

Therefore Compression reinforcement is not required

$$Z = \frac{d}{2}\left(1 + \sqrt{1 - 3.53k}\right) = 0.99d > 0.95d \; use \; 0.95d$$

$$A_{s,req} = \frac{M_{xx}}{0.87fyk * 0.95d} = \frac{77 * 10^6}{0.87 * 400 * 0.95 * 609} = 382mm^2$$

Minimum and maximum area of reinforcement

$$A_{s,min} = Max\begin{cases} \frac{0.26fctmbtd}{fyk} = \frac{0.26*2.9*1700*609}{400} = 1951mm^2 \\ 0.00013btd = 0.00013 * 1700 * 609 = 134.5 \end{cases}$$ .....[3] $Art.9.2.1.1$ (1)

$A_{st,max}$=0.04*Ac=0.04*1700*1700=115600mm^2……..[3] Art.9.2.1.1 (3)

$$A_{s,} = 1951mm^2$$

$$a_s = \frac{\pi d^2}{4} = \frac{3.14 * 12^2}{4} = 113.09mm^2$$

$$N = \frac{A_S}{as} = \frac{1951mm^2}{113.09mm^2} = 17.25 = 18\emptyset 12$$

$$Spacing = \frac{b * as}{Asreq} = \frac{1700 * 113.04}{1951mm^2} = 98.5mm = 100mm$$

- **Traverse bar calculation**

$$\boldsymbol{K} = \frac{M_{xx}}{fck * b * d^2} = \frac{73 * 10^6}{30 * 1700 * 597^2} = 0.0038 < Kbcl = 0.167$$

Therefore Compression reinforcement is not required

$$Z = \frac{d}{2}\left(1 + \sqrt{1 - 3.53k}\right) = 0.99d > 0.95d \; use \; 0.95d$$

$$A_{s,req=} \frac{M_{xx}}{0.87fyk * 0.24d} = \frac{73 * 10^6}{0.87 * 400 * 0.95 * 597} = 369mm^2$$

Minimum and maximum area of reinforcement

$$A_{s,min}=Max\begin{cases} \frac{0.26fctmbtd}{fyk} = \frac{0.26*2.9*1700*597}{400} = 1913mm^2 \\ 0.00013btd = 0.00013 * 1700 * 609 = 134.5 \end{cases}$$ .....[3] $Art.9.2.1.1$ (1)

A_{st},max=0.04*Ac=0.04*1700*1700=115600mm^2……...[3] Art.9.2.1.1 (3)

$$A_{s,} = 1913mm^2$$

$$a_{s=} \frac{\pi d^2}{4} = \frac{3.14 * 12^2}{4} = 113.09mm^2$$

$$N = \frac{A_S}{as} = \frac{1913mm^2}{113.09mm^2} = 16.9 = 18Ø12$$

$$Spacing = \frac{b * as}{Asreq} = \frac{1700 * 113.04}{1913} = 100mm$$

9.4 Vertical shear

At the face of the column, d=609mm=0.609m

0.609+0.65+0.4=1.66m

Average pressure at critical section $=190+\left(\frac{1.66}{1.7}\right)*18=207\,{}^{KN}/_{m^2}$

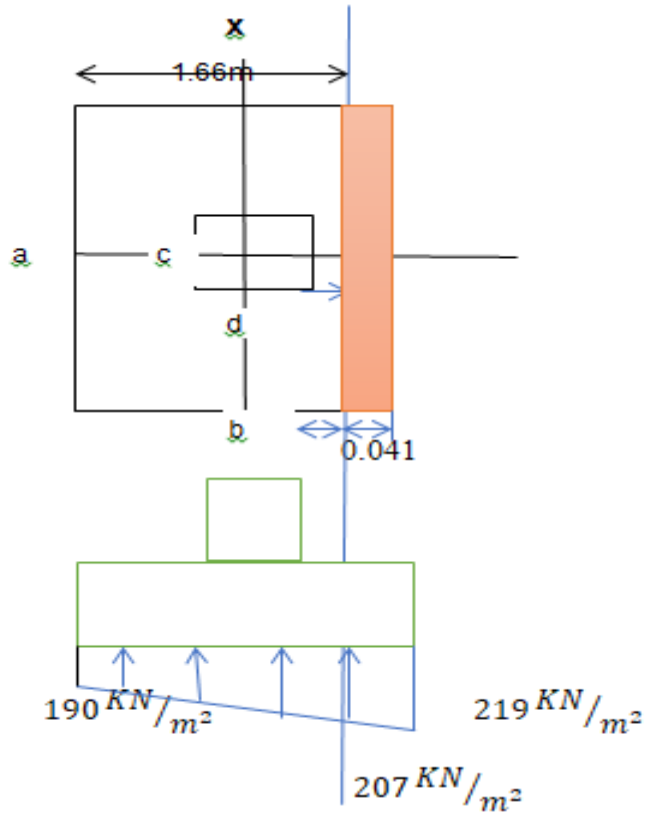


Figure 43 Vertical shear of foundation

$Design\ shear V_{ED}=207*0.041*1.7=8.48*1.7=14.42KN$

- **Vertical shear**

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

$V_{Rd,c}=(v_{min}+k_1\sigma_{cp})\,b_w d$ [1]

Where f_{ck} is in MPa

$k = 1+\sqrt{\frac{200}{d}} \le 2.0$ with d in mm$=1+\sqrt{\frac{200}{609}}=1.57<2$ *ok*

$\rho_1 = \frac{A_{sl}}{b_w d} \le 0.02$

$=\frac{1591}{1.7*609}=0.0015$

K1=0.15

$\sigma_{cp}=0$

$$CRd,c = \frac{0.18}{1.5} = 0.12$$

$$V_{min} = 0.035 * k^{3/2} f_{ck}{}^{1/2} = 0.035 * 1.97 * 5.47 = 0.37$$

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

$$VRd,c = max\begin{cases}(0.12 * 1.57\sqrt[3]{100 * 0.0015 * 30} + 1.5 * 0) * 1700 * 609 = 32.4 \\ (0.036 + 0.15 * 0) * 1700 * 609 = 37.2\end{cases}$$

$$VRd,c = 37.2KN > V_{ED=14.42.}KN \ldots\ldots OK!!!!!!$$

9.5 Punching shear calculation

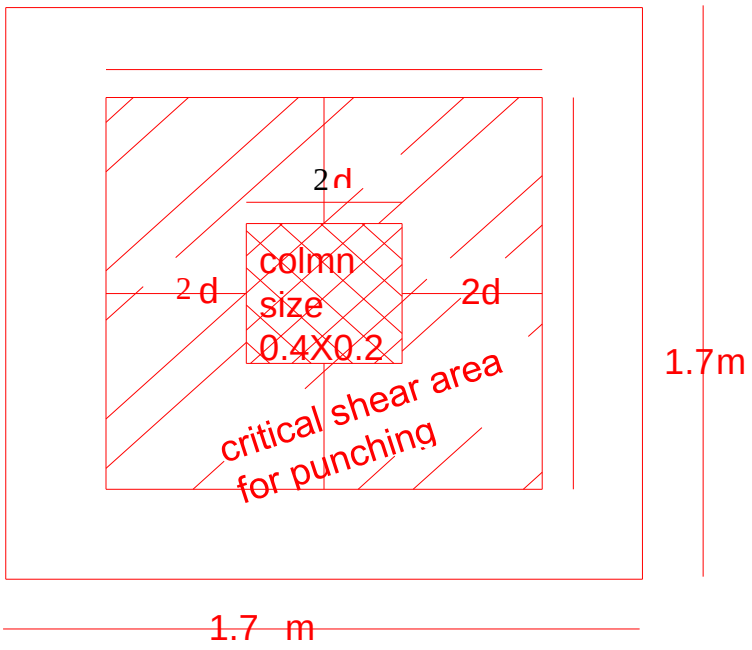


Figure 44 Punching shear of the foundation

Calculating punching shear at 2d from the face of the column

$Average\ d = \frac{609+597}{2} = 603mm$, 2d=1206mm

Control perimeter

U=2*(400+200)+(2*π * 1206) =8774mm

Area within perimeter

$$A = (0.4 * 0.2) + (2 * 0.4 * 1.206) + (2 * 0.2 * 1.206) + (3.14 * 1.206^2) = 6.1m^2$$

$V_{ED} = 204 * ((1.7 * 1.7) * -6.1))$=654Kn

Punching shear stress:

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

Where $\beta = 1 + k\left(\frac{M_{Ed}}{V_{Ed}}\right)\left(\frac{u_1}{W_1}\right)$

W1=$0.5* 400^2 + 400 * 200 + 4 * 200 * 603^2 + 16 * 603^2 + 2 * 3.14 * 603 * 400$

$W1 = 7.6m^2$

K=from table k=0.65 for C1/C2=2

$$\beta = 1 + 0.65\left(\frac{11.4 * 10^6}{654 * 10^3}\right) * \frac{8774}{7.6 * 10^6 m^2} = 1.02$$

$$v_{ED} = \frac{1.02 * 654 * 10^6}{8774 * 603} = 0.12$$

- **punching shear resistance**

$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ with d in mm=$1+\sqrt{\frac{200}{609}} = 1.57 < 2 \quad ok$

$V_{Rd,c}=V_{min} = 0.035 * k^{3/2} f_{ck}{}^{1/2} = 0.035 * 1.57^{3/2} 30^{1/2}$

$= 0.38\, {}^{N}/_{mm^2} > v_{ED}(0.12) \ldots\ldots\ldots\ldots OK!!!!!$

Maximum punching shear at column perimeter

$$V_{ED,max} = 1149.4kN$$

Column perimeter $u_o = 2(400 + 200)mm = 1200mm$

K=from table k=0.65 for C1/C2=2

- **Punching shear stress,**

W1=$0.5* 400^2 + 400 * 200$

$W1 = 0.16 * 10^6 mm^2$

$$\beta = 1 + 0.65\left(\frac{11.4 * 10^6}{1149.4 * 10^3}\right) * \frac{1200}{0.16 * 10^6 m^2} = 1.04$$

$$v_{ED} = \frac{1.04 * 10^6}{1200 * 603} = 1.5\, {}^{N}/_{mm^2}$$

Maximum shear resistance

$$v_{ED} = 0.5\left[0.6 * \left(1 - \frac{f_{ck}}{250}\right) * \left(\frac{f_{ck}}{1.5}\right)\right] = 0.5\left[0.6 * \left(1 - \frac{30}{250}\right) * \left(\frac{30}{1.5}\right)\right]$$

$$v_{ED} = 5.3\,{}^{N}/_{mm^2} > v_{ED}{=}1.5\,{}^{N}/_{mm^2} \ldots\ldots\ldots\ldots OK!!!$$

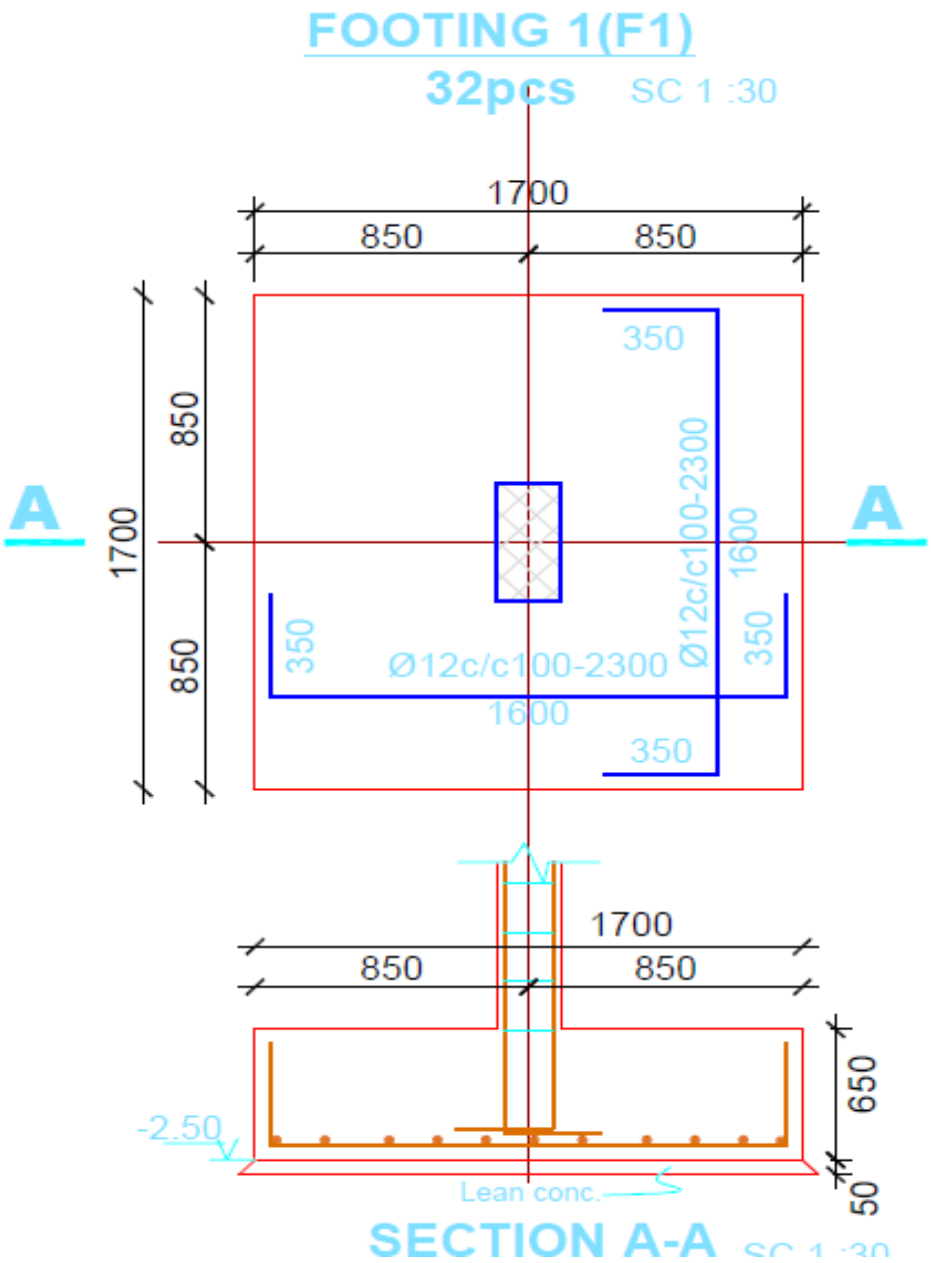


Figure 45 Foundation reinforcement detail

Conclusion

For designing and analysis of this G+3 building, ES EN code of 2015 was the main and fundamental reference for any relevant assumptions carried out in this project papers. Besides gratitude for EBCS editors, we conclude that they were thoroughly investigating the actual physical and environmental future of the country incorporating with universally accepted Engineering standards and formulae.

On analysis of this project many tedious calculation works were compiled with ETABS NL V9.6and the result was taken as it is for designing purpose, as the adequacy of each building elements was understandable by the program and can generate the output results with acceptable sign convention. From this what we conclude is that if this program is not available it is so difficult to complete this project within this time constraint.

In accordance with our design and analysis, the structure is economical and safe against different load application, but for actual implementation as the soil bearing capacity for the foundation was taken from relevant assumption, it is better if further soil exploration and modification is made for design of foundations accordingly.

Recommendation

Recommendation towards the design,

Truly speaking this project make as to come up with relating our class subjects to real design.

While designing structure, everywhere in the world, it must be per building code of standard of the country in which the structure is going to be built. This is because the actual physical and environmental future of the country is investigated and universally accepted engineering standards and formula provided in the code.

b. Recommendation towards the construction site,

- All the works especially the concrete work should be done and supervised with great care to attain class-I work (Job-quality requirement must be satisfied)
- All the structural members should be built as per the design.
- The quality of material to be used for construction should fulfill the specification given in the design.

c. Recommendation to our department,

- Even though, if there is a lot of peoples for the accomplishment of this project, there was the things that were try to put limitation and made negative effects on our other tasks.
- Finally, as it was recognized analysis and design of any engineering structure is supported with computer programming, hence we recommend that it had better if computer programming for civil engineers is given as a subject in order to make the students familiar with different software.

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[2] Dr. Ing. Adil Z, "ES EN 1991-2015 wind action training," Addis Ababa University, AAiT. SCEE, 2018.

[3] Ethiopian ministry of construction, *ES EN 2 1992-2015*. Ethiopia, 2015.

[4] Dr. Ing. Adil Z, "ES EN slab design example," Addis Ababa University, AAiT. SCEE, 2018.

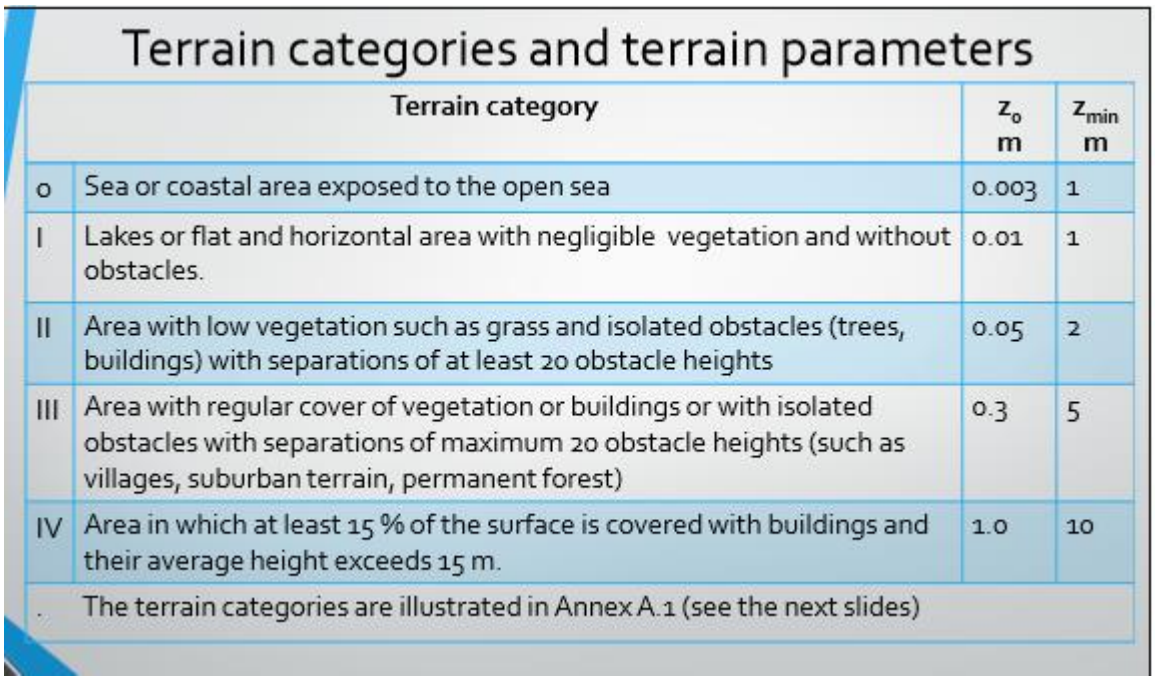
[5] Mr debisa G, "design of solid slabs," Wolkite University, Feb-2020.

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[7] Ethiopian ministry of construction, *ES EN 8 1992-2015*. Ethiopia, 2015.

Appendix A Terrain categories for roof design

Terrain categories [2]



Terrain categories and terrain parameters

	Terrain category	z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0.003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles.	0.01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0.05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0.3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m.	1.0	10
.	The terrain categories are illustrated in Annex A.1 (see the next slides)		

Appendix B recommended structural classification

Minimum cover, Cmin,b, requirements with regard to bond [3]

Table 4.2: Minimum cover, $c_{min,b}$, requirements with regard to bond

Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (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.	

Table 4.3N: Recommended structural classification

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 4.4N: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

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

Table 4.5N: Values of minimum cover, $c_{min,dur}$ requirements with regard to durability for prestressing 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	SD3 / XS3
S1	10	15	20	25	30	35	40
S2	10	15	25	30	35	40	45
S3	10	20	30	35	40	45	50
S4	10	25	35	40	45	50	55
S5	15	30	40	45	50	55	60
S6	20	35	45	50	55	60	65

Appendix C Live loadings on buildings

Table 6.1 - Categories of use

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

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1.5 to 2.0	2.0 to 3.0
- Stairs	2.0 to 4.0	2.0 to 4.0
- Balconies	2.5 to 4.0	2.0 to 3.0
Category B	2.0 to 3.0	1.5 to 4.5
Category C		
- C1	2.0 to 3.0	3.0 to 4.0
- C2	3.0 to 4.0	2.5 to 7.0 (4.0)
- C3	3.0 to 5.0	4.0 to 7.0
- C4	4.5 to 5.0	3.5 to 7.0
- C5	5.0 to 7.5	3.5 to 4.5
Category D		
- D1	4.0 to 5.0	3.5 to 7.0 (4.0)
- D2	4.0 to 5.0	3.5 to 7.0

Appendix D Moment coefficient

considered	Values of l_y/l_x								β_{sx} for all values of l_y/l_x
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
Interior panels									
Negative moment at continuous edge	0.031	0.037	0.042	0.046	0.050	0.053	0.059	0.063	0.032
Positive moment at mid-span	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
One short edge discontinuous									
Negative moment at continuous edge	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.067	0.037
Positive moment at mid-span	0.029	0.033	0.036	0.039	0.041	0.043	0.047	0.050	0.028
One long edge discontinuous									
Negative moment at continuous edge	0.039	0.049	0.056	0.063	0.068	0.073	0.082	0.089	0.037
Positive moment at mid-span	0.030	0.036	0.042	0.047	0.051	0.055	0.062	0.067	0.028
Two adjacent edges discontinuous	0.030	0.036	0.042	0.o047	0.051	0.055	0.062	0.067	0.028
Negative moment at continuous edge	0.047	0.056	0.063	0.069	0.074	0.078	0.087	0.093	0.045
Positive moment at mid-span	0.036	0.042	0.047	0.051	0.055	0.059	0.065	0.070	0.034
Two short edges discontinuous									
Negative moment at continuous edge	0.046	0.050	0.054	0.057	0.060	0.062	0.067	0.070	-
Positive moment at mid-span	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	0.034
Two long edges discontinuous									
Negative moment at continuous edge	-	--	-	-	-	-	-	-	0.045
Positive moment at mid-span	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	0.034
Three edges discontinuous (one long edge continuous)									
Negative moment at continuous edge	0.057	0.065	0.071	0.076	0.081	0.084	0.092	0.098	-
Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.063	0.069	0.074	0.044
Three edges discontinuous (one short edge continuous)									

Appendix E Shear force coefcient

Type of Panel and location	β_{vx} for values of l_y/l_x								β_{vy}
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
Four edges continuous									
Continuous edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
One short edge discontinuous									
Continuous edge	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
Discontinuous edge	-	-	-	-	-	-	-	-	0.24
One long edge discontinuous									
Continuous edge	0.36	0.40	0.44	0.47	0.49	0.51	0.55	0.59	0.36
Discontinuous edge	0.24	0.27	0.29	0.31	0.32	0.34	0.36	0.38	-
Two adjacent edges discontinuous									
Continuous edge	0.40	0.44	0.47	0.50	0.52	0.54	0.57	0.60	0.40
Discontinuous edge	0.26	0.29	0.31	0.33	0.34	0.35	0.38	0.40	0.26
Two short edges discontinuous									
Continuous edge	0.40	0.43	0.45	0.47	0.48	0.49	0.52	0.54	-
Discontinuous edge	-	-	-	-	-	-	-	-	0.26
Two long edges discontinuous									
Continuous edge	-	-	-	-	-	-	-	-	0.40
Discontinuous edge	0.26	0.30	0.33	0.36	0.38	0.40	0.44	0.47	-
Three edges discontinuous (one long edge discontinuous)									
Continuous edge	0.45	0.48	0.51	0.53	0.55	0.57	0.60	0.63	-
Discontinuous edge	0.30	0.32	0.34	0.35	0.36	0.37	0.39	0.41	0.29
Three edges discontinuous (one short edge discontinuous)									
Continuous edge	-	-	-	-	-	-	-	-	0.45
Discontinuous edge	0.29	0.33	0.36	0.38	0.40	0.42	0.45	0.48	0.30
Four edges discontinuous									
Discontinuous edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

Appendix F Basic span/depth ratio

Table 7.4N: 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	11.0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	11.3	18	26
Interior span of beam or one-way or two-way spanning slab	11.5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	11.2	17	24
Cantilever	0.4	6	8

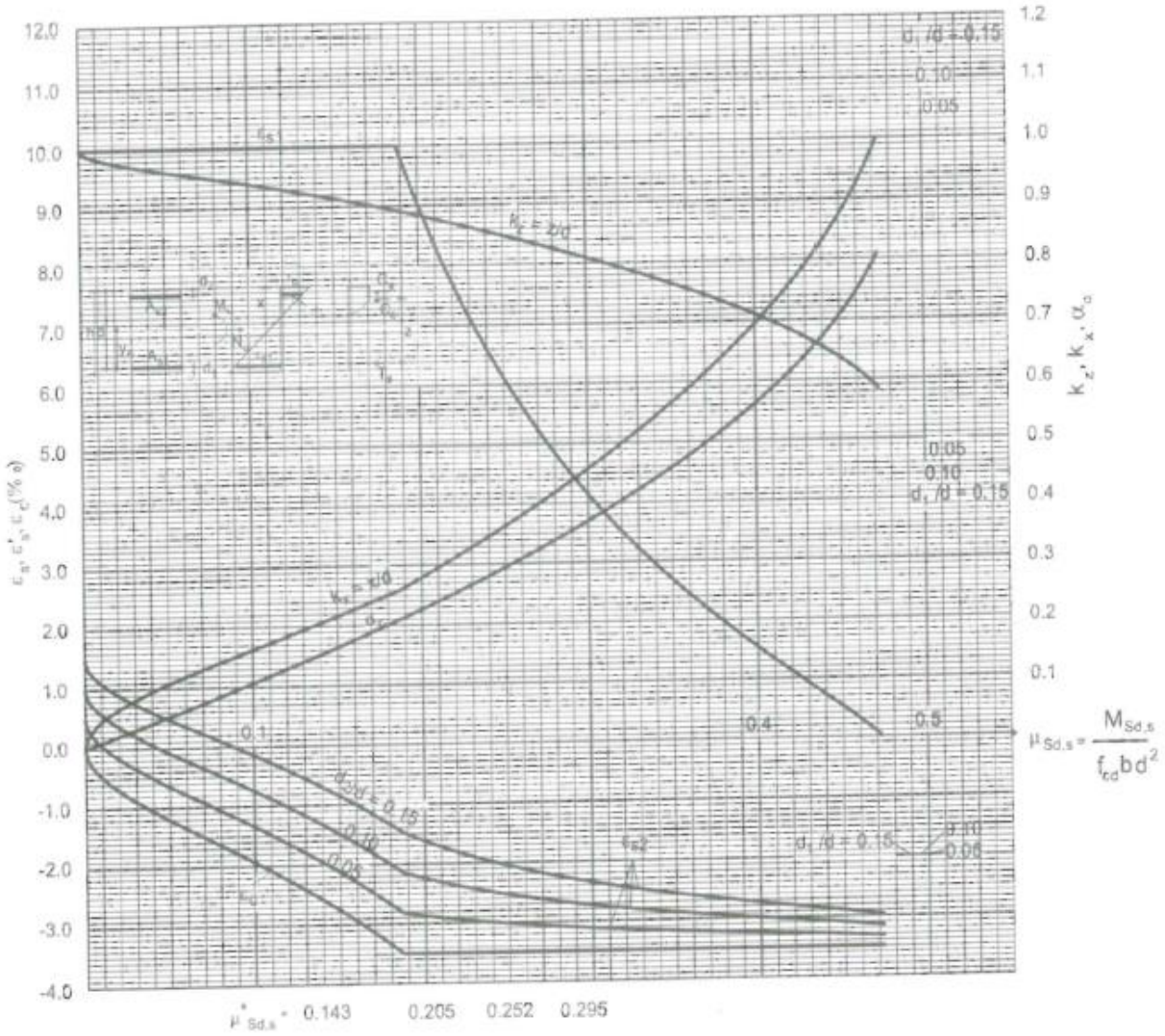
Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.
Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.
Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory

Appendix G General design table

4.1.1 General Design Table 1a: Rectangular Section - Moment with or without Axial Load, with or without Compression Reinforcement

k_m					k_s							Redist-ribution
C15	C20	C25	C30	C40	S300	S400	S460	ε_c	ε_{s1}	k_x	k_z	
15	17	19	21	24	3.95	2.96	2.58	0.94	10	0.09	0.97	
19	22	25	27	31	3.99	2.99	2.60	1.27	10	0.11	0.96	
23	26	29	32	37	4.04	3.03	2.63	1.59	10	0.14	0.95	
26	30	33	37	42	4.08	3.06	2.66	1.92	10	0.16	0.94	
29	33	37	40	47	4.12	3.09	2.69	2.24	10	0.18	0.93	
31	35	40	43	50	4.17	3.13	2.72	2.56	10	0.20	0.92	
31.12	35.94	40.18	44.02	50.83	4.18	3.13	2.72	2.63	10	0.208	0.918	30%
33	38	42	46	53	4.21	3.16	2.75	2.89	10	0.22	0.91	
34	40	44	49	56	4.26	3.19	2.78	3.22	10	0.24	0.90	
36	42	46	51	59	4.31	3.23	2.81	3.50	9.7	0.26	0.89	
37.36	43.13	48.23	52.83	61	4.36	3.27	2.84	3.50	8.7	0.288	0.880	20%
37	43	48	53	61	4.36	3.27	2.84	3.50	8.6	0.29	0.88	
39	45	50	55	63	4.41	3.30	2.87	3.50	7.7	0.31	0.87	
40	46	52	56	65	4.46	3.34	2.91	3.50	6.9	0.34	0.86	
41	47	53	58	67	4.51	3.38	2.94	3.50	6.2	0.36	0.85	
41.42	47.83	53.47	58.58	67.64	4.53	3.39	2.95	3.50	6.0	0.368	0.847	10%
42	49	54	60	69	4.56	3.42	2.98	3.50	5.6	0.38	0.84	
43	50	56	61	71	4.62	3.46	3.01	3.50	5.1	0.41	0.83	
44	51	57	63	72	4.67	3.51	3.05	3.50	4.6	0.43	0.82	
44.79	51.72	57.83	63.35	73.15	4.71	3.53	3.07	3.50	4.3	0.448	0.814	

Appendix H general design chart 1



Appendix I important classes for buildings

Table 4.3 Importance classes for buildings

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

NOTE Importance classes I, II and III or IV correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EN 1990:2002, Annex B.

Appendix J ground types

Table 3.1: Ground types

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			