Civil Engineering Department
Wolkite University, College of Engineering and Technology

Structural design of B+G+5 mixed use commercial building using EBCS 2015

A B.Sc. thesis/project (CENG5281) submitted as a partial fulfilment of the requirements for the Degree of Bachelor Science in Civil Engineering

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As members of the examining board of the final B.Sc. open defense, we verify that we have read and evaluated the final BSc thesis/project prepared by Mekbib Turga, Selamawit Kefale, Mulu Girma, Kena Bayisa, Seifu Asfew, Lensa Assefa and Blen Girma entitled Structural design of B+G+5 mixed use commercial building using EBCS 2015, and recommended for acceptance as a fulfillment of the requirement of B.Sc. in Civil Engineering

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Executive Summary

This project is about the structural design and analysis of B+G+5 mixed use commercial building considering all the effects the building receives in its service year. Structural design involves proportioning of structural members of the building and providing reinforcement which results economical. In addition to the structural design and analysis. In short, the main structural designs included the paper is as follows. The roof is composed of a flat RC roof the design deals with analysis of the external forces that the roof is exposed. Then, we have selected an adequate strength against the applied wind, dead and live loads that they are subjected to. Our buildings floors are all constructed with a solid RC slab. We have designed the slab for serviceability and ultimate limit state. Therefore, we have selected a depth which satisfies deflection requirement and provided flexural and shear reinforcement. The circulation system in our building is facilitated by a u-shaped having 3 flight stair type. It has been designed by analyzing the stair for all loads it is subjected to through its service life and provided with adequate reinforcement. The lateral load analysis includes both wind and earthquake. It consists of lateral load analysis which are wind and earthquake load analysis by ETABS 2017 by using several load combinations and also frames were analyzed using ETABS 2017. After generating the analysis result, we designed beams and columns by assuming a preliminary cross section and finally checking with respect to the maximum shear and flexural moments. Finally, we have provided shear and flexural reinforcements. We have also designed a shear wall for wall and also, we have designed isolated footing type is selected for our foundation. The design was made by taking the loads coming from the super structure through columns. Finally, we generated detail drawings for slab reinforcement, beam reinforcement, column reinforcement and footing reinforcement. Throughout the whole process, different software will be used such as: Auto-cad, MS – office (word, excel,) Analysis & design software (ETABS 2017 and SAP 2000 v14)

Acknowledgement

First of all we would like to give our deepest praise to almighty God for his great kindness to us from start to the end of the project.

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At last but not least our heartfelt gratitude and love for our group members, classmate and friends who shares different ideas while we get problems.

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List of Symbols and Acronyms

- EBCS-Ethiopian building code standard
- EGA-Enhanced Graphics Adapter
- ϕ-Diameter of bars
- fck – compressive characteristic strength of concrete
- fctd- tensile design strength
- fctk- tensile characteristic strength of concrete
- Fcd-compressive design strength of concrete
- Y_c– concrete factor of safety
- fyk – characteristic strength of steel
- fyd – design strength of steel
- Wk - Wind Load
- Pd - Design Load
- t_f - Thickness of a Flange
- tw-thickness of a web
- Ac - Gross sectional Area
- P – Load, V – Shear, T – Torsion, M - Moment
- Ix & Iy - Moment of Inertia in the X and in the Y respectively
- Le- Effective Length
- DL - Dead Load, LL - Live Load
- αxs & αys - Support BM coefficients for rectangular slabs supported on four sides in the x and y
- ΔM - Support Moment difference between two adjacent slabs
- Madj - Adjusted Moment
- C - Span Moment factors for adjustment
- ρmin – minimum steel ratio
- As,min - Minimum required area of steel
- deff - effective depth
- As - Area of topping reinforcement
- S - Spacing
- d(T) - Design Spectrum at period T
- C_β - Design Response factor
- ys – Steel factor of safety
- A_c: Gross Area of concrete
- A_s: Area of tensile reinforcement
- A_s: Area of compressive reinforcement bar
- b, h: Dimension of rectangular section (width and depth respectively)
- CALT: Altitude factor
- C_d: Dynamic coefficient
- CDIR: Direction factor

- C_e: Exposure coefficient
- C_p: Pressure coefficient
- C_r: Roughness coefficient
- C_T: Topographic coefficient
- t CTEM: Temperature coefficient
- d: Effective depth
- D: Gross Depth
- E: Modulus of elasticity
- E_c: Modulus of elasticity of concrete
- f_{cy} Maximum compressive strain of concrete
- E_s: Modulus of elasticity of steel
- f_{cd}: Design Compressive strength of concrete in compression
- f_{ck}: Characteristic cylindrical compressive strength of concrete
- f_{ctd}: Tensile strength of concrete
- f_{yd}: Design yield strength of Reinforcement Bar in Tension and Compression
- f_{yk}: Characteristic strength of Reinforcement Bar in Tension
- HCB: Hollow Concrete Block
- I: Second Moment of Inertia
- K: Equivalent roughness
- KT: Terrain factor
- qref: Reference mean velocity pressure
- r: Radius of gyration
- RC: Reinforced Concrete
- t: Plate thickness
- Vm: Mean wind velocity
- V_{ref}: Relative wind velocity factor
- Z: Height above the ground
- Z_{min}: Minimum height
- Z_o: Roughness length
- W_e: external pressure
- C_{pe}: external pressure coefficient
- C_{pi}: internal pressure coefficient

- Qk: live load
- G_k: dead load
- F_b: base shear
- X_m: center of mass in x axis
- Y_m: center of mass in y axis
- L_y: long dimension
- L_x: short direction
- M_{ys}: support moment in the longer direction of panel
- M_{xs}: support moment in the shorter direction of panel
- M_{xf}: field moment in the shorter direction of the panel
- M_{yf}: field moment in the longer direction of panel

1. Introduction

1.1 General

Structural Design is the art and science of proportioning and dimensioning structural members based on responses values obtained. With reference to minimum Codal provision, the depth of a member, spacing of reinforcement, area of reinforcement are decided to achieve special requirements of design like safety, strength, durability, economy as well as Aesthetic.

Structural Analysis is the process by which an engineer determines the responses of a structure for a specified loading condition to achieve basic requirements of design. Those structural responses include shear force, bending moment, deflection, reaction, rotation and etc. To determine those structural responses linear elastic analysis are used in this project.

1.2 Background of the project

A. Project Site Description

This project is expected to be constructed at Adama town, Ethiopia. Adama is a town located in the Great Rift Valley zone, where geographical and geological environment needs great concern for design. Basic information about Adama city from national map and Google earth is as follows.

Table 1. 1Basic Background Information Of Proposed Project Site

S.N	Information	Description
1	City Type	
2	Population Size	
3	Wind Category	Category-IV
4	Earth Quack Category	Category-IV
5	Altitude	
6	Longitude, E	39.2682
7	Latitude, N	8.5386
8	Soil Type	
9	Temperature	
10	Humidity	
11	Ground Water Location	
12	Annual Rain Fall	

No table of figures entries found. Our project has B+G+5 it is 9 stories mixed used commercial building that is around 497.9 m2 area its basement floor is used as Parking, shop and store(1 parking, 3 shops and 1 store), Ground floor is used as shop, café, reception room and Game house (4 shops, 1 café, 1 reception room and 4 game houses), first floor is used as restaurant, bed room and store(1 restaurant, 6 bed room and 1 store) ,second floor to fourth floor have the same functions since they all are typical. Their function is same(12 bed rooms and 2 double bed rooms), Fifth floor is used for Laundry and store(1 laundry and 1 store). From architectural plan our roof type is mono pitched, duo pitch and slab roof.

This project deals about the structural analysis and design of B+G+5 building for mixed used considering all the internal and external effects according to ES EN 2015.

B. Purpose of building

The building going to analyze and design has mixed use commercial applications like shopping, office, parking, store, restaurant…

1.3 Objective

1.3.1 Main objective

As an Structural Engineer we are expected to convert architectural engineers' (given drawing) drawing in to real world building. Our main objectives are design safe, durable and economical structural design.

1.3.2 Sub-objectives

our selected project has the following objectives

- To improve the analysis and design skill of graduators
- To develop the habit of working together.
- To appreciate practical use of design codes and design software
- To make understanding on structural planning and detailing
- Structural modeling

1.4 Structural Design Philosophy and Methods

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

1. Working Stress Method , WSM

This was the traditional and currently an outdated method of design primary for reinforced concrete, and also for structural steel and timber design.

2 Ultimate Load Method, ULM

In this method, the stress condition at the state of impending collapse of the structure is analyzed, and the non-linear stress-strain curves of concrete and steel are made use of. The design stresses used are the ultimate strength of materials and for safety the loads are magnified by load factors, defined as the ratio of ultimate load to working load.

3 Limit States Method, LSM

Unlike WSM, which is based on service load conditions alone, and unlike ULM, which based on ultimate load condition alone, LSM aims for a comprehensive and rational solution to the design problem, by considering safety at ultimate loads and serviceability at working loads.

The limit states for reinforced concrete structures can be divided into three basic groups.

- **Ultimate Limit State.** This involves a structural collapse of part or all of the structure. Such limit state should have a very low probability of occurrence, because it may lead to loss of life and major financial losses.
- **Serviceability Limit States.** These involve disruption of the functional use of the structure, but no collapse occurs. Because there is less danger of loss of life, a higher probability of occurrence is generally tolerated than in case of an ULS.
- **Special Limit States.** This class of limit states involve damage or failure due to abnormal conditions or abnormal loadings and includes:
  - ✓ Damage or collapse in extreme earthquakes,
  - ✓ Structural effects of fire, explosions, or vehicular collisions,
  - ✓ Structural effects of corrosion or deterioration, and

1.5 Design Specifications and Constants

Purpose – Mixed use building

1.5.1 Material used and Properties

According to the architectural plan, various materials are recommended for different parts of the building. For the calculation of self-weight, their specification is taken from the standards. Generally the following materials are used in this specific building project.

- ✓ Class of concrete – Class I Work
- ✓ Steel
- ✓ Masonry
- ✓ HCB
- ✓ Finishing material
- ✓ RHS for Roof truss and Purlin
- ✓ EGA- 300 for roof cover is used.
- ✓ Soil condition – sandy soil

Reinforced concrete

- Unit weight of normal concrete =24KN/m3
- Unit weight of reinforced concrete =25KN/m3
- Partial safety factor, γc = 1.5, Partial safety factor; Υs=1.15
- C20/25 for super structure, C25/30 for foundation
- Design concrete strength, fcd = $\frac{\alpha cc * fck}{\gamma c} = \frac{0.85*20}{1.5}$ = 11.33Mpa
- Design tensile strength, fctd = $\frac{\alpha ct * fctk,0.05}{\gamma c} = 1.031$Mpa
- Fctm = 2.21Mpa for C20/25, Fctm = 2.49Mpa for C25/30

- Design concrete strength, fcd = $\frac{\alpha cc*fck}{\gamma c} = \frac{0.85*24}{1.5}$ = 13.6Mpa
- Design tensile strength, fctd = $\frac{\alpha ct*fctk,0.05}{\gamma c} = 1.165\text{Mpa}$

Reinforcement (steel)

- Grade of steel; S-460
- Fyd = $\frac{fyk}{\gamma s} = \frac{460}{1.15}$ = 400Mpa
- Modulus of elasticity; Es=200Gpa; Ec=30Mpa
- Poison's ratio=0.2

1.5.2 Design Aids used for this paper

Approach- Limit state design method
References (Code) = ES EN (Ethiopian Standard based on Euro Norms). {Megersa 2021}
Euro Code nowadays is being practiced in most countries around the world and Ethiopia is one of those countries who has shown interest in adopting the Euro Code.

The codes serve at least two distinct functions:

✓ Ensure adequate structural safety, by specifying certain essential minimum requirements for design.

✓ Ensure a measure of consistency among different designers.

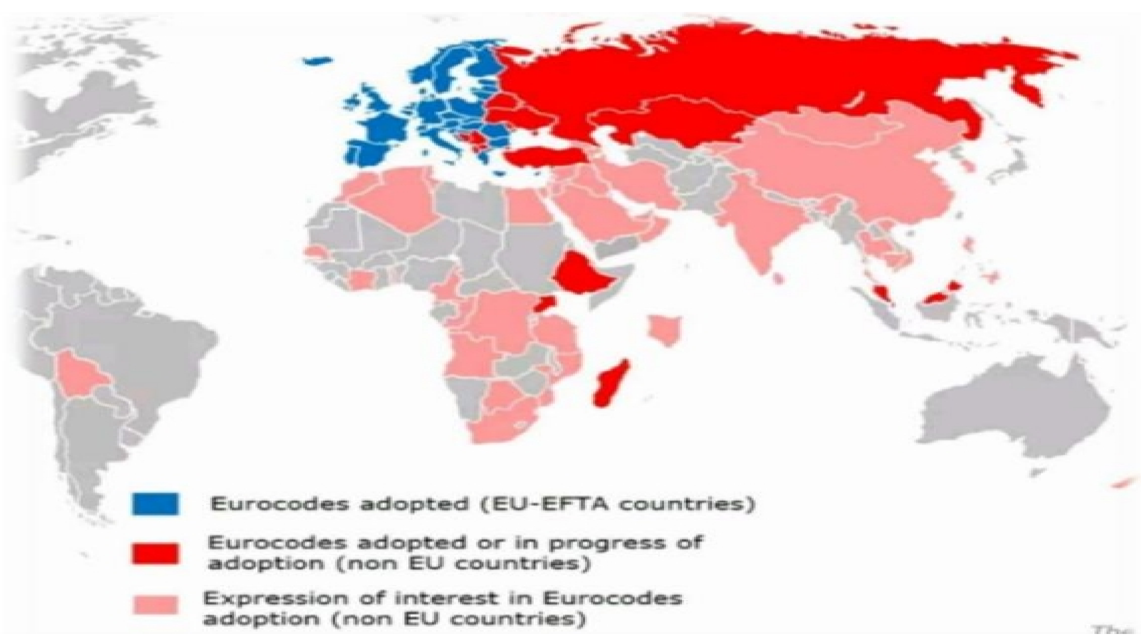


Figure 1. 1 European Norm practices

Table 1. 2 ES EN Basis of design

ES EN 1990	ES EN 0	Basis of design (ES EN 0)
ES EN 1991	ES EN 1	Actions on structures (ES EN 1)
	Part 1-1	General actions – Densities, self-weight and imposed loads
	Part 1-2	General actions on structures exposed to fire
	Part 1-3	General actions – Snow loads
	Part 1-4	General actions – Wind loads
	Part 1-5	General actions – Thermal actions
	Part 1-6	Actions during execution
	Part 1-7	Accidental actions from impact and explosions
	Part 2	Traffic loads on bridges
	Part 3	Actions induced by cranes and machinery
	Part 4	Actions in silos and tanks
ES EN 1992	ES EN 2	Design of concrete structures (ES EN 2)
	Part 1-1	General rules and rules for buildings
	Part 1-2	General rules – structural fire design
	Part 2	Reinforced and prestresses concrete bridges
	Part 3	Liquid retaining and containing structures
ES EN 1993	ES EN 3	Design of steel structures
ES EN 1994	ES EN 4	Design of composite steel and concrete structures
ES EN 1995	ES EN 5	Design of timber structures
ES EN 1996	ES EN 6	Design of masonry structures
ES EN 1997	ES EN 7	Geotechnical design
ES EN 1998	ES EN 8	Earthquake resistant design of structures
ES EN 1999	ES EN 9	Design of aluminum alloy structures

1.6 Basic Structural Components

According to the architectural plan of our project the following components of building are expected to be analyzed and designed in such a way to carry, resist and transfer loads interactively to maintain the overall stability of the whole building**.** But due to time limitation some components are not deigned.

- Roof
- Beam
- Column
- Slab
- Stair
- Lateral Shear wall
- Masonry wall
- Basement structure

- Foundation

1.7 Design Code Used

In order to perform accurate analysis and design to achieve architectural and structural requirements of the project, the following national and international design codes and standards are referred,

- ✓ EBCS-1995 Design Standards
- ✓ ES EN 2015 Design Standards

1. **Design Software**

To simplify our work as well as to achieve the accuracy of the analysis and design of the project, ETABS and SAP design software is used. Other computer skills like AutoCAD, Ms Word, Ms excel are also used to prepare the document**.**

2. **Design Load Considered**

Based on the function, location and material to be used in the project, the following expected loads are considered during analysis and design referring the design standards.

- ✓ Dead Load
- ✓ Live Load
- ✓ Wind Load
- ✓ Earth Quack Load
- ✓ Lateral Earth Pressure
- ✓ Imposed load

1.8 Methodology

The required data for structural analysis are collected from architectural plan, design codes, design standards and also reasonable assumptions are taken based on engineering judgment. Based on this preliminary data, step by step analysis and design are conducted. Based on analysis and design results various accuracy checks are done referring Codal design recommendation. The project document will be prepared using Ms Word and any supporting Microsoft office applications and software outputs. Finally the conclusion and recommendations will be included about the project.

2 Analysis And Design Of Roof

2.1 Introduction

Wind is simply a moving air with phenomenon of great complexity of many flow situations arising from the interaction of wind with surrounding terrain and the proposed structure to be design. It is one of the main environmental loads considered in the design of building structures in wind prone areas.

- Velocity of air
- Height of the structure
- Density of air
- Type of roof
- wind incidence angle

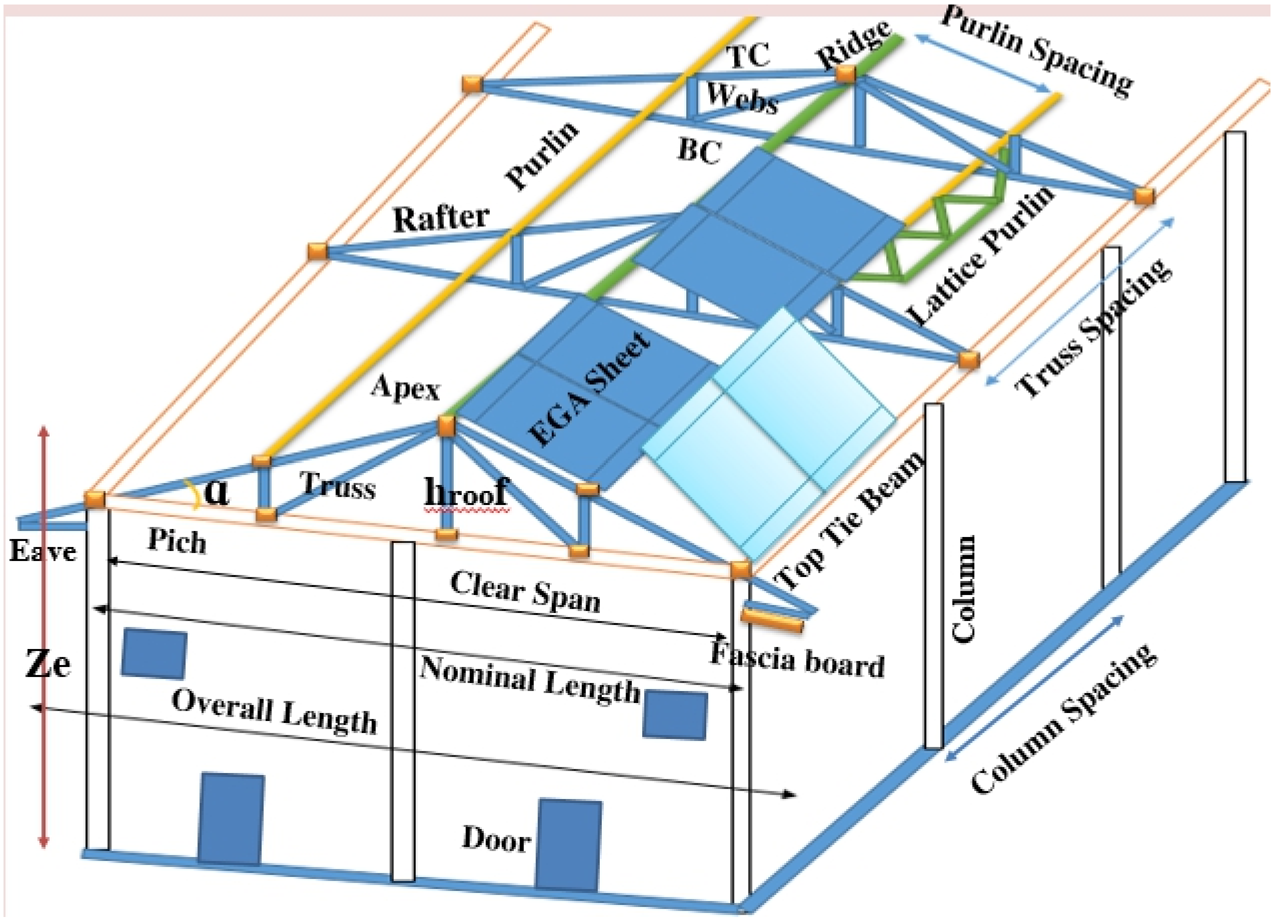
Table 2. 1 General selection criteria for roof

S.N	Performance Criteria	RS	RC	RI	RL	RD	RF
1	Strength &stability	✓			✓	✓	✓
2	Fire resistance	✓	✓	✓	✓		
3	Life span	✓	✓	✓	✓	✓	✓
4	Maintainace cost	✓	✓	✓	✓	✓	✓
5	Initial cost	✓	✓	✓	✓	✓	✓
6	Life cycle cost	✓	✓	✓	✓	✓	✓
7	Sustainability	✓	✓	✓			
8	Ease of installation	✓	✓	✓	✓		
9	Freedom from maintainace	✓	✓	✓	✓	✓	✓
10	Sound resistance		✓	✓	✓		
11	Thermal performance		✓	✓	✓		
12	Weather resistance		✓		✓	✓	
13	Weight		✓	✓			
14	Security				✓		
15	Thickness			✓			
RS=Roof Structure RL= Roof light		RC=Roof Covering RD= Roof Drainage			RI=Roof Insulation RF=Roof Feature		

Roof System

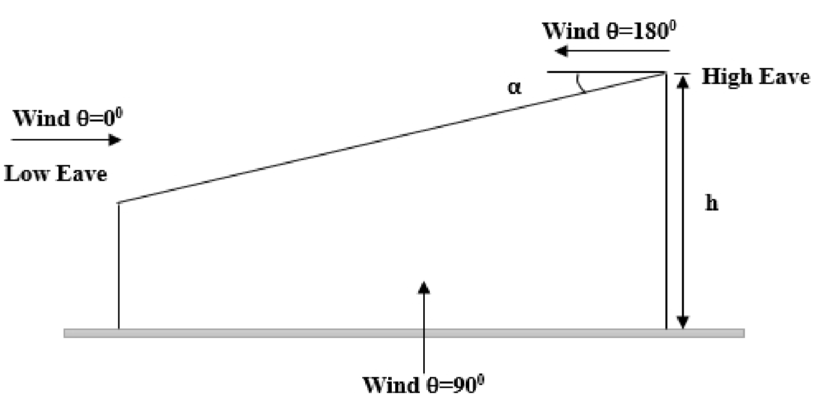
Roof is the most upper surface of any building that protects the user from rain, snow, sunlight. Different types of roofs are Mono-pitch, duo-pitched, hipped, gable, sky light, multi span, slab roof etc. The selection of roof type depends on the shape of building, economy, aesthetics, material type, constructability. From architectural plan our roof type is mono pitched, duo-pitched and slab roof.

Typical Components of duo pitched Roof System

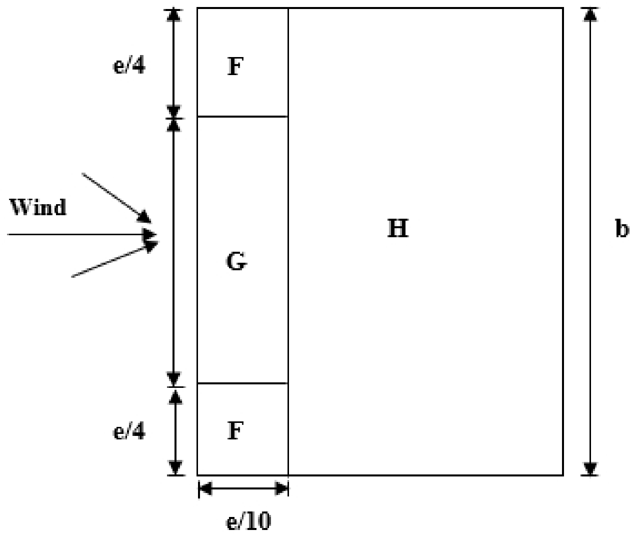


3Figure 2. 1 Typical Components of duo pich Roof System

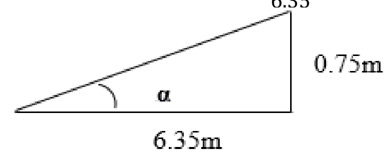
Roof 1: Monopich Roof Analysis and Design



Monopich Roof wind Analysis, wind direction Θ = 0°



- ✓ Height of roof h=16.5m
- ✓ Pitch height = 0.75m
- ✓ Width along cross wind, b=21.31m,
- ✓ Width parallel to wind=6.35m
- ✓ Pitch angle $\alpha = \tan^{-1}[\frac{0.75}{6.35}] = 6.74^0$



- ✓ e =b or 2h whichever is smaller

b=21.31m & 2h = 33m

Therefore e=21.31m

e/4=21.31/4=5.33m

Figure 2. 2 mono pitched roof wind analysis for $\Theta=0^0$ and $\Theta=180^0$ Wind Direction

2.2 Analysis of wind load on the roof

2.2.1 Wind Parameter analysis to monopitched roof

I. **Fundamental Wind Velocity, Vb,0**

The fundamental wind velocity, $V_{b,0}$ is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights which is mostly taken as **22m/sec.**

II. **Basic Wind Velocity, V_b**

The basic wind velocity V_b is determined according to ES EN 1991-1-4:2015 expression 4.1.

$V_b = C_{dir} * C_{season} * V_{,0}$

Where: $V_{b,0}$ is the fundamental value of the basic wind velocity;

C_{dir} is the directional factor; its recommended value is 1. ES EN-1991-1-4:2015 sec-4.2

C_{season} is the seasonal factor, its recommended value is 1 ES EN-1991-1-4:2015 sec-4.2

Therefore, $V_b = 1*1*22m/sec = 22m/sec$

III. **Mean Wind Velocity, $V_m(z)$**

The mean wind velocity $V_m(z)$ at a height Z above the terrain depends on the terrain roughness and topography and on basic wind velocity, Vb, and should be determined using ES EN 1991-1-4:2015 section 4.3.1(1) P, expression (4.3)

$V_m(z) = C_r(z) \times C_o(z) \times V_b$

Where: $C_r(z)$ is the roughness factor, and

$C_o(z)$ is the topography factor.

-According to ES EN 1991-1-4:2015 sec-4.3.1 the topography factor can be taken as 1 unless specified. The roughness factor accounts the variability of mean wind velocity of the size of the structure due to

- Height above ground level
- The ground roughness of the terrain up wind of the structure on the direction considered (ES EN 1991-1-4:2015 section 4.3.2(1)) the topography factor is given by:

Cr (z) =$Kr*\ln(\frac{z}{zo})$, $for\ Zmin \leq Z \leq Zmax$ or

Cr (z)=$(Zmin)$ for $Z \leq Zmin$

Where, Kr is the terrain factor depending on roughness length Zo.

The terrain factor Kr should be determined using ES EN 1991-1-4:2015 expression (4.5).

$$Kr=0.19(\frac{Z0}{Zo,II})^{0.07}$$

Where, $Z_{0,\ II}$ 0.05 (for terrain category II, ES EN 1991-1-4:2015, Table 4.8),

Zmin is minimum height depend on terrain category, and

Zmax is maximum height is to be taken 200m unless otherwise defined

Zo is Roughness length depends on terrain category.

We have categorized Adama under terrain category IV in which at least 15 % of the city surface is covered with buildings and their average height exceeds 15 m. According to ES EN 1991-1-4:2015, Table 4.1, values of roughness length (Zo) and minimum height (Zmin) are 1m and 10m respectively.

Table 4.1 Terrain categories and terrain parameters

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

The total height of our building from the ground level to roof top is 16.5m

Zo = 1m Zo, $_{II}$ = 0.05m Zmin = 10m Z = 16.5m Zmax = 200m

Since, Zmin ≤ Z ≤Z_{max} $10m \leq 16.5\ m \leq 200m$

Kr = $0.19*(\frac{1}{0.05})^{0.07}$ = **0.234**

$$Cr(z) = k_r*\ln(\frac{Z}{Z0}) = 0.234*\ln(\frac{16.5}{1}) = 0.656$$

Therefore, the Mean Wind Velocity Vm (Z) = Cr(Z) *Co (Z) *Vb = 0.656*1*22 **=14.44m/s**

IV. **Wind Turbulence**

According to ES EN 1991-1-4:2015, section 4.4(11) is the turbulence intensity $l_v(z)$ at height Z is defined as the standard deviation of the turbulence divided by mean wind velocity. And it should be determined using ES EN 1991-1-4:2015 expression (4.7).

$l_v(z)=\frac{\sigma v}{Vm(z)}=\frac{Kr}{Co(z)*\ln\left(\frac{Zo}{Z0,II}\right)}$ for Zmin<Z<Zmax

$l_v(\ )=$ lv(Z) for Z<Zmin

σ$_v$ is the standard deviation.

The Standard deviation is given by ES EN 1991-1-4:2015 expression 4.6. = * *

=0.234×22 / ×1=5.148

Therefore Wind Turbulence is lv(16.5m)=$\frac{\sigma v}{Vm(16.5)}=\frac{5.148}{14.44}$= **0.357**

V. Peak Velocity Pressure,qp(z)

The peak velocity pressure qp(z) at height z, which includes mean and short term velocity fluctuations, should be determined by ES EN 1991-1-4:2015 Expression (4.8)

qp (z) =[1+7*lv(z)]*$\frac{1}{2}$* *$V_m^2(z)$ or qp (z) =Ce(z)*q_b

Where

ρis the density of air depends on attitude, temperature, and barometric pressure of the region

Ce(z) is exposure factor determined by ES EN 1991-1-4:2015

qb is the basic velocity pressure, determined from ES EN 1991-1-4:2015 expression 4.9.

=$\frac{1}{2}$* *V ^{2}=1/2*1.25*22^2= **302.5kg/ms^2**

According to ES EN 1991-1-4:2015, the recommended value of air density is 1.25kg/m^3.

qp(z) = [1+7*lv(z)]*$\frac{1}{2}$**$V_m^2(z)$*

qp (16.5m) = (1+7*0.357*)*1/2*1.25*14.44^2=455.99N/m2= **0.456KN/m2**

2.2.2 Wind Pressure on Surface

I. External wind pressure

External wind pressure on the surfaces, is obtained from ES EN 1991-1-4:2015 expression 5.1.

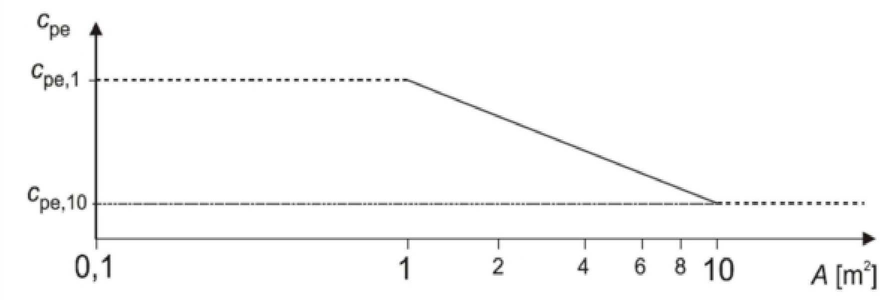
= ( )*

Where, ( ) is the peak velocity pressure,

Zeis the reference height for the external pressure, and

Cpe(e) is the pressure coefficient for the external pressure

The external pressure coefficients Cpe for buildings and parts of buildings depend on the size of the loaded area A. The external pressure coefficients are given for loaded areas **A** of 1**m^2** and **10m^2** in ES EN 1991-1-4:2015 table 7.4 for the appropriate building configurations as Cpe,1, for local & Cpe,10, for overall coefficients, respectively (ES EN 1991-1-4:2015 Sec.7.2.1



The figure is based on the following:

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

Figure 7.2 — Recommended procedure for determining the external pressure coefficient c_{pe} for buildings with a loaded area A between 1 m^2 and 10 m^2

= ,1 for A $\leq 1m^2$

= ,1− (, 1− , 10) $\log_{10}$ for $1m^2 < A < 10m^2$

= ,10 for $A \geq 10m^2$

Where Cpe, 1 is external pressure coefficient for loaded area A of 1 m^2

Cpe, 10 is external pressure coefficient for loaded area A of 10 m^2, and

A is loaded area of roof zones

II. Internal Pressure, Cpi

The internal pressure coefficient can be calculated using the following formula, if it is difficult to estimate μ accurately Cpi=+0.2 and Cpi=-0.3 is taken.

$$\mu = \frac{\sum \text{area of openings where } c_{pe} \text{ is negative or - 0.0}}{\sum \text{area of all openings}}$$

- ✓ Close to the openings at the leeward and parallel side (internal pressure), μ =0 and Cpi = **+0.2**
- ✓ Close all the openings at the windward direction (internal suction). μ = 1.0 and Cpi = **-0.3**

All area of subdivided zone is greater than $10m^2$,Cpe=Cpe,10, Table 7.3a CES 145 and interpolated value of Cpe,10 for 6.74^0 roof pich angle between 5^0 and 15^0 is taken for 0^0 wind direction.

Table 7.3a — External pressure coefficients for monopitch roofs

Pitch Angle α	Zone for wind direction $\theta = 0°$						Zone for wind direction $\theta = 180°$					
	F		G		H		F		G		H	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5°	-1,7	-2,5	-1,2	-2,0	-0,6	-1,2	-2,3	-2,5	-1,3	-2,0	-0,8	-1,2
	+0,0		+0,0		+0,0							
15°	-0,9	-2,0	-0,8	-1,5	-0,3		-2,5	-2,8	-1,3	-2,0	-0,9	-1,2
	+0,2		+0,2		+ 0,2							
30°	-0,5	-1,5	-0,5	-1,5	-0,2		-1,1	-2,3	-0,8	-1,5	-0,8	
	+0,7		+0,7		+0,4							
45°	-0,0		-0,0		-0,0		-0,6	-1,3	-0,5		-0,7	
	+0,7		+0,7		+0,6							
60°	+0,7		+0,7		+0,7		-0,5	-1,0	-0,5		-0,5	
75°	+0,8		+0,8		+0,8		-0,5	-1,0	-0,5		-0,5	

Table 1. 7Table 2. 2 External coefficients of mono-pitch for 6.74^0 pitch angle

piche angle	Cpe	F	G	H
5^0	Cpe,10-	-1.7	-1.2	-0.6
	Cpe, 10+	0	0	0
6.74^0	Cpe,10-	-1.56	-1.13	-0.547
	Cpe, 10+	0.135	0.135	0.135
15^0	Cpe,10-	-0.9	-0.8	-0.3
	Cpe, 10+	0.2	0.2	0.2

Wnet=We-Wi=qp(Ze)*Cpe- qp(Zi)*Cpi =qp(Z)(Cpe-Cpi) taking Ze=Zi=Z, depends on values of both Cpe and Cpi can positive or negative pressure coefficients and we have for possible vales of net pressure, among those the maximum positive and negative pressure is taken for design.

Wnet = qp(z)*[Cpe(-) – Cpi(+)]
Wnet = qp(z)*[Cpe(-) – Cpi(-)]
Wnet = qp(z)*[Cpe(+) – Cpi(+)]

Table 1. 8Table 2. 3 net wind pressure result for mono pitched roof for 0 degree wind direction

Wnet=qp(z)*(Cpe-Cpi) =0.456 *(Cpe-Cpi)KN/m^2

Zone	**F**	**G**	**H**
Area,m^2	11.36	22.7	306.2
Cpe,-	-1.56	-1.13	-0.547
Cpe,+	0.135	0.135	0.135
Cpi,+	0.2	0.2	0.2
Cpi,-	-0.3	-0.3	-0.3
Cpe(-) - Cpi,+	-1.76	-1.33	-0.747
Cpe(-) - Cpi,-	-1.26	-0.83	-0.247
Cpe(+) - Cpi,+	-0.065	-0.065	-0.065
Cpe(+) - Cpi,-	0.435	0.435	0.435
Wnet	**-0.803**	-0.606	-0.341
Wnet	-0.575	-0.378	-0.113
Wnet	-0.03	-0.03	-0.03
Wnet	**0.198**	**0.198**	**0.198**

The maximum net wind pressure is taken considering the magnitude regardless of sign, for 0^0 degree wind direction maximum net positive pressure occurs at all surfaces **Wnet,+= +0.198 KN/m^2** and the maximum net negative pressure occurs at zone **F**,**Wnet,-= -0.803KN/m^2.**

Monopich Roof wind Analysis, wind direction ϴ = 90°

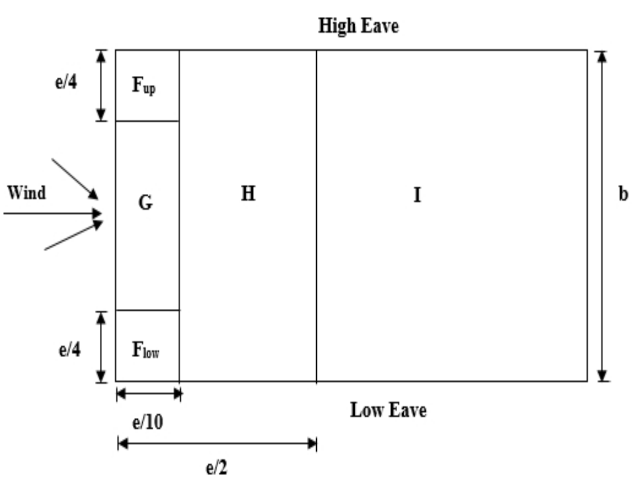
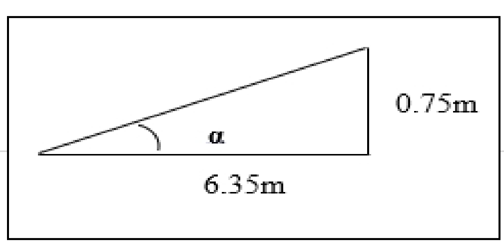


Figure 1. 5Figure 2. 3 mono pitched roof wind ana

- ✓ Height of roof h=16.5m
- ✓ Pitch height = 0.75m
- ✓ Width along cross wind, b=6.35m,
- ✓ Width parallel to wind=21.31m
- ✓ Pitc
- ✓
- ✓
- ✓
- ✓

h angle α = $\tan^{-1}[\frac{0.75}{6.35}]$ = 6.74^0

- ✓ e =b or 2h whichever is smaller



- Height of roof h=16.5m
- Pitch height = 0.75m
- Width along cross wind, b=6.35m,
- Width parallel to wind=21.31m
- Pitch angle $\alpha = \tan^{-1}[\ 0.75\ 6.35\ ] = 6.74^{o}$
- e =b or 2h whichever is smaller b=6.35m & 2h = 33m, Therefore e=6.35m e/4=6.35/4=1.588m e/10=6.35/10=0.635m

Table 7.3b — External pressure coefficients for monopitch roofs

Pitch Angle α	Zone for wind direction θ = 90°									
	F_{up}		F_{low}		G		H		I	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5°	-2,1	-2,6	-2,1	-2,4	-1,8	-2,0	-0,6	-1,2	-0,5	
15°	-2,4	-2,9	-1,6	-2,4	-1,9	-2,5	-0,8	-1,2	-0,7	-1,2
30°	-2,1	-2,9	-1,3	-2,0	-1,5	-2,0	-1,0	-1,3	-0,8	-1,2
45°	-1,5	-2,4	-1,3	-2,0	-1,4	-2,0	-1,0	-1,3	-0,9	-1,2
60°	-1,2	-2,0	-1,2	-2,0	-1,2	-2,0	-1,0	-1,3	-0,7	-1,2
75°	-1,2	-2,0	-1,2	-2,0	-1,2	-2,0	-1,0	-1,3	-0,5	

Since the areas of zone **F_{up},F_{low} and G** is between $1m^2$ and $10m^2$,the external pressure coefficient Cpe for those zone is calculated as follow using the following formula.

$$Cpe=Cpe,1-(Cpe,1-Cpe,10)\log 10A \quad \text{for } 1m2<A<10m2$$

Zone F_{up}, $Cpe=Cpe,1-(Cpe,1-Cpe,10)\log A$ =-2.6-(-2.6-(-2.1)) log2.256= -2.43
Zone Flow, $Cpe=Cpe,1-(Cpe,1-Cpe,10)\log A$ =-2.4-(-2.4-(-2.1)) log2.256= -2.29
Zone G, $Cpe=Cpe,1-(Cpe,1-Cpe,10)\log A$ =-2-(-2-(-1.8)) log4.531= -1.87

Table 2. 4 Cpe for mono pitch roofs for 90^0 wind direction and 6.74^0 pitched angle

A	Cpe,10	Fup	Flow	G	H	I
5^0	Cpe,10-	-2.1	-2.1	-1.8	-0.6	-0.5
	Cpe, 10+	0	0	0	0	0
6.74^0	Cpe,10-	-2.152	-2.013	-1.817	-0.635	-0.535
	Cpe, 10+	0	0	0	0	0

15^0	Cpe,10-	-2.4	-1.6	-1.9	-0.8	-0.7
	Cpe, 10+	0	0	0	0	0

A	Cpe,1	Fup	Flow	G	H	I
5^0	Cpe,1-	-2.6	-2.4	-2	-1.2	-0.5
	Cpe, 1+	0	0	0	0	0
6.74^0	Cpe,1-	-2.652	-2.4	-2.087	-1.2	-0.621
	Cpe, 1+	0	0	0	0	0
15^0	Cpe,1-	-2.9	-2.4	-2.5	-1.2	-1.2
	Cpe, 1+	0	0	0	0	0

Therefore, the values of Cpe for those divided zones area between 1m2 and 10m2 can be calculated as follow using,$Cpe = Cpe,1 - (Cpe,1 - Cpe,10)\log A$

zone	Cpe 10,+	Cpe 1,+	A	logA	Cpe +
Fup	0	0	1.0084	0.003633	0
Flow	0	0	1.0084	0.003633	0
G	0	0	2.0155	0.304383	0
H	0	0	16.129	1.207607	0
I	0	0	20.161	1.304512	0

zone	Cpe 10-	Cpe 1 -	A	logA	Cpe-
Fup	-2.152	-2.652	1.0084	0.003633	-2.65
Flow	-2.013	-2.4	1.0084	0.003633	-2.398
G	-1.817	-2.087	2.0155	0.304383	-2.005
H	-0.635	-1.2	16.129	1.207607	-0.518
I	-0.535	-0.621	20.161	1.304512	-0.509

Table 2. 5 Net wind pressure result for mono pitched roof for 90 degree wind direction

Wnet= qp(Z)*(Cpe-Cpi) =0.456 *(Cpe-Cpi) KN/m^2

Zone	**F_{up}**	**F_{low}**	**G**	**H**	**I**
Area,m^2	1.0084	1.0084	2.0155	16.129	20.161

Cpe,-	-2.65	-2.398	-2.005	-0.518	-0.509
Cpe,+	0	0	0	0	0
Cpi,+	0.2	0.2	0.2	0.2	0.2
Cpi,-	-0.3	-0.3	-0.3	-0.3	-0.3
Cpe(-) - Cpi,+	--2.85	-2.598	-2.205	-0.718	-0.709
Cpe(-) - Cpi,-	-2.35	-2.098	-1.705	-0.218	-0.209
Cpe(+) - Cpi,+	-0.2	-0.2	-0.2	-0.2	-0.2
Cpe(+) - Cpi,-	0.3	0.3	0.3	0.3	0.3
Wnet	**-1.23**	-1.185	-1.005	-0.327	-0.323
Wnet	-1.072	-0.956	-0.777	-0.099	-0.095
Wnet	-0.0912	-0.0912	-0.0912	-0.0912	-0.0912
Wnet	**0.1368**	**0.1368**	**0.1368**	**0.1368**	**0.1368**

For 90^0 wind direction maximum net positive pressure occurs at all surfaces **Wnet,+ = +0.1368 KN/m2** and the maximum net negative pressure occurs at zone **Fup, Wnet,-= -1.23KN/m2.**There for the net design wind pressure for mono pitched roof is

Wnet= +0.1368 KN/m^2 Pressure **Wnet= -1.23KN/m^2 Suction**

This indicates the 90^0 is critical and governing wind direction for design.

2.3 Analysis and design of lattice purlin

Purlins are beams used on trusses to support the sloping roof system between the adjacent trusses. RHS, Channels, angle sections, and cold formed C- or Z-sections are widely used as purlins. They are placed in an inclined position over the main rafters of the trusses. To avoid bending in the top chords of roof trusses, it is theoretically desirable to place purlins only at panel points.

2.3.1 Purlin loading

I. Wind load on purlin

The wind load on purlin is the wind that is transferred from the roof cover; already calculated,

Maximum wind pressure = +0.1368 KN/m^2

Maximum wind Suction pressure = -1.23KN/m^2

II. Dead load on purlin

In purling the dead load arises from the weight of EGA sheet and the self-weight of purlin itself.

Selection EGA sheet

The EGA sheet is the top cover of the roof and is selected from kality metal products factory catalogue. Let we choose EGA 300 corrugated sheet as roof cover in our building.

Self-Weight of EGA sheet

The self-weight of EGA-300 sheet of thickness 0.8mm is 6.28kg/m=0.0628KN/m to determine the dead load per area of the weight should be divided by the width of the sheet=0.823m

Unit weight=weight*g/ (A) =6.28*10/ (800) =78.5 KN/m3

$$Dead\ load = \frac{0.0628}{width\ of\ sheet} = \frac{0.0628}{0.823} = 0.763KN/m2$$

I. Imposed load on purlin

In ES EN 1991-1.1:2015 table 6.9 roofs are categorized according to their accessibility into three categories and this are roofs not accessible except for normal maintenance and repair (category H), roofs accessible with occupancy according to categories A to D (category I) and roofs accessible for special services, such as helicopter landing areas (category K). In our project roof type is assumed to be category H. Imposed loads for roofs of category H is within the range 0.00 KN/m^2 to 1.0 KN/m^2 for qk and Qk may be selected within the range 0.9 KN to 1.5 KN (ES EN 1991-1.1:2015 Table 6.10).

The recommended values are: qk= 0.4KN/m^2 and Qk= 1.0 KN is taken.

Load Combination for purlin spacing determination

The purlin spacing and carrying capacity is decided based consideration of DL,LL,WL and their critical combination from deflection/strength requirement as follow.

Combination I: Positive Pressure.

- pitch angle of roof α=6.74^0
- W_L= +0.1368 KN/m2, perpendicular to the roof
- To make it a vertical load, W_L= +0.1368 KN/m2*Cos6.74^0 = **0.1359KN/m^2**

Combination II: Negative Pressure.

W_L= -1.3789KN/m2, perpendicular to the roof to make it a vertical load.

W_L= -1.3789KN/m2 * Cos6.74^0= **-1.3694KN/m^2**

Combination III: WL, LL, DL coming together

$$\text{Max}\begin{cases} Ed2 = 1.35Gk + \varphi o * 1.5Qk + 1.5Qk, wind \\ Ed1 = 1.35Gk + 1.5Qk + \varphi * 1.5Qk, wind \end{cases}$$

For roof design φ_o and φ are taken to be 0

$$\text{Max}\begin{cases} Ed2 = 1.35 * 0.763 + 0 * 1.5 * 0.4 + 1.5 * 0.1359 = 1.234KN/m2 \text{.......for } positive \\ Ed1 = 1.35 * 0.763 + 1.5 * 0.4 + 0 * 1.5 * 0.1359 = \mathbf{1.63KN/m2} \text{.........for } positive \end{cases}$$

$$\text{Max}\begin{cases} Ed2 = 1.35 * 0.763 + 0 * 1.5 * 0.4 + 1.5 * (-1.3694) = -1.024KN/m2.. \text{for } negative \\ Ed1 = 1.35 * 0.763 + 1.5 * 0.4 + 0 * 1.5 * (-1.3694) = 1.63KN/m2 \text{...for } negative \end{cases}$$

Among those load **-1.729KN/m²** for negative and **0.7KN/m²** for positive pressure design is taken to determine purlin spacing using interpolation from kaliti manual EGA-300 for t=0.8mm.

Purlin spacing for negative pressure

2.17= 1.5m

1.729= x

1.59= 1.75m

Therefore, the spacing of purlin is **1.73m** for negative pressure.

Purlin spacing for positive pressure

0.65= 2.75m

0.7= ?

0.78= 2.5m

The purlin spacing is **2.65m** for positive pressure

Therefore, the governing purlin spacing is = min $\begin{cases}1.69m\\2.65m\end{cases}$ $\ldots.. take\ 1.69m\ for\ our\ design$

The spacing of purlin in the mono pich roof is 1.69m.

Table 2.6 The roof cover material specification from kality steel manual

Roof cover parameter	Values
EGA sheet type	EGA-300
Purling spacing	1.69m
Thickness	0.8mm
Width of EGA sheet	0.823m
Area	$800mm^2$
Weight	6.28kg/m
Unit weight	$78.5KN/m^3$

Carrying Capacity	0.703KN/m^2
Moment of inertia, I	108533mm^4
Section modulus, Z	3138mm^3

Taking a maximum wind surface load of -1.729KN/m^2 and purling spacing of 1.69m, the load carrying capacity of EGA-300 having thickness of 0.8mm is 0.703KN/m^2 is taken which is greater than the maximum wind surface load on our roof.

Wind Load

Wind load acts perpendicular to the ridge, but Live and Dead Load are acted in the direction of gravity.in our case wind loads are acted at an angle of 6.74^0 from the normal of rafter.

Suction pressure= -1.3694KN/m2*1.73m= -2.3691 KN/m

Positive pressure= +0.1368 KN/m2*1.73m= 0.236KN/m

Self-weight of EGA=0.0628KN/m^2*purlin spacing =0.0628 KN/m2*1.73m =0.109KN/m

UDL of live load =qk*C/C distance of successive purlin =0.4*1.73KN/m=0.692KN/m

Load perpendicular to the rafter

Dead load (DL) = 0.109KN/m *Cos6.74^0 = 0.1081KN/m

Live load (UDL) =0.692KN/m * Cos6.74^0= 0.687KN/m (uniformly distributed load)

Live load (point load) = 1KN * Cos6.74^0= 0.99KN (concentrated Load)

2.3.2 Load combination for mono-pitched roof According to ES EN 1990:2015 section 6.4.3.1(1) P

The combination of effects of these actions should be based on the design value of the leading variable action and the design combination values of accompanying variable actions. Therefore various combination of ultimate limit state for persistent and transient design situations are used for design.

Values of ψ factor

Recommended value of ψ factored for the most common actions is obtained from ES EN 1990:2015 table A1.1 as follows: -

For actions of imposed loads category H: roofs $\psi_0 = 0$

For actions of wind loads on building $\psi_0 = 0.6$

Design value of actions in persistent and transient design situation

The load combination for persistent and transient design situations is given in ES EN 1990:2015 A1.2 (B) expression 6.10 as follows: -

$Ed=\gamma Gj,+ \gamma Q1Qk1+\gamma Qi\psi 0,iQk,i$

Where, γGj, is the factor of safety for dead load (recommended 1.35)

$Gkjsup$ is dead load

$\gamma Q1$ is the factor of safety of the leading variable (recommended 1.5)

$Qk1$ is leading variable imposed load

$Qk,$ is accompanying variable imposed load

$\psi_0,$ is multiplier for accompanying variable imposed load

Case 1: Only Dead Load and Imposed Load

Combination 1, Ed=1.35$Gkjsup$+1.5$Qk1$($distributed$)

=1.35*0.1081+1.5*0.687= 1.177KN/m

Combination 2, Ed=1.35$Gkjsup$+1.5$Qk1$($concentrated$)

=1.35*0.1081+1.5*1KN = 0.1459KN/m+1.5KN

Case 2: imposed load leading variable and wind load accompanying variable

Combination 3, Ed=1.35$Gkjsup$+1.5$Qk1$($distributed$)+1.5∗0.6Qk,i($compression$)

=1.35*0.1081+1.5*0.687+1.5*0.6*0.236= 1.3888KN/m

Combination 4, Ed=1.35$Gkjsup$+1.5$Qk1$($concentrated$)+1.5∗0.6Qk,i($compression$)

=1.35*0.1081+1.5*1KN+1.5*0.6*0.236= 0.3584KN/m +1.5KN

Combination 5, Ed=1.35$Gkjsup$+1.5$Qk1$($distributed$)+1.5∗0.6Qk,i($suction$)

=1.35*0.1081+1.5*0.687+1.5*0.6*(-2.3691) = -0.9558KN/m

Combination 6, Ed=1.35$Gkjsup$+1.5$Qk1$($concentrated$)+1.5∗0.6Qk,i($suction$)

=1.35*0.1081+1.5*1KN+1.5*0.6*(-2.3691) = -1.9863KN/m +1.5KN

Case 3: wind load leading variable and imposed load accompanying variable

Commination 7, Ed**=1.35**$Gkjsup$**+1.5**$Qk1$**(**$compression$**)+1.5∗0***Qk,i**(**$distributed$**)**

=1.35*0.1081+1.5*0.236 + 0 = 0.499KN/m

Combination 8, Ed**=1.35**$Gkjsup$**+1.5**$Qk1$**(**$compression$**)+1.5∗0***Qk,i**(**$concentrated$**)**

=1.35*0.1081+1.5*0.236+0 = 0.499KN/m

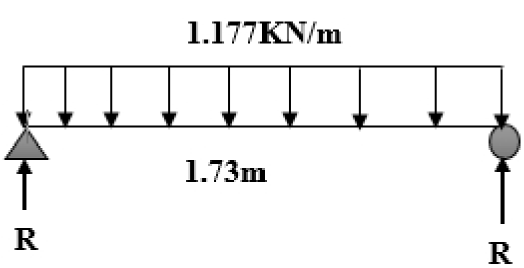
Combination 9, Ed**=1**$Gkjsup$**+1.5**$Qk1$**(**$suction$**)+1.5∗0*** Qk,i**(**$distributed$**)**

=1*0.1081+1.5*(-2.3691) + 0 = -3.446KN/m

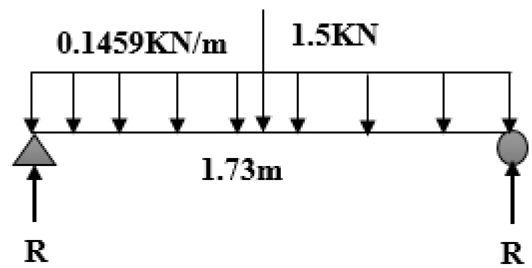
Combination 10, Ed**=1**$Gkjsup$**+1.5**$Qk1$**(**$suction$**)+1.5∗0***Qk,i**(**$concentrated$**)**

=1*0.1081+1.5*(-2.3691)+0=-3.446KN/m

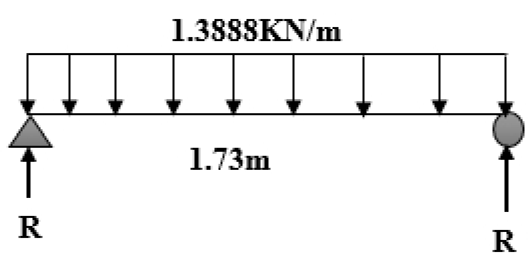
Loading for various combinations



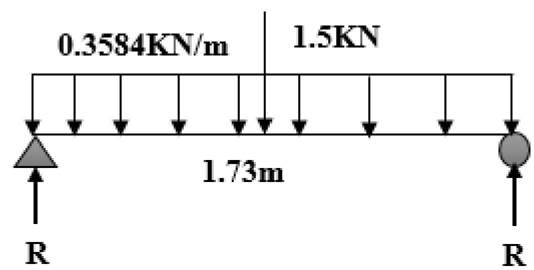
Combination 1



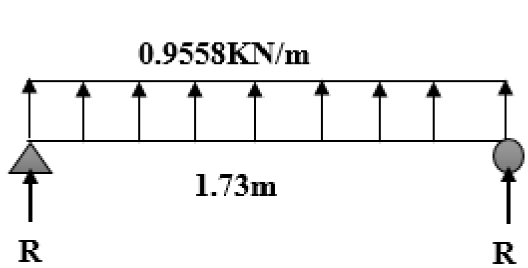
Combination 2



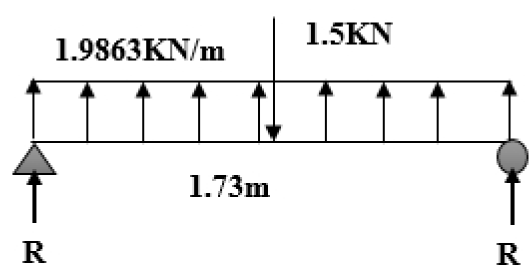
Combination 3



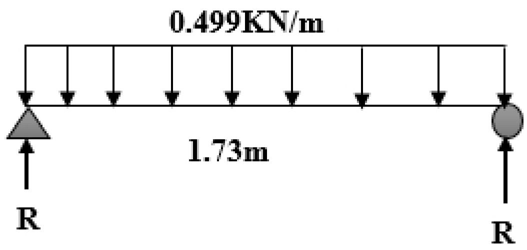
Combination 4



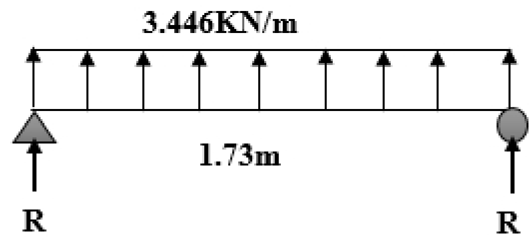
Combination 5



Combination 6



Combination 7,8



Combination 9,10

Figure 2. 4 Loadings for various combinations

Table 2. 7 Final moment and reaction values of mono pitched roof purline

Combination	Ed	M_{max}	R
1	1.177KN/m	$Wl^2/8$=0.441KNm	Wl/2=1.018KN
2	0.1459KN/m+1.5KN	$Wl^2/8$+wl/2=1.352KNm	Wl/2+w/2=0.876KN
3	1.3888KN/m	$Wl^2/8$=0.5196KNm	Wl/2=1.21KN
4	0.3584KN/m +1.5KN	$Wl^2/8$+wl/2=1.432KNm	Wl/2+w/2=1.06KN
5	-0.9558KN/m	$Wl^2/8$= -0.3576KNm	Wl/2= -0.827KN
6	-1.9863KN/m +1.5KN	$Wl^2/8$+wl/2= 0.654KNm	Wl/2+w/2= -0.738KN
7	0.499KN/m	$Wl^2/8$=0.1867KNm	Wl/2=0.432KN
8	0.499KN/m	$Wl^2/8$=0.1867KNm	Wl/2=0.432KN
9	-3.446KN/m	$Wl^2/8$= -1.2892KNm	Wl/2= -2.984KN
10	-3.446KN/m	$Wl^2/8$= -1.2892KNm	Wl/2= -2.984KN

N.B. W is UDL load, w is point load l is purlin spacing

From all the above combination the following maximum critical value is considered for design.

- ✓ Maximum positive reaction= +1.21KN, Combination 3
- ✓ Maximum negative reaction= -2.984KN, Combination 9
- ✓ Maximum positive moment= +1.432KNm, Combination 4
- ✓ Maximum negative moment= -1.2892KNm, Combination 9

2.3.3 Serviceability limit state Verification (deflection requirement)

Check for roof cover section capacity

From the above select the maximum reaction and moment

According to ES EN 1993 part1-2 Hot- rolled products of structural steel with grade of Fe-460 is taken. Hence fy=460N/mm2 and fu=550N/mm2

Section modulus for Z=M/Fy= 1.318KNm/460N/mm^2=2865.2mm^3

For the EGA sheet 300 Section modulus for Z=3138mm^3

Since 2865mm^3<3138mm^3… Hence ok, the roof cover section can carry maximum moment

Check for Moment Capacity

M_{plrd} =wpl*fy/ϒmo =3138*460/1=1.44KNm

1.44kN.m > 1.318kN.m……………………..Hence OK

Check for Deflection

The capacity of the cover for the deflection =4.88mm for EGA sheet 300

Deflection (D) =5wl^4/ (384EI) +PL^3/48EI………………..from basic structural analysis

D =5*3.591*1.44/ (384*210000*108533) +1.51*1.4^3/ (48*210000*108533)

D =0mm<4.88mm……………..OK

Therefore, the EGA sheet that we select can resist the applied load safely.

Check for adequacy of section

According to ES,EN 1993 part1-2 Hot- rolled products of structural steel with grade of Fe-460 is taken. Hence fy=460N/mm^2 and fu=550N/mm^2

The section used at kaliti profile=50*30*3

Section property

Use RHS 50x30x3 H=50mm B=30mm t= 3mm

Section classification according to ES,EN1993 design of steel structure maximum width to thickness ratio for compression part

C=b-3t =30-3(3) =21mm

d=h-3t=50-3(3)=41

ε=√235 /fy=0.71 for fy=460

For web=d/tw=41/3=13.67<=33*0.71 23.43 class 1

For flange=C/tf=21/3=7<=72*0.71=51.12 class 1

Therefore the section is class-I

II Mpl,Rd **flexural resistance**

= wpl*fy/γmo

6.57cm^3*460N/mm^2/1.0=3.02KNm

Mmax=wl^2/8 =3.229*2.71^2/8=2.96KNm Mpl,Rd>Mmax ok

III.shear resistance

Plastic shear resistance

Vpl ,Rd=Av(fy/√3) /γmo

A=4.34cm^2

Av=h*tw=50*3*1.092=163.8mm^2

VPL,Rd=163.8*(460√3)/1=43.5KN ,Vmax=4.375KN OK

Shear buckling

N_{ED}/Nb,rd<=1

Nb,rd=xAfy /γm1 for class 1,2,3

X=1/Φ+√(Φ^2-λ^2) but x<=1.0

Where x=0.5(1+x(λ)+-0.2)+λ^2)

From table 6.2ES EN 1993 PAGE 70, for hollowsection hot finished and buckling about any axis s460 the buckling curve is ″ao″

For buckling curve ″ao″the imperfection factor αis 0.13 (table 6.1ES EN page 64)

$\lambda = \frac{\sqrt{A*fy}}{Ncr} = \frac{\sqrt{4340*460}}{468.23} = 3.02$

Φ=0.5(1+0.13(3.02-0.2)+3.02^2)=5.24

$X = \frac{1}{5.24+\sqrt{5.24-3.24}} = 0.150$

$Nrbd = \frac{0.150*4340*460}{1.0} = 209.62KN$

Nrbd>NED (209.62KN>4.375KN) ………..OK

Deflection (serviceability) check

According to ES EN 1995 table 5.1, the deflection values for a general roof are

L=2.71m

*Δ*max=2710/200=13.55mm

Δ(Lmax)=27710/250=10.84mm

Live load case

*Δ*max=2710/200=13.55mm

For DL+LL case

Δ(Lmax)=27710/250=10.84mm

Imposed load deflection

γll=(5×wL^2)/(384×EIy)

=(5×3.46^2*2.71^2)/(2.1*10^5*5.94)=2.65*10^{5}mm

Therefore the deflection is less than the maximum limit, it is safe

2.3.4 Procedure for analysis of lattice purlin

- The analysis is done for both the positive and negative reaction cases.
- The critical reactions calculated above are applied as uniformly distributed load on the purlin.

- Then, the uniformly distributed load is turned to concentrated load at center by multiplying it by the length of the purlin.
- The concentrated load on each node is then determined by dividing the concentrated load at center by the number of nodes.
- The effect of self-weight of the purlin is included in the SAP analysis by defining the section.
- We use a lattice purlin having a length, L=4.5m

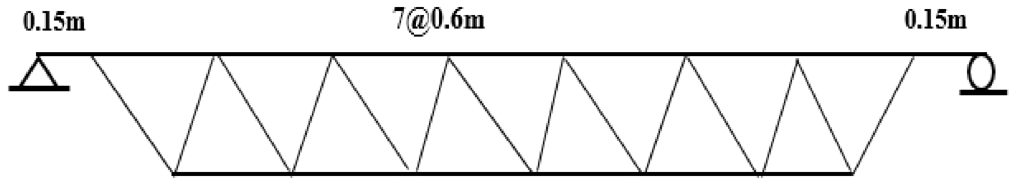
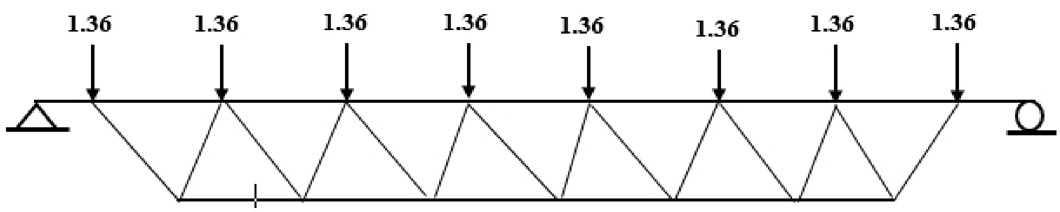


Figure 2.4 lattice Purlin Lay out

Since the purlin carries the load from adjacent EGA sheet section, the reaction from the critical loading should be multiplied by two.

A) Positive reaction, R(+ve)

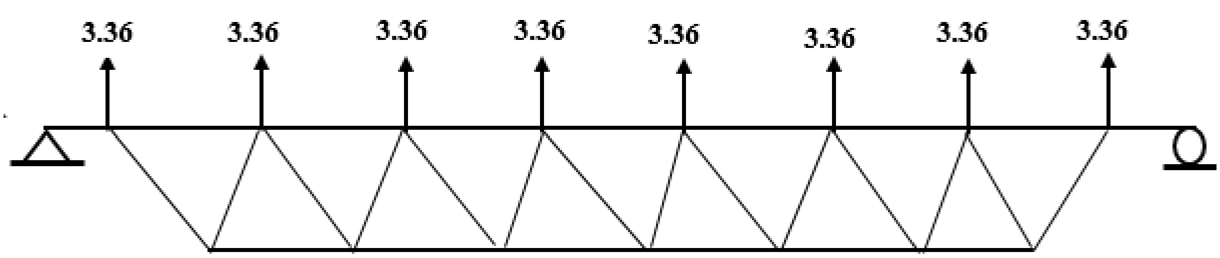
- R (+ve) = 1.21KN/m
- 2R(+ve)=2.42 KN/m
- L=4.5m
- R concentrated=2.42 KN/m*4.5= 10.89KN
- R on each node=R concentrate/no of node = 10.89 KN / 8= 1.36 KN



7Figure 2. 5 Lattice Purlin Layout of mono pitched roof

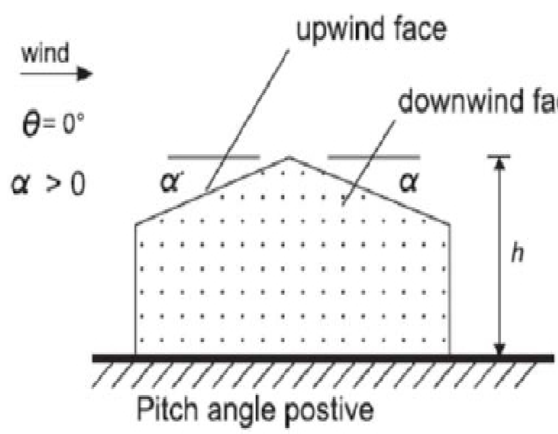
B) Negative reaction, R(-ve)

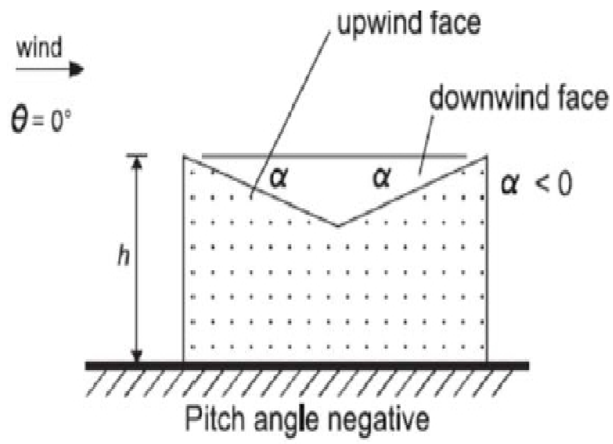
- R (-ve) = -2.984KN/m
- 2R(-ve)=-5.97KN/m
- Truss spacing,L=4.5m
- R concentrated at the center= -5.97KN/m*4.5m = -26.87KN
- R on each node=R concentrated/No of node = -26.87 KN/8 = -3.36KN



8Figure 2. 6 Duo-pitch Roof Analysis and Design

A duo-pitch roof is a two sided positive or negative sloped/pitched roof with reference to the ridge. in our project the pitched angle is positive.





2.4 Duo-pitch Roof Analysis for 0^0 wind direction

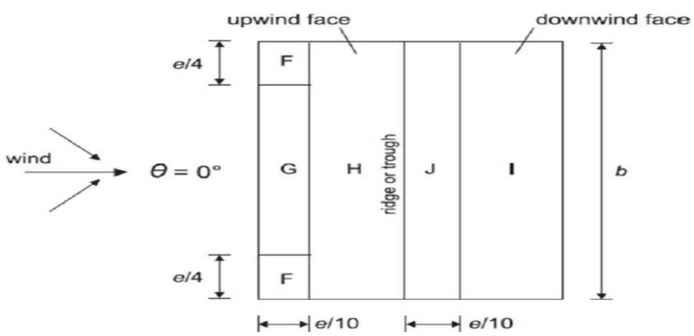
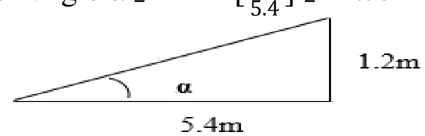


Figure 2. 7 duo-pitch roof layout for 0^0 Wind Directi

- ✓ Height of roof h=19.95m
- ✓ Pitch height = 1.2m
- ✓ Roof plan=10.8m*6.75m
- ✓ Width along cross wind b=6.75m,
- ✓ Width parallel to wind=10.8m
- ✓ Pitch angle α = $\tan^{-1}\left[\frac{1.2}{5.4}\right]$ = 12.53^0



- ✓ e =b or 2h whichever is smaller

b=6.75m & 2h = 39.9m, Therefore e=6.75m

e/4=6.75/4=1.688m

- Height of roof h=16.5m
- Pitch height = 0.75m
- Width along cross wind, b=6.35m,
- Width parallel to wind=21.31m
- Pitch angle α=tan −1 [0.75 6.35]= 6.74^0
- e =b or 2h whichever is smaller b=6.35m & 2h = 33m, Therefore e=6.35m e/4=6.35/4=1.588m e/10=6.35/10=0.635m

2.4.1 Wind Parameter analysis for duo-pitch roof

I. Fundamental Wind Velocity, Vb,0

With similar concept as mono pitched roof analysis, fundamental wind velocity, *V*b,0 is taken as **22m/sec.**

II. Basic Wind Velocity,Vb

The basic wind velocity Vb, is determined according to ES EN 1991-1-4:2015 expression 4.1.

$$Vb=Cdir*Cseason*Vb,=\mathbf{1*1*22}\,\boldsymbol{m/sec}=\mathbf{22}\,\boldsymbol{m/sec}$$

III. Mean Wind Velocity,Vm(z)

The mean wind velocity Vm(z) at a height Z above the terrain is determined using ES EN 1991-1-4:2015 section 4.3.1(1) P, expression (4.3)

$$Vm(z)=Cr(z)\times Co(z)\times Vb$$

According to ES EN 1991-1-4:2015 sec-4.3.1 the orography factor can be taken as 1 unless specified. And the roughness factor is determined as follow

$\mathrm{Cr}(z)=Kr*\ln(\frac{z}{zo}), for\ Zmin\leq Z\leq Zmax$ or

$(z)=(Zmin)\ for\ Z\leq Zmin$

Where, Kr is the terrain factor depending on roughness length Zo.

The terrain factor Kr should be determined using ES EN 1991-1-4:2015 expression (4.5).

$$Kr=0.19(\frac{Z0}{Zo,II})^{0.07}$$

We have categorized Adama under terrain category IV, According to ES EN 1991-1-4:2015, Table 4.1, values of roughness length (Zo) and minimum height (Zmin) are 1m and 10m respectively.

The total height of our building from the ground level to roof top is h=Z=19.95m

Zo = 1m Zo, II = 0.05m

Zmin = 10m Z = 19.95m Zmax = 200m

$Zmin \leq Z \leq Z_{max}$ $10m \leq 19.95\ m \leq 200m$

$\mathrm{Kr} = 0.19*(\frac{1}{0.05})^{0.07} =$ **0.234**

$\mathrm{Cr(z)} = k_r*\ln(\frac{Z}{Z0}) = 0.234*\ln(\frac{19.95}{1}) =$ **0.70**

Therefore, the Mean Wind Velocity Vm(Z) = Cr(Z) *Co (Z) *Vb = 0.7*1*22 **=15.4m/s**

IV. Wind Turbulence

According to ES EN 1991-1-4:2015, section 4.4(11) is the turbulence intensity lv(z) at height Z is defined as the standard deviation, σ_v of the turbulence divided by mean wind velocity. And it should be determined using ES EN 1991-1-4:2015 expression (4.7).

$lv(z)=\frac{\sigma v}{Vm(z)}=\frac{Kr}{Co(z)*\ln(\frac{Zo}{Z0,II})}$ for Zmin < Z <Zmax

$lv(z)= lv(Zmin)$ for Z < Zmin

Taking the recommended value turbulence factor K1 as 1,the Standard deviation is given by ES EN 1991-1-4:2015 expression 4.6.

$$\sigma v=Kr*Vb*K_l=0.234\times 22m/sec\times 1=5.148$$

Therefore, Wind Turbulence is $lv(19.95m)=\frac{\sigma v}{Vm(19.95)}=\frac{5.148}{15.4}=$ **0.334**

V. Peak Velocity Pressure, qp(z)

The peak velocity pressure qp(z) at height z, is determined by ES EN 1991-1-4:2015 Expression (4.8)

$$qp(z) = [1+7*lv(z)]*\frac{1}{2}*\rho*V_m^2(z) \quad \text{or} \quad qp(z) = Ce(z)*q_b$$

Where, ρ is the density of air=1.25kg/m^3

Ce(z) is exposure factor determined by ES EN 1991-1-4:2015

qb is the basic velocity pressure, determined from ES EN 1991-1-4:2015 expression 4.9.

$$qb = \frac{1}{2}*\rho*Vb^2 = 1/2*1.25*22^2 = \mathbf{302.5kg/ms^2}$$

Therefore, the Peak Velocity Pressure, qp(z) is determined as,

$$qp(z) = [1+7*lv(z)]*\frac{1}{2}*\rho*V_m^2(z)$$

qp (19.95m) = (1+7*0.334*)*1/2*1.25*15.4^2 = 494.775N/m^2 = **0.495KN/m^2**

2.4.2 Wind Pressure on Surface

Cpe for duo pitch roofs for 0^0 wind direction and 12.53^0 pitched angle						
α	Cpe	F	G	H	I	J
5^0	Cpe, 10+	0	0	0	-0.6	0.2
	Cpe,10-	-1.7	-1.2	-0.6	-0.6	-0.6
	Cpe, 1+	0	0	0	-0.6	0.2

A. External wind pressure,We

The pressure acting on the external surfaces of the roof is given according to ES EN 1991-1-4:2015 expression 5.1. $We=(Ze)*Cpe$

Where, $qp\ (Ze)$ is the peak velocity pressure,

Ze is the reference height for the external pressure, and

Cpe(Ze) is the pressure coefficient for the external pressure

The external pressure coefficients, Cpe for buildings and parts of buildings depend on the size of the loaded area A, and given with the following formula and (ES EN 1991-1-4:2015 section 7.2.1.

$Cpe=Cpe,1$ for A $\leq 1m^2$

$Cpe=Cpe,1-(Cpe,1-Cpe,10)\log_{10}A$ for $1m^2<A<10m^2$

$Cpe=Cpe,10$ for $A\geq 10m^2$

Where $Cpe, 1$ is external pressure coefficient for loaded area A of 1 m^2

$Cpe, 10$ is external pressure coefficient for loaded area A of 10 m^2, and

A is loaded area of roof zones

Since roof pitch angle is between 5^0 and 15^0 which is 12.53^0 we should calculate the value of Cpe for pitch angle 12.53^0 using interpolation.

	Cpe, 1-	-2.5	-2	-1.2	-0.6	-0.6
12.53^0	Cpe, 10+	0.150602	0.150602	0.150602	-0.14819277	0.049398
	Cpe,10-	-1.09759	-0.8988	-0.3741	-0.44939759	-0.9012
	Cpe, 1+	0.150602	0.150602	0.150602	-0.14819277	0.049398
	Cpe, 1-	-2.12349	-1.62349	-0.52229	-0.44939759	-1.27771
15^0	Cpe, 10+	0.2	0.2	0.2	0	0
	Cpe,10-	-0.9	-0.8	-0.3	-0.4	-1
	Cpe, 1+	0.2	0.2	0.2	0	0
	Cpe, 1-	-2	-1.5	-0.3	-0.4	-1.5

Table 2. 8 wind pressure on surfaces for 90^0 wind direction and 12.53^0 pitch angle

Cpe for duo-pitch roofs for 90 degree wind direction and 12.53 degree pitch angle

pitch angle	Cpe	F	G	H	I
5	Cpe,10-	-1.6	-1.3	-0.7	-0.6
	Cpe, 1-	-2.2	-2	-1.2	-0.6
12.53	Cpe,10-	-1.3741	-1.3	-0.6247	-0.5246988
	Cpe, 1-	-2.0494	-2	-1.2	-0.5246988
15	Cpe,10-	-1.3	-1.3	-0.6	-0.5
	Cpe, 1-	-2	-2	-1.2	-0.5

zone	Cpe 10,+	Cpe 1,+	A	logA	Cpe +
F	0.1506	0.1506	1.139	0.056523724	0.1506
G	0.1506	0.1506	2.277	0.357363031	0.1506
J	0.0494	0.0494	4.556	0.658583715	0.0494

zone	Cpe 10-	Cpe 1 -	A	logA	Cpe-
F	-1.0976	-2.1235	1.139	0.056523724	-2.0655
G	-0.8988	-1.6235	2.277	0.357363031	-1.3645
J	-0.9012	-1.2777	4.556	0.658583715	-1.0298

Since the areas of zone **F**, **G** and **J** for wind direction 0^0 are between $1m^2$ and $10m^2$,Cpe for those zone is calculated as follow using the following formula.

$$Cpe=Cpe,1-(Cpe,1-Cpe,10)\log 10A \quad \text{for } 1m^2<A<10m^2$$

Zone F_{up}, $Cpe,+=Cpe,1-(Cpe,1-Cpe,10)\log A$ =0.1506-(0.1506-(0.1506)) log1.139= 0.15
Zone F_{low}, $Cpe,-=Cpe,1-(Cpe,1-Cpe,10)\log A$ =-2.1235-(-2.1235-(-1.0976)) log1.139= -2.07
Zone G, $Cpe,+=Cpe,1-(Cpe,1-Cpe,10)\log A$ =0.1506-(0.1506-(0.1506)) log2.277= 0.15
Zone G, $Cpe,-=Cpe,1-(Cpe,1-Cpe,10)\log A$ = -1.6235-(-1.6235-(-0.8988)) log2.277= -1.36
Zone J, $Cpe,+=Cpe,1-(Cpe,1-Cpe,10)\log A$ =0.0494-(0.0494-(0.0494)) log4.556= 0.05
Zone J, $Cpe,+=Cpe,1-(Cpe,1-Cpe,10)\log A$ =-1.277-(-1.277-(-0.9012)) log4.556= -1.03

B. Internal Wind Pressure, Wi

The pressure acting on the internal surfaces of the roof is given by $\mathbf{Wi=(Zi)*Cpi}$

The internal pressure coefficient, Cpi is assumed the following values considering the two extreme cases as Cpi= +0.2 and Cpi= -0.3 is taken.

- ✓ Close to the openings at the leeward and parallel side (internal pressure), μ =0 and Cpi = **+0.2**
- ✓ Close all the openings at the windward direction (internal suction). μ = 1.0 and Cpi = **-0.3**

All area of subdivided zone is greater than $10m^2$,$Cpe=Cpe,10$, Table 7.3a CES 145 and take roof pich angle value as 12.53^0 to find Cpe for 0^0 wind direction on mono-pitch roof.

Wnet=We-Wi=qp(Ze)*Cpe - qp(Zi)*Cpi =qp(Z)(Cpe-Cpi) taking Ze=Zi=Z, depends on values of both Cpe and Cpi can positive or negative pressure coefficients and we have four possible vales of net pressure, among those the maximum positive and negative pressure is taken for design.

Wnet = qp(z)* [Cpe(-) – Cpi(+)]

Wnet = qp(z)* [Cpe(-) – Cpi(-)]

Wnet = qp(z)* [Cpe(+) – Cpi(+)]

Wnet = qp(z)* [Cpe(+) – Cpi(-)]

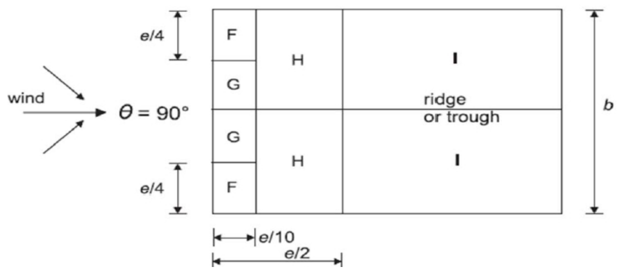
Table 2. 9 Net wind pressure of mono pitch roof for 6.74^0 pitch angle

Wnet= qp(z)*(Cpe-Cpi) =0.495 *(Cpe-Cpi)KN/m^2

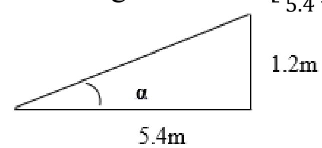
Zone	**F**	**G**	**H**	**I**	**J**
Area,m^2	1.139	2.277	31.894	31.894	4.556
Cpe,-	-2.07	-1.36	-0.4	-0.45	-1.03
Cpe,+	0.15	0.15	0.15	-0.15	0.05
Cpi,+	0.2	0.2	0.2	0.2	0.2
Cpi,-	-0.3	-0.3	-0.3	-0.3	-0.3
Cpe(-) - Cpi,+	-2.27	-1.56	-0.6	-0.65	-1.23
Cpe(-) - Cpi,-	-1.77	-1.06	-0.1	-0.15	-0.73
Cpe(+) - Cpi,+	-0.05	-0.05	-0.05	-0.35	-0.15
Cpe(+) - Cpi,-	0.45	0.45	0.45	0.15	0.35
Wnet	**-1.1237**	-0.7722	-0.297	-0.32175	-0.6089
Wnet	-0.8762	-0.5247	-0.0495	-0.07425	-0.3614
Wnet	-0.0248	-0.0248	-0.02475	-0.17325	-0.0743
Wnet	**0.22275**	0.22275	0.22275	0.07425	0.17325

For 0^0 degree wind direction maximum net positive pressure is Wnet,+= **0.22275KN/m^2** and the maximum net negative pressure is, Wnet,- = **-1.1237KN/m^2**.

Duo pitch Roof Analysis for 90^0 wind direction



- ✓ Height of roof h=19.95m
- ✓ Pitch height = 1.2m
- ✓ Roof plan=10.8m*6.75m
- ✓ Width along cross wind,b=10.8m,
- ✓ Width parallel to wind=6.75m
- ✓ Pitch angle $\alpha = \tan^{-1}[\frac{1.2}{5.4}] = 12.53^0$



- ✓ e =b or 2h whichever is smaller

b=10.8m & 2h = 39.9m, Therefore e=10.8m

e/4=10.8/4=2.7m

F igure 2. 9 Wind Direction $\Theta=90^0$

- Height of roof h=19.95m
- Pitch height = 1.2m
- Roof plan=10.8m*6.75m
- Width along cross wind,b=10.8m,
- Width parallel to wind=6.75m
- Pitch angle $\alpha = \tan -1$ [1.2 5.4] $= 12.53^o$
- e =b or 2h whichever is smaller b=10.8m & 2h = 39.9m, Therefore e=10.8m e/4=10.8/4=2.7m e/10=10.8/10=1.08m

Since the areas of zone **F and G** for wind direction 90^0 are between $1m^2$ and $10m^2$,Cpe for those zone is calculated as follow using the following formula.

$$Cpe = Cpe,1 - (Cpe,1 - Cpe,10) \log 10A \quad \text{for } 1m^2 < A < 10m^2$$

Zone F, $Cpe,+ = Cpe,1 - (Cpe,1 - Cpe,10) \log A$ =0-(0-(0)) log2.916= 0

Zone F, $Cpe,- = Cpe,1 - (Cpe,1 - Cpe,10) \log A$ =-2.049-(-2.049-(-1.374)) log2.916= -1.74

Zone G, $Cpe,+ = Cpe,1 - (Cpe,1 - Cpe,10) \log A$ =0-(0-(0)) log5.832= 0

Zone G, $Cpe,- = Cpe,1 - (Cpe,1 - Cpe,10) \log A$ = -1.62-(-1.62-(-0.9)) log5.832= -1.18

Table 2. 10 Cpe for duo pitch roof for 90^0wind direction &12.53^0 pitched angle

Cpe for duo pitch roof for 90^0wind direction &12.53^0 pitched angle					
A	Cpe	F	G	H	I
5^0	Cpe,10-	-1.6	-1.3	-0.7	-0.6
	Cpe, 1-	-2.2	-2	-1.2	-0.6

12.53^0	Cpe,10-	-1.3741	-1.3	-0.6247	-0.5246988
	Cpe, 1-	-2.0494	-2	-1.2	-0.5246988
15^0	Cpe,10-	-1.3	-1.3	-0.6	-0.5
	Cpe, 1-	-2	-2	-1.2	-0.5

zone	Cpe 10-	Cpe 1 -	A	logA	Cpe-
F	-1.374096	-2.049398	2.916	0.46479	-1.7355
G	-1.3	-2	5.832	0.76582	-1.4639

Wnet=qp(z)*(Cpe-Cpi) =0.495 *(Cpe-Cpi)KN/m^2

Zone	**F**	**G**	**H**	**I**
Area,m^2	2.916	5.832	46.656	14.56
Cpe,-	-1.74	-1.46	-0.63	-0.53
Cpe,+	0	0	0	0
Cpi,+	0.2	0.2	0.2	0.2
Cpi,-	-0.3	-0.3	-0.3	-0.3
Cpe(-) - Cpi,+	-1.94	-1.66	-0.83	-0.73
Cpe(-) - Cpi,-	-1.44	-1.16	-0.33	-0.23
Cpe(+) - Cpi,+	-0.2	-0.2	-0.2	-0.2
Cpe(+) - Cpi,-	0.3	0.3	0.3	0.3
Wnet	**-0.9603**	-0.8217	-0.4109	-0.3614
Wnet	-0.7128	-0.5742	-0.1634	-0.1139
Wnet	-0.099	-0.099	-0.099	-0.099
Wnet	**0.1485**	0.1485	0.1485	0.1485

For 90^0 degree wind direction maximum net positive pressure is Wnet,+= **0.22275KN/m²** and the maximum net negative pressure is ,Wnet,- = **-0.9603KN/m²**.

There for considering the maximum of the two principal wind direction, net design wind pressure are,

Wnet,+= 0.22275KN/m2 Pressure Wnet,- = -1.1237KN/m2Suction

This indicates 90^0 is critical wind direction for suction design and 0^0 is critical for pressure design.

Analysis and design of lattice purlin

Purlins are beams used on trusses to support the sloping roof system between the adjacent trusses. RHS, Channels, angle sections, and cold formed C- or Z-sections are widely used as purlins. They are placed in an inclined position over the main rafters of the trusses.

2.5 Purlin loading

I. Wind load on purlin

Maximum wind pressure = +0.22275KN/m^2

Maximum wind Suction pressure = -1.1237KN/m^2

II. Dead load on purlin

In purling the dead load arises from the weight of EGA sheet and the self-weight of purlin itself.

Selection EGA sheet

The EGA sheet is the top cover of the roof and is selected from kality metal products factory catalogue. Let we choose EGA 300 corrugated sheet as roof cover in our building.

III. Self-Weight of EGA sheet

The self-weight of EGA-300 sheet of thickness 0.8mm is 6.28kg/m=0.0628KN/m to determine the dead load per area of the weight should be divided by the width of the sheet=0.823m

Unit weight=weight*g/ (A) =6.28*10/ (800) =78.5 KN/m3

$$Dead\ load = \frac{0.0628}{width\ of\ sheet} = \frac{0.0628}{0.823} = 0.0763KN/m2$$

IV Imposed load on purlin

Similar to mono pich roof category H roof type is assumed with recommended values imposed loadas

QK=1 KN (concentrated load)

qk=0.4 KN/m^2(Uniformly Distributed Load)

LoadCombination for purlin spacing determination

The purlin spacing and carrying capacity is decided based consideration of DL,LL,WL and their critical combination from deflection/strength requirement as follow.

Combination I: Positive Pressure.

- Since we have a pitch angle of $\alpha=12.53^0$
- W_L= +0.22275KN/m^2, perpendicular to the roof

- To make it a vertical load, W_L= +0.22275KN/m2 *Cos12.53^0 = **0.2174KN/m^2.**

Combination II: Negative Pressure.

W_L= -1.1237KN/m^2, perpendicular to the roof to make it a vertical load.

W_L= -1.1237KN/m^2 *Cos12.53^0 = **-1.097KN/m^2**

Combination 3: DL,LL,WL Coming together

$$\text{Max}\begin{cases} Ed2 = 1.35Gk + \varphi o * 1.5Qk + 1.5Qk, wind \\ Ed1 = 1.35Gk + 1.5Qk + \varphi * 1.5Qk, wind \end{cases}$$

For roof design φ_o and φ are taken to be 0

$$\text{Max}\begin{cases} Ed2 = 1.35 * 0.0763 + 0 * 1.5 * 0.4 + 1.5 * 0.2174 = 0.429KN/m2 \text{.......... for } positive \\ Ed1 = 1.35 * 0.0763 + 1.5 * 0.4 + 0 * 1.5 * 0.2174 = 0.703KN/m2 \text{..... for } positive \end{cases}$$

$$\text{Max}\begin{cases} Ed2 = 1.35 * 0.0763 + 0 * 1.5 * 0.4 + 1.5 * (-1.097) =- 1.542\text{KN/m2} \text{....... for } negative \\ Ed1 = 1.35 * 0.0763 + 1.5 * 0.4 + 0 * 1.5 * (- 1.097) = 0.703KN/m2 \text{.........for } negative \end{cases}$$

To decide purlin spacing **-1.542KN/m^2** is maximum negative pressure and **0.7KN/m^2** maximum positive pressure is considered. Using this load and thickness of EGA, the purlin spacing is determined by using interpolation from the previous kaliti manual.

2.17= 1.5m

1.54= ?

1.59= 1.75m

There for, the spacing of purlin is **1.73m** for positive pressure

Purlin spacing for positive pressure

0.65= 2.75m

0.7= ?

0.78= 2.5m

The purlin spacing is **2.65m** for positive pressure Therefore, the governing purlin spacing is = min $\begin{cases} 1.73m \\ 2.65m \end{cases} \text{.....} take\ 1.73m\ for\ our\ design$

The spacing of purlin in the duo pitch roof is 1.73m.

Table 1. 16Table 2.11 Roof cover material property

Roof cover parameter	Values
EGA sheet type	EGA 300
Purling spacing	1.73m
Thickness	0.8mm
Width of EGA sheet	0.823m
Area	800mm^2
Weight	6.28kg/m
Unit weight	78.5KN/m3
Carrying Capacity	1.63 KN/m2
Moment of inertia, I	108533mm^4
Section modulus, Z	3138mm^3

Taking a maximum wind surface load of **-1.097KN/m²** and purling spacing of 1.73m, the load carrying capacity of EGA 300 having thickness of 0.8mm is 1.63KN/m² .which is greater than the maximum wind surface load on our roof.

Wind Load

Suction pressure= -1.1237KN/m² *1.73m=-1.944KN/m

Positive pressure= 0.22275 KN/m²*1.73m=0.385KN/m

Wind load acts perpendicular to the ridge, but Live and Dead Load are acted in the direction of gravity.in our case wind loads are acted at an angle of 12.53^0 from the normal of rafter.

Self-weight of EGA=0.0628KN/m²*purlin spacing =0.0628 KN/m2*1.73m =0.109KN/m

UDL of live load =qk*c/c distance of successive purlin =0.4*1.73KN/m=0.692KN/m

Load transferred perpendicular to the rafter

- ✓ Dead load (DL) = 0.109KN/m *Cos12.53^0 = 0.0164KN/m
- ✓ Live load (UDL) =0.692KN/m * Cos12.53^0 = 0.676KN/m (uniformly distributed load)
- ✓ Live load (point load) = 1KN * Cos12.53^0 = 0.98KN (concentrated Load)
- ✓ Suction pressure= -1.1237KN/m² *1.73m= -1.944KN/m
- ✓ Positive pressure= 0.22275 KN/m²*1.73m = 0.385KN/m

2.5.1 Load combination for duo-pitch roof According to ES EN 1990:2015 section 6.4.3.1(1) P

Case 1: Only Dead Load and Imposed Load

Combination 1, $Ed=1.35Gkjsup+1.5Qk1(distributed)$

=1.35*0.109+1.5*0.692= 1.185KN/m

Combination 2, $Ed=1.35Gkjsup+1.5Qk1(concentrated)$

=1.35*0.109+1.5*1.0 = 0.147KN/m+1.5KN

Case 2: imposed load leading variable and wind load accompanying variable

Combination 3, $Ed=1.35Gkjsup+1.5Qk1(distributed)+1.5*0.6Qk,i(compression)$

=1.35*0.109+1.5*0.692+1.5*0.6*0.385= 1.53KN/m

Combination 4, $Ed=1.35Gkjsup+1.5Qk1(concentrated)+1.5*0.6Qk,i(compression)$

=1.35*0.109+1.5*1KN+1.5*0.6*0.385= 0.494KN/m +1.5KN

Combination 5, $Ed=1.35Gkjsup+1.5Qk1(distributed)+1.5*0.6Qk,i(suction)$

=1.35*0.109+1.5*0.692+1.5*0.6*(-1.944) = -0.565KN/m

Combination 6, $Ed=1.35Gkjsup+1.5Qk1(concentrated)+1.5*0.6Qk,i(suction)$

=1.35*0.109+1.5*1KN+1.5*0.6*(-1.944) = -1.61KN/m +1.5KN

Case 3: wind load leading variable and imposed load accompanying variable

Commination 7, $Ed=1.35Gkjsup+1.5Qk1(compression)+1.5*0*Qk,i(distributed)$

=1.35*0.109+1.5*0.385 + 0 = 0.725KN/m

Combination 8, $Ed=1.35Gkjsup+1.5Qk1(compression)+1.5*0*Qk,i(concentrated)$

=1.35*0.109+1.5*0.385+0 = 0.725KN/m

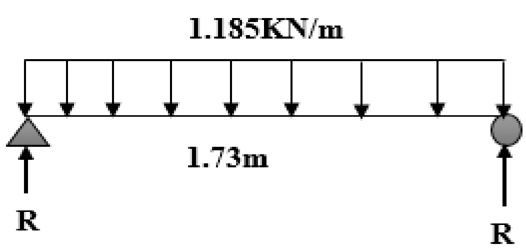
Combination 9, $Ed = 1Gkjsup + 1.5Qk1(suction) + 1.5*0*\ Qk,i(distributed)$

$= 1*0.109 + 1.5*(-1.944) + 0 = -2.807KN/m$

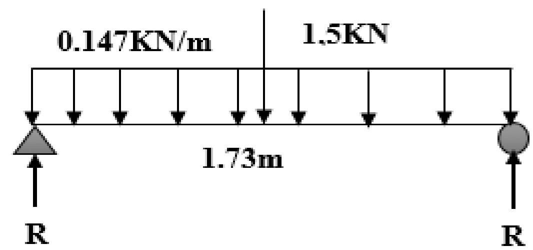
Combination 10, $Ed = 1Gkjsup + 1.5Qk1(suction) + 1.5*0*Qk,i(concentrated)$

$= 1*0.109 + 1.5*(-1.944) + 0 = -2.807KN/m$

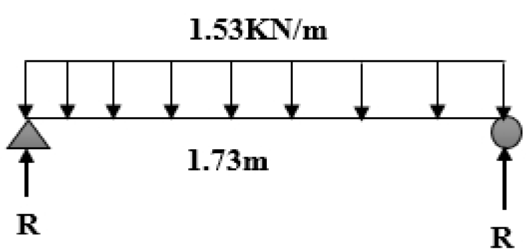
Loading for various combinations



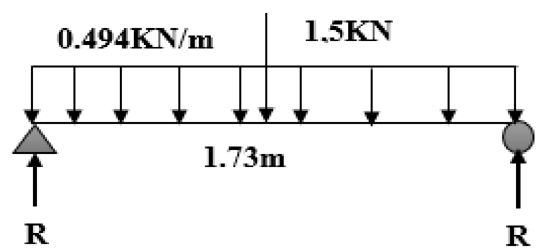
Combination 1



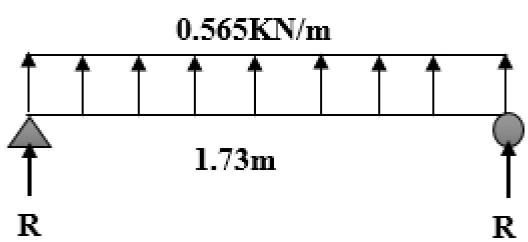
Combination 2



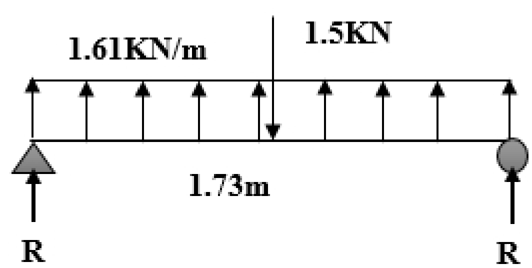
Combination 3



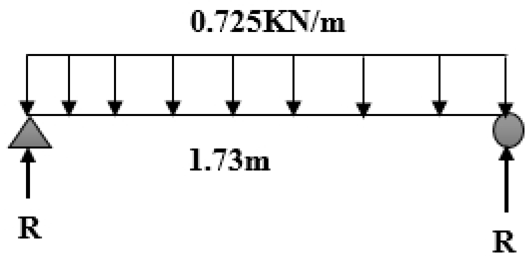
Combination 4



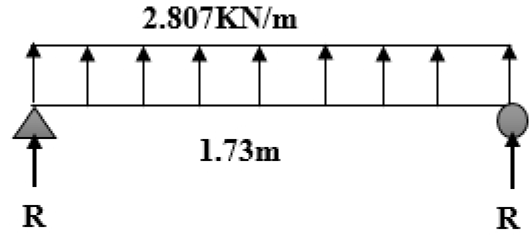
Combination 5



Combination 6



Combination 7,8



Combination 9,10

Figure 2. 9 Loadings for various combinations for duo pitch

Table 2.12 Load combination for duo-pitch

Combination	**Ed**	**M_{max}**	R
1	1.185KN/m	$Wl^2/8$=0.443KNm	Wl/2=1.025KN
2	0.147KN/m+1.5KN	$Wl^2/8$+wl/2=1.35KNm	Wl/2+w/2=0.877KN
3	1.53KN/m	$Wl^2/8$=0.573KNm	Wl/2=1.33KN
4	0.494KN/m +1.5KN	$Wl^2/8$+wl/2=1.483KNm	Wl/2+w/2=1.177KN
5	-0.565KN/m	$Wl^2/8$= -0.212KNm	Wl/2= -0.489KN
6	-1.61KN/m +1.5KN	$Wl^2/8$+wl/2= 0.695KNm	Wl/2+w/2= -0.643KN
7	0.725KN/m	$Wl^2/8$=0.271KNm	Wl/2=0.627KN
8	0.725KN/m	$Wl^2/8$=0.271KNm	Wl/2=0.627KN
9	-2.807KN/m	$Wl^2/8$= -1.05KNm	Wl/2= -2.43KN
10	-2.807KN/m	$Wl^2/8$= -1.05KNm	Wl/2= -2.43KN

N.B. W is UDL load, w is point load L is purlin spacing

From all the above combination the following maximum critical value is considered for design.

- ✓ Maximum positive reaction= +1.33KN, Combination 3
- ✓ Maximum negative reaction= -2.43KN, Combination 9
- ✓ Maximum positive moment= +1.483KNm, Combination 4

- ✓ Maximum negative moment= -1.05KNm, Combination 9

2.5.2 Analysis and Design of duo-pitched roof Lattice purlin

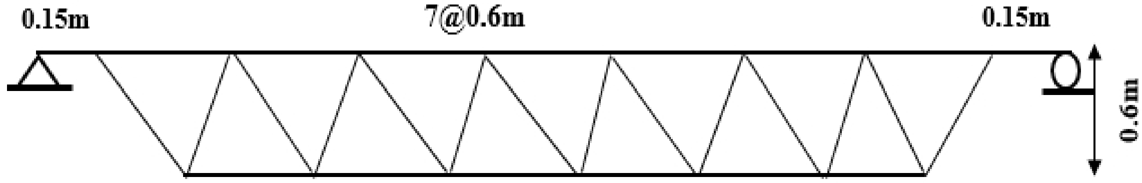
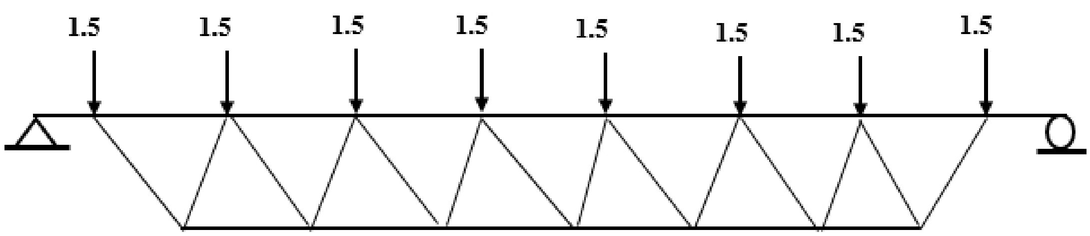


Figure 2. 10 lattice Purlin Lay out for duo-pitch

Since the purlin carries the load from adjacent EGA sheet section, the reaction from the critical loading should be multiplied by two.

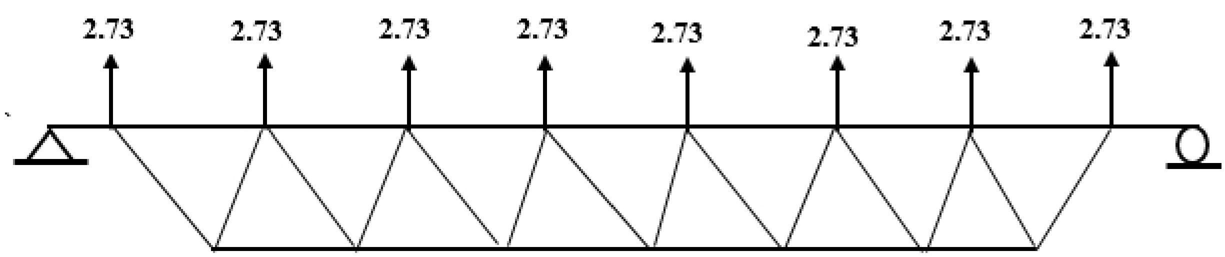
C) Positive reaction, R(+ve)

- R (+ve) = 1.33KN/m
- 2R(+ve)=2.66KN/m
- L=4.5m
- R concentrated=2.66 KN/m*4.5= 11.97KN
- R on each node=R concentrate/no of node = 11.97 KN / 8= 1.496KN=1.5KN



D) Negative reaction, R(-ve)

- R (-ve) = -2.43KN,
- 2R(-ve)=-4.86KN/m
- L=4.5m
- R concentrated at the center= -4.86KN/m*4.5m = -21.87KN
- R on each node=R concentrated/No of node = -21.87 KN/8 = -2.7337KN=-2.73KN



3 Analysis and Design of Slab

3.1 Introduction

A reinforced concrete slab is a broad, flat plate, usually horizontal, with top and bottom surfaces parallel or nearly so. It may be supported by reinforced beams (and is usually cast monolithically with such beams), by masonry or reinforced concrete walls, by structural steel members, directly by columns, or continuously by ground.

Type of slab

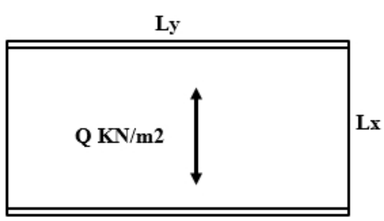
A slab subjected to dominantly UDLs may be considered to be one-way spanning if either:

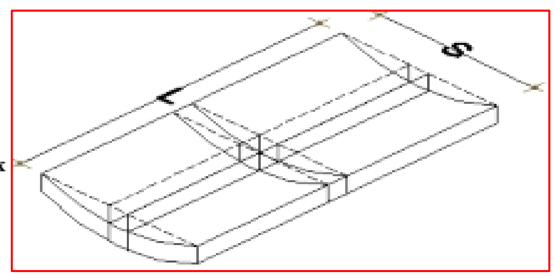
- It possesses two free (unsupported) and sensibly parallel edges, or
- It is the central part of a sensibly rectangular slab supported on four edges with a ratio of the longer to shorter span greater than 2.(ES EN 1992-1-1:2015,section 5.3(5))

The first step in slab design is classifying it as one way or two-way using their span ratio Ly/Lx.

One Way Slab

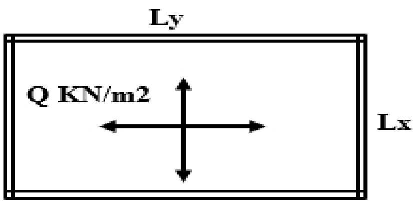
- ✓ Transfer loads in the shorter axis
- ✓ Supported in two direction only
- ✓ Main reinforcement is provided parallel to the shorter axis
- ✓ Shrinkage reinforcement is provided parallel to longer axis
- ✓ $Ly/Lx \leq 2$, Ly is longer side
- ✓ Bending occurs in shorter axis

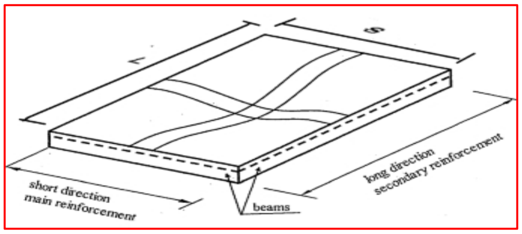




Two Way Slab

- ✓ Load is transfer in both axis
- ✓ Supported in all four direction
- ✓ Reinforcement is provided in both direction (in shorter & then in longer axis)
- ✓ $Ly/Lx<2$, Ly is longer side
- ✓ Bending occurs in both axis
- ✓ Is more stiffer than one way slab
- ✓ Mmax & Δmax is less than that of one way





The slabs are designed by following the procedures stated bellow.

a) Step:- Depth determination:

The minimum depth required for the slab can be calculated from the minimum depth required for deflection.

Concrete cover

$$c_{min} = \max \{c_{min,b}; c_{min,dur}; 10 \text{ mm}\}$$

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

Where:

$c_{min,b}$ minimum cover due to bond requirement,

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

are also given.

$$D_{overall} = d + c + \phi/2$$

b) Step:- analysis of the design load.

Design load (Pd) = 1.35DL + 1.5LL………… [1]

dead load(DL) analysis design load

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 over all depth times unit weight of concrete.

Live load (LL) analysis

a) Step:- analysis of the design moment:

Analysis of the design moment will be done as for two-way solid slabs and for one way solid slabs the calculation will be performed as 1m wide beam.

b) Step-: Moment adjustment:

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

c) Step:-reinforcement details:

After calculation of the design moment reinforcement detailing follows the appropriate rebar spacing.

Note: For the purpose of construction simplicity and monolithic construction the governing overall depth has been taken.

$\emptyset = 12mm$

$Ab = \frac{\pi d^2}{4}$

Spacing, $S = \frac{b * Ab}{As}$

The slabs are designed by following the procedures stated bellow.

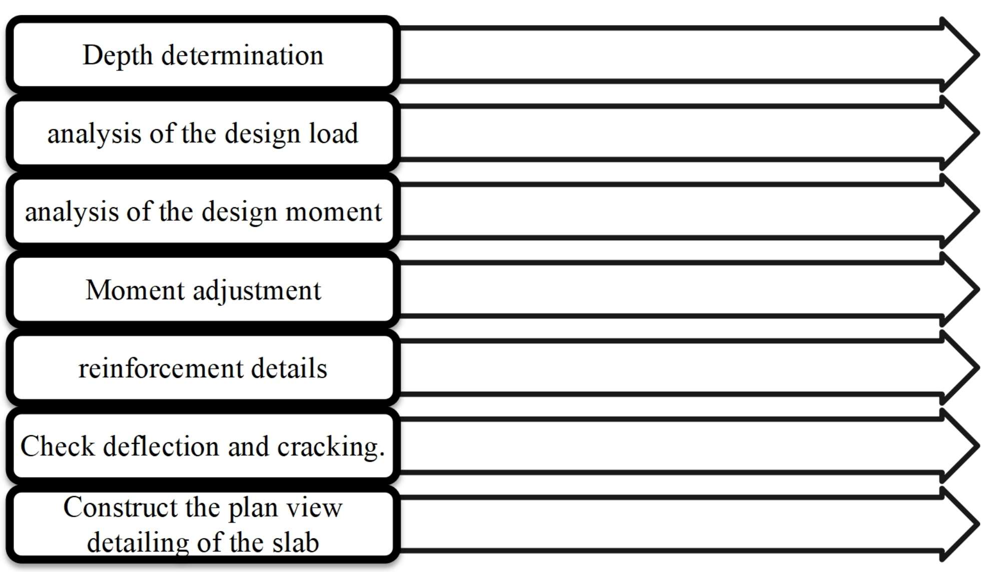


Figure 3. 1 designed procedures for slabs

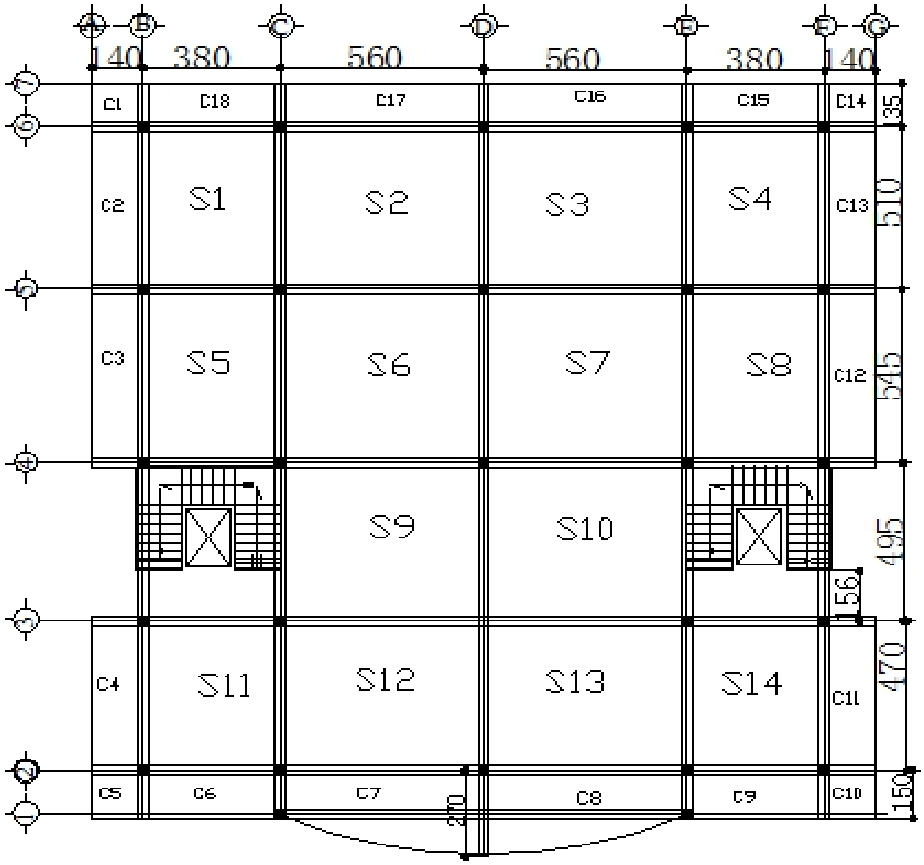


Figure 3.2 Second to Fourth Slab Layout

3.2 Design for Concrete Cover of slab

The nominal cover shall be specified on the drawings and given by, Cnom = Cmin+ΔCdev

ΔCdev is the deviation or reduction in concrete cover due to accurate pre-fabrication of members which are given in country's national code.

Cmin shall be provided in order to ensure: protection of steel against corrosion (durability), adequate fire resistance and safe transmission of bond forces b/n steel & concrete

The greater value for **Cmin** satisfying the requirements for both bond and environmental conditions shall be used.

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

where:

cmin,b minimum cover due to bond requirement,

cmin,dur minimum cover due to environmental conditions,

Δcdur,γ additive safety element,

Δcdur,st reduction of minimum cover for use of stainless steel,

Δcdur,add reduction of minimum cover for use of additional protection, $\mathbf{C_{min,b}}$ **,** for safe bond force transmission C_{min} should not less than $\mathbf{C_{min,b}}$.

- ✓ $\mathbf{C_{min,dur}}$ minimum cover due to environmental condition for different exposure classes
- ✓ ΔC_{dur}=0 ..ES EN 1992-1-1:2015 sec 4.4.1.2(6),

- ✓ ΔC_{dur}, st=0….ES EN 1992-1-1:2015 section 4.4.1.2(7), (for stainless steel reinforcement).
- ✓ ΔC_{dur}, add=0…ES EN 1992-1-1:2015 sec 4.4.1.2(7), (for not considering additional protection).

Therefore after simplification of the above condition

$$\textbf{Cmin} = \textbf{Max}\ \{Cmin,\ 10mm,\ Cmin,\ dur\}$$

The minimum cover required for bond is given by ES EN 1992-1-1:2015, Table 4.2.

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.	

And Assume, Cmin, bond=bar diameter=10mm

Cmin, dur is the minimum cover value for reinforcement in normal weight concrete take account of environmental condition for different exposure classes.

Table 4.4N: Values of minimum cover, cmin,dur, requirements with regard to durability for reinforcement steel

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

We are going to design our building

- Maximum aggregate size of 20mm
- Consider structural class S4
- Consider Exposure class XC1 which is for dry environment. ES EN 1992-1-1:2015 Table 4.1.

According to ES EN 1992-1-1:2015, section 4.4.1.2(5)

Note: Structural classification and values of cmin,dur for use in a Country may be found in its National Annex. The recommended Structural Class (service life of 50 years) is 4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class are given in Table 4.3N. The recommended minimum Structural Class is 1.

According to ES EN 1992-1-1:2015, Annex E, Table E.1N, for exposure class of XC1 take a minimum concrete grade of C-20/25 and ϕ12mm diameter of bar for slab.

Cover for bond and durability

$Cmin$ = max ($10mm$, $10mm$,$10mm$), $\boldsymbol{Cmin}$**=10**$\boldsymbol{mm}$

The nominal cover should be computed according to ES EN 1992-1-1:2015, Expression 4.1 by adding the minimum cover for bond and durability (Cmin) and the allowance in design for deviation (Δc, dev). $Cnom = Cmin + \Delta Cdev$

the allowance in design for deviation may be reduced. **10mm≥ΔC, dev≥5mm.**

Where it can be assured that a very accurate measurement device is used for monitoring and non-confirming members are rejected the allowance in design for deviation, Δc, dev may be reduced.

10mm≥Δc, dev≥0mm.

For our case we are going to use the allowance in design deviation to be 10mm which is given by ES EN 1992-1-1:2015, section 4.4.13(1) P because we don't have accurate measurement devices and quality assurance system. **ΔC, dev** $= 10mm$

$Cnom = Cmin + \Delta C, dev = 10mm + 10mm = \mathbf{20mm}$

Therefore, use a concrete cover of 20mm for the slab.

3.3 Slab Depth Determination

According to ES EN 1992-1-1:2015 section 7.4 the minimum depth of the slab should satisfy the serviceability requirement.

We have used the limited span to depth ratio according to equation 1 or 2 shown below the limiting span/depth ratio may be estimated using Expressions 1 and 2(ES EN 1992-1-1:2015 section 7.4.2) below and multiplying this by correction factors to allow for the type of reinforcement used and other variables. And it is given by ES EN 1992-1-1:2015 expression (7.16a).

$$\frac{l}{d} = K[11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho} + 3.2\sqrt{fck} * (\frac{\rho o}{\rho} - 1)^{3/2}],\ if\ \rho \leq \rho o \quad \textbf{Case 1}$$

$$\frac{l}{d} = K[11 + 1.5\sqrt{fck} * \frac{\rho o}{\rho - \rho'} + \frac{1}{12}\sqrt{fck} * (\frac{\rho'}{\rho})^{3/2}],\ if\ \rho > \rho o \quad \textbf{Case 2}$$

Where:

$\frac{l}{d}$ Is the limit span/depth

K Is the factor to take into account the different structural systems;

Table 7.4N: Basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural System	K	Concrete highly stressed $\rho = 1,5\%$	Concrete lightly stressed $\rho = 0,5\%$
Simply supported beam, one- or two-way spanning simply supported slab	1,0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1,3	18	26
Interior span of beam or one-way or two-way spanning slab	1,5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1,2	17	24
Cantilever	0,4	6	8
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.			

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

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

ρ´ Is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers); and

fck Is cylindrical characteristics compressive strength of the concrete in MPa units.

According to ES EN 1992-1-1:2015 section 7.4.2, Table 7.4N, the recommended values of k are given according to the structural system and the value of ρ for concrete slightly stressed is 0.5%.

The reference reinforcement ratio (ρo) for cylindrical characteristics concrete strength of 25Mpa (C-20/25) is

Serviceability Depth Calculation

Grade of concrete,C25=25Mpa, Fck =20mpa

Grade of steel fyk=460Mpa

$\rho o = \sqrt{20} * 10^{-3} = 4.47 * 10^{-3} = 0.447\% \approx 0.5\%$

$\rho o = 0.5\%$ reference steel ratio

ρ=0.5% for lightly stressed concrete

since, $\rho \leq \rho o$, 0.5%≤0.5% **Case 1 is used for slab depth determination**

Sample depth calculation

For slab 1, Slab-1

$\frac{l}{d}$ = k[11+1.5√20∗$\frac{0.5}{0.5}$ +3.2√20∗($\frac{0.5}{0.5}$ −1)$^{3/2}$]*[$\frac{500}{460}$] =19.25k , where K=1.5 since this panel is interior two way slab

$\frac{lx}{d}$ =28.87 for slab 1 Lx=3.8m

d=197.42mm

Slab depth, D = $d + (bar\ diameter)/2 + concrete\ cover$

D = 197.42mm + 10mm/2 + 20mm=222.42mm ≈ ***225mm***

Similar calculation is done for all floors with different values of Lx and K, the maximum governing depth is obtained at Slab 6 or slab 7 as d=283.15mm,

D=283.15mm + 10mm/2 + 20mm=308.15mm ≈ **310*mm*** is governing design depth for all floor.

Table 1. 18Table 3. 1 depth determination of Slab

Slab N.	Ly	Lx	Ly/Lx	Slab type	Slab confgrn	Support	k	ρ_o	ρ	Lx/d	dcal	Dcal	Dprovid	Dmax	LL	DLslab	Plength	PDL	TDL	%PL	Ed	Analysis mthd
S1	5.1	3.8	1.34	TW	NS	IS	1.5	0.5	0.5	19.25	197.42	222.42	225	310	3	9.17	7.95	68.07	245.78	21.93	16.88	Strip Mthd
S2	5.6	5.1	1.10	TW	NS	IS	1.5	0.5	0.5	19.25	264.96	289.96	290	310	3	9.17	5.79	49.57	311.47	12.19	16.88	Strip Mthd
S3	5.6	5.1	1.10	TW	NS	IS	1.5	0.5	0.5	19.25	264.96	289.96	290	310	3	9.17	5.79	49.57	311.47	12.19	16.88	Strip Mthd
S4	5.1	3.8	1.34	TW	NS	IS	1.5	0.5	0.5	19.25	197.42	222.42	225	310	3	9.17	7.9	67.64	245.35	21.82	16.88	Strip Mthd
S5	5.45	3.8	1.43	TW	NS	ES	1.3	0.5	0.5	18.28	207.92	232.92	235	310	3	9.17	8.15	69.78	259.69	21.23	16.88	Strip Mthd
S6	5.6	5.45	1.03	TW	NS	IS	1.5	0.5	0.5	19.25	283.15	308.15	310	310	3	9.17	10.21	87.41	367.28	18.64	16.88	Strip Mthd
S7	5.6	5.45	1.03	TW	NS	IS	1.5	0.5	0.5	19.25	283.15	308.15	310	310	3	9.17	10.21	87.41	367.28	18.64	16.88	Strip Mthd
S8	5.45	3.8	1.43	TW	NS	ES	1.3	0.5	0.5	18.28	207.92	232.92	235	310	3	9.17	8.75	74.91	264.83	22.44	16.88	Strip Mthd
S9	5.6	4.95	1.13	TW	NS	IS	1.5	0.5	0.5	19.25	257.17	282.17	285	310	3	9.17	4.95	42.38	296.57	10.90	16.88	Strip Mthd
S10	5.6	4.95	1.13	TW	NS	IS	1.5	0.5	0.5	19.25	257.17	282.17	285	310	3	9.17	4.95	42.38	296.57	10.90	16.88	Strip Mthd
S11	4.7	3.8	1.24	TW	NS	IS	1.5	0.5	0.5	19.25	197.42	222.42	225	310	3	9.17	1.96	16.78	180.56	6.99	18.15	Coff mthd
S12	5.6	4.7	1.19	TW	NS	IS	1.5	0.5	0.5	19.25	244.18	269.18	270	310	3	9.17	4.5	38.53	279.88	10.48	16.88	Strip Mthd
S13	5.6	4.7	1.19	TW	NS	IS	1.5	0.5	0.5	19.25	244.18	269.18	270	310	3	9.17	4.5	38.53	279.88	10.48	16.88	Strip Mthd
S14	4.7	3.8	1.24	TW	NS	IS	1.5	0.5	0.5	19.25	197.42	222.42	225	310	3	9.17	1.96	16.78	180.56	6.99	18.15	Coff mthd
S15	3.8	1.56	2.44	OW	CS	ES	0.4	0.5	0.5	13.90	112.22	137.22	140	310	2	9.17	0	0.00	54.36	0.00	15.38	Static mthd
S16	3.8	1.56	2.44	OW	CS	ES	0.4	0.5	0.5	13.90	112.22	137.22	140	310	2	9.17	0	0.00	54.36	0.00	15.38	Static mthd
C5	1.5	1.4	1.07	TW	CS	ES	0.4	0.5	0.5	13.90	100.71	125.71	130	310	2	9.17	0	0.00	19.26	0.00	15.38	Strip Mthd
C6	3.8	1.5	2.53	OW	CS	ES	0.4	0.5	0.5	13.90	107.91	132.91	135	310	2	9.17	0	0.00	52.27	0.00	15.38	Static mthd
C7	5.6	2.7	2.07	OW	CS	ES	0.4	0.5	0.5	13.90	194.23	219.23	215	310	2	9.17	0	0.00	138.65	0.00	15.38	Static mthd
C8	5.6	2.7	2.07	OW	CS	ES	0.4	0.5	0.5	13.90	194.23	219.23	215	310	2	9.17	0	0.00	138.65	0.00	15.38	Static mthd
C9	3.8	1.5	2.53	OW	CS	ES	0.4	0.5	0.5	13.90	107.91	132.91	135	310	2	9.17	0	0.00	52.27	0.00	15.38	Static mthd
C10	1.5	1.4	1.07	TW	CS	ES	0.4	0.5	0.5	13.90	100.71	125.71	130	310	2	9.17	0	0.00	19.26	0.00	15.38	Strip Mthd
C11	4.7	1.4	3.36	OW	CS	ES	0.4	0.5	0.5	13.90	100.71	125.71	130	310	2	9.17	1.35	11.56	71.90	13.36	15.38	Static mthd
C12	5.45	1.4	3.89	OW	CS	ES	0.4	0.5	0.5	13.90	100.71	125.71	130	310	2	9.17	1.2	10.27	80.24	10.57	15.38	Static mthd
C13	5.1	1.4	3.64	OW	CS	ES	0.4	0.5	0.5	13.90	100.71	125.71	130	310	2	9.17	1.5	12.84	78.32	13.64	15.38	Static mthd
C14	1.4	1.35	1.04	TW	CS	ES	0.4	0.5	0.5	13.90	97.12	122.12	125	310	2	9.17	0	0.00	17.33	0.00	15.38	Strip Mthd
C15	3.8	1.35	2.81	OW	CS	ES	0.4	0.5	0.5	13.90	97.12	122.12	125	310	2	9.17	0	0.00	47.04	0.00	15.38	Static mthd
C16	5.6	1.35	4.15	OW	CS	ES	0.4	0.5	0.5	13.90	97.12	122.12	125	310	2	9.17	0	0.00	69.33	0.00	15.38	Static mthd
C17	5.6	1.35	4.15	OW	CS	ES	0.4	0.5	0.5	13.90	97.12	122.12	125	310	2	9.17	0.82	7.02	76.35	7.54	16.63	Static mthd
C18	3.8	1.35	2.81	OW	CS	ES	0.4	0.5	0.5	13.90	97.12	122.12	125	310	2	9.17	0.82	7.02	54.06	10.72	15.38	Static mthd
C1	1.35	1.4	0.96	TW	CS	ES	0.4	0.5	0.5	13.90	100.71	125.71	130	310	2	9.17	0	0.00	17.33	0.00	15.38	Strip Mthd
C2	5.1	1.4	3.64	OW	CS	ES	0.4	0.5	0.5	13.90	100.71	125.71	130	310	2	9.17	1.5	12.84	78.32	13.64	15.38	Static mthd
C3	5.45	1.4	3.89	OW	CS	ES	0.4	0.5	0.5	13.90	100.71	125.71	130	310	2	9.17	1.2	10.27	80.24	10.57	15.38	Static mthd
C4	4.7	1.4	3.36	OW	CS	ES	0.4	0.5	0.5	13.90	100.71	125.71	130	310	2	9.17	1.35	11.56	71.90	13.36	15.38	Static mthd

CS= Cantilver slab NS=normal slab ES=End span SSD=Simply supported slab b LL= live load DL=dead load IS=interior span OW=one way TW= two way

Loads on slabs are both permanent loads and imposed loads. The permanent loads are loads that arise from the self-weight of the slab, cement screed, floor finishing material, plastering and ceiling, partition walls and partition wall finishing materials.

Slab S6 depth sample calculation, 5.6m*5.45m

Let concrete grade fcu =C25, fck=20Mpa and steel grade fyk=400mpa

$p_o = \sqrt{f_{ck}} * 10^{-3} = \sqrt{20} * 10^{-3}$ = 0.0045≈0.005 reference reinforcement ratio,

ρ = 0.5% =0.005 For lightly stressed concrete

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

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

K=1.5, for interior span, factor to account different structural system, Table 7.4N

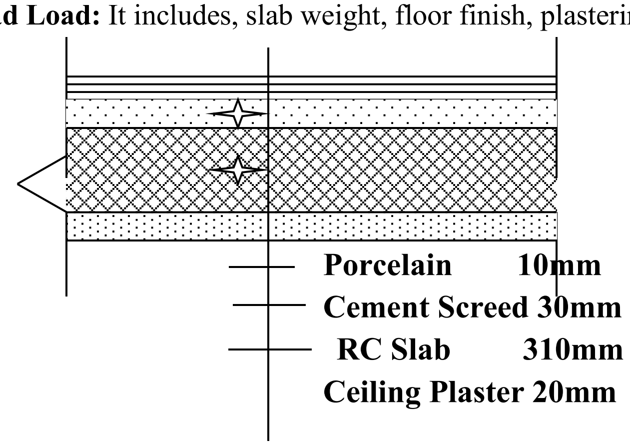
$$\frac{l}{d} = 1.5\left[11 + 1.5\sqrt{20} * \frac{0.005}{0.005} + 3.2\sqrt{20} * \left(\frac{0.005}{0.005} - 1\right)^{\frac{3}{2}}\right] * \frac{500}{460}$$

d=283.15mm

For all slabs similar calculation is done based on their span and support condition and the maximum depth is take as design service depth.

2.6 Slab Design Load Calculation

Dead Load: It includes, slab weight, floor finish, plastering, cement screed, partition



Floor finishing material may vary according to floor function. Referring the architectural plan, the dead load of each floor is calculated by multiplying unit weight of floor material with their respective unit weight as follow.

Table 3. 2 Floor finishing material may vary according to floor function

Material	ϒ(KN/m^3)	Thickness (m)	DL (KN/m^2)
Porcelain	27	0.01	0.27
Cement screed	23	0.03	0.69
Self-weight	25	0.31	7.75
Ceiling Plaster	23	0.02	0.46
		Total DL	**9.17**

Live Load: it is an imposed load where its value is taken from the code based on building and floor function as follow..

Loaded area	Qk (KN/m^2)	Qk (KN)
Normal Floor	3	2
Stair	3	3
Balcony	3	2
Cantilever floor	2	2

Partition loads: are floor dividing wall in which their self-weight on the floor is calculated as,

Partition wall weight = t_w * L* H*γ_{HCB}+2*t_p*L*H*γ_{pl}

Where

t_w Thickness of the partition wall(m);

H is the height of the partition wall

L length of partition wall over the slab

γ_{HCB} unit weight of HCB 14KN/m3

γ_{pl} Unit weight of plastering 23 KN/m3

Partition wall load $= \frac{1.35*PDL}{1.35(DLslab+PDL)+1.5LL} * 100 \geq 10\%$ *the slab is design by strip method*

Partition wall load $= \frac{1.35*PDL}{1.35(DLslab+PDL)+1.5LL} * 100 < 10\%$ *the slab is design by cofficient method*

And the load is distributed over slab Area as $PDL_{distributed}$=PDL/Lx*Ly

The new slab design load is Ed=1.35(DLslab+ $PDL_{distributed}$) +1.5LL

Sample partition wall effect calculation

Slab 1

Thickness of wall = 15cm

Length of wall = 7.95m

Height of wall = 3.15m

Thickness of plastering=20mm

Unit weight of plastering=23KN/m3

Unit weight of HCB=14KN/m3

Partition wall load on slab-1 = tw * L* H*γ_{HCB}+2*t_p*L*H*γ_{pl}

Partition wall load on slab-1 = 0.15*7.95*3.15*14+2*0.02*7.95*3.15*23 = 68.07KN

$\% = \frac{1.35*68.07}{[1.35*68.07]+[1.35*9.17*5.1*3.8]+[1.5*3*5.1*3.8]} = 21.93\% \geq 10\%$

It has structural effect so this slab should be calculated by strip method.

Slab 11

Thickness of wall = 15cm

Length of wall = 1.96m

Height of wall = 3.15m

Thickness of plastering=20mm

Unit weight of plastering=23KN/m3

Unit weight of HCB=14KN/m3

Partition wall load on slab-1, PDL = tw * L* H*γ_{HCB}+2*t_p*L*H*γ_{pl}

Partition wall load on slab-1, PDL = 0.15*1.96*3.15*14+2*0.02*1.96*3.15*23 = 16.78KN

$\% = \frac{1.35*16.78}{[1.35*16.78]+[1.35*9.17*5.1*3.8]+[1.5*3*5.1*3.8]} = 6.99\% < 10\%$

Therefore, this slab should be designed by coefficient method and the Partition load is distributed over slab area and added with slab dead load.

Dead load of this slab=DLslab + PDL/Ly*Lx=9.17+16.78/4.7*3.8=10.12KN

Design load of this slab Ed=1.35DL+1.5LL=1.35*10.12+1.5*3=18.15KN/m2

Similar calculation can be done and slab design method is shown in the table below. Two way cantilever slab is design by strip method, one way cantilever slab is designed by principle of static.

3.4 Method of Slab Analysis

Various simplified methods have been adopted for determining moments, shear and reactions. The methods that are accepted because of satisfying equilibrium conditions are:

1. Coefficient method of analysis
2. Yield line method of analysis
3. Strip method of analysis

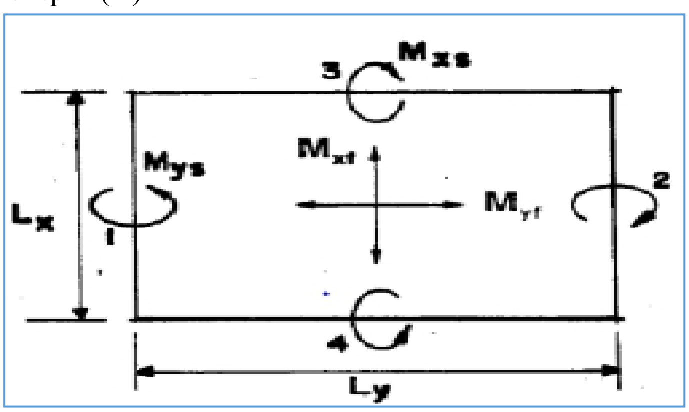
3.4.1 Analysis of two-way slab using coefficient method

Coefficient method of analysis is the most widely used and easiest method of analysis. In slabs where the corners are prevented from lifting, and provision for torsion is made, the maximum design moments per unit width are given by the following equations.

$Mx = \alpha x * Pd * Lx^2$ and $My = \alpha y * Pd * Lx^2$

Where
Mx and My are Bending moments in the x and y direction of slab respectively
αx and αy Moment coefficient based on supported edge conditions

Ed Design load on the slab (KN/m^2)
Lx Length of the shorter span (m)



T***he subscript represents***

- ✓ **S=** support
- ✓ **f=** field/span

- ✓ **x=** direction of shorter span
- ✓ **y=** direction of longer span

For those slabs strictly their partition load effect<10% S 11 and S14 and for those their partition load effect is close to 10% (S9,S10,S12,S13)coefficient method of analysis is used as follow.

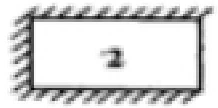
For Slab-11

Ly/Lx=4.7m/3.8m=1.24

Ed= 18.15KN/m2

Lx=3.8m

Support condition-2, discontinuous on the shorter side



From EBCS table-1 of moment coefficient for rectangular slab for Ly/Lx=1.24 is obtained from interpolation between Ly/Lx=1.2 and Ly/Lx=1.3 as follow for both S11 and S14

Ly/Lx	Axs	axf	ays	ayf
1.2	0.048	0.036	0.039	0.029
1.24	0.0496	0.037	0.039	0.029
1.3	0.052	0.039	0.039	0.029

Support Moment

Mxs = αxs* Ed *Lx^2 =0.0496*18.15*3.8^2 = 12.99KNm

Mys= αys*Ed*Lx^2 = 0.039*18.15*3.8^2 = 10.22KNm

Field moment

Mxf =αxf* Ed *Lx^2 =0.037*18.15*3.8^2 = 9.69KNm

Myf=αyf*Ed*Lx^2 = 0.029*18.15*3.8^2 = 7.6KNm

For slab 14

Ly/Lx=4.7m/3.8m=1.24

Ed= 18.15KN/m2

Lx=3.8m

Support condition-2, discontinuous on the shorter side

Support Moment

Mxs = αxs* Ed *Lx^2 =0.0496*18.15*3.8^2 = 12.99KNm

Mys= αys*Ed*Lx^2 = 0.039*18.15*3.8^2 = 10.22KNm

Field moment

Mxf =αxf* Ed *Lx^2 =0.037*18.15*3.8^2 = 9.69KNm

Myf=αyf*Ed*Lx^2 = 0.029*18.15*3.8^2 = 7.6KNm

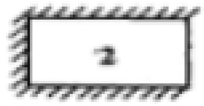
For slab 9

Ly/Lx=5.6m/4.95m=1.13

Ed= 18.94KN/m2

Lx=4.95m

Support condition-2, discontinuous on the shorter side



From EBCS table-1 of moment coefficient for rectangular slab for Ly/Lx=1.13 is obtained from interpolation between Ly/Lx=1.1 and Ly/Lx=1.2 as follow for both S9 and S10.

Ly/Lx	Axs	axf	ays	ayf
1.1	0.044	0.033	0.039	0.029
1.13	0.0452	0.034	0.039	0.029
1.2	0.048	0.036	0.039	0.029

Support Moment

Mxs = αxs* Ed *Lx^2 =0.0452*18.94*4.95^2 = 20.98KNm

Mys= αys*Ed*Lx^2 = 0.039*18.94*4.95^2 = 18.1KNm

Field moment

Mxf =αxf* Ed *Lx^2 =0.034*18.94*4.95^2 = 15.78KNm

Myf=αyf*Ed*Lx^2 = 0.029*18.94*4.95^2 = 13.46KNm

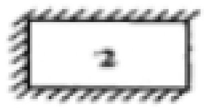
For slab 10

Ly/Lx=5.6m/4.95m=1.13

Ed= 18.94KN/m2

Lx=4.95m

Support condition-2, discontinuous on the shorter side



Support Moment

Mxs = αxs* Ed *Lx^2 =0.0452*18.94*4.95^2 = 20.98KNm

Mys= αys*Ed*Lx^2 = 0.039*18.94*4.95^2 = 18.1KNm

Field moment

Mxf =αxf* Ed *Lx^2 =0.034*18.94*4.95^2 = 15.78KNm

Myf=αyf*Ed*Lx^2 = 0.029*18.94*4.95^2 = 13.46KNm

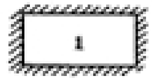
For slab 12

Ly/Lx=5.6m/4.7m=1.19

Ed= 18.86KN/m2

Lx=4.7m

Support condition-1, continuous on all sides



From EBCS table-1 of moment coefficient for rectangular slab for Ly/Lx=1.19 is obtained from interpolation between Ly/Lx=1.1 and Ly/Lx=1.2 as follow for both S12 and S13.

Ly/Lx	Axs	axf	ays	ayf
1.1	0.037	0.028	0.032	0.024
1.19	0.042	0.032	0.032	0.024
1.2	0.042	0.032	0.032	0.024

Support Moment

Mxs = αxs* Ed *Lx^2 =0.042*18.86*4.7^2 = 17.5KNm

Mys= αys*Ed*Lx^2 = 0.032*18.86*4.7^2 = 13.33KNm

Field moment

Mxf =αxf* Ed *Lx^2 =0.032*18.86*4.7^2 = 13.33KNm

Myf=αyf*Ed*Lx^2 = 0.024*18.86*4.7^2 = 10KNm

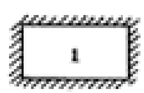
For slab 13

Ly/Lx=5.6m/4.7m=1.19

Ed= 18.86KN/m2

Lx=4.7m

Support condition-1, continuous on all sides



Support Moment

Mxs = αxs* Ed *Lx^2 =0.042*18.86*4.7^2 = 17.5KNm

Mys= αys*Ed*Lx^2 = 0.032*18.86*4.7^2 = 13.33KNm

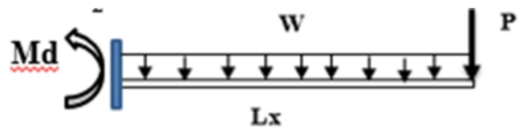
Field moment

Mxf =αxf* Ed *Lx^2 =0.032*18.86*4.7^2 = 13.33KNm

Myf=αyf*Ed*Lx^2 = 0.024*18.86*4.7^2 = 10KNm

3.4.2 Analysis of One way cantilever slab

One way cantilever slab is designed as cantilever beam with 1m width along the shorter span of slab. The loading is A UDL load over 1m strip and qKN point load at the end of slab due to cantilever barrier wall of assumed height 2m and 15cm thickness.



- A UDL slab load throughout the shorter span, $M_1=WLx^2/2$
- A point load at the end of slab due to partition wall $M_2=P*Lx$

Where, $W(KN/m^2)=1.35(DLslab+PDL/(Lx*Ly))+1.5LL$

$P(KN/m)=1.35DL$ due to wall load at edge

The total design moment of cantilever slab is **$Md=M_1+M_2=$** KNm/m at the support

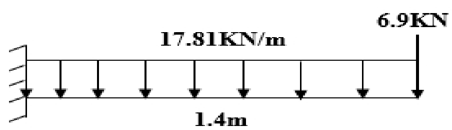
The effect of partition Dead load on cantilever slab is also considered and distributed over slab area.

C-2, C-3,C-4

UDL=Ed=17.81KN/m

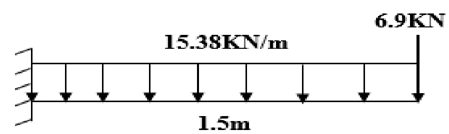
Point load, q=2m*0.15m*14KN/m3+2m*0.02m*23 KN/m3=5.12KN/m.

Design point load qd=1.35*5.12=6.9 KN/m



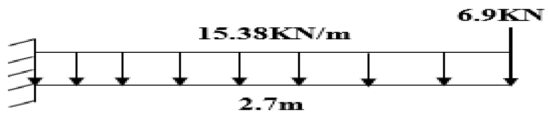
- $M_1=WLx^2/2=17.81*1.4^2/2=17.45$KNm/m
- $M_2=P*Lx=6.9*1.4=9.66$KNm/m
- Md= **M_1+M_2= 27.11KNm/m** at the support
- R= 17.81*1.4+6.9=31.84KN/m

C-6, C-9



- $M_1=WLx^2/2=15.38*1.5^2/2=17.3$KNm/m
- $M_2=P*Lx=6.9*1.5=10.35$KNm/m
- Md= **M_1+M_2= 27.65KNm/m** at the support
- R= 15.38*1.5+6.9=29.97KN/m

C-7,C-8



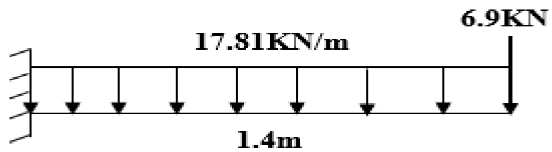
- $M_1=WLx^2/2$ =15.38*2.7^2/2=56.96KNm/m
- M_2=P*Lx=6.9*2.7=18.63KNm/m
- Md= **M_1+M_2= 75.59KNm/m** at the support
- R= 15.38*2.7+6.9=48.43KN/m

C-11, C-12,C-13

UDL=Ed=17.81KN/m

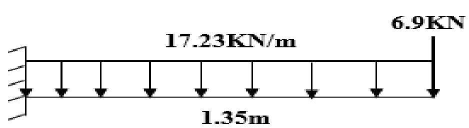
Point load, q=2m*0.15m*14KN/m3+2m*0.02m*23 KN/m3=5.12KN/m.

Design point load qd=1.35*5.12=6.9 KN/m



- $M_1=WLx^2/2$ =17.81*1.4^2/2=17.45KNm/m
- M_2=P*Lx=6.9*1.4=9.66KNm/m
- Md= **M_1+M_2= 27.11KNm/m** at the support
- R= 17.81*1.4+6.9=31.84KN/m

C-15, C-16,C-17, C-18



- $M_1=WLx^2/2$ =17.23*1.35^2/2=15.7KNm/m
- M_2=P*Lx=6.9*1.35=9.32KNm/m
- Md= **M_1+M_2= 25.02KNm/m** at the support
- R= 17.23*1.35+6.9=30.16KN/m

3.4.3 Analysis of slab using strip method

The strip method is a lower bound approach, based on satisfaction of equilibrium requirements everywhere in the slab in contrast to yield line analysis. By the strip method (sometimes referred to as the equilibrium theory, a moment field is first determined that full fills equilibrium requirements, after which the reinforcements in the slab at each point is designed for this moment field. It gives result on the safe side which is certainly preferable in practice

- It is a design method, by which the needed reinforcement can be calculated.
- It encourages the designer to vary the reinforcement in logical way, leading to an economical arrangement of steel, as well as a safe design

It is **Analysis of two way cantilever slab using strip method**

In this typical project two way cantilever slab supported adjacent sides are designed by strip method. Those slabs are **C-1, C-5, C-10 and C-14.**The general strip division for uniformly loaded rectangular slab continues in two adjacent sides and discontinuous in the two adjacent side is given below.

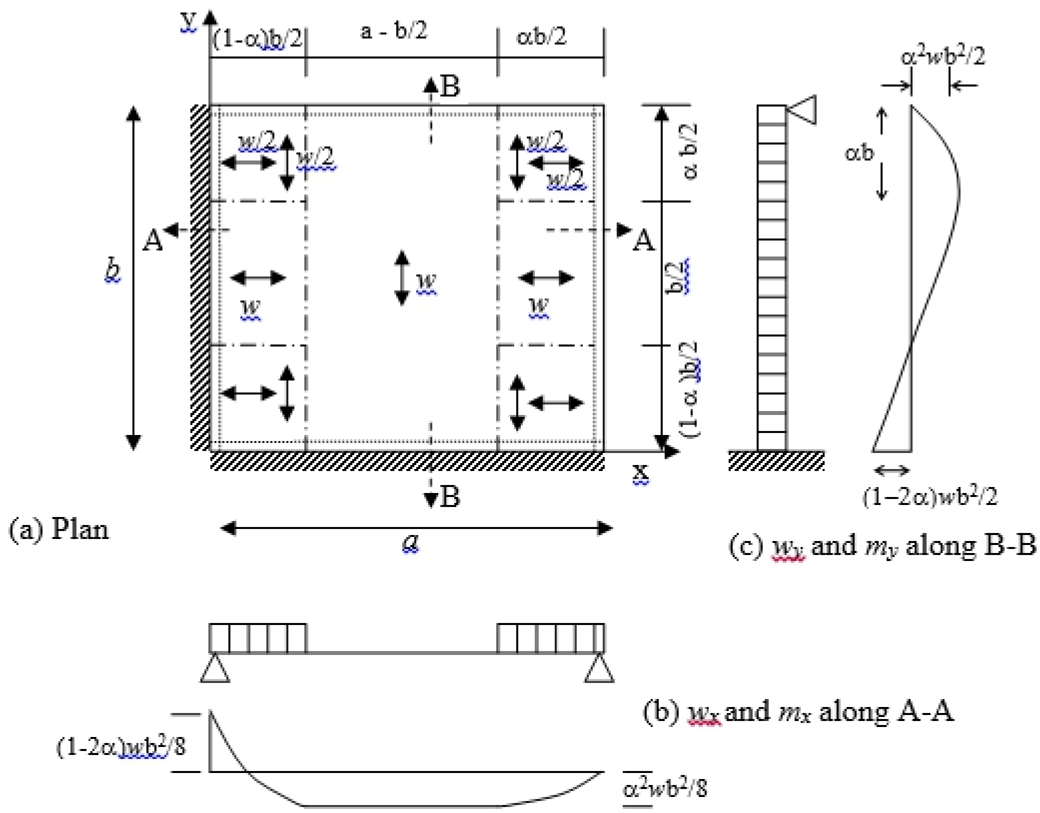
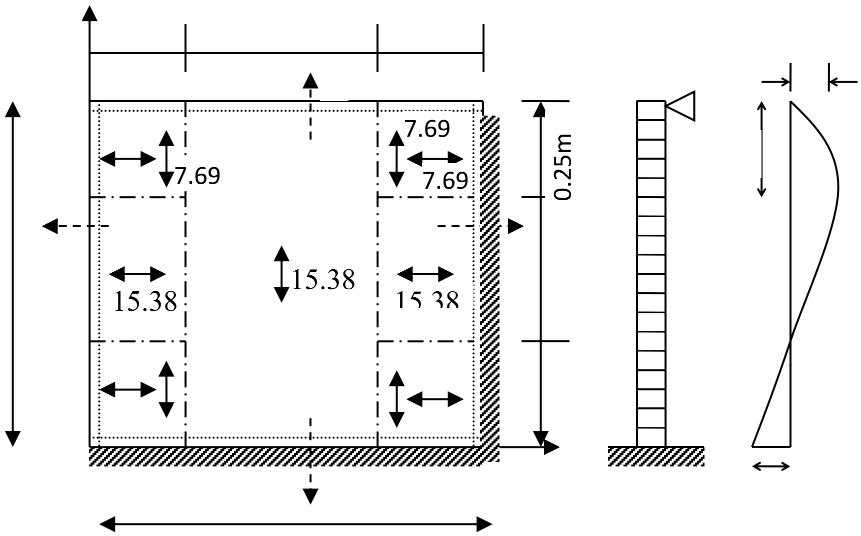
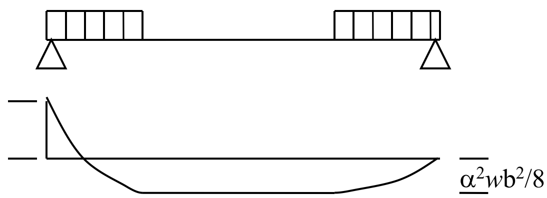


Figure 1. 13Figure 3.2 Strip division for slab with fixed edge and continuity edge

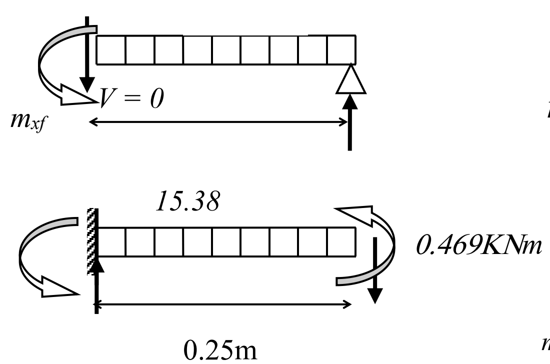
Cantilever Slab, C-1

- ✓ W=Ed=15.38KN/m2
- ✓ W/2=Ed/2=7.69KN/m2
- ✓ a=1.4m b=1.35m b is the smaller of the two span
- ✓ Assume support moment/span moment $m_{xs}/m_{xf} = 2$
- ✓ α= 0.366 chosen in this range 0.35 ≤ α ≤ 0.39 corresponding with negative to positive moment ratio from 2.5 to 1.5 range recommended by Hillerborg.
- ✓ α should be less than 0.5.
- ✓ The edge strips widths is greater than b/4 at fixed end & less than b/4 at simple end.





Moment X-Direction Middle Strips: Section A-A



Positive moment in the span

$$m_{xf} = \frac{\alpha^2 wb^2}{8} = \frac{0.366^2 * 15.38 * 1.35^2}{8} = 0.469KNm$$

Negative moment at the left support

$$m_{xs} = \frac{(1-2\alpha)wb^2}{8} = \frac{(1-2*0.366)*15.38*1.35^2}{8} = 0.94KNm$$

Observing, the absolute of the negative moment at a support plus the span moment= the "cantilever" moment. $= \frac{\alpha^2 wb^2}{8} + \frac{(1-2\alpha)wb^2}{8} = \frac{(1-\alpha)^2 wb^2}{8}$ =1.41KNm

Now, the ratio of negative to positive moments in the x-direction middle strip is: $\frac{m_{xs}}{m_{xf}} = \frac{1-2\alpha}{\alpha^2}$

As per Hillerborg general rule for fixed edges, the support moment should be about 1.5 to 2.5 times the span moment in the same strip.⇒ Assume m_{xs}/m_{xf} = 2 so $2\alpha^2 + 2\alpha - 1 = 0 \Rightarrow \alpha = 0.366$.

Higher values should be chosen for longitudinal strips that are largely unloaded and in such cases a ratio of support to span moment of 3 to 4 may be used. However A_{smin} may govern for such high ratios with too small positive moment.

Moment in the X-Direction Edge Strips:

Note that they are one half of those in the middle strips because load is half.

Mxf= Mxf middle strip/2=0.469/2=0.235KNm

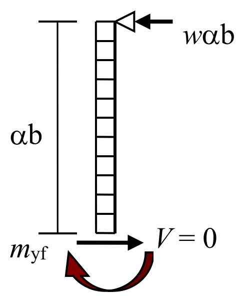
Mxs= Mxs middle strip/2=0.94/2=0.47KNm

Moment in the Y- Direction Middle Strips: Section B-B

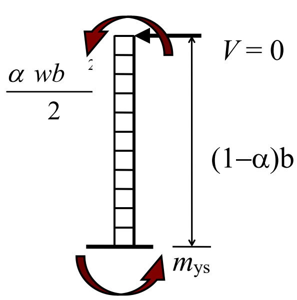
It is reasonable to choose the same m_{xs}/m_{xf} = 2 in the y- direction as in the x- direction before,

where : $\frac{m_{xs}}{m_{xf}} = \frac{1-2\alpha}{\alpha^2}$, $m_{xs}/m_{xf} = 2$, $\alpha = 0.366$

→ Choose the distance from the right support to maximum moment section as αb [the Cantilever span = (1- α)b ⇒ $m_{ys} = (1-2\alpha)wb^2/2$]

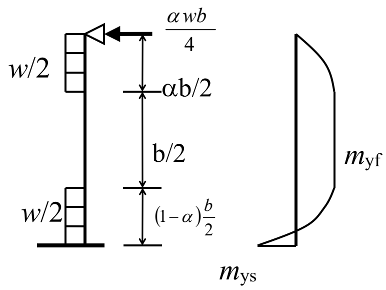


$$m_{yf} = \alpha^2 \frac{wb^2}{2} = 0.366^2 * \frac{15.38*1.35^2}{2} = 1.88KNm$$



$$m_{ys} = (1-2\alpha)\frac{wb^2}{2} = (1-2*0.366)*\frac{15.38*1.35^2}{2} = 3.76KNm$$

Moment in the Y-Direction Edge Strips:



$$m_{yf} = \frac{w\alpha b}{2}\left(\frac{\alpha b}{2}\right) - \frac{w}{2}\frac{\alpha b}{2}\frac{\alpha b}{2} = \frac{w(\alpha b)^2}{16} = 0.24\,KNm$$

Cantilever moment

$$m_{ys} = \frac{w}{2}(1-\alpha)\frac{b}{2}.(1-\alpha)\frac{b}{4} - \alpha^2\frac{wb^2}{16}$$

$$= (1-\alpha)^2\frac{wb^2}{16} - \alpha^2\frac{wb^2}{16} = (1-2\alpha)\frac{wb^2}{16} = 0.47\,KNm$$

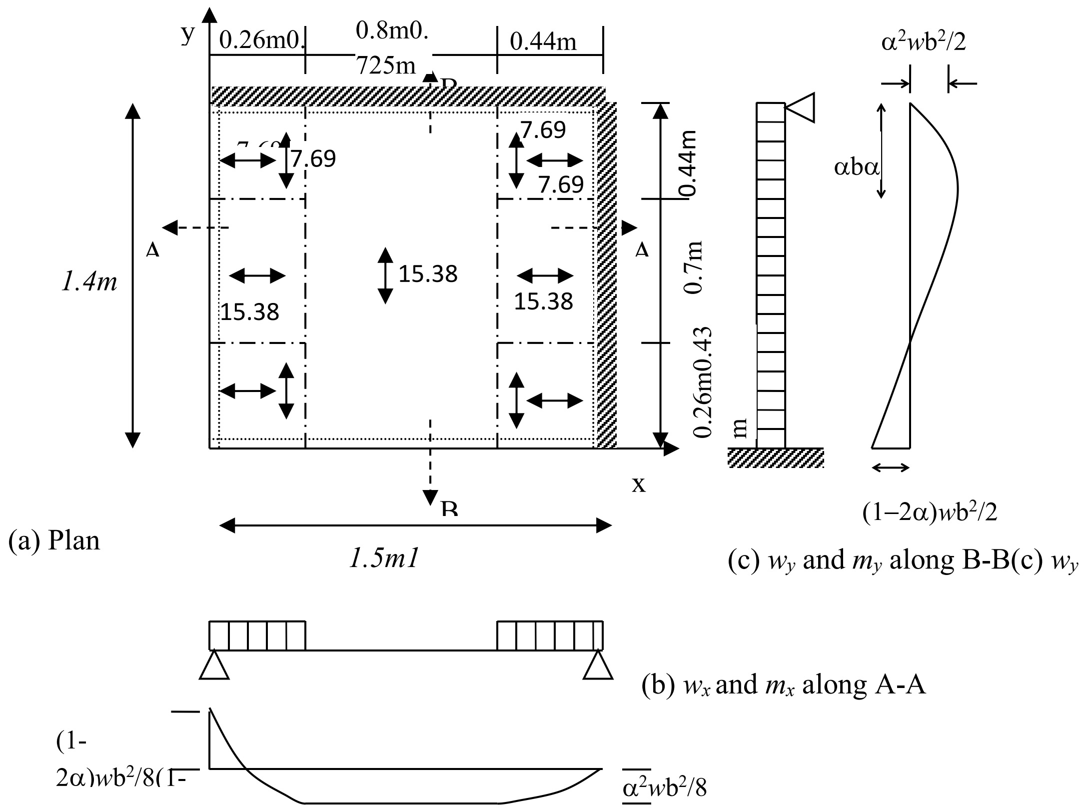
One-eighth of those in y- direction middle

For example, if it is decided that support moment is to be twice the span moments, the value of α= 0.366 and the negative and positive moments in the central strip in the y- direction are respectively $0.134wb^2$ and $0.067wb^2$. In the middle strip in the x- directions, moments are one-fourth those values; and in the edge strips in both directions, they are one-eighth of those values.

Cantilever Slab, C-5

- ✓ W=Ed=15.38KN/m2
- ✓ W/2=Ed/2=7.69KN/m2
- ✓ a=1.5m b=1.4m, b is the smaller span
- ✓ Assume support moment/span moment $m_{xs}/m_{xf} = 2$
- ✓ α= 0.366 chosen in this range 0.35 ≤ ≤ 0.39 corresponding with negative to positive moment ratio from 2.5 to 1.5 range recommended by Hillerborg.
- ✓ α should be less than 0.5.

The edge strips widths is greater than b/4 at fixed end & less than b/4 at simple end.



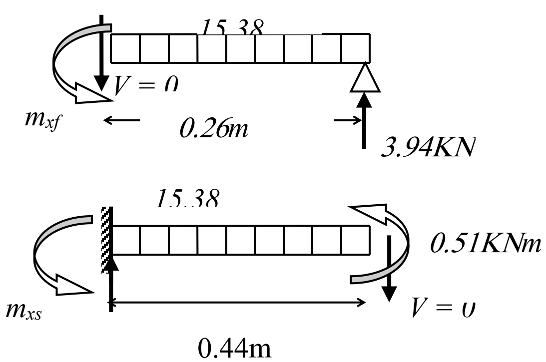
(a) Plan

(c) w_y and m_y along B-B(c) w_y

(b) w_x and m_x along A-A

Moment X-Direction Middle Strips: Section A-A

Positive moment in the span



$$m_{xf} = \frac{\alpha^2 wb^2}{8} = \frac{0.366^2 * 15.38 * 1.4^2}{8} = 0.51 KNm$$

Negative moment at the left support

$$m_{xs} = \frac{(1-2\alpha)wb^2}{8} = \frac{(1-2*0.366)*15.38*1.4^2}{8} = 1.01 KNm$$

Observing, the absolute of the negative moment at a support plus the span moment= the "cantilever" moment. = $\frac{\alpha^2 wb^2}{8} + \frac{(1-2\alpha)wb^2}{8} = \frac{(1-\alpha)^2 wb^2}{8}$ =1.52KNm

Now, the ratio of negative to positive moments in the x-direction middle strip is: $\frac{m_{xs}}{m_{xf}} = \frac{1-2\alpha}{\alpha^2}$

As per Hillerborg general rule for fixed edges, the support moment should be about 1.5 to 2.5 times the span moment in the same strip.⇒ Assume m_{xs}/m_{xf} = 2so $2\alpha^2 + 2\alpha - 1 = 0 \Rightarrow \alpha = 0.366$.

Higher values should be chosen for longitudinal strips that are largely unloaded and in such cases a ratio of support to span moment of 3 to 4 may be used. However A_{smin} may govern for such high ratios with too small positive moment.

Moment in the X-Direction Edge Strips:

Note that they are one half of those in the middle strips because load is half.

Mxf= Mxf middle strip/2=0.51/2=0.26KNm

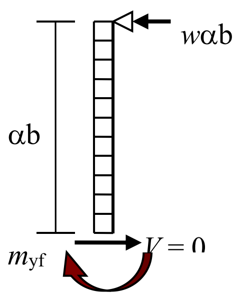
Mxs= Mxs middle strip/2=1.01/2=0.51KNm

Moment in the Y- Direction Middle Strips: Section B-B

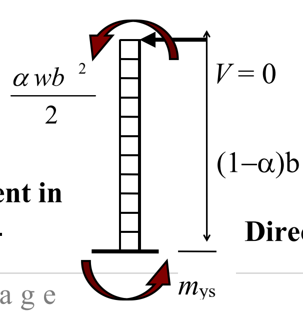
It is reasonable to choose the same m_{xs}/m_{xf} = 2 in the y- direction as in the x- direction before,

where : $\frac{m_{xs}}{m_{xf}} = \frac{1-2\alpha}{\alpha^2}$, $m_{xs}/m_{xf} = 2$, $\alpha = 0.366$

→ Choose the distance from the right support to maximum moment section as αb [the Cantilever span = (1- α)b ⇒ $m_{ys} = (1-2\alpha)wb^2/2$]

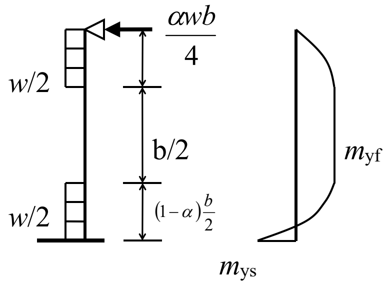


$$m_{yf} = \alpha^2 \frac{wb^2}{2} = 0.366^2 * \frac{15.38*1.4^2}{2} = 2.02KNm$$



$$m_{ys} = (1-2\alpha)\frac{wb^2}{2} = (1-2*0.366)*\frac{15.38*1.4^2}{2} = 4.04KNm$$

Moment in the Y- Direction Edge Strips:



$$m_{yf} = \frac{w\alpha b}{2}\left(\frac{\alpha b}{2}\right) - \frac{w}{2}\frac{\alpha b}{2}\frac{\alpha b}{2} = \frac{w(\alpha b)^2}{16} = 0.25KNm$$

Cantilever moment

$$m_{ys} = \frac{w}{2}(1-\alpha)\frac{b}{2}.(1-\alpha)\frac{b}{4} - \alpha^2\frac{wb^2}{16}$$

$$= (1-\alpha)^2\frac{wb^2}{16} - \alpha^2\frac{wb^2}{16} = (1-2\alpha)\frac{wb^2}{16} = 0.51KNm$$

One-eighth of those in y- direction middle

Moment in the X-Direction Edge Strips:

Note that they are one half of those in the middle strips because load is half.

Mxf= Mxf middle strip/2=0.51/2=0.26KNm

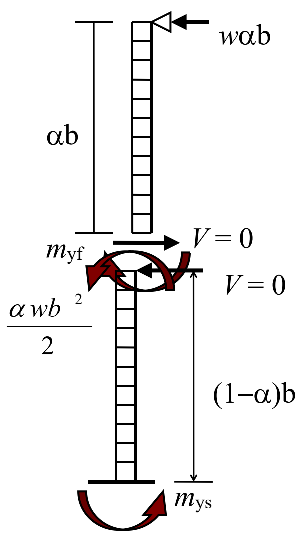
Mxs= Mxs middle strip/2=1.01/2=0.51KNm

Moment in the Y- Direction Middle Strips: Section B-B

It is reasonable to choose the same $m_{xs}/m_{xf} = 2$ in the y- direction as in the x- direction before,

where : $\frac{m_{xs}}{m_{xf}} = \frac{1-2\alpha}{\alpha^2}$, $m_{xs}/m_{xf} = 2$, $\alpha = 0.366$

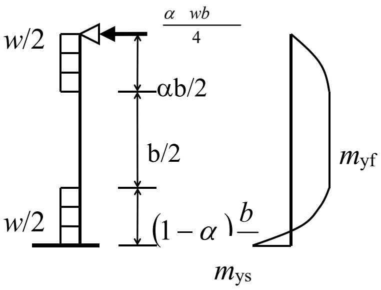
→ Choose the distance from the right support to maximum moment section as αb [the Cantilever span = (1- α)b ⇒ $m_{ys} = (1-2\alpha)wb^2/2$]



$$m_{yf} = \alpha^2\frac{wb^2}{2} = 0.366^2 * \frac{15.38*1.4^2}{2} = 2.02KNm$$

$$m_{ys} = (1-2\alpha)\frac{wb^2}{2} = (1-2*0.366)*\frac{15.38*1.4^2}{2} = 4.04KNm$$

Moment in the Y-Direction Edge Strips:



$$m_{yf} = \frac{w\alpha b}{2}\left(\frac{\alpha b}{2}\right) - \frac{w}{2}\frac{\alpha b}{2}\frac{\alpha b}{2} = \frac{w(\alpha b)^2}{16} = 0.25KNm$$

Cantilever moment

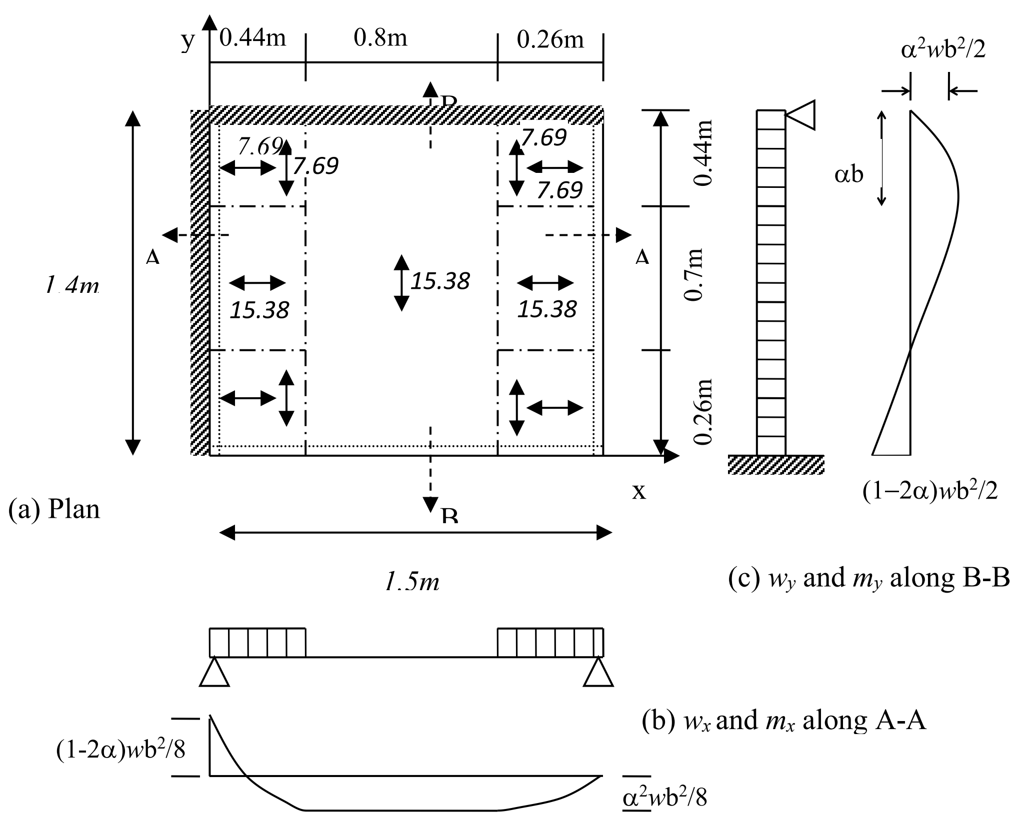
$$m_{ys} = \frac{w}{2}(1-\alpha)\frac{b}{2}.(1-\alpha)\frac{b}{4} - \alpha^2\frac{wb^2}{16}$$

$$= (1-\alpha)^2\frac{wb^2}{16} - \alpha^2\frac{wb^2}{16} = (1-2\alpha)\frac{wb^2}{16} = 0.51KNm$$

One-eighth of those in y- direction middle strip

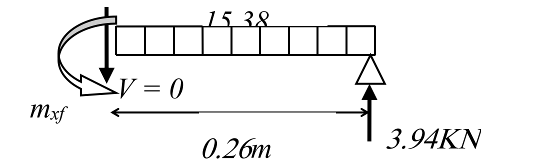
Cantilever Slab, C-10

- ✓ W=Ed=15.38KN/m2
- ✓ W/2=Ed/2=7.69KN/m2
- ✓ a=1.5m b =1.4m b is the smaller of the two span
- ✓ Assume support moment/span moment $m_{xs}/m_{xf} = 2$
- ✓ α= 0.366 chosen in this range 0.35 ≤ α ≤ 0.39 corresponding with negative to positive moment ratio from 2.45 to 1.45 range recommended by Hillerborg.
- ✓ α should be less than 0.5.
- ✓ The edge strips widths is greater than b/4 at fixed end & less than b/4 at simple end.

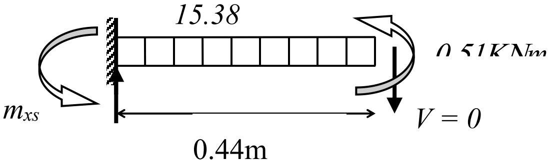


Moment X-Direction Middle Strips: Section A-A

Positive moment in the span



$$m_{xf} = \frac{\alpha^2 wb^2}{8} = \frac{0.366^2 * 15.38 * 1.4^2}{8} = 0.51 KNm$$



Negative moment at the left support

$$m_{xs} = \frac{(1-2\alpha) wb^2}{8} = \frac{(1-2*0.366)*15.38*1.4^2}{8} = 1.01 KNm$$

Observing, the absolute of the negative moment at a support plus the span moment= the "cantilever" moment. $= \frac{\alpha^2 wb^2}{8} + \frac{(1-2\alpha) wb^2}{8} = \frac{(1-\alpha)^2 wb^2}{8}$ =1.52KNm

Now, the ratio of negative to positive moments in the x-direction middle strip is: $\frac{m_{xs}}{m_{xf}} = \frac{1-2\alpha}{\alpha^2}$

Moment in the X-Direction Edge Strips:

Note that they are one half of those in the middle strips because load is half.

Mxf= Mxf middle strip/2=0.51/2=0.26KNm

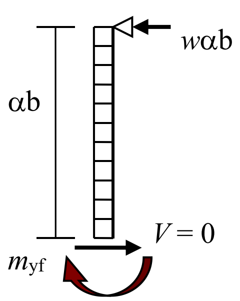
Mxs= Mxs middle strip/2=1.01/2=0.51KNm

Moment in the Y- Direction Middle Strips: Section B-B

It is reasonable to choose the same $m_{xs}/m_{xf} = 2$ in the y- direction as in the x- direction before,

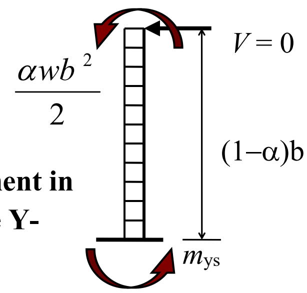
where : $\frac{m_{xs}}{m_{xf}} = \frac{1-2\alpha}{\alpha^2}$, $m_{xs}/m_{xf} = 2$, $\alpha = 0.366$

→ Choose the distance from the right support to maximum moment section as αb [the Cantilever span = (1- α)b ⇒ $m_{ys} = (1-2\alpha)wb^2/2$]

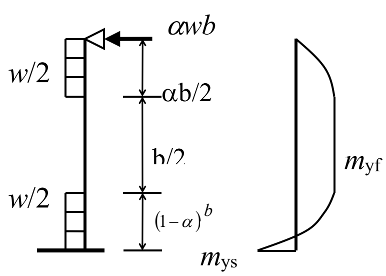


$$m_{yf} = \alpha^2 \frac{wb^2}{2} = 0.366^2 * \frac{15.38*1.4^2}{2} = 2.02 KNm$$

Moment in the Y- Direction Edge Strips:



$$m_{ys}=(1-2\alpha)\frac{wb^2}{2}=(1-2*0.366)*\frac{15.38*1.4^2}{2}=4.04KNm$$



$$m_{yf}=\frac{w\alpha b}{2}\left(\frac{\alpha b}{2}\right)-\frac{w}{2}\frac{\alpha b}{2}\frac{\alpha b}{2}=\frac{w(\alpha b)^2}{16}=0.25KNm$$

Cantilever moment

$$m_{ys}=\frac{w}{2}(1-\alpha)\frac{b}{2}.(1-\alpha)\frac{b}{4}-\alpha^2\frac{wb^2}{16}$$

$$=(1-\alpha)^2\frac{wb^2}{16}-\alpha^2\frac{wb^2}{16}=(1-2\alpha)\frac{wb^2}{16}=0.51KNm$$

One-eighth of those in y- direction middle strip

Cantilever Slab, C-14

- ✓ W=Ed=15.38KN/m2
- ✓ W/2=Ed/2=7.69KN/m2
- ✓ a=1.4m b=1.35m b is the smaller of the two span
- ✓ Assume support moment/span moment $m_{xs}/m_{xf}=2$
- ✓ α= 0.366 chosen in this range 0.35 ≤ ≤ 0.39 corresponding with negative to positive moment ratio from 2.5 to 1.5 range recommended by Hillerborg.
- ✓ α should be less than 0.5.

The edge strips widths is greater than b/4 at fixed end & less than b/4 at simple end.

Higher values should be chosen for longitudinal strips that are largely unloaded & in such cases m_{xs}/m_{xf} of 3 to 4 may be used. However A_{smin} may govern this high ratio with too small positive moment.

Moment in the X-Direction Edge Strips:

Note that they are one half of those in the middle strips because load is half.

Mxf= Mxf middle strip/2=0.469/2=0.235KNm

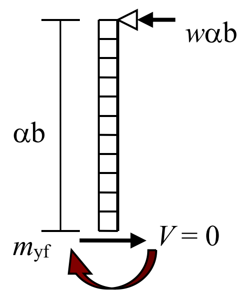
Mxs= Mxs middle strip/2=0.94/2=0.47KNm

Moment in the Y- Direction Middle Strips: Section B-B

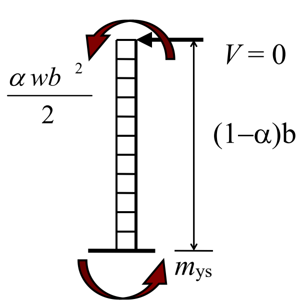
It is reasonable to choose the same $m_{xs}/m_{xf} = 2$ in the y- direction as in the x- direction before,

where : $\frac{m_{xs}}{m_{xf}} = \frac{1-2\alpha}{\alpha^2}$, $m_{xs}/m_{xf} = 2$, $\alpha = 0.366$

$\rightarrow$ Choose the distance from the right support to maximum moment section as αb [the Cantilever span = $(1-\alpha)b \Rightarrow m_{ys} = (1-2\alpha)wb^2/2$]

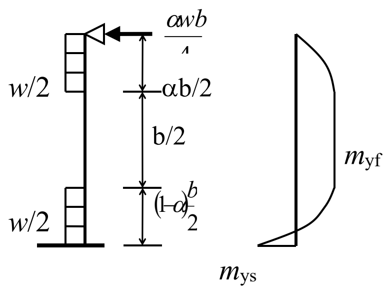


$$m_{yf} = \alpha^2 \frac{wb^2}{2} = 0.366^2 * \frac{15.38*1.35^2}{2} = 1.88KNm$$



$$m_{ys} = (1-2\alpha)\frac{wb^2}{2} = (1-2*0.366)*\frac{15.38*1.35^2}{2} = 3.76KNm$$

Moment in the Y-Direction Edge Strips:



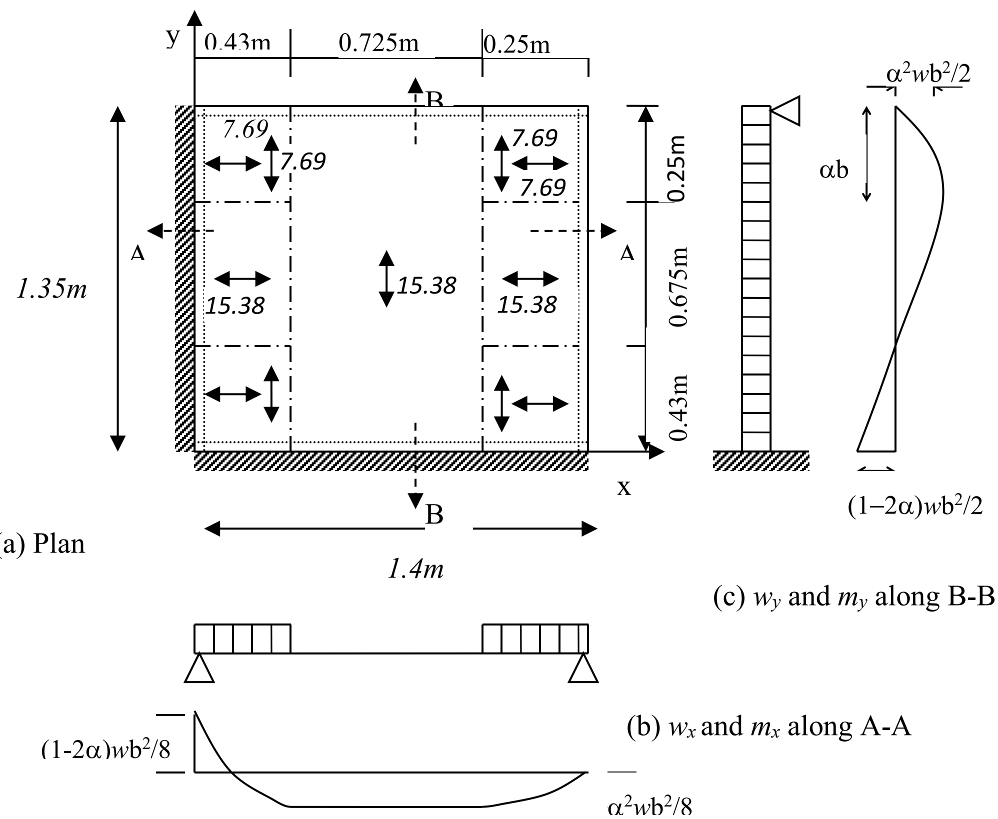
$$m_{yf} = \frac{w\alpha b}{2}\left(\frac{\alpha b}{2}\right) - \frac{w}{2}\frac{\alpha b}{2}\frac{\alpha b}{2} = \frac{w(\alpha b)^2}{16} = 0.24\,KNm$$

Cantilever moment

$$m_{ys} = \frac{w}{2}(1-\alpha)\frac{b}{2}.(1-\alpha)\frac{b}{4} - \alpha^2\frac{wb^2}{16}$$

$$= (1-\alpha)^2\frac{wb^2}{16} - \alpha^2\frac{wb^2}{16} = (1-2\alpha)\frac{wb^2}{16} = 0.47\,KNm$$

One-eighth of those in y- direction middle strip



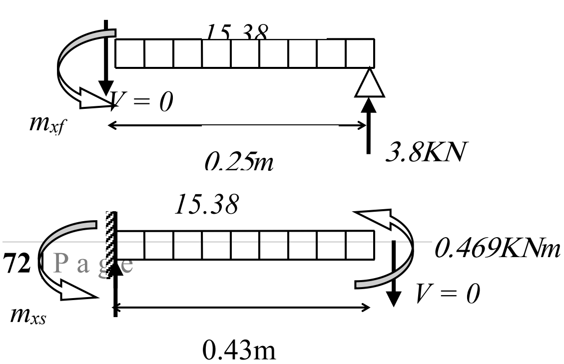
(a) Plan

(c) w_y and m_y along B-B

(b) w_x and m_x along A-A

Moment X-Direction Middle Strips: Section A-A

Positive moment in the span



$$m_{xf} = \frac{\alpha^2 wb^2}{8} = \frac{0.366^2 * 15.38 * 1.35^2}{8} = 0.469KNm$$

Negative moment at the left support $m_{xs} = \frac{(1-2\alpha)wb^2}{8} = \frac{(1-2*0.366)*15.38*1.35^2}{8} = 0.94KNm$

Observing, the absolute of the negative moment at a support plus the span moment= the "cantilever" moment. $= \frac{\alpha^2 wb^2}{8} + \frac{(1-2\alpha)wb^2}{8} = \frac{(1-\alpha)^2 wb^2}{8}$ =1.41KNm

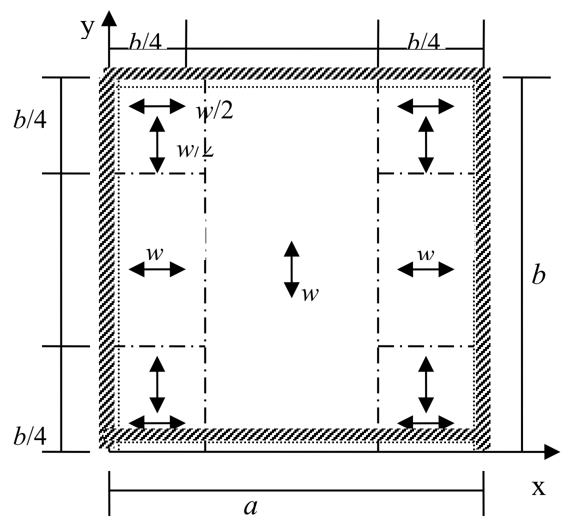
Now, the ratio of negative to positive moments in the x-direction middle strip is: $\frac{m_{xs}}{m_{xf}} = \frac{1-2\alpha}{\alpha^2}$

Analysis of two way slab using strip method

The two wan slab in which the partition dead load is greater than 10% of the self-weight of that slab is designed by strip method as follow. From our floor structural layout S1,S2,S3,S4,S5,S6,S7,S8 ,are designed by strip method observing their support condition.

Analysis of S2 and S3 Analysis

- ✓ Both are rectangular floor continuous in all four sides
- ✓ Ly=5.6m,Lx=5.1m therefore, b=5.1m
- ✓ Design load w=Ed=16.88KN/m2 at middle strip
- ✓ Design load w/2=Ed/2=8.44KN/m2 around corner strip



Rectangular slab with discontinuity lines parallel to the sides

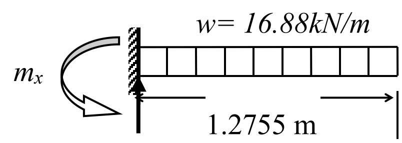
Figure 3. 3 strip division for rectangular slab fixed in all sides

<u>Strips in the slabs</u>

- ✓ Edge strip width = b/4 = 5.1/4 = 1.275m
- ✓ corners strips are loaded 50% and middle strips are loaded 100%
- ✓ A ratio of support moment to the span moment of 2 is used
- ✓ Negative moment=2/3*cantilever moment, positive moment=1/3* cantilever moment along X middle and edge strip and Y edge strip

- ✓ Negative moment=2/3*simply supported moment, positive moment=1/3* simply supported moment along Y middle strip

<u>Calculation of moments</u>



X direction middle strip along A-A:

Cantilever moment: m_x = 16.88 * 1.275^2/2 = 13.72 kNm

Negative Moment: m_{xs} = 13.72* 2/3 = 9.15 kNm

Positive moment: m_{xf} = 13.72 * 1/3 =4.57 kNm

X direction edge strip:

Cantilever moment: m_x = 8.44* 1.275^2/2 = 6.86 kNm

Negative Moment: m_{xs} = 6.86 * 2/3 = 4.57 kNm

Positive moment: m_{xf} = 6.86* 1/3 = 2.29 kNm

Y direction middle strip along B-B:

Simply supported span moment: m_y = 16.88* 5.1^2/8 = 54.88 kNm

Negative Moment: m_{ys} = 54.88* 2/3 = 36.59kNm

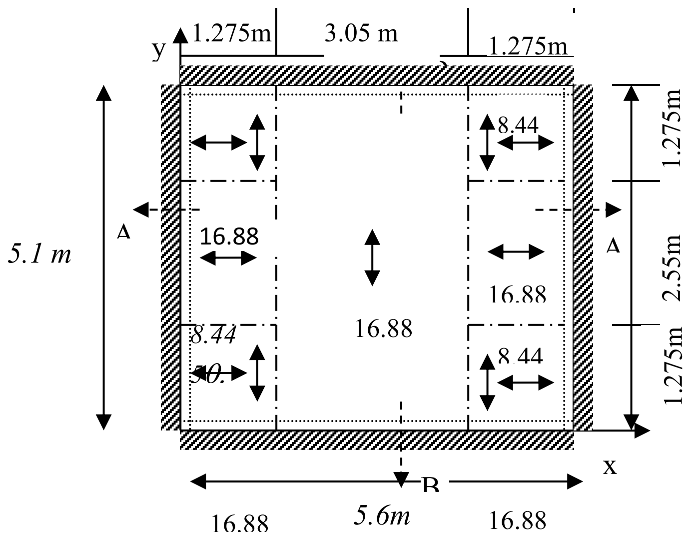
Positive moment: m_{yf} = 54.88 * 1/3 = 18.29kNm

Y direction edge strip:

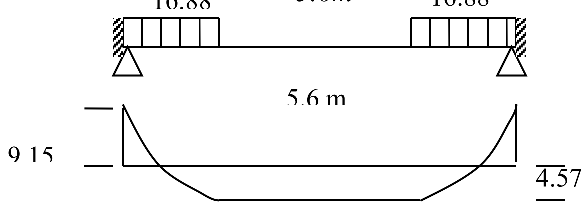
Cantilever moment: m_y = 8.44* 1.275^2/2 = 6.86kNm

Negative Moment: m_{ys} = 6.86 * 2/3 = 4.57kNm

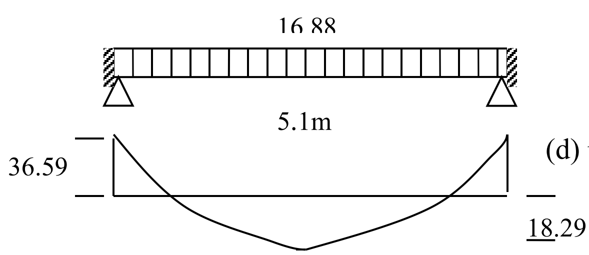
Positive moment: m_{yf} = 6.86* 1/3 = 2.29 kNm



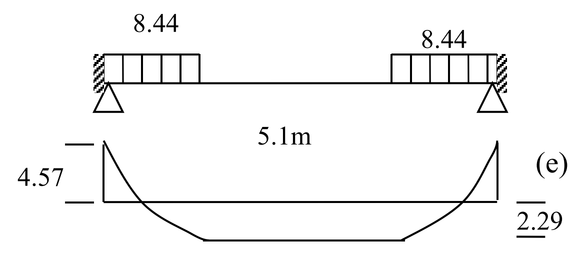
(a) Plan



(b) w_x and m_x along A-A



(d) w_y and m_y along B-B



(e) w_x and m_x, w_y and m_y along Edge strip

Analysis of S6 and S7 Analysis

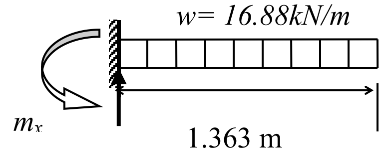
- ✓ Both are rectangular floor continuous in all four sides
- ✓ Ly=5.6m,Lx=5.45m therefore, b=5.45m
- ✓ Design load w=Ed=16.88KN/m2 at middle strip
- ✓ Design load w/2=Ed/2=8.44KN/m2 around corner strip

Strips in the slabs

- ✓ Edge strip width = b/4 = 5.45/4 = 1.363m
- ✓ corners strips are loaded 50% and middle strips are loaded 100%
- ✓ A ratio of support moment to the span moment of 2 is used
- ✓ Negative moment=2/3*cantilever moment, positive moment=1/3* cantilever moment along X middle and edge strip and Y edge strip
- ✓ Negative moment=2/3*simply supported moment, positive moment=1/3* simply supported moment along Y middle strip

Calculation of moments

X direction middle strip along A-A:



Cantilever moment: m_x = 16.88 * $1.363^2/2$ = 15.68 kNm

Negative Moment: m_{xs} = 15.68 * 2/3 = 10.45 kNm

Positive moment: m_{xf} = 15.68 * 1/3 = 5.23 kNm

X direction edge strip:

Cantilever moment: m_x = 8.44* $1.363^2/2$ = 7.84 kNm

Negative Moment: m_{xs} = 7.84 * 2/3 = 5.23 kNm

Positive moment: m_{xf} = 7.84* 1/3 = 2.61 kNm

Y direction middle strip along B-B:

Simply supported span moment: m_y = 16.88* $5.45^2/8$ = 62.67 kNm

Negative Moment: m_{ys} = 62.67* 2/3 = 41.78kNm

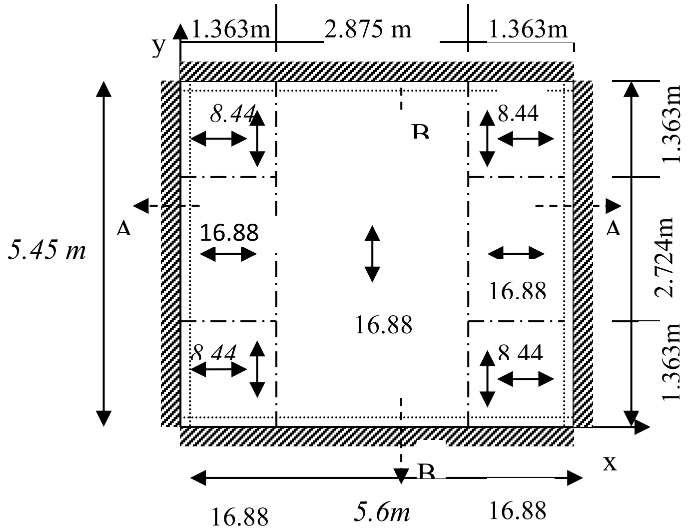
Positive moment: m_{yf} = 62.67 * 1/3 = 20.89kNm

Y direction edge strip:

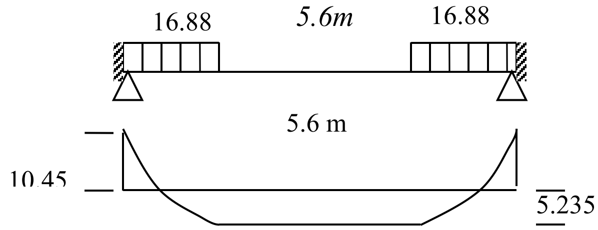
Cantilever moment: m_y = 8.44* $1.363^2/2$ = 7.84kNm

Negative Moment: m_{ys} = 7.84 * 2/3 = 5.23 kNm

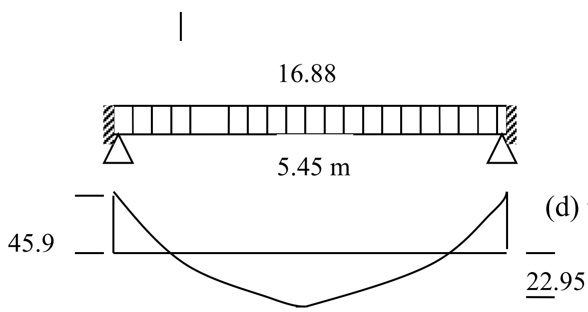
Positive moment: m_{yf} = 7.84* 1/3 = 2.61 kNm



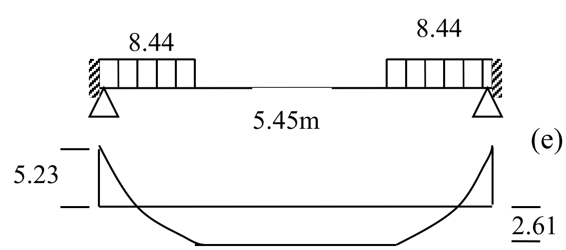
(a) Plan



(b) w_x and m_x along A-A



(d) w_y and m_y along B-B



(e) w_x and m_x, w_y and m_y along Edge

Analysis of slab S1 and S4

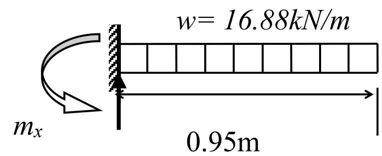
- ✓ Both are rectangular floor continuous in all four sides
- ✓ Ly=5.1m,Lx=3.8m,therefore, b=3.8m
- ✓ Design load w=Ed=16.88KN/m2 at middle strip
- ✓ Design load w/2=Ed/2=8.44KN/m2 around corner strip

<u>Strips in the slabs</u>

- ✓ Edge strip width = b/4 = 3.8/4 = 0.95m
- ✓ corners strips are loaded 50% and middle strips are loaded 100%
- ✓ A ratio of support moment to the span moment of 2 is used
- ✓ Negative moment=2/3*cantilever moment, positive moment=1/3* cantilever moment along X middle and edge strip and Y edge strip
- ✓ Negative moment=2/3*simply supported moment, positive moment=1/3* simply supported moment along Y middle strip

<u>Calculation of moments</u>

X direction middle strip along A-A:



Cantilever moment: m_x = 16.88 * $0.95^2/2$ = 7.62 kNm

Negative Moment: m_{xs} = 7.62 * 2/3 =5.08 kNm

Positive moment: m_{xf} = 7.62 * 1/3 = 2,54 kNm

X direction edge strip:

Cantilever moment: m_x = 8.44* $0.95^2/2$ = 3.81 kNm

Negative Moment: m_{xs} = 3.81 * 2/3 = 2.54 kNm

Positive moment: m_{xf} = 3.81* 1/3 = 1.27 kNm

Y direction middle strip along B-B:

Simply supported span moment: m_y = 16.88* $3.8^2/8$ = 30.47 kNm

Negative Moment: m_{ys} = 30.47* 2/3 =20.32kNm

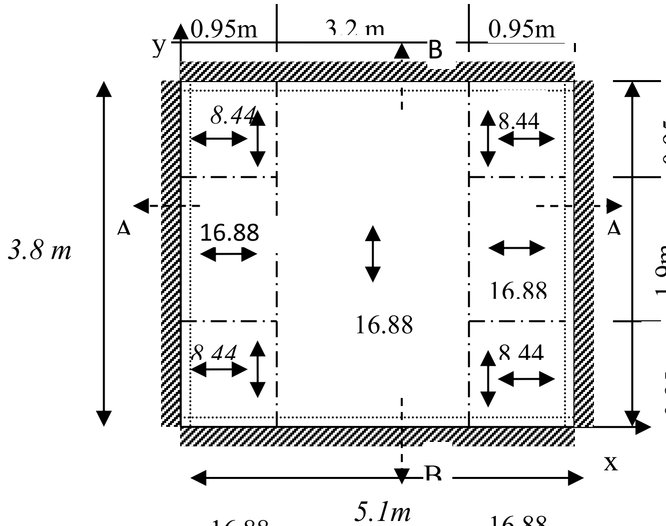
Positive moment: m_{yf} = 30.47 * 1/3 = 10.16kNm

Y direction edge strip:

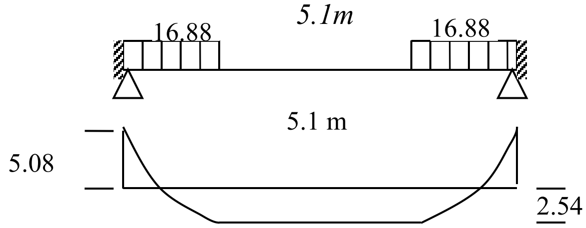
Cantilever moment: m_x = 8.44* $0.95^2/2$ = 3.81 kNm

Negative Moment: m_{xs} = 3.81 * 2/3 = 2.54 kNm

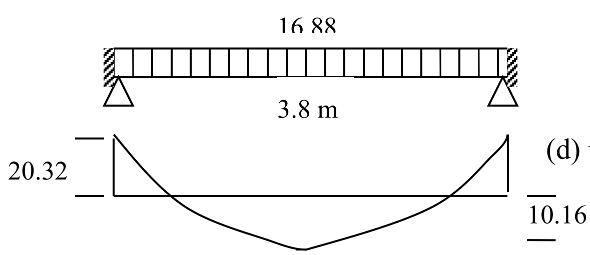
Positive moment: m_{xf} = 3.81* 1/3 = 1.27 kNm



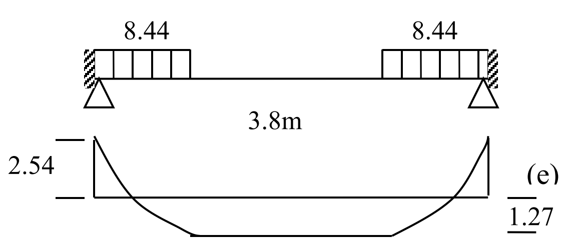
(a) Plan



(b) w_x and m_x along A-A



(d) w_y and m_y along B-B



(e) w_x and m_x, w_y and m_y along Edge strip

Rectangular Slab with free edge in short span direction

Consider a strip along A-A in the x direction. Summing moments about the left end, with unknown support moment m_{xs},

$$m_{xs} + \frac{wb^2}{32} - \frac{kwb}{4}\left(a - \frac{b}{8}\right) = 0 \qquad \text{From which, } k = \frac{1 + 32 m_{xs}/wb^2}{8(a/b) - 1}$$

Once m_{xs} is selected and k is known, it is easily shown that maximum span moment occurs when,

$$X = (1 - k)*b/4$$

And it has a value, $m_{xf} = \frac{kwb^2}{32}\left(\frac{8a}{b} - 3 + k\right)$

The moments in the x direction edge strips are one-half of those in the middle strip.

Y- Direction middle strip along C-C: Simply supported span Moment = $wb^2/8$

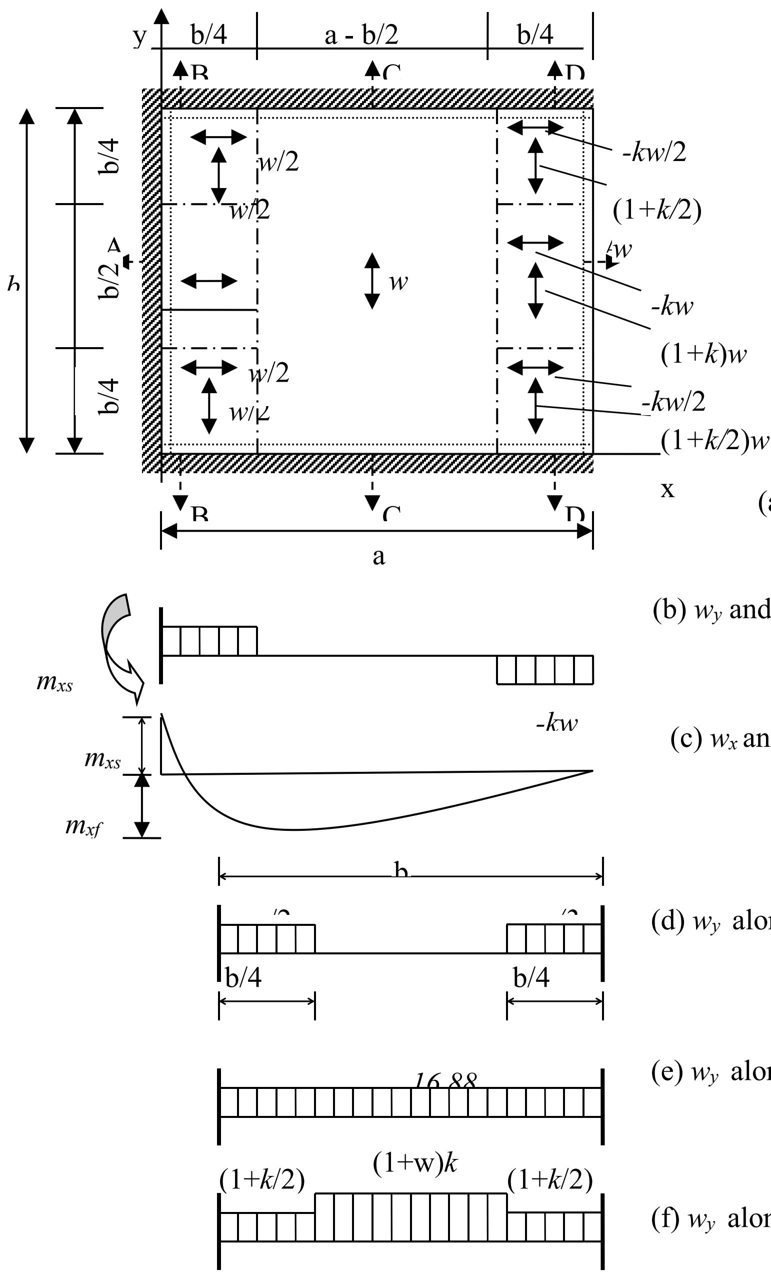
Adopting a ratio of support to span moment of 2,

$$m_{xs} = \frac{wb^2}{12} \qquad \& \qquad m_{xf} = \frac{wb^2}{24}$$

Moments along sections B-B and D-D can also be found by the same principles for the corresponding load values, with appropriate ratios of negative and positive moments

For Strip along A-A in the x direction, Mxs/Mxf=2

- ✓ $m_{xs} = \frac{wb^2}{12} = \frac{16.88*3.8^2}{12} = 20.31 KNm$
- ✓ $k = \frac{1 + 32 m_{xs}/wb^2}{8(a/b) - 1} = \frac{1 + 32*20.31/16.88*3.8^2}{8(5.45/3.8) - 1}$ =**0.35**
- ✓ $m_{xf} = \frac{wb^2}{24} = \frac{16.88*3.8^2}{24} = 40.61 KNm$
- ✓ X= (1 – k)*b/4=(1 – 0.35)*3.8/4=0.6175m



(a) Plan

(b) w_y and m_y along B-B

(c) w_x and m_x along A-A

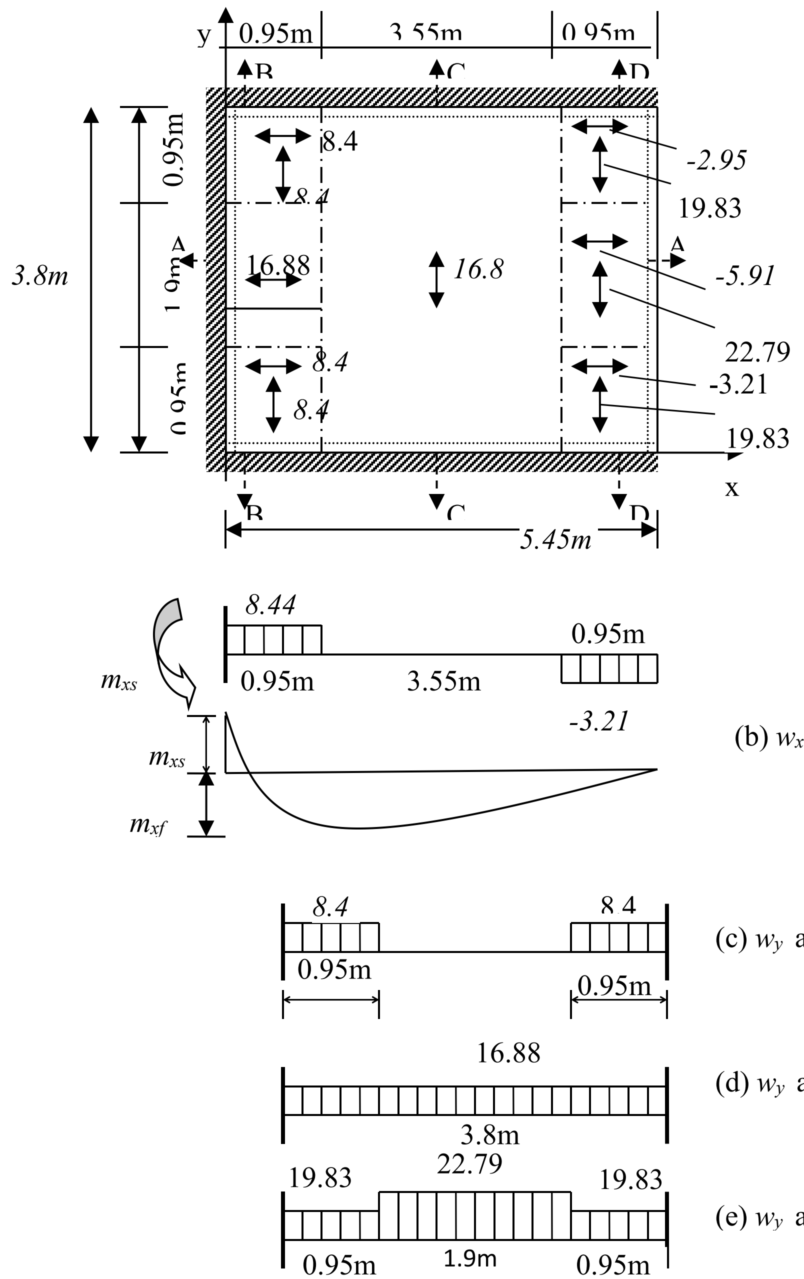
(d) w_y along B-B

(e) w_y along C-C

(f) w_y along D-D

Analysis of slab S5 and S8

- ✓ Both are rectangular floor discontinuous in one of the shorter sides
- ✓ Ly=5.45m,Lx=3.8m,therefore, b=3.8m
- ✓ Design load w=Ed=16.88KN/m2 at middle strip
- ✓ Design load w/2=Ed/2=8.44KN/m2 around corner strip



(a) Plan

(b) w_x and m_x along A-A

(c) w_y along B-B

(d) w_y along C-C

(e) w_y along D-D

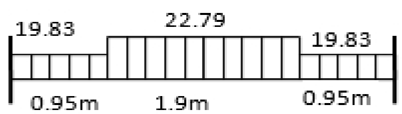
.

Y-Edge Strip moments

A. wy along B-B

- ✓ Cantilever moments-8.44*0.95^2/2=3.81KNm
- ✓ Negative moment, Mys=2/3*3.81=2.54KNm
- ✓ Positive moment, Myf=1/3*3.81=1.27Nm

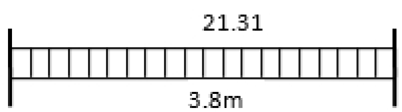
B**.** wy along D-D



By using weight average method

$= \frac{(19.83*0.95) + (22.79*1.9) + (19.83*0.95)}{0.95+1.9+0.95} = 21.31$ KN/m

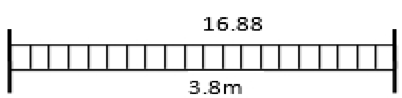
Now treat it as simply supported beam and find the moments



- ✓ Simply supported moment=wl^2/8=21.31*3.8^2/8=38.5KNm
- ✓ Negative moment, Mys=2/3*38.5=25.67KNm
- ✓ Positive moment, Myf=1/3*38.5=12.83KNm

Y-Middle Strip moments

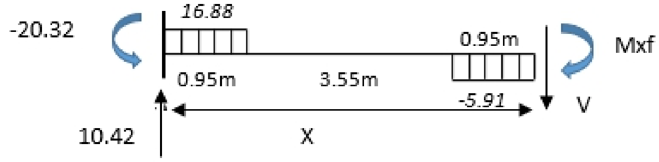
A. wy along C-C



- ✓ Simply supported moment=wl^2/8=16.88*3.8^2/8=30.5KNm
- ✓ Negative moment, Mys=2/3*30.5=20.34KNm
- ✓ Positive moment, Myf=1/3*30.5=10.17KNm

X-Middle Strip moments along A-A

Mxs=16.88*0.95^2/2-5.91*0.95*4.975= -20.32KNm



Let we find maximum span moment at X distance from support to far end load.

Mx+20.32+10.42*X-16.04*(X-0.95/2)+5.91*(X-4.5)*(X-4.975) =0

$Mx=-5.91x^2+61.62x-160.24$

To get location for max positive moment differentiate Mx wrt x and equate to 0.

Vx= -11.82X+61.62   at Vx=0 moment is maximum

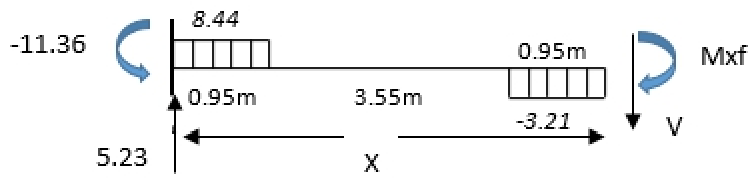
-11.82X+61.62=0, X=5.21m

Mxf @ x=5.21m

Mxf= $-5.91x^2+61.62X-160.24$= **0.38KNm**

X-Edge Strip moments bottom corner strip

Mxs=$8.44*0.95^2/2-3.21*0.95*4.975$= **-11.36KNm**



Mx+11.36+5.23*X-8.02*(X-0.95/2)+3.21*(X-4.5)*(X-4.975) =0

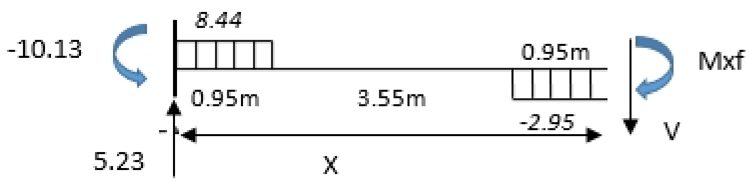
$Mx=-3.21x^2+33.21X-87$

Vx= -6.42X+33.21, X=5.17m at Vx=0

Mxf = $-3.21x^2+33.21X-87$= **1.1KNm**

X-Edge Strip moments top corner strip

Mxs=$8.44*0.95^2/2-2.95*0.95*4.975$= -10.13KNm



Mx+10.13+5.23*X-8.02*(X-0.95/2)+2.95*(X-4.5)*(X-4.975) =0

$Mx+2.91X^2-30.36X+79.1=0$

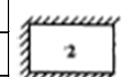
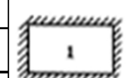
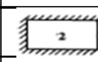
Vx=5.82X-30.36 at Vx=0, X=5.22m

Mxf @ x=5.22m Mxf= $-2.91X^2+30.36X-79.1$, Mxf=0.087KNm

3.5 Support Moment Adjustment between adjacent slabs

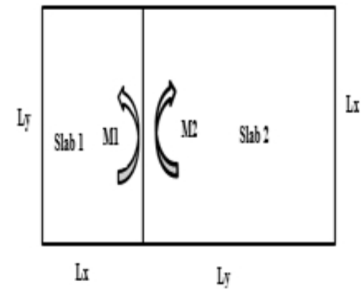
Table unadjusted support and span moment from coefficient method of analysis

Table 1. 20 Table 3. 3 Moment coefficient for two way slab

		moment coffiecients					moment value ,KNm				
Slab N.	Ly/Lx	axs	axf	ays	ayf	Ed	Mxs	Mxf	Mys	Myf	S.type
S9	1.13	0.045	0.034	0.039	0.029	18.94	20.98	15.78	18.1	13.46	
S10	1.13	0.045	0.034	0.039	0.029	18.94	20.98	15.78	18.1	13.46	
S11	1.24	0.05	0.037	0.039	0.029	18.15	12.99	9.69	10.22	7.6	
S12	1.19	0.042	0.032	0.032	0.024	18.86	17.5	13.33	13.33	10	
S13	1.19	0.042	0.032	0.032	0.024	18.86	17.5	13.33	13.33	10	
S14	1.24	0.05	0.037	0.039	0.029	18.15	12.99	9.69	10.22	7.6	
C6	2.53	-	-	-	-	15.38	27.65	-	-	-	-
C7	2.07	-	-	-	-	15.38	75.59	-	-	-	-
C8	2.07	-	-	-	-	15.38	75.59	-	-	-	-
C9	2.53	-	-	-	-	15.38	27.65	-	-	-	-
C11	3.36	-	-	-	-	17.75	27.11	-	-	-	-
C12	3.89	-	-	-	-	17.20	27.11	-	-	-	-
C13	3.64	-	-	-	-	17.81	27.11	-	-	-	-
C17	4.15	-	-	-	-	15.38	25.02	-	-	-	-
C18	4.15	-	-	-	-	15.38	25.02	-	-	-	-
C2	3.64	-	-	-	-	17.81	27.11	-	-	-	-
C3	3.89	-	-	-	-	17.81	27.11	-	-	-	-
C4	3.36	-	-	-	-	17.81	27.11	-	-	-	-

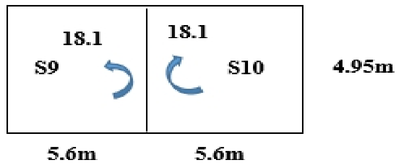
- If **ΔM=0%** no need of adjustment
- If **ΔM =100%** take larger M for design
- If **ΔM < 10%** use simple averaging
- If **ΔM≥10%**redistribute based on slab stiffness
- $\mathbf{M_L}$ is moment in the left
- $\mathbf{M_R}$ is moment on the right
- **K=/1/Lx** stiffness of slab
- $\mathbf{M_d}$ Design Moment
- **Lx** is the span of panel considered
- **K=1/Lx** stiffness of slab
- $\mathbf{M_d}$ Design Moment
- **Lx** is the span of panel considered

$$\Delta M = \frac{M1 - M2}{M1} * 100$$



Support Condition	Coeff.	Values of L_y/L_x								Long span coefficients, α_{ys} and α_{yf} for all values of L_y/L_x
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
1	α_{xs} α_{xf}	0.032 0.024	0.037 0.028	0.042 0.032	0.046 0.035	0.050 0.037	0.053 0.040	0.059 0.044	0.063 0.048	0.032 0.024
2	α_{xs} α_{xf}	0.039 0.029	0.044 0.033	0.048 0.036	0.052 0.039	0.055 0.041	0.058 0.043	0.063 0.047	0.067 0.050	0.039 0.029
3	α_{xs} α_{xf}	0.039 0.030	0.049 0.036	0.056 0.042	0.062 0.047	0.068 0.051	0.073 0.055	0.082 0.062	0.089 0.067	0.039 0.030
4	α_{xs} α_{xf}	0.047 0.036	0.056 0.042	0.063 0.047	0.069 0.051	0.074 0.055	0.078 0.059	0.087 0.065	0.093 0.070	0.047 0.036
5	α_{xs} α_{xf}	0.046 0.034	0.050 0.038	0.054 0.040	0.057 0.043	0.060 0.045	0.062 0.047	0.067 0.050	0.070 0.053	- 0.034
6	α_{xs} α_{xf}	- 0.034	- 0.046	- 0.056	- 0.065	- 0.072	- 0.078	- 0.091	- 0.100	0.045 0.034
7	α_{xs} α_{xf}	0.057 0.043	0.065 0.048	0.071 0.053	0.076 0.057	0.081 0.060	0.084 0.063	0.092 0.069	0.098 0.074	- 0.044
8	α_{xs} α_{xf}	- 0.044	- 0.054	- 0.063	- 0.071	- 0.078	- 0.084	- 0.096	- 0.105	0.058 0.044
9	α_{xf}	0.056	0.065	0.074	0.081	0.087	0.092	0.103	0.111	0.056

Sample adjustment calculation

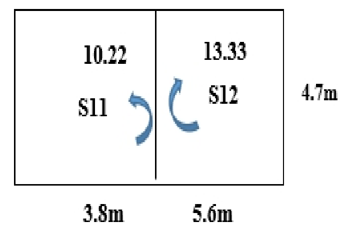
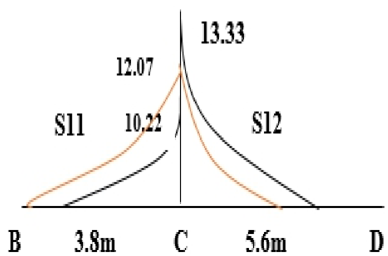


$\Delta M = \frac{18.1-18.1}{18.1} * 100 = 0\%$

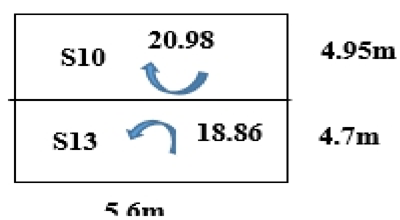
Use simple averaging, Madj=18.1KNm

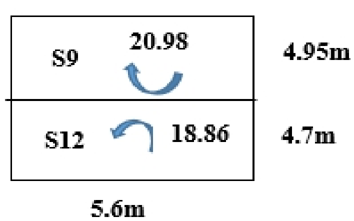
If the adjacent moment difference is >10% then the support moment is adjusted based on slab stiffness.

Table 3. 4 Moment adjustment for slab

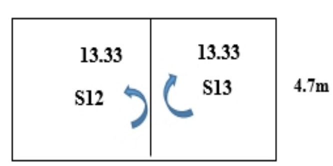
	ML	MR	ΔM	ΔM,%	10%larg M	Slab configuration
Values	10.22	13.33	3.11	23.33	1.333	
Member	BC	CD				
Length	3.8	5.6				
K	0.263	0.179				
DF	0.596	0.404				
FEM	-10.22	13.33				
Balancing	-1.853	-1.257				
∑	-12.073	12.073				
adjusted Moment, **Ms**	**12.073KNm**					
in S11 support moment is increased and in S12 support moment is decreased and span moment will increase						

Δ





ΔM

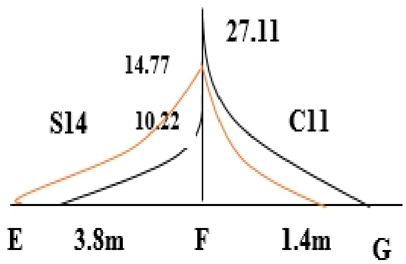


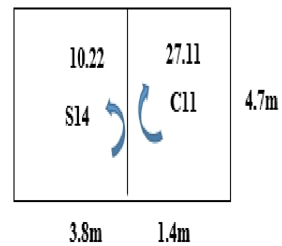
20.98−18.86

20.98−18.86

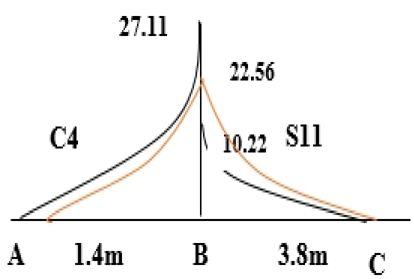
$\Delta M = \frac{13.33-13.33}{13.33} * 100 = 0\% \leq 10\%$

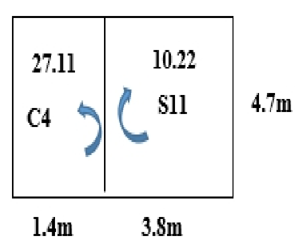
	ML	MR	ΔM	ΔM,%	10%larg M	Slab configuration
Values	10.22	27.11	16.89	62.30	2.711	
member	EF	FG				
Length	3.8	1.4				
K	0.263	0.714				
DF	0.269	0.731				
FEM	-10.22	27.11				
Balancing	-4.547	-12.343				
∑	-14.77	14.77				
adjusted Moment,**Ms**	**14.77KNm**					
S14 support moment is increased and in C11 support moment is decreased and span moment will increase						



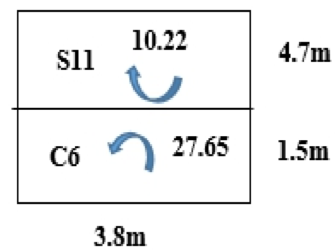
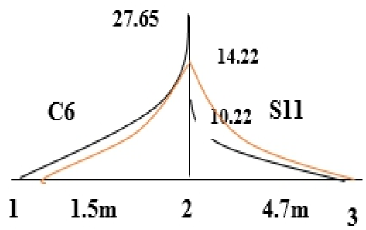


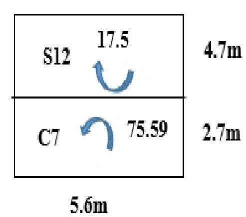
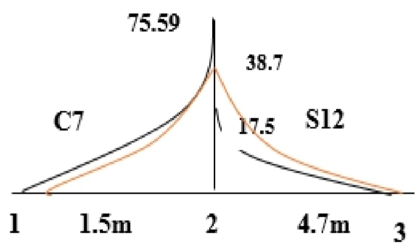
	ML	MR	ΔM	ΔM,%	10%larg M	Slab configuration
Values	27.11	10.22	16.89	62.30	2.721	
member	AB	BC				
Length	1.4	3.8				
K	0.714	0.263				
DF	0.731	0.269				
FEM	-10.22	27.11				
Balancing	-12.343	-4.547				
∑	-22.56	22.56				
adjusted Moment,**Ms**	**22.56KNm**					



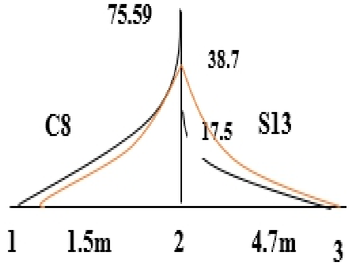


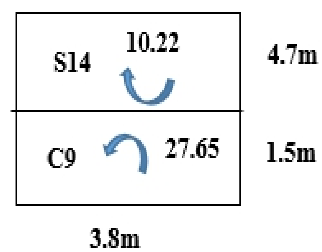
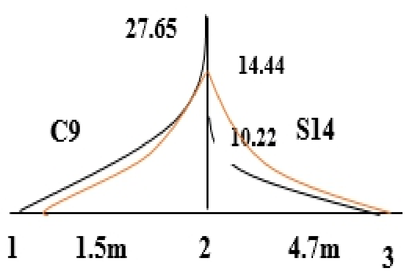
in C4 support moment is decreased and span moment will increase in S11 support moment is increased

	ML	MR	ΔM	ΔM,%	10%larg M	Slab configuration
Values	10.22	27.65	17.43	63.04	2.765	
member	12	23				
Length	1.5	4.7				
K	0.667	0.213				
DF	0.758	0.242				
FEM	27.65	-10.22				
Balancing	-13.213	-4.217				
∑	14.44	-14.44				
adjusted Moment,**Ms**	**14.44KNm**					
in C6 support moment is decreased and span moment will increase in S11 support moment is decreased						

	ML	MR	ΔM	ΔM,%	10%larg M	Slab configuration
Values	75.59	17.5	58.09	76.85	7.559	
member	12	23				
Length	2.7	4.7				
K	0.370	0.213				
DF	0.635	0.365				
FEM	75.59	-17.5				
Balancing	-36.895	-21.195				
∑	38.70	-38.70				
adjusted Moment, **Ms**	**38.7KNm**					
in C7 support moment is decreased and span moment will increase in S12 support moment is decreased						

	ML	MR	ΔM	ΔM,%	10%larg M	Slab configuration
Values	75.59	17.5	58.09	76.85	7.559	

member	12	23		
Length	2.7	4.7		
K	0.370	0.213		
DF	0.635	0.365		
FEM	75.59	-17.5		
Balancing	-36.895	-21.195		
∑	38.70	-38.70		
adjusted Moment, **Ms**		**38.7KNm**		
in C8 support moment is decreased and span moment will increase in S13 support moment is decreased				

	ML	MR	ΔM	ΔM,%	10%larg M	Slab configuration
Values	27.65	10.22	17.43	63.04	2.765	
member	12	23				
Length	1.5	4.7				
K	0.667	0.213				
DF	0.758	0.242				
FEM	27.65	-10.22				
Balancing	-13.213	-4.217				
∑	14.44	-14.44				
adjusted Moment,**Ms**	**14.44KNm**					
in C9 support moment is decreased and span moment will increase in S14 support moment is decreased						

3.5.1 Span Moment Adjustment of a panel

If the moment in the adjusted support decreases, the span moment should be increased to compensate the change in support moment. The design moment of span is calculated as

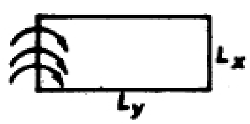
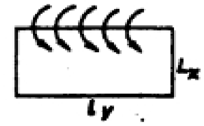
$$\mathbf{M_{xd} = M_{xf} + C_x\,\Delta M} \qquad \textbf{and} \qquad \mathbf{M_{yd} = M_{yf} + C_y\,\Delta M}$$

- ΔM is change in moment in all supports
- Cx & Cy are coefficients for adjustment of span moment (EBCS-2, 1995 Table A-2)

Change in moment	Initial moment	Adjusted span moment
ΔMxf= CxΔMxs+CxΔMys	(Mxf)i	Mxf=Mxfi+ ΔMxf

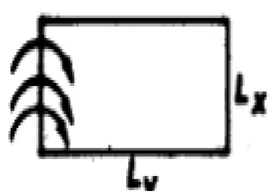
ΔMyf=CyΔMxs+CyΔMys (Myf)i Myf=Myfi+ ΔMyf

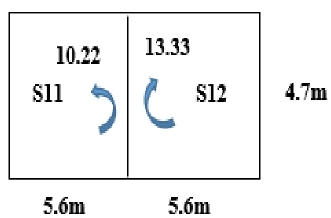
Table A-2 Factors for Adjusting Span Moments m_{xf} and m_{yf}

L_y/L_x				
	c_x	c_y	c_x	c_y
1.0	0.380	0.280	0.280	0.380
1.1	0.356	0.220	0.314	0.374
1.2	0.338	0.172	0.344	0.364
1.3	0.325	0.135	0.373	0.350
1.4	0.315	0.110	0.398	0.331
1.5	0.305	0.094	0.421	0.310
1.6	0.295	0.083	0.443	0.289
1.7	0.285	0.074	0.461	0.272
1.8	0.274	0.066	0.473	0.258
1.9	0.258	0.060	0.481	0.251
2.0	0.238	0.055	0.484	0.248

Between Slab S11 and S12

- ✓ Ly/Lx=5.6/4.7=1.19=1.2
- ✓ ΔMxs=3.11KNm
- ✓ ΔMys=58.09KNm
- ✓ Mxfi=13.33KN
- ✓ Myfi=10KNm
- ✓ Cx=0.338
- ✓ Cy=0.172





ΔMxf= CxΔMxs+CxΔMys=0.338*3.11+58.09*0.172=11KNm
ΔMyf=CyΔMxs+CyΔMys=0.172*3.11+58.09*0.338=21KNm
Mxf=Mxfi+ ΔMxf=13.33KN+11KNm=**24.33KNm**
Myf=Myfi+ ΔMyf=10KNm+21KNm=**31KNm**

The remaining support moment adjustment is done between two way slab and one way cantilever slab, where adjusted moment is decrease than the previous support moment in the cantilever is decreased which mean need span moment adjustment. But since cantilever slab has no span moment, no need of span moment adjustment.

3.5.2 Design of slab for maximum moment

Material used and geometry

- ✓ fyk= 460Mpa, fyd=400mpa
- ✓ fcu=25Mpa, fck=20mpa fcd=11.33Mpa

- ✓ d=280mm
- ✓ ϕmain bar=10mm
- ✓ $as=\frac{\pi*\phi^2}{4}=78.5mm^2$
- ✓ concrete cover is 20mm

Check adequacy of the section for flexure

For one way cantilever slab analyzed by static method taking maximum moment

b=1000mm
Msd=75.59KNm slab C7 and C8

$$d=\sqrt{\frac{Msd}{0.2952*fcd*b}}=\sqrt{\frac{75.59*10^6}{0.2952*11.33*1000}}=150mm<280mm.........OK$$

For slab analyzed by strip method taking maximum moment

b=2875mm
Msd=41.78KNm Slab 6 and slab 7

$$d=\sqrt{\frac{Msd}{0.2952*fcd*b}}=\sqrt{\frac{41.78*10^6}{0.2952*11.33*2875}}=66mm<280mm.........OK$$

For slab analyzed by coefficient method, taking final adjusted max moment

b=1000mm
Msd=41.78KNm Slab 6 and slab 7

$$d=\sqrt{\frac{Msd}{0.2952*fcd*b}}=\sqrt{\frac{41.78*10^6}{0.2952*11.33*2875}}=66mm<280mm.........OK$$

Effective depth of two way slab

The effective depth of two way slab in two orthogonal direction measured from compression face to tension reinforcement center is given below

$dx= D-\frac{\phi}{2}-cover=310-\frac{10}{2}-20=285mm$

$dy= D-\frac{\phi}{2}-\phi-cover=310-\frac{10}{2}-10-20=275mm$

Where

D is total depth of the slab,

Dx is the effective depth of main reinforcement,

Dy is the effective depth of transverse reinforcement

Design for main reinforcement bar (principal reinforcement)

Design moment = Msd= 58.14KNm

- ✓ The steel ratio is calculated as follow

$$\rho=[1-\sqrt{1-\frac{2Mu}{fcdbd^2}}]\frac{fcd}{fyd}$$

$\therefore \rho = [1-\sqrt{1-\frac{2*41.78*10^6}{2875*280^2*11.33}}]\frac{11.33}{400} = 0.00047$

- ✓ The minimum geometric steel ratio of main reinforcement in slab shall not be less than $\rho_{min}=\frac{0.5}{fyk}=\frac{0.5}{460}=0.001086$

  Since $\rho_{min} > \rho$, 0.001086 >0.00047and proceed area of steel calculation using value of ρ_{min}
- ✓ Area of steel is calculated as As=ρmin*b*d   b=2875mm, strip of slab

  ∴ Area of steel ($As_{calculated}$) = ρmin bd=0.001086*2875*20=874mm^2

- ✓ spacing of reinforcement S= a_s*b/As

  ∴ $S_{calculated}=\frac{as*b}{As}=\frac{78.5*2875}{874}$=258mm,
- ✓ Smax= 400mm from the code

  So provided $S_{provided}$=150mm

Reinforcement Design for slab analyzed by coefficient method

1. **Depth check for flexure, Mu=19.92KNm**

$$d = \sqrt{\frac{Msd}{0.2952*fcd*b}} = \sqrt{\frac{19.92*10^6}{0.2952*11.33*1000}} = 77mm<280mm\text{.........OK}$$

2. **Minimum area of main reinforcement**

$$ASmin=Max\begin{cases}\frac{0.26fctm*b*d}{fyd}\\0.0013b*d\end{cases}$$

fctm=2.12*ln(1+$\frac{fcm}{10}$)

fcm=fck+8Mpa=20+8=28Mpa

fctm=2.2 for C25/20 from concrete strength class table

$$AS_{min}=Max\begin{cases}\frac{0.26*2.2*1000*280}{400} = 245.12mm^2\\0.0013bt*d=0.0013*1000*149.05=193.77mm^2\end{cases}$$

Asmin=254.12mm^2

3. **Maximum area of main reinforcement**

The maximum area of reinforcement according to ES EN 1992-1-1:2015 section 9.2.1.1(3) is

As,max=0.004AC, where Ac is the area of concrete.

As,max= 0.004*310mm*1000mm=1240mm^2

Since, $As_{calculated}$ > Asmax   therefore, provide $As_{provided}$=1240mm^2

Spacing for main reinforcement

$S_{calculated}=\frac{as*b}{AS\ provided}=\frac{78.5*1000}{1240}$= 63.3mm

- ✓ In areas with concentrated loads or maximum moment those provisions become respectively

$$S_{max} = min\begin{cases} 2 * 310 = 620mm \\ 250mm \end{cases} \rightarrow for\ principal\ reinforcement$$

Therefore, Smax= 250mm for primary reinforcement

$S_{provided}$= 150mm

Since $S_{provided}$<Smax, 150mm<250mm, so provide C/C spacing of primary reinforcement Sprovided=150mm

4. Secondary Transverse Reinforcement

According to ES EN 1992-1-1:2015 section 9.3.1.1 Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slabs. Ast=20%As, provided.

Ast=0.2∗1240mm^2 =248mm^2

$Ast_{provided}$=248mm^2

Spacing for secondary reinforcement

- ✓ $S_{calculated} = \frac{as*b}{AS\ provided} = \frac{78.5*1000}{248}$= 316.5mm
- ✓ $S_{max} = min\begin{cases} 3D = 3 * 310 = 930mm \\ 400mm \end{cases} \rightarrow for\ secondary\ reinforcement$
- ✓ **Therefore, Smax=400mm**
- ✓ **Therefore $S_{provided}$=300mm**

Since, $S_{calculated}$ >Smax, 930mm>400mm, provide C/C spacing of secondary reinforcment little bit less than Smax. Therfore provide Sprovided 300mm for secondary reinforcment

Therefore, provide Ø10 C/C 150mm for main reinforcement bar and provide Ø10 C/C 300mm for secondary reinforcement bar.

5 Defelection Check

$\Delta max = \frac{5wl^4}{384EI}$, deflection is checked based on unfactored service load

- ✓ **Lx=4.95mm for slab S9**
- ✓ W =Edservice=13.7KN/m
- ✓ $I = \frac{bD^3}{12} = \frac{1000*310^3}{12}$ = 2.5*$10^{12}$$mm^4$
- ✓ Ecm=22(fcm/10)$^{0.3}$=22*(28/10)$^{0.3}$=30GPa
- ✓ $\Delta max = \frac{5*13.7*4.91^4}{384*30*10^6*2.5*10^{12}}$ = 0.01438mm
- ✓ Allowable deflection as per EBCS-1995 code, $\delta = \frac{Le}{200} = \frac{4950mm}{250}$ = 19.8mm
- ✓ Δmax = 0.01438mm<δ= 19.8mm…………..ok

The depth,spacing,deflection and area of reinforcment for all floors analysed by static strip and cofficient method are calculated and checked in excel table attached.

3.6 Reinforcment Detailing of slabs

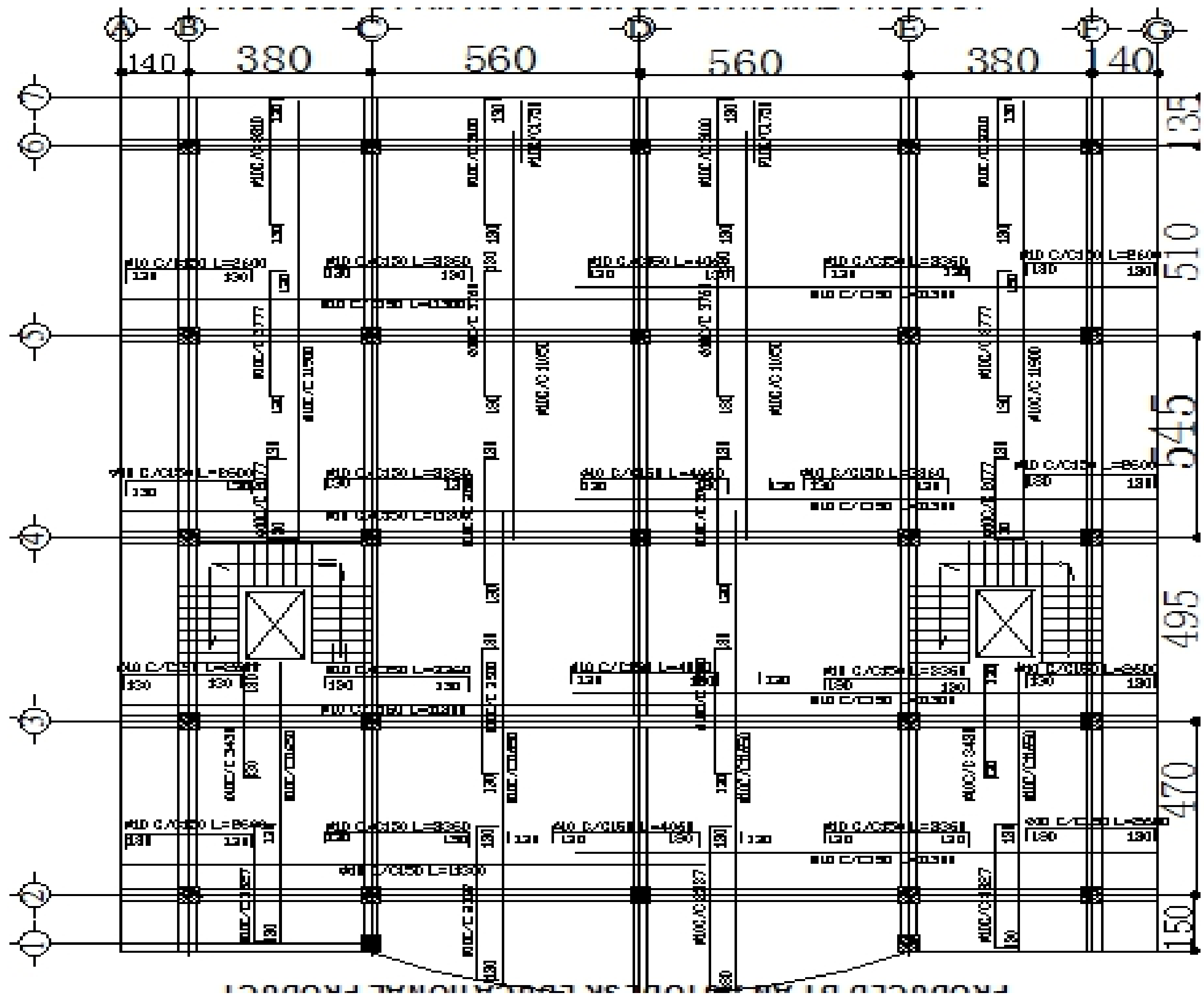
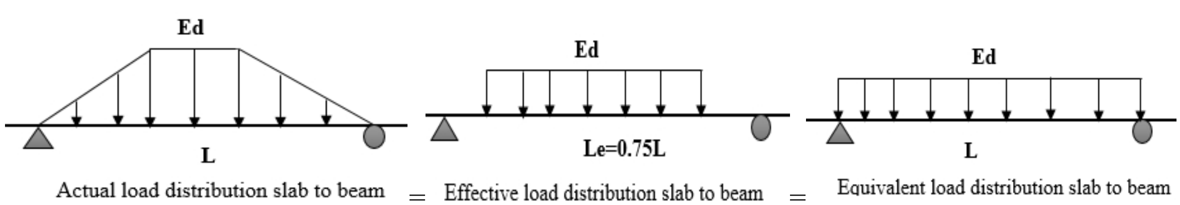
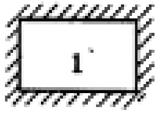
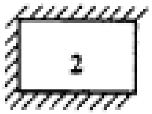
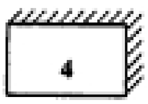
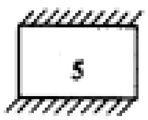
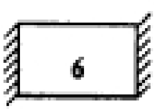
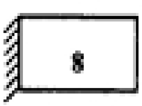
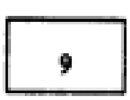


Figure 3. 4 Reinforcement detailing of slabs

3.7 Load transfer from slab to beam.

- ✓ The load from the slab to beam is assumed to be distributed over 75% of beam length, (Le=0.75L), in the form of triangular or trapezoidal patterns but to make it fully distributed beam span it should be multiplied by *k=0.92*



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

4 Analysis And Design of Stair Case

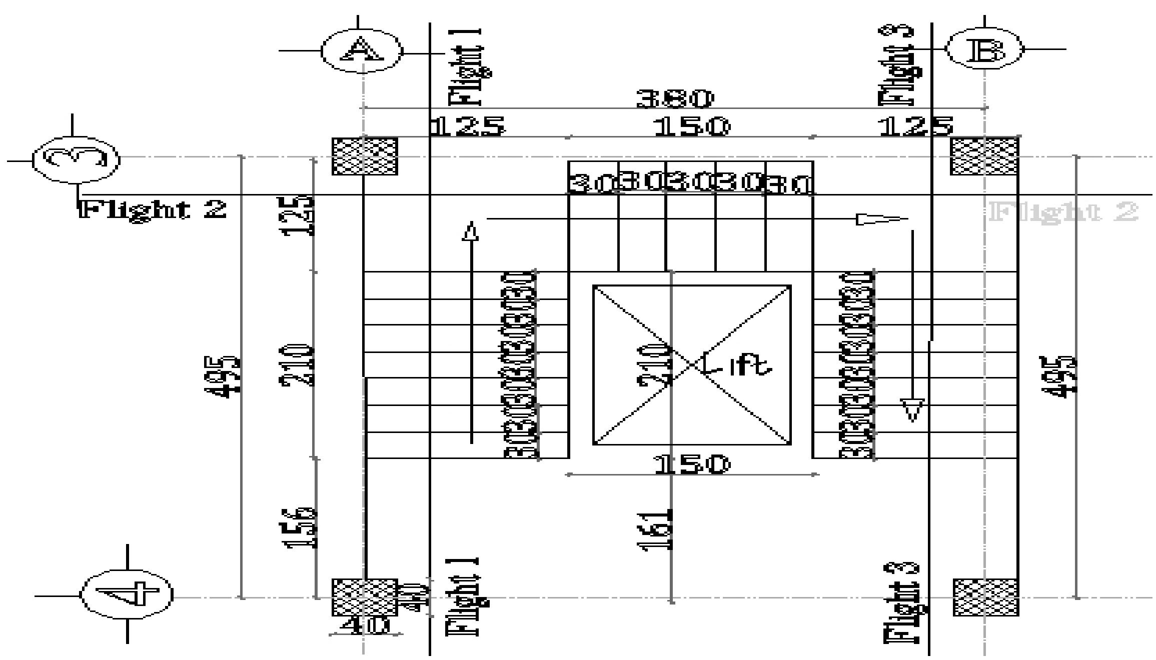
4.1 Introduction

In this typical building a dog legged stair with two landing between floors are given. Keeping the material and floor height similar, we can consider three possible stair for analysis and design based on their support condition.

- **Case A:** Stair case connecting basement floor to ground floor
- **Case B:** Stair case connecting typical floors
- **Case C:** Stair case connecting the last floor and floor below it

▪ Design procedures

a) 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
b) Loading: which determines the total load in the stair and landing
c) Analysis: determines moment and shear forces based on the analyzed moment
d) 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.
e) Reinforcement provision: using the computed moments, number and area of reinforcement bars determined.
f) Detailing: the arrangement of reinforcement.



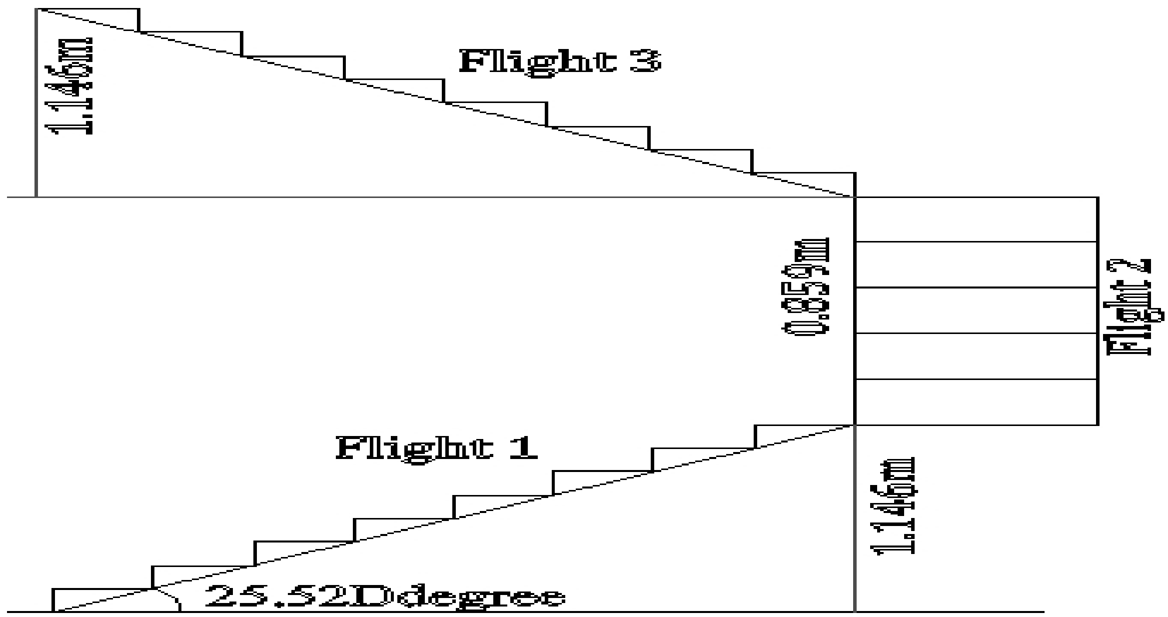


Figure 4. 1 plan and cross section layout of stair case

4.2 Analysis of staircase case geometry

Number of riser (N) for flight 1= (tread + 1) = (7 + 1) = 8

Number of riser (N) for flight 2= (tread + 1) = (5 + 1) = 6

Number of riser (N) for flight 3 = (tread + 1) = (7 + 1) = 8

The height of each typical floor = 3.15m

Number of riser N_R=8+8+6=22

Number of tread,N_T= 7+7+5=19

Height of riser = $\frac{H}{N} = \frac{3.15}{22}$ = 14.32cm

Inclination of stair case, θ=$\tan^{-1}(\frac{14.32}{30})$ = 25.52°

Check $N_T + N_R \leq 45$ 19+22≤45=41≤45…………OK

$N_T + 2N_R$ = 19+2*22=61 should be between 50 and 70 hence …………OK

Width of flight/stair (W) =1.25m

Height of flight 1,H=1.146m

Height of flight 2, H=0.859m

Height of flight 3, H=1.146m

Depth calculation from deflection requirement

We have two possible ways for depth calculation

$\frac{l}{d}$=k[11+1.5$\sqrt{fck}*\frac{\rho o}{\rho}$ +3.2$\sqrt{fck}*(\frac{\rho o}{\rho}-1)^{3/2}$], $if\ \rho \le \rho o$

$\frac{l}{d}$=k[11+1.5$\sqrt{fck}*\frac{\rho o}{\rho - \rho'}+\frac{1}{12}\sqrt{fck}*(\frac{\rho'}{\rho})^{3/2}$] ,$if\ \rho > \rho o$

The above equation is given for f_{yk} =500Mpa and C30/37 and if a steel grade other than 500mpa is used the above two formula is multiplied by ($\frac{500}{f_{yk}\left(\frac{A_{st.req}}{A_{st,prov}}\right)}$) to obtained the modified depth.

Let concrete grade fcu=C25, fck=20Mpa

Let steel grade fyk=400mpa

$p_o = \sqrt{f_{ck}} * 10^{-3} = \sqrt{20} * 10^{-3}$ =0.0045≈0.005 reference reinforcement ratio,

ρ = 0.5% =0.005 For lightly stressed concrete

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

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

K=1.5, for interior span, factor to account different structural system, Table 7.4N

$$\frac{l}{d} = 1.5\left[11 + 1.5\sqrt{20} * \frac{0.005}{0.005} + 3.2\sqrt{20} * \left(\frac{0.005}{0.005} - 1\right)^{\frac{3}{2}}\right] * \frac{500}{400}$$

$$\frac{l}{d} = 33.21$$

For flight -1 (L= 4.95m), d = $\frac{4.95}{33.21}$ = 149.05mm

For flight -2 (L= 3.8m), d = $\frac{3.8}{33.21}$ = 114.44mm

For flight -3 (L= 4.95m), d = $\frac{4.95}{33.21}$ = 149.05mm

Taking the maximum deflection depth

$$d = \text{Max}\begin{cases}149.05mm\\114.44mm\\149.05mm\end{cases}$$

Therefore, flight 1 gives governing depth, d = 149.05mm, Then

$D=d+\frac{1}{2}Ø+cover$

Where d=149.5mm, Ø=10, assume $cover$=20mm similar to slab

D=149.5mm + $\frac{1}{2}$*10mm +20mm =174.5mm≅175mm

4.3 Analysis of Stairs Loading

Table 4. 1 Material used with their unit weight and thickness

Material type	Unit weight(KN/m2)	Thickness (cm)
Cement screed	23	3
Concrete	25	16
Plastering	17	2
Marble	27	2
Porcelain	27	1

Calculation of dead load using 1m width Flight 3

Dead load on landing

- Landing slab = D*b* γcon = 0.175*1*25 = 4.38KN/m
- Plastering ,2cm = tp*b* γpl = 0.02*1*17 = 0.46KN/m
- Cement screed,3cm = tcs*b*γcs = 0.03*1*23 = 0.69KN/m
- Porcelain,1cm = tp*b* γp = 0.01*1*27 = 0.27KN/m
- Marble,2cm=tm*b* γm=0.02*1*27=0.54KN/m

Total Dead Load = **5.8KN/m**

And $L.L = 3kN/m^2 * 1m = 3kN/m$

Design load on landing

Ed = 1.35*Gk + 1.5Qk = 1.35*5.8 + 1.5*3 = **12.33KN/m**

Dead load on stairs

- Load on slab = $\frac{D*b*\gamma con}{Cos\theta} = \frac{0.175*1*25}{Cos25.52^{0}}$ = 4.85KN/m
- Load on step = $\frac{½*\text{riser height}*\text{width of thread}*\text{width of stair}*\gamma con*\text{number of riser}}{projected\ length}$

  $= \frac{1/2*0.1432*1*0.3*25*8}{2.1}$ =2.05KN/m
- Marble ,2cm = t* γmar = 0.02*27 = 0.54KN/m^2
- Cement screed,3cm = $tcs*\gamma cs$ = 0.03*23 = 0.69KN/m^2
- On thread = (0.54+0.69) = 1.23*1= 1.23KN/m
- On riser = $\frac{(0.54+0.69)*\text{riser height}*\text{stair width}*\text{number of riser}}{projected\ length} = \frac{(0.54+0.69)*0.1432*1*8}{2.1}$ = 0.67KN/m
- Plastering,2cm = $\frac{0.02*17*1}{Cos25.52}$= 0.38KN/m

Total dead load = 4.85+2.05+0.54+0.69+1.23+0.67+0.38 = 10.41KN/m

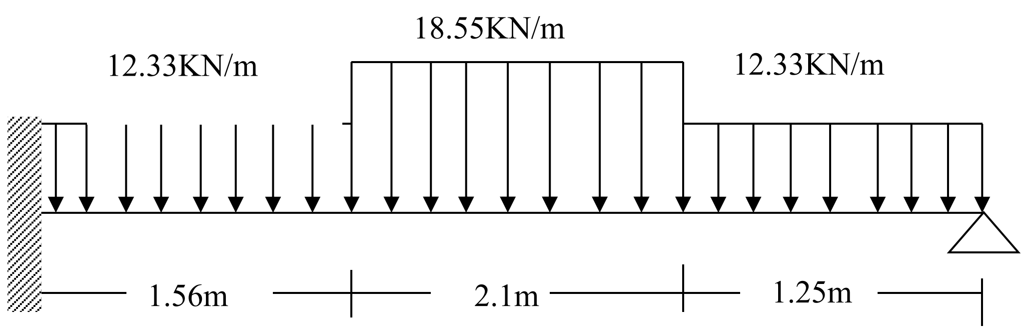
And $L.L = 3kN/m^2 * 1m = 3kN/m$

Design Load on the flight

Ed = 1.35*Gk + 1.5Qk= 1.35*10.41+ 1.5*3 =**18.55KN/m**

The analysis and design of staircase can be done like one way slab taking 1m strip along the flight

4.4 Analysis of Stair Case Responses



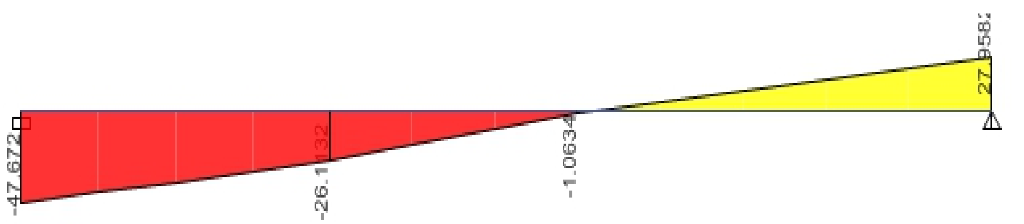


Figure 1. 16Figure 4. 2 Shear Force Diagram, SFD

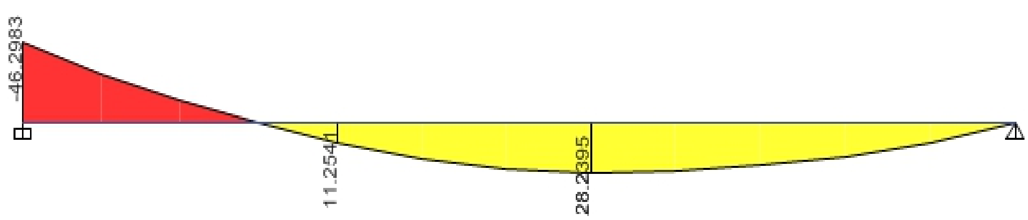


Figure 1. 17 Figure 4. 3 Bending Moment Diagram, BMD

Design shear = 47.67KN/m

Design moment(-ve) = 46.29KNm

Design moment (+ve)=28.74KNm

Load transfer from flight 3 to flight 2

W = $\frac{27.958}{1.25*2} = 11.18KN/\text{m}$

Calculation of dead load using 1m width Flight 2

Dead load on landing

- Landing slab = D*b* γcon = 0.175*1*25 = 4.38KN/m

- Plastering ,2cm = tp*b* γpl = 0.02*1*17 = 0.46KN/m
- Cement screed,3cm = tcs*b*γcs = 0.03*1*23 = 0.69KN/m
- Porcelain,1cm = tp*b* γp = 0.01*1*27 = 0.27KN/m

Total dead load = 5.8KN/m

And $L.L = 3kN/m^2 * 1m = 3kN/m$

Design load on landing

Ed = 1.35*Gk + 1.5Qk= 1.35*5.8 + 1.5*3 = **12.33KN/m**

Total Ed on landing =12.33+11.18 = **23.5KN/m**

Dead load on stairs

- Load on slab = $\frac{D * b * \gamma con}{Cos\theta} = \frac{0.175*1*25}{Cos25.52^{o}}$ = 4.85KN/m
- Load on step = $\frac{½ * \text{riser height} * \text{width of thread} * \text{width of stair} * \gamma\text{con} * \text{number of riser}}{projected\ length}$

  $= \frac{1/2*0.1432*1*0.3*25*6}{1.5}$ =2.15KN/m
- Marble ,2cm = t* γmar = 0.02*27 = 0.54KN/m^2
- Cement screed,3cm = $tcs*\gamma cs$ = 0.03*23 = 0.69KN/m
- On thread = (0.54+0.69) = 1.23*1 = 1.23
- On riser = $\frac{(0.54+0.69) * \text{riser height} * \text{stair width} * \text{number of riser}}{projected\ length}$

  $= \frac{(0.54+0.69) * 0.1432*1*6}{1.5}$ = 0.71KN/m
- Plastering,2cm = $\frac{0.02*17*1}{Cos25.52}$= 0.38KN/m

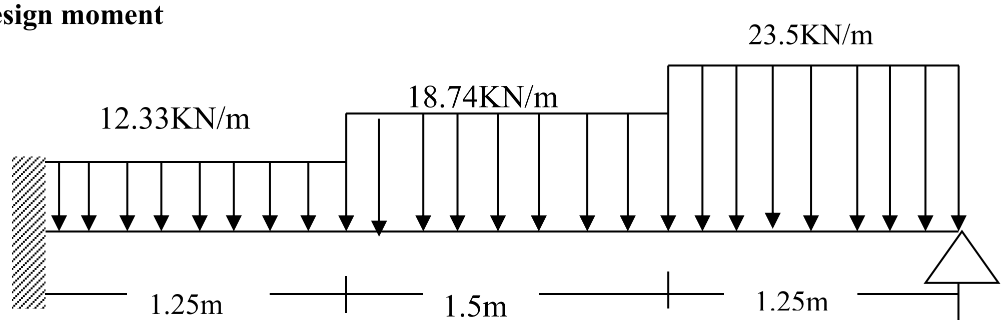
Total dead load = 4.85+2.15+0.54+0.69+1.23+0.71+0.38= 10.55KN/m

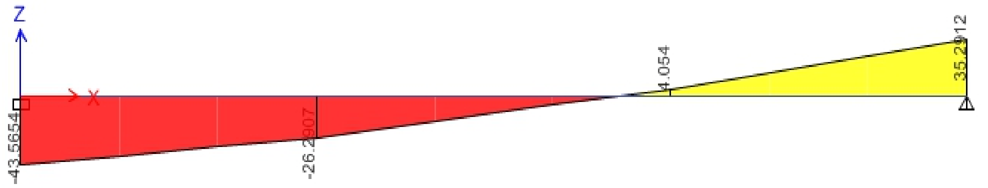
And $L.L = 3kN/m^2 * 1m = 3kN/m$

Design load flight

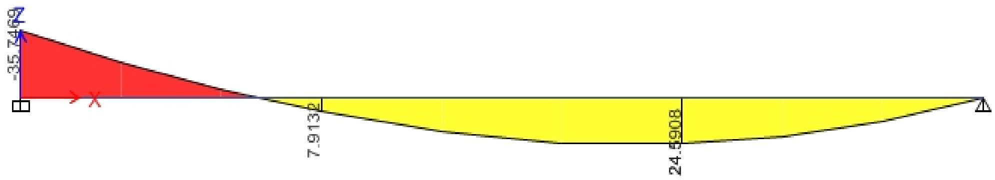
Ed = 1.35*Gk + 1.5Qk= 1.35*10.55+ 1.5*3 = **18.74KN/m**

Design moment





Shear Force Diagram, SFD



Bending Moment Diagram, BMD

Design shear = 43.56KN/m

Design moment (-ve) =35.75KNm

Design moment (+ve) =24.59KNm

4.5 Load transfer from flight 2 to flight 1

W = $\frac{35.29}{1.25*2} = 14.2KN/$m

Calculation of dead load using 1m width Flight 1

Dead load on landing

- Landing slab = D*b* γcon = 0.175*1*25 = 4.38KN/m
- Plastering ,2cm = tp*b* γpl = 0.02*1*17 = 0.46KN/m
- Cement screed,3cm = tcs*b*γcs = 0.03*1*23 = 0.69KN/m
- Porcelain,1cm = tp*b* γp = 0.01*1*27 = 0.27KN/m

Total dead load = 5.8KN/m

And $L.L = 3kN/m^2 * 1m = 3kN/m$

Design load on landing

Ed = 1.35*Gk + 1.5Qk= 1.35*5.8 + 1.5*3 = **12.33KN/m**

Total Ed on landing =12.33+14.2 = **26.5KN/m**

Dead load on stairs

- Load on slab = $\frac{D*b*\gamma con}{Cos\theta} = \frac{0.175*1*25}{Cos25.52^{o}}$ = 4.85KN/m
- Load on step = $\frac{½*\text{riser height}*\text{width of thread}*\text{width of stair}*\gamma con*\text{number of riser}}{projected\ length}$

  $= \frac{1/2*0.1432*1*0.3*25*8}{2.1}$ =2.05KN/m
- Marble ,2cm = t* γmar = 0.02*27 = 0.54KN/m^2
- Cement screed,3cm = $tcs*\gamma cs$ = 0.03*23 = 0.69KN/m

- On thread = (0.54+0.69) = 1.23*1 = 1.23
- On riser = $\frac{(0.54+0.69)* \text{ riser height}*\text{stair width}* \text{ number of riser}}{\textit{projected length}}$

$$= \frac{(0.54+0.69)* 0.1432*1* 8}{2,1} = 0.67\text{KN/m}$$

- Plastering,2cm = $\frac{0.02*17*1}{\text{Cos}25.52}$= 0.38KN/m

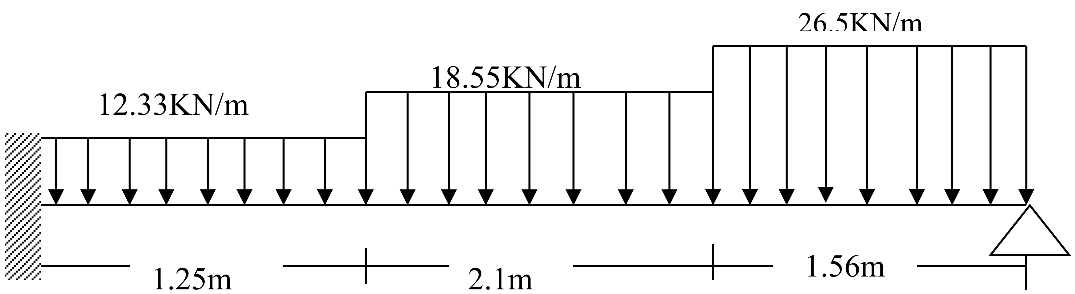
Total dead load = 4.85+2.05+0.54+0.69+1.23+0.67+0.38= 10.41KN/m

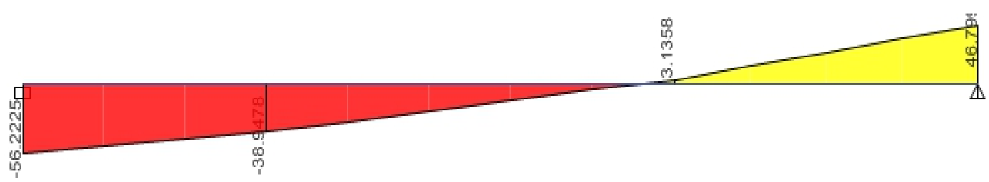
And $L.L = 3kN/m^2*1m = 3kN/m$

Design load flight

Ed = 1.35*Gk + 1.5Qk= 1.35*10.41+ 1.5*3 = **18.55KN/m**

Design Moment





Shear Force Diagram, SFD

Bending Moment Diagram, BMD

Design shear = 56.22KN/m

Design moment (-ve) =58.14KNm

Design moment (+ve) =38.54KNm

The stair case can be designed for maximum negative moment near to the support, maximum positive moment at the soffit/flight of stair and maximum shear/reaction near to the support.For the sake of reasonable simplification we have chosen flight on for design since it has maximum negative moment and shear among the three flights

4.6 Design of Stair Case

Taking the maximum of among the three moments we can check the adequacy of stair depth

Designing Flight 1 since it is governing

Check adequacy of the section for flexure

$d = \sqrt{\frac{Msd}{0.2952*fcd*bw}}$

$d = \sqrt{\frac{58.14*10^6}{0.2952*11.33*1000}}$ =131.85mm<149.05mm………OK

Design of staircase for flexure

Material used and geometry

fyk= 400Mpa, fyd=347.82mpa

fcu=25Mpa,   fcd=11.33Mpa

d=149.05mm

bt=1000mm

Using reinforcement Ø 10 then $as=\frac{\pi*\phi^2}{4}=78.5mm^2$

Design for main reinforcement bar (principal reinforcement), for flight 1

Design moment = Msd= 58.14KNm

- ✓ The steel ratio is calculated as follow

$$\rho = [1 - \sqrt{1 - \frac{2Mu}{fcdbd^2}}]\frac{fcd}{fyd}$$

:. $\rho = [1-\sqrt{1 - \frac{2*29.6*10^6}{1000*149.05^2*11.33}}]\frac{11.33}{400} = 0.0087$

- ✓ The minimum geometric steel ratio of main reinforcement in slab shall not be less than $\rho_{min}=\frac{0.5}{fyk} = \frac{0.5}{460} = 0.001086$
- ✓ Then check $\rho_{min} < \rho$, 0.00125<0.0087and proceed area of steel calculation using value of ρ
- ✓ if $\rho_{min} > \rho$, then use ρ_{min} for area of steel calculation
- ✓ Area of steel is calculated as As=ρbd   b=1000mm, 1m strip of stair slab

  :. Area of steel ($As_{calculated}$) = ρbd=0.0087*1000*149.05=1297mm^2

- ✓ $a_s = \Pi*\phi^2/4$ area of single reinforcement with diameter ϕ

Assume ϕ of bar = 12mm, as= $\frac{12^2}{4}$ *3.14 = 113mm^2

✓ spacing of reinforcement S= a_s*b/As

:. $S_{calculated} = \frac{as*b}{As} = \frac{113*1000}{1297}$=87.12mm, take 85mm

Check for minimum and maximum reinforcement area

Minimum area of main reinforcement

$$ASmin=Max \begin{cases} \frac{0.26fctm*b*d}{fyd} \\ 0.0013bt*d \end{cases}$$

fctm=2.12*ln(1+$\frac{fcm}{10}$)

fcm=fck+8Mpa=20+8=28Mpa

fctm=2.2 for C25/20 from concrete strength class table

$$AS_{min}=Max \begin{cases} \frac{0.26*2.2*1000*149.05}{347.82} = 245.12mm^2 \\ 0.0013bt*d=0.0013*1000*149.05=193.77mm^2 \end{cases}$$

Asmin=254.12mm^2

Maximum area of main reinforcement

The maximum area of reinforcement according to ES EN 1992-1-1:2015 section 9.2.1.1(3) is

As,max=0.004AC, where Ac is the area of concrete.

As,max= 0.004∗175mm∗1000mm=700mm^2

Since, $As_{calculated}$>Asmax therefore, provide $As_{provided}$=700mm^2

Spacing for main reinforcement

$S = \frac{as*b}{AS\ provided} = \frac{113*1000}{700}$= 161.4mm

$S_{provided}$= 150mm

✓ In areas with concentrated loads or maximum moment those provisions become respectively

$$S_{max} = min\begin{cases} 2h \\ 250mm \end{cases} \rightarrow for\ principal\ reinforcement$$

$$S_{max} = min\begin{cases} 2*175 = 350mm \\ 250mm \end{cases} \rightarrow for\ principal\ reinforcement$$

Therefore, Smax= 250mm for primary reinforcement

Since $S_{provided}$<Smax, 160mm<250mm, so provide C/C spacing of primary reinforcement S provided=160mm

Flight 2, Secondary Transvers Reinforcement

According to ES EN 1992-1-1:2015 section 9.3.1.1 Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slabs. In areas near supports transverse reinforcement to principal top bars is not necessary where there is no transverse bending moment.

Staircase is treated as one-way slab. Therefore, secondary transverse reinforcement for staircase is Ast=20%As, provided.{ES EN 1992}

Ast=0.2∗700mm^2 =140mm^2

$Ast_{provided}$=140mm^2

Spacing forsecondary reinforcement

$S_{calculated}=\frac{as*b}{AS\ provided}=\frac{113*1000}{140}=807mm$

$$S_{max}=min\begin{cases}3h=3*175=525mm\\ 400mm\end{cases}\rightarrow for\ secondary\ reinforcement$$

Therefore, Smax=400mm

Since, $S_{calculated}$>Smax, 807mm>400mm, provide C/C spacing of secondary reinforcment little bit less than Smax. Therfore provide Sprovided=250mm for secondary reinforcment

Therefore, provide Ø12 C/C 150mm for main reinforcement bar and provide Ø12 C/C 250mm for secondary reinforcement bar.

Design Of Staircase for Shear

According to ES EN 1992-1-1:2015 section 6.2.1 For member's subject to predominantly uniformly distributed loading the design shear force need not to be checked at a distance less than d from the face of the support. Any shear reinforcement required should continue to the support. In addition, it should be verified that the shear at the support does not exceed$V_{Rd,max}$.

Check if the VRd,max greater than VEd at the support

According ES EN 1992-1-1:2015 section 6.2.3 equation 6.9 the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts is calculated as follows:

$$VRd,max=\frac{\alpha cw*bw*z*v1*fcd}{(cot\theta+tan\theta)}$$

Where

- αcw is a coefficient taking account of the state of the stress in the compression chord
- bw=1m=1000mm width of the slab to be analysis, mm
- V1 is a strength reduction factor for concrete cracked in shear
- θ is the angle b/n the concrete compression strut and beam axis perpendicular to shear force
- Z is inner lever arm, for member with constant depth, corresponding to the bending moment in element under consideration. In shear analysis of reinforced concrete without axial force.mm

αcw=1 $fornon\text{−}prestressed\ structures$

Z=0.9d=0.9∗149.05mm=134.15mm

V1=0.6, for fck≤ 60Mpa

The recommended value of cot θ is $1 \leq \cot\theta \leq = 2.5$, when we take $\cot\theta = 2$, $\theta = 26.5^0$

Maximum Design Shear Resistance VEdat the support

For flight 1 $= 56.22KN$

Maximum Design Shear Resistance

$$VRd,max = \frac{1*1000*134.15*0.6*11.33}{(2+0.498)} = 365KN$$

VRd,max = 365KN $> VEd = 56.22KN$……………………..OK

According to ES EN 1992-1-1:2015 in regions of the member where **VRd,c** ≥**VEd** no need of calculating shear reinforcement is necessary.

Check if **VRd,c** is greater than **VEd** at **d** distance from the face of the support

According to ES EN 1992-1-1:2015 section 6.2.2 equation (6.2.a and 6.2.b)

$$VRd,c = \max\begin{cases} CRd,c * k * \left(100 * \rho 1 * fck\right)^{1/3}) + k1 * \sigma cp * bw * d \\ (vmin + k1 * \sigma cp) * bw * d \end{cases}$$

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

$$K1 = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{149.05}} = 2.16 \leq 2$$

K1 = 2 since it is ≥2, its maximum value is 2 $\rho 1 = \frac{As}{bw*d} \leq 0.02$

For flight 1 ($A_{sprovided} = 700mm^2$)

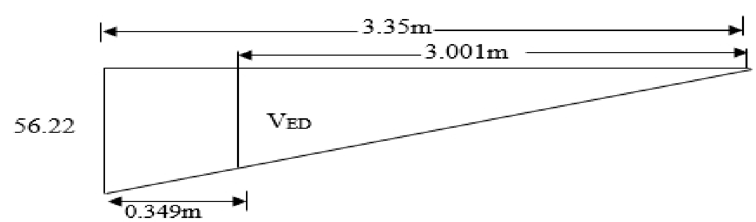
$$\rho 1 = \frac{As}{bw * d} = \frac{700}{1000 * 149.05} = 0.00469 \leq 0.02 \ldots\ldots\ldots OK$$

$\delta cp = \min\begin{cases} NED/Ac \\ 0.2fcd \end{cases} = 0$ in both cases since we have no axial load effect on stair, $N_{ED} = 0$

Or $k1 * \sigma cp = 0$ since we don't have axial load effect on the stair

$Vmin = 0.035*k^{3/2}*fck^{1/2} = 0.035*2^{3/2}*20^{1/2} = 0.44$

VEd from d distance from the face of the support



Using similarity of triangle

$\frac{56.22}{3.35} = \frac{VEd}{3.001}$ from this VEd = 50.36KN

VRd,c = max $\begin{cases} 0.12 * 2 * (100 * 0.00469 * 20)^{\frac{1}{3}} = 16.03 \\ (0.44 + 2 * 0) * 1000 * 149.05 = 65.58\text{KN} \end{cases}$

VRd,c = max $\begin{cases} 16.03KN \\ 65.58\text{KN} \end{cases}$

VRd,c =65.58KN

VRd,c =65.58KN >VEd = 56.22KN……………….OK

Therefore, no need of shear reinforcement and minimum reinforcement is required for our staircase because according to ES EN 1992-1-1:2015 section 6.2.1. If **VRd**,c≥**VEd** no shear reinforcement is required and the minimum shear reinforcement may be omitted in members such as slabs (solid, ribbed) where transverse redistribution of loads is possible.

Check for deflection

Δmax = $\frac{5wl\ \text{^}4}{384EI}$, deflection is checked based on unfactored service load

W =DL+LL= 5.8+3 = 8.8KN/m

Le = 1.25+2.1+1.56= 4.91m

I = $\frac{bD^3}{12} = \frac{1000*175^3}{12}$ = 4.466*10^{-3}mm^4

Ecm=22(fcm/10)$^{0.3}$=22*(28/10)$^{0.3}$=30GPa

Δmax = $\frac{5*8.8*4.91\ \text{^}4}{384*30*10^6*4.466*10\text{^}-3}$ = 0.497mm

Allowable deflection as per EBCS-1995 code, δ = $\frac{Le}{200} = \frac{4910mm}{200}$ = 24.55mm

Δmax = 0.497mm<δ= 24.55mm…………..ok

4.7 Load transfer from staircase to beam

Load transferred from stair to beam in the form of KN/m is taken from end reaction/shear value for beam with L=3.8m from flight 1 and for beam with L=4.95m from flight 2.

Supporting Beam	R(KN)	Total load
L= 3.8m	56.22KN/m	213.64KN
L= 4.95m	43.5KN/m	214.58KN

5 Frame Analysis

5.1 Introduction

Frame is an interconnected structure of beam and column expected to carry, resist and transfer a combination of lateral and gravity loads to the foundation.

Analysis procedure

The steps to analyze the structure described below.

1. Select the Base Units and Design Codes: - we select the base unit to metric SI and design codes of Euro Code
2. Set up Grid Lines: - set up number and spacing of grid lines in both x and y direction by using Architectural drawing data.
3. Define Story Levels: - define story height and label them
4. Define material Properties: - define material properties that we used in overall design process. For initial we define C-25 concrete and S-400 rebar according to material data's on [2]

A. Concrete: -

Grade - C25

Modules of elasticity – 20 GPa

Weight per unit volume- 25kg/m3.

Poisson's ratio, u – is the ratio between lateral strains and longitudinal strains in a material subjected to loading, Poisson's ratio of concrete varies between 0.1 for high strength mix and 0.2 for weak mixes. It is normally taken as 0.15 for strength design and 0.2 for serviceability criteria.

Coefficient of thermal expansion: - when free to deform concrete will expand and contract due to fluctuations in temperature. An average value for coefficient of the coefficient of thermal expansion of concrete is about 10 millionths per degree Celsius (10*10-6 /c).

The software itself calculates shear modulus, the formula for shear modulus=E/2(1+U).

B. Rebar

Grade – S400

Weight per unit volume = 78.6KN/m3

Modulus of elasticity =200GPa=200000MPa

Coefficient of Thermal expansion=0.0000117

5. Define Section properties: - for initial we define column, beam and slab sections.
    a. Beam: - Cross section 300x500 and top tie beam 300x300
    b. Column: - Cross section 500x500 and 400x400
    c. Slab: - depth of 200mm
6. Draw Structural Objects: - draw the structural objects included in architectural drawing
7. Define Load Patterns: - define the loads on the structure

The general load combinations with the consideration of seismic effect are**,** New code ES EN1998:2015

comb 1	1.35DL	1.5LL			linear add
comb 2	DL	LL			linear add
comb 3	DL	0.6LL	EQXL	0.3EQYL	linear add
comb 4	DL	0.6LL	EQXR	0.3EQYL	linear add
comb 5	DL	0.6LL	-EQXL	0.3EQYL	linear add
comb 6	DL	0.6LL	-EQXR	0.3EQYL	linear add
comb 7	DL	0.6LL	EQXL	0.3EQYR	linear add
comb 8	DL	0.6LL	EQXR	0.3EQYR	linear add
comb 9	DL	0.6LL	-EQXL	0.3EQYR	linear add
comb 10	DL	0.6LL	-EQXR	0.3EQYR	linear add
comb 11	DL	0.6LL	EQXL	-0.3EQYL	linear add
comb 12	DL	0.6LL	EQXR	-0.3EQYL	linear add
comb 13	DL	0.6LL	-EQXL	-0.3EQYL	linear add
comb 14	DL	0.6LL	-EQXR	-0.3EQYL	linear add
comb 15	DL	0.6LL	EQXL	-0.3EQYR	linear add
comb 16	DL	0.6LL	EQXR	-0.3EQYR	linear add
comb 17	DL	0.6LL	-EQXL	-0.3EQYR	linear add
comb 18	DL	0.6LL	-EQXR	-0.3EQYR	linear add
comb 19	DL	0.6LL	EQYL	0.3EQXL	linear add
comb 20	DL	0.6LL	EQYR	0.3EQXL	linear add

comb 21	DL	0.6LL	-EQYL	0.3EQXL	linear add
comb 22	DL	0.6LL	-EQYR	0.3EQXL	linear add
comb 23	DL	0.6LL	EQYL	0.3EQXR	linear add
comb 24	DL	0.6LL	EQYR	0.3EQXR	linear add
comb 25	DL	0.6LL	-EQYL	0.3EQXR	linear add
comb 26	DL	0.6LL	-EQYR	0.3EQXR	linear add
comb 27	DL	0.6LL	EQYL	-0.3EQXL	linear add
comb 28	DL	0.6LL	EQYR	-0.3EQXL	linear add
comb 29	DL	0.6LL	-EQYL	-0.3EQXL	linear add
comb 30	DL	0.6LL	-EQYR	-0.3EQXL	linear add
comb 31	DL	0.6LL	EQYL	-0.3EQXR	linear add
comb 32	DL	0.6LL	EQYR	-0.3EQXR	linear add
comb 33	DL	0.6LL	-EQYL	-0.3EQXR	linear add
comb 34	DL	0.6LL	-EQYR	-0.3EQXR	linear add
comb 35	1.35DL	1.5LL	0.9WL		linear add
comb 36	1.35DL	1.5LL	-0.9WL		linear add
comb 37	1.35DL	1.5WL			linear add
comb 38	comb 3-18				envelope
comb 39	comb 19-34				envelope
comb 40	comb 1-37				envelope

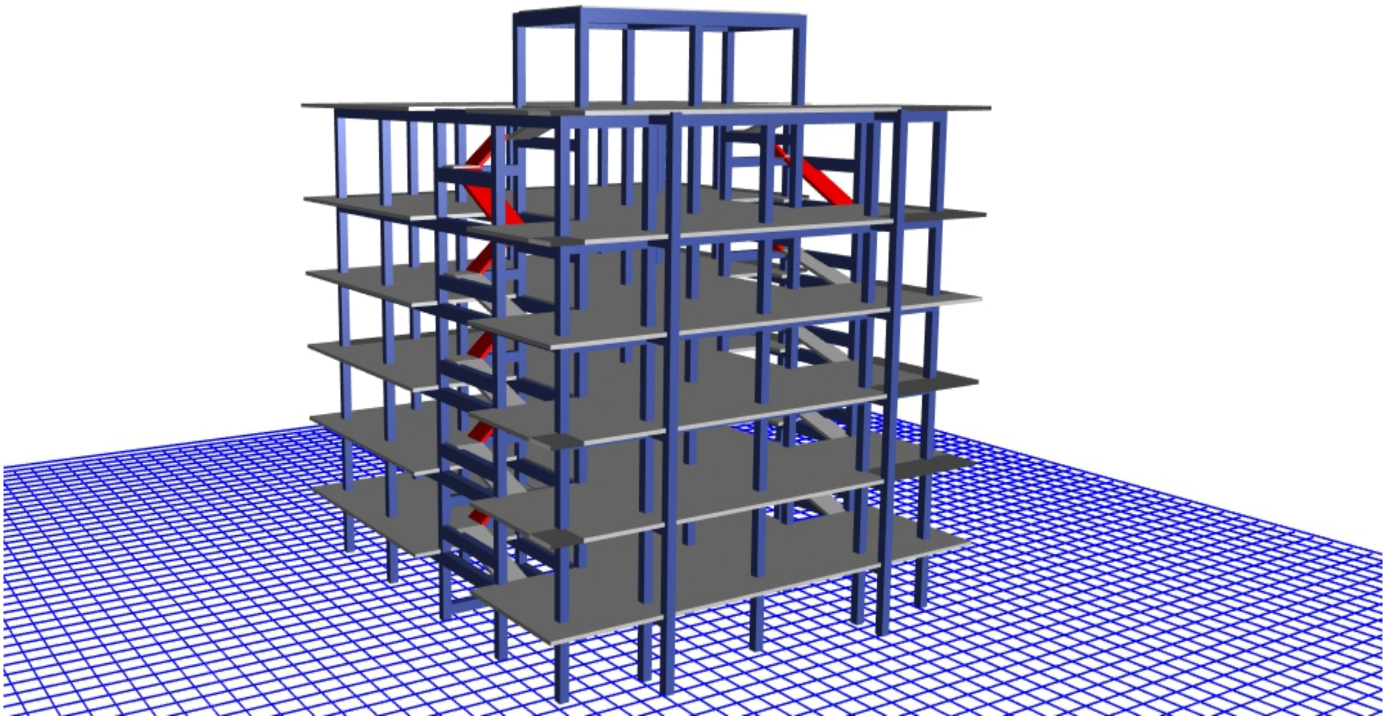


Figure 5. 1 The model of the structure from ETABS out put

3D Frame Modeling Analysis Result

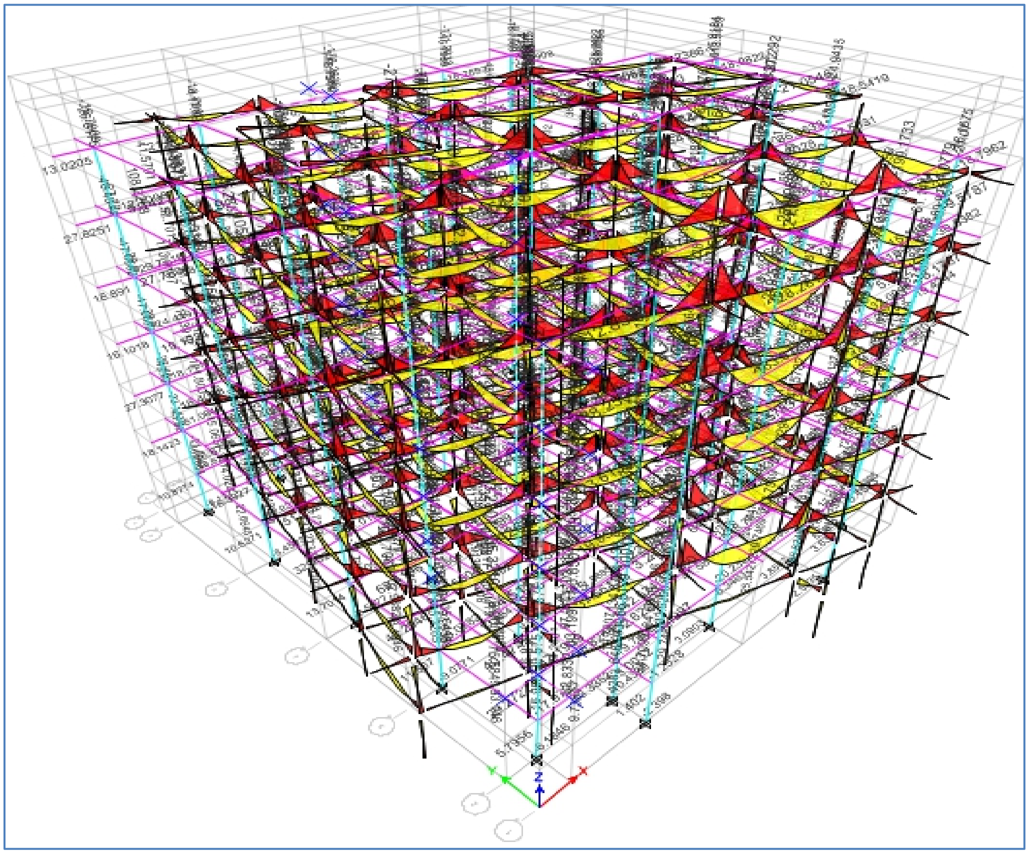


Figure 5. 2 3D Frame Modeling Analysis Result

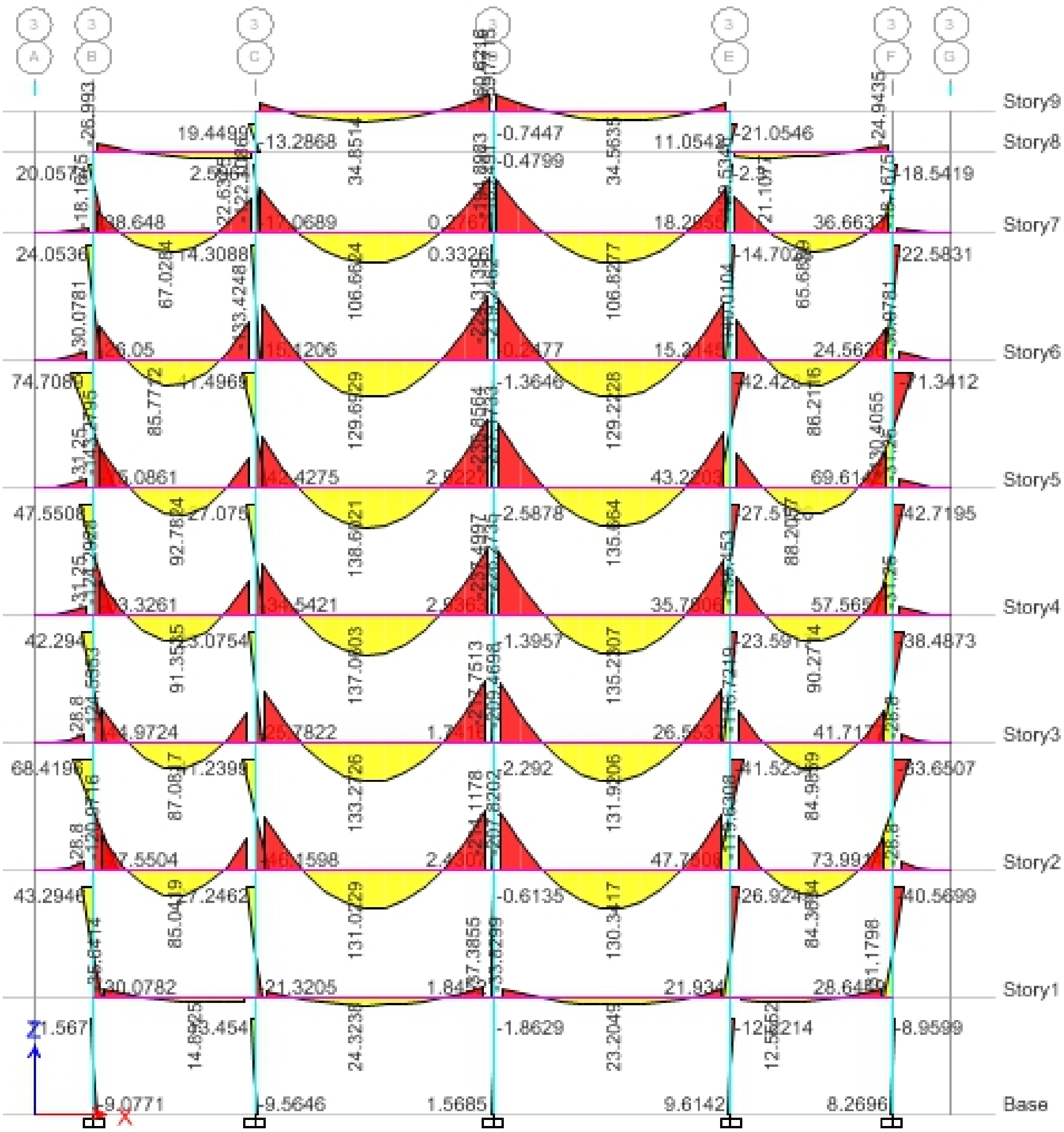


Figure 5. 3 Axis 3-3 And Bending Moment Diagram, M 3-3

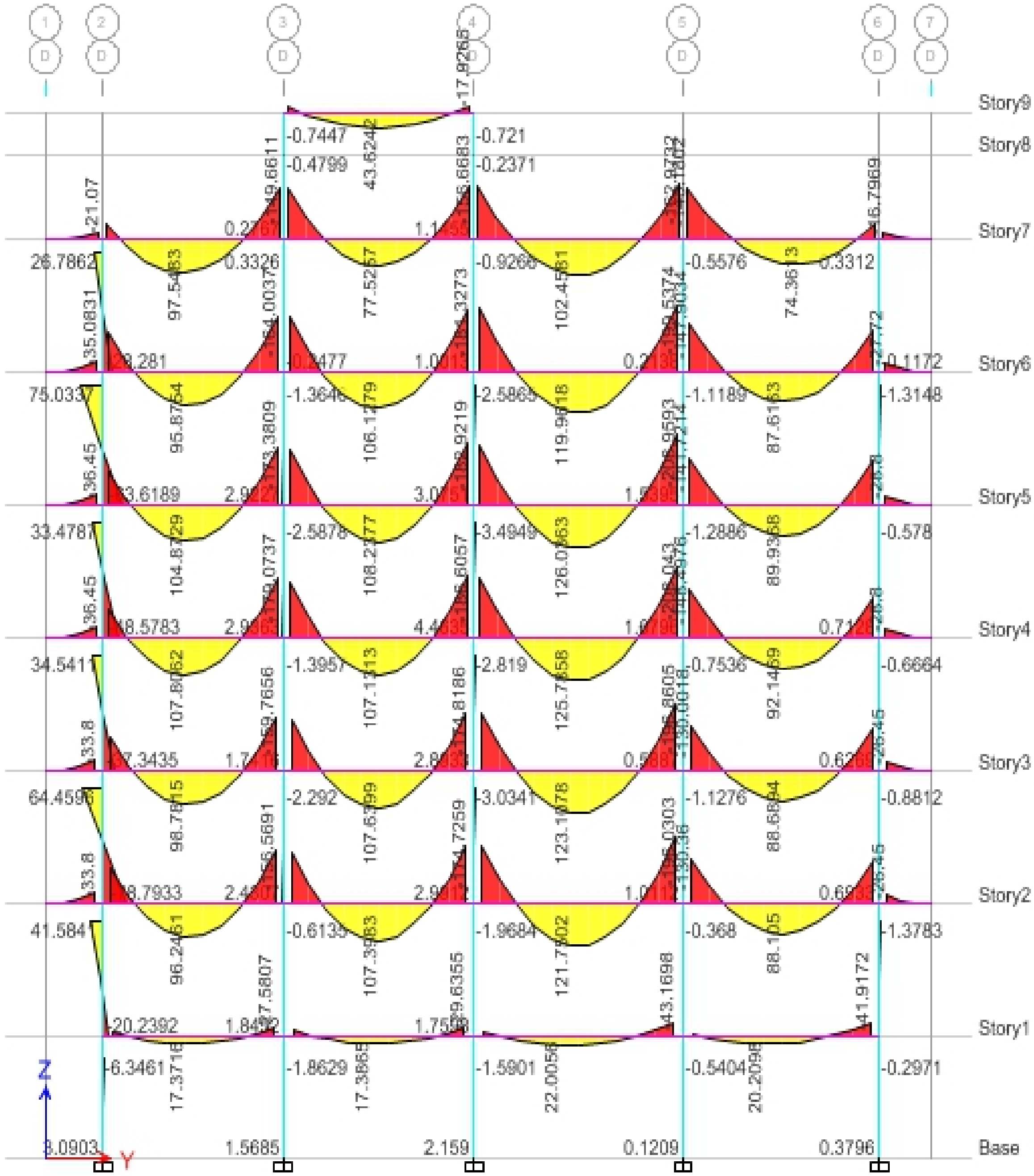


Figure 5.4:- Axis D-D Bending Moment Diagram, M 3-3

Figure 5. 5 Axis 3-3 And Axis D-D Axial Force Diagram

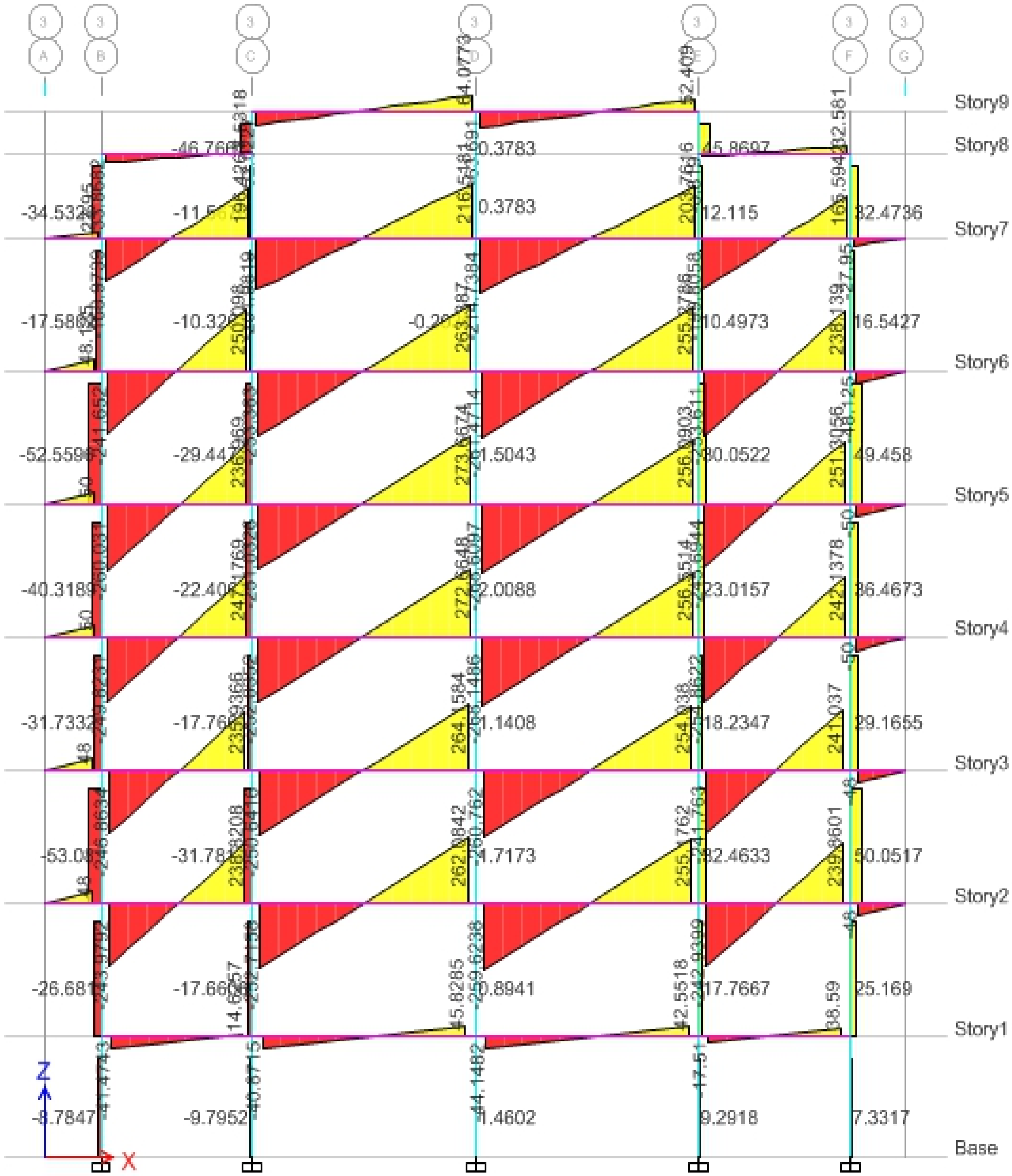


Figure 5. 6 Axis 3-3 Beam Shear Force Diagram

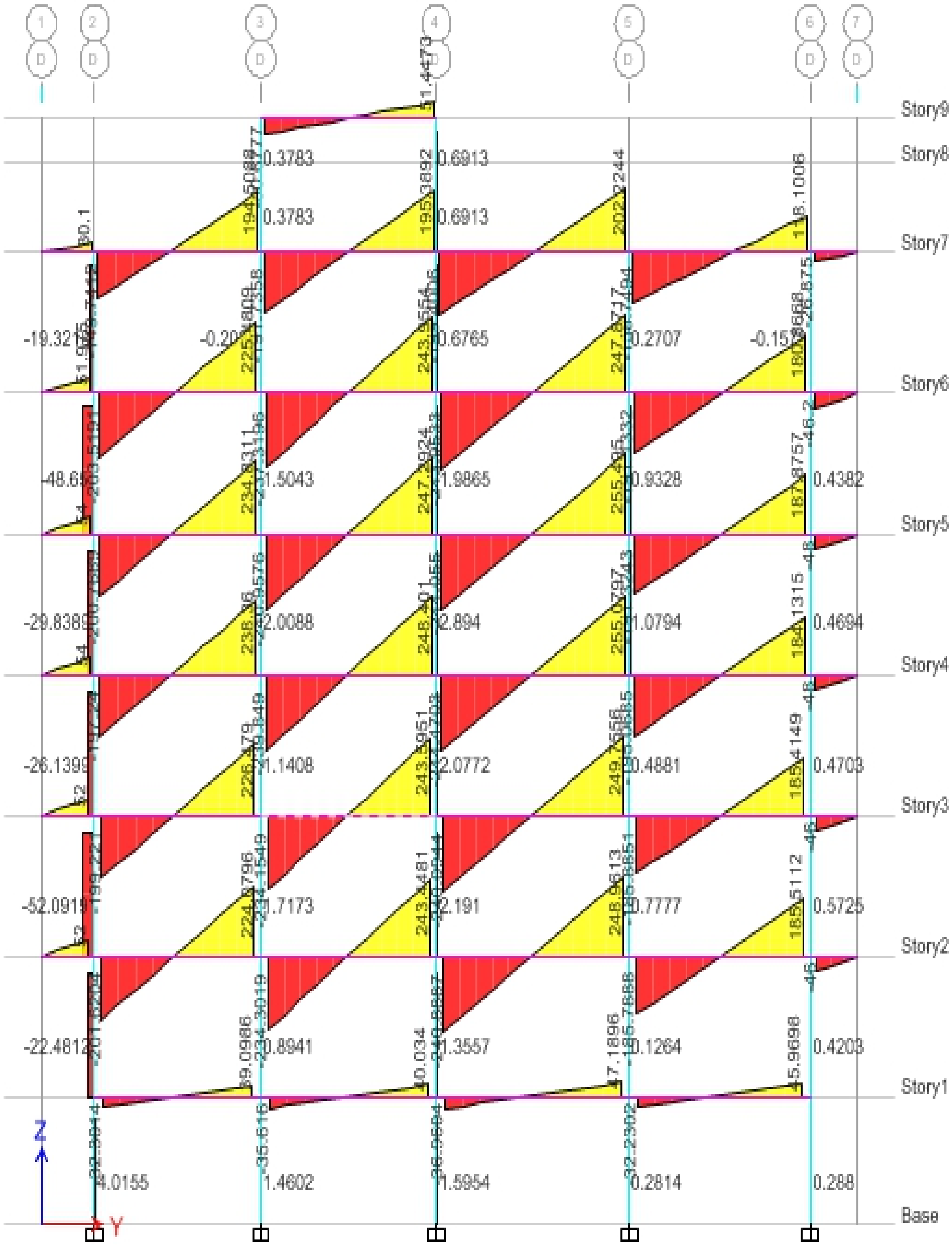


Figure 5. 7 Axis D-D Beam Shear Force Diagram

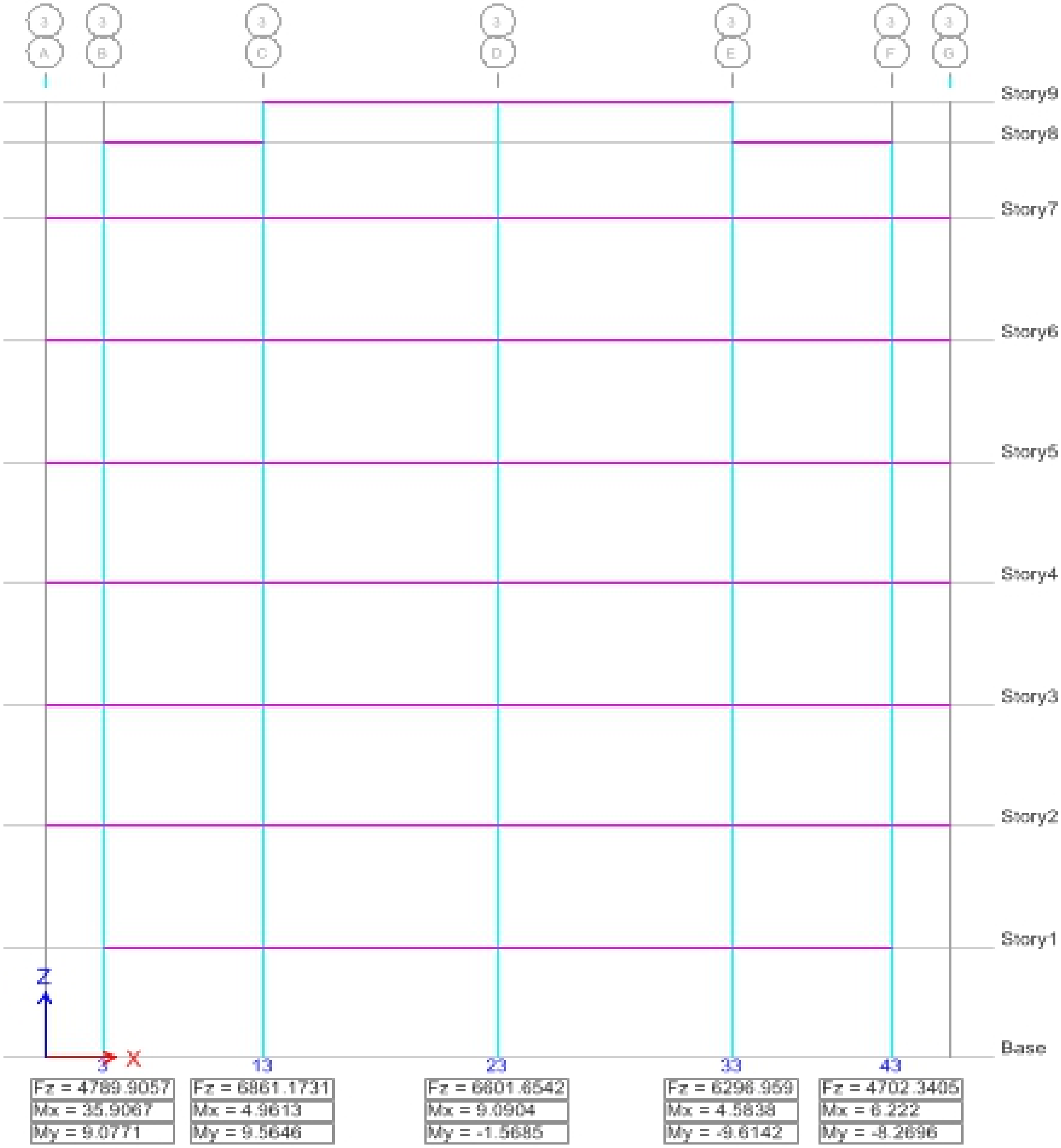


Figure 5. 8 Axis 3-3 Support Reaction/Moment Value

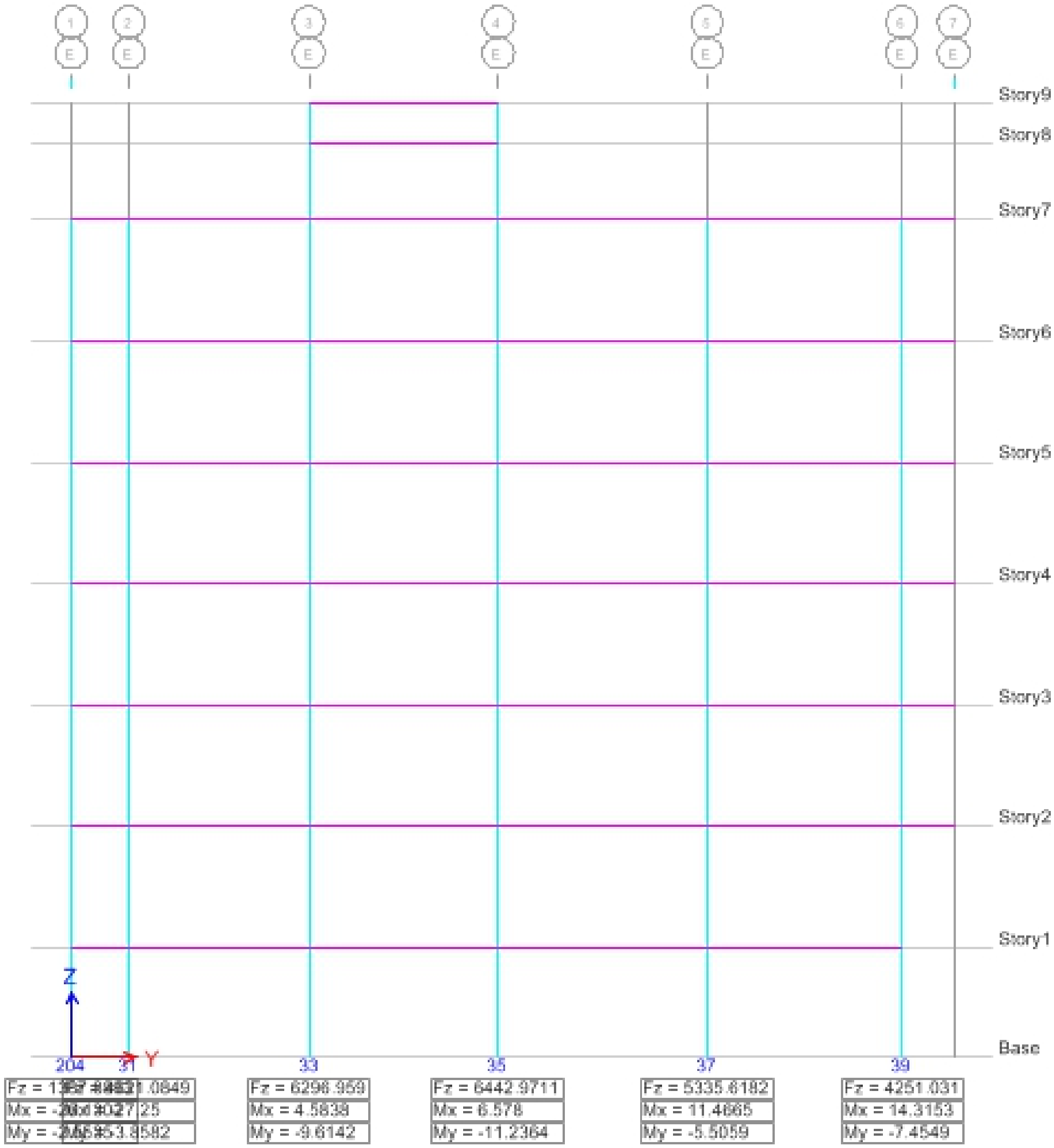


Figure 5. 9 Axis E-E Support Reaction/Moment Value

6 Beam Analysis and Design

6.1 Introduction

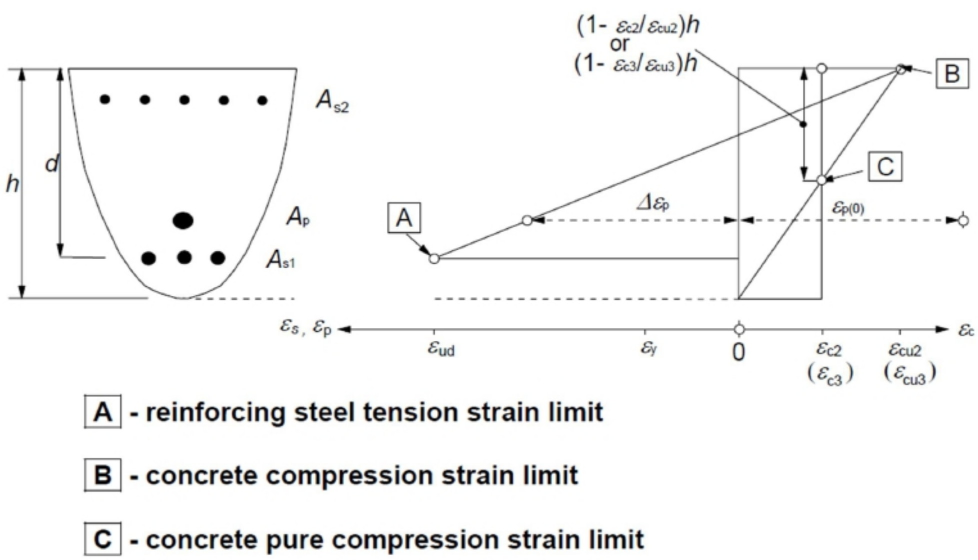
A beam is a structural element that is capable of withstanding load primarily by resisting bending, due to external loads, own weight, span and external reactions.

And the design of beams is in accordance with ES EN 1992-1.1:2015.

6.1.1 Basic principles and assumptions of beam design

Basic principles and assumptions at ultimate limit state described on ES EN 1992-1.1:2015 section 6.1(2) P.

1. Internal stress resultants such as bending moments, shear forces etc. at any section of the member are in equilibrium with the external action effects.
2. Plane sections before bending remains plane after bending.
3. The strain in bonded reinforcement or bonded pre-stressing tendons, whether in tension or in compression, is the same as that in the surrounding concrete.
4. The tensile strength of concrete is ignored.
5. The stresses in the concrete in compression are derived from the design stress/strain relationship given in ES EN 1992-1.1:2015 section 3.1.7 stress-strain relations for the design of cross-sections.



6.1.2 Preliminary analysis and beam sizing

Beam dimensions required are: - [2, p. 2015]

1. Cover to the reinforcement
2. Breadth (b)
3. Effective depth (d)
4. Overall depth (h)

Design for Cover

according to ES EN 1992-1.1:2015 section 4.4.1 and ES EN 1992-1.2:2015 section 5.6, therefore the nominal concrete cover for the beam is designed for a design service life of 50 years, normal quality control, maximum aggregate size of 20mm, HR fire resistance and exposure class of XC1 (dry or permanently wet) as follows.

$$Cmin = \max\begin{cases} \text{Cmin, bond} \\ \text{Cmin, dur} \\ 10\text{mm} \end{cases}$$

Cmin, bond for the longitudinal reinforcement and shear reinforcement is equal to the diameter of the longitudinal reinforcement and the shear reinforcement respectively. The longitudinal and shear reinforcement we used in this project are Ø16mm and Ø8mm respectively. Therefore, Cmin, bond for the longitudinal reinforcement and shear reinforcement is 16mm and 8mm.

Cmin,dur, according to ES EN 1992-1.1:2015 table 4.4N, for a structural class –IV and exposure XC1 (dry or permanently wet). The minimum concrete cover is 15mm. this applies for both longitudinal and shear reinforcements.

Longitudinal reinforcement, $Cmin = max\begin{cases} 16\text{mm} \\ 15\text{mm} \\ 10\text{mm} \end{cases}$

$Cmin = 16mm$, allowing for in design deviation, Δc,dev=10mm

The nominal concrete cover, $Cnom = cmin + \Delta cdev = 16mm + 10mm = 26mm$

Shear reinforcement, $Cmin = max\begin{cases} 8\text{mm} \\ 15\text{mm} \\ 10\text{mm} \end{cases}$

$Cmin = 15mm$, allowing for in design deviation, Δc, dev=10mm

The nominal concrete cover, $Cnom = \text{C}min + \Delta\text{C}dev = 15mm + 10mm = 25mm$ It can be seen from the above calculation that the nominal concrete cover for the shear reinforcement governs. Therefor the provided nominal cover for our beam is, $\boldsymbol{Cnom = 25mm.}$

Check for Fire Resistance

According to ES EN 1992-1.2:2015 table 5.6 for standard fire resistance of R60, the recommended bmin =200mm and a (nominal cover) =12mm. therefore the nominal concrete cover, Cnom=25mm provided is also satisfactory for R60 fire resistance.

Depth and Width of beam

The deformation of a beam member should be in way that does not affect its appearance and functionality. This can be checked by limiting the span/depth ratio, according to the [ES EN 1992-1-1:2015]expression (7.16a) on the Table 7-1 below.

Effective depth calculation

Depth calculation, for beam 3-4 Axis D of 3rd storey Lx=4.95m

Let concrete grade fcu=C25, fck=20Mpa and steel grade fyk=400mpa

$p_o = \sqrt{f_{ck}} * 10^{-3} = \sqrt{20} * 10^{-3}$ =0.0045≈0.005 reference reinforcement ratio,

ρ = 0.5% =0.005 For lightly stressed concrete

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

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

K=1.5, for interior span, factor to account different structural system, Table 7.4N

$$\frac{l}{d} = 1.5\left[11 + 1.5\sqrt{20} * \frac{0.005}{0.005} + 3.2\sqrt{20} * \left(\frac{0.005}{0.005} - 1\right)^{\frac{3}{2}}\right] * \frac{500}{460}$$

D $_{cal}$=257mm, D $_{cal}$=257+24/2+25+8=302mm, D $_{Provided}$=400mm

For all beams similar calculation is done based on their span and support condition and the maximum depth is take as design service depth of beam.
Required minimum width for fire resistance (B) = 200mm

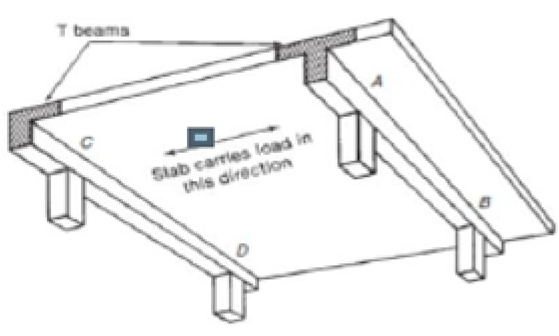
Provided width (B) = 250mm > 200mm……………………………. Ok

6.1.3 Analysis of beam section (bending moment and shear force)

The beam is designed for maximum negative moment, maximum positive moment and maximum shear force obtained from ETABS-2016 analysis result along a selected axis of typical floor

Design of beam section for ultimate limit state

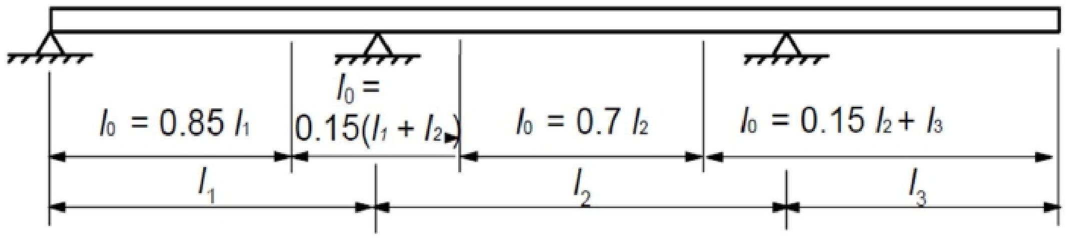
During construction, since slabs and beams are monolithically casted, the slab serves as the top flange of the beams, as indicated by the shading in the figure, and the beam may be designed as either a T beam or an inverted L beam depending whether the beam is an interior beam which has flange on both sides or an exterior beam which has flange on one side only.



Effective width of flange

Assuming beams along axis3-4 of an interior floor T beam of 3rdstorey. In T beams the effective flange width, over which uniform conditions of stress can be assumed, depends on the web and flange dimensions, the type of loading, the span, the support conditions and traverse reinforcement (ES EN 1992-1.2:2015 section 5.3.2.1(1) P.

According to ES EN 1992-1.1:2015 sec 5.3.2.1(1) P, the effective width of a flange should be based on the distance lo between points of zero moments, obtained from ES EN 1992-1.1:2015 figure .



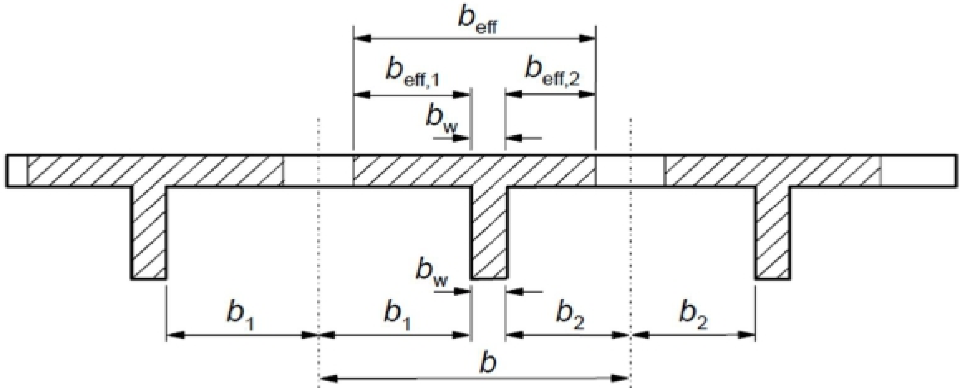
The effective flange width with b $_{eff}$ for a T beam is driven by ES EN 1992-1.1:2015 expression 5.7: -

$beff=\sum beff,i+bw\leq b$

Where

$beff$, $i=0.2bi+0.1lo\leq 0.2lo$ and

$beff,i\leq bi$



Along axis D considering beam 3-4

Beam 2-3, l1=4.7m, lo=0.85l1=3.995m, lo=0.15(l1+l2)=1.4475m

Beam 3-4,l2=4.95m,l0=0.72l2=3.564m

Beam 4-5,l3=5.45m,lo=0.15l2+l3=6.1925m

The Effective flange width T- beams should be smaller of the following two values as per old EBCS code

1. $be = bw + \frac{le}{5}$ =300+4950/5=1290mm
2. be = Actual width of top slab extending between the centers of adjacent spans=4950mm

   Therefore, **beff=1290mm**

Support moment at axis 3-4 3rd storey axis D

Design moment, Msd = 182KNm

Tensile reinforcement bar, Ø = 24mm

Reinforcement bar for compression, Ø = 16mm

Effective depth, d = 400mm – 25mm – 8mm – 24/2mm = 355mm
Effective width, beff= 1290mm
Effective web, bw= 300mm
Rectangular beam section is considered because it is at the support. But according to ES EN 1992-1.1:2015 section 9.2.1.2(2) an intermediate supports of continuous beams, the total area of tension reinforcement As1 can be spread over the effective width of the flange and part of it may be placed on the web. But this may not be necessary if the width of the web is enough to place the tensile reinforcement with adequate spacing.
$\mu sd = Msd/(fcd*b*d^2)$.
$\mu sd = (182*10^6 Nmm)/(11.33 N/mm^2*300mm*(355mm)^2)$.
$\mu sd = 0.0298 \leq 0.295$
This indicates that the section is designed as singly reinforced beam.
Using μsd=0.0298, kz=0.982…. $from\ design\ table$ 2.2 $for C12/25-C50/60$
z = Kz*d = 0.982*359mm = 352.54mm
$As1 = \frac{Msd}{fyd*Z}$
$As1 = 36.35*10^6*Nmm/(400\frac{N}{mm^2}*mm*352.54) = 257.77\ mm^2$

Check for maximum and minimum reinforcement limits

Minimum reinforcement, $As, min = max\begin{cases}\frac{0.26*fctm*bt*d}{fyk}\\ 0.0013bt*d\end{cases}$

$$As, min = max\begin{cases}\frac{0.26*2.83\frac{N}{mm^2}*300mm*359mm}{460\frac{N}{mm^2}} = 172.27mm^2\\ 0.0013*300mm*359mm = 140.01mm^2\end{cases}$$

$As, min = 172.27\ mm^2 < As1 = 257.77mm^2$……………………………………. ok

$As, max = 0.04AC$ = 0.04*300mm*400mm

$As, max = 4800mm2 > As1 = 257.77mm^2$………………………………………ok
Therefore, $As1 = 257.77mm^2$
Number of bars in tension, n $= \frac{As1}{ast} = \frac{257.77mm^2}{\pi*16^2/4} = 1.28 \cong 2$

Provide 2Ø16 tensile reinforcements
Check for longitudinal reinforcement spacing

-According to ES EN 1992-1.1:2015 section 8.2(1) P, the spacing of bars shall be in such that concrete can be placed and compacted satisfactory for the development of adequate bond. The clear distance between individual parallel or horizontal layers of parallel bars should not be less than the maximum of k1 *bar diameter, (dg +k2) or 20mm where dg is the maximum size of aggregate (in our case 20mm). the recommended value of k1 and k2 are 1 and 5mm respectively.
Spacing required = max (1*16mm, 20mm+5mm, 20mm)
Spacing required = max (16mm, 25mm, 20mm)
Spacing required = 25mm

Spacing available = (300mm-2*25mm-2*8mm-2*16mm)/1

Spacing available =202mm > Spacing required = 25mm……………………………………. ok

Span moment on axis 3-4 3rd storey axis D

Design moment, Msd = 108 KNm

Tensile reinforcement bar, ∅ = 16mm

Reinforcement bar for compression, ∅ = 14mm

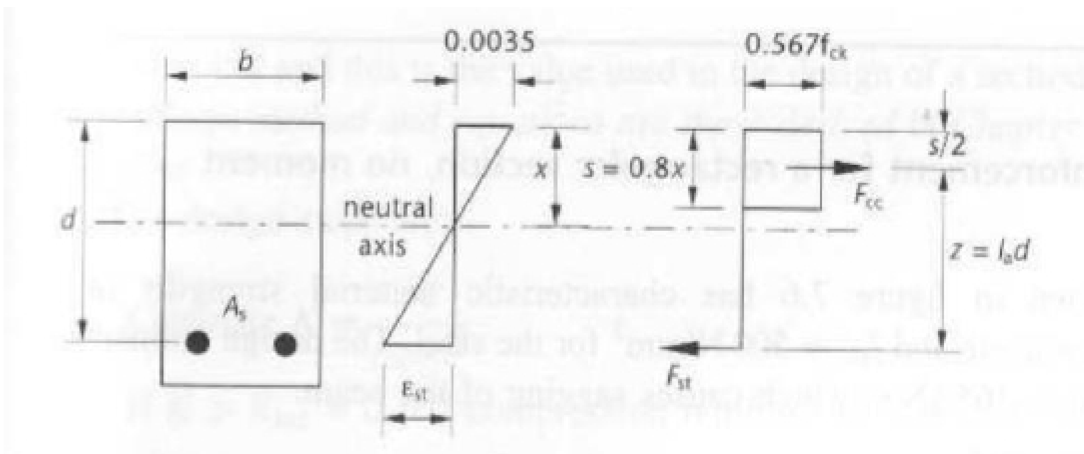
Effective depth, d = 400mm – 25mm – 8mm – 24/2mm = 355mm

Effective width, beff = 1290mm

Effective web, bw = 300mm

Since the beam at this span is resisting a sagging bending moment, the first thing to do is to check whether the beam should be designed as rectangular or T beam. To do this we have to determine

the neutral axis using a simplified stress-strain distribution called rectangular stress-strain distribution curve as shown in figure below



$Msd = 0.8x*fcd*beff*(d-0.4x)$

$108*10^6\ Nmm = 0.8x*11.33\ N/mm^2*1730mm*(359mm-0.4x)$

$x^2-892.82x+2319.73=0$

x=2.605$mm < tf$ =250mm, the beam is designed as a rectangular beam

$\mu sd=Msd/(fcd*b*d2)$.

$\mu sd=(14.55*10^6Nmm)/(11.33N/mm^2*300mm*(359mm)^2)$.

$\mu sd = 0.0332 \leq 0.295$

This indicates that the section is designed as singly reinforced beam.

$Kz = 0.98$

$z = 0.98d = 0.98*359mm = 351.82mm$

$As1=14.55*(10)^6*Nmm/(400N/(mm^2)*351.82mm)=103.39mm^2$

Check for maximum and minimum reinforcement limits

As, min=172.27mm^2 >$As1$= 103.39mm^2…………………………………not ok, therefore

$As1$ = As, min=172.27mm^2
As, max=0.04AC = 0.04*300mm*400mm
As, max=4800$mm2$ >$As1$= 172.27mm^2 ……………………………………… ok

Therefore, $As1$= 172.27$mm2$

Number of bars in tension, n =$\frac{As1}{ast}$ = $\frac{172.27mm^2}{\pi*16^2/4}$ =1.28≅2

Provide 2∅16 tensile reinforcements

Check for longitudinal reinforcement spacing
Spacing required = 25mm
Spacing available = (300mm-2*25mm-2*8mm-2*16mm)/1

Spacing available =202mm > Spacing required = 25mm……………………………… ok

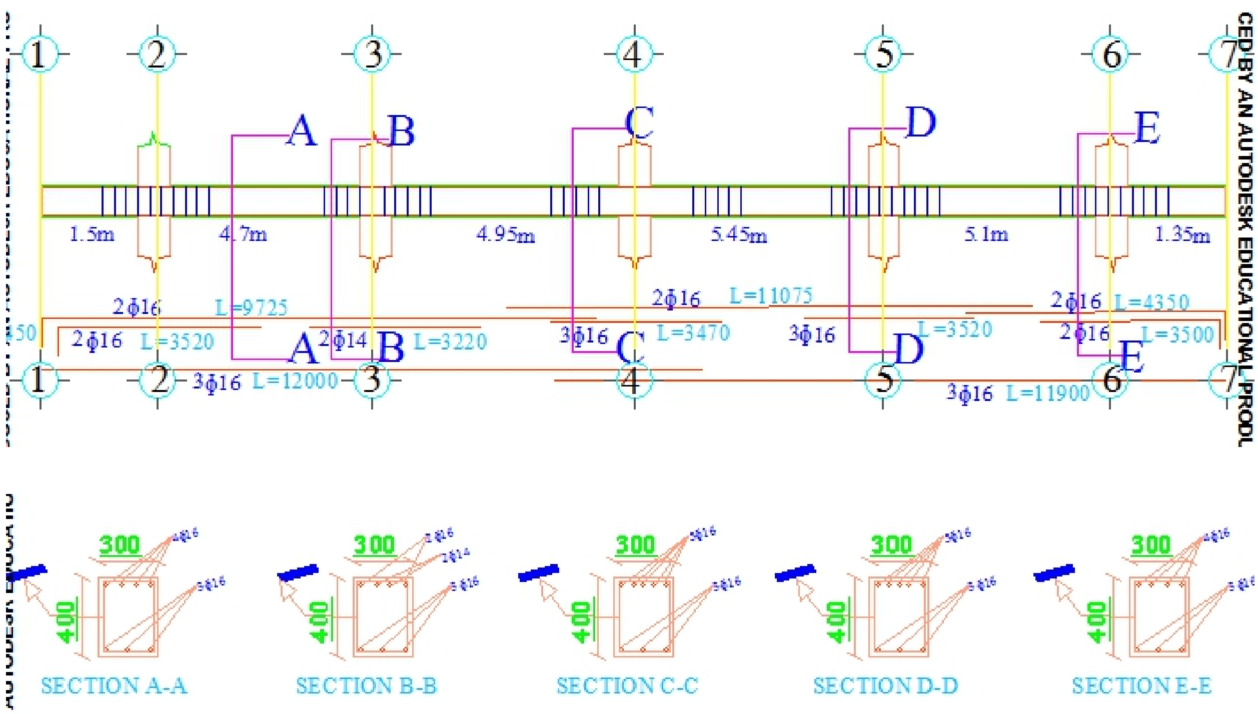


Figure 6. 1 detailing of beam

7 Design of Column

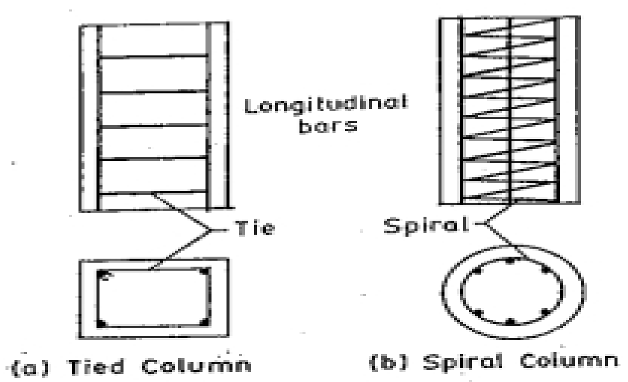
A column is a vertical structural member supporting axial compressive loads, with or without moment.

7.1 Classification of columns

a) **Based on lateral reinforcement**

1. 'Tied columns' in which the main longitudinal bars are confined within closely spaced lateral ties.

2. 'Spiral columns' having main longitudinal reinforcements enclosed within closely spaced and continuously wound spiral reinforcement.



b) **Based on type of loading**

1. Axially loaded columns: are columns which are not exposed to any moment in any direction, which is the only action will be the axial load.

2. Uniaxial columns: are columns which are highly exposed to bending moment in one of the direction rather than both directions in addition to the axial load.

3. Biaxial columns: are columns which are subjected to moment in both direction besides axial load.

c) **Based on degree of slenderness**

1. Short column: are columns for which the strength is governed by the strength the materials and the geometry of the cross section in short columns, second-order effects are negligible.

2. Slender column: when the unsupported length is long lateral deflection shall be so high that the moments shall increase and weaken the column. Such a column, whose axial load carrying capacity is significantly reduced by moment resulting from lateral deflection of the column, is referred to as slender column.

Braced and unbraced columns

A column may be considered braced if the lateral loads, due to wind for example, are resisted by shear walls or some other form of bracing rather than by the column. A column may be considered to be unbraced if the lateral loads are resisted by the sway action of the column.

Second order effects on columns

According to ES EN 1992-1-1:2015 section 5.8 second order effects are additional action caused by structural deformation. Second order effects may be ignored if they are less than 10% of the corresponding first order or satisfies the following criteria.

Simplified criteria for second order effects

Slenderness criteria

According to EN ES 1992-1-1:2015 section 5.8.3.1 second order effects may be ignored if the slenderness λ is below a certain λlim value.

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

Where

$= \frac{1}{(1+0.2\varphi ef)}$(if$\varphi ef$ is not known ,A =0.7 may be used),

$B = \sqrt{1+2\omega}$ (if ω is not known, B =1.1 may be used),

C= 1.7−rm (if rm is not known, =0.7 may be used),

Effective creep ratio ,

Mechanical reinforcement ratio,

$n = \frac{NEd}{(Ac*fcd)}$; relative normal force ,

$rm = \frac{M01}{M02}$; moment ratio ,and

$M01$, $M02$ are the first order of moments,$|M02| \geq |M01|$.

In cases with biaxial bending, the slenderness criterion may be checked separately for each direction. Depending on the outcome of this check, second order effects

(a) may be ignored in both directions,

(b) should be taken into account in one direction, or

(c) should be taken into account in both directions.

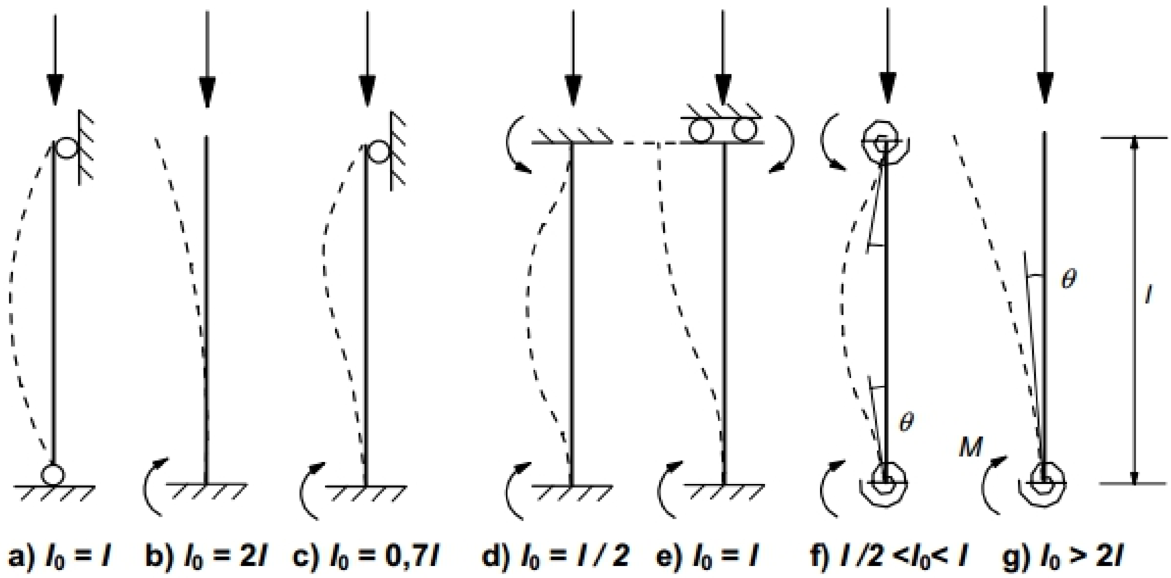
Slenderness and effective length

The slenderness ratio is defined as follows:

$\lambda = \frac{lo}{i}$Where

lo is the effective length, and

i is the radius of gyration of the un cracked concrete section



Examples of different buckling modes and corresponding effective

For compression members in regular frames, the slenderness criterion should be checked with an effective length $l0$ determined in the following way.

Braced members

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

Un Braced members

$$lo=l*max\begin{cases}\sqrt{\left(1+\frac{10*k1*k2}{k1+k2}\right)}\\ \left(1+\frac{k1}{1+k1}\right)*(1+\frac{k2}{1+k2})\end{cases}$$

Where

$k1,2$ are the relative flexibilities of rotational restraints at ends 1 and 2 respectively,

$k=(\frac{\theta}{M})*(\frac{EI}{l})$

is the rotation of restraining members for bending moment M,

is the bending stiffness of compression members, and

is the clear height of compression member between end restraints.

Note: k=0 is the theoretical limit for rigid rotational restraint, and k=∞ represents the limit for no restraint at all. Since fully rigid restraint is rare in practice, a minimum value of 0.1 is recommended for $k1$and $k2$.

Method of analysis

If $\lambda>\lambda lim$ the second order effect must be analyzed using the following method.

a. Second order analysis based on nominal stiffness, and

b. Method based on estimation of curvature.

Design procedure

Concrete cover design

Cover is designed according to [2]

a. Exposure class = XC1, for Dry or permanently wet Environment like Concrete inside buildings with low air humidity and Concrete permanently submerged in water. [Appendix B, Table B-2][2]

b. Minimum strength class = C20/25, for exposure class XC1[Appendix B, Table B-3]

c. Minimum concrete cover for bond/durability

d. $Cmin = max\begin{Bmatrix} Cmin, B \\ Cmin, Dur \\ 10mm \end{Bmatrix} = max\begin{Bmatrix} 20mm \\ 10mm \\ 10mm \end{Bmatrix} = 20mm$

   i. $C_{min,\,b}$= 20 mm, assumed diameter of bar

   ii. Cmin, dur = 10 mm,

      1. Cmin=20mm

The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E

   i. and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1

e. Nominal cover

   a. $C_{nom} = C_{min} + \delta c_{dev}$ = 20mm + 10mm = 30mm

      i. Δc_{dev} = 10mm, the value of Δcdev for use in a Country may be found in its National Annex. The recommended value is 10 mm.

f. Minimum cover Design for Fire = 20mm

g. Governing concrete cover = 30mm

Design for cover

Cmin,dur, according to ES EN 1992-1.1:2015 table 4.4N, for a structural class of four and exposure XC1 (dry or permanently wet). The minimum concrete cover is 15mm. this applies for both longitudinal and shear reinforcements.

Longitudinal reinforcement, $Cmin = max\begin{cases} 20mm \\ 15mm \\ 10mm \end{cases}$

$Cmin$=20mm, allowing for in design deviation, Δc,dev=10mm

The nominal concrete cover, $Cnom = cmin + \Delta cdev$ =20mm+10mm = $\mathbf{30mm}$

7.2 Longitudinal reinforcement

The general procedure followed to calculate longitudinal reinforcement of column are:
1. calculate first order moment
2. calculate the effective length and radius of gyration
3. calculate the slenderness ratio and slenderness limit and check for second order effect
4. calculate the accidental eccentricity
5. calculate equivalent first order moment
6. calculate equivalent first order moment with addition of accidental eccentricity moment
7. If slenderness limit is greater than slenderness ratio calculates the longitudinal reinforcement using the moment and in 6 if not use one of the two second order analysis to calculate the additional moment due to second order effect and add to the moment in 6 to calculate longitudinal reinforcement.
8. Calculate Asusing moment in 7 and axial force.
9. Check for As, and As

First order moment

First order moment are moments coming from ETABS analysis of the building and those moments are due to the load effect of the structure.

Table 7. 1 First order moment and axial force from ETABS for column

Column	Moment	$M3$=My(kNm)	$M2$=Mx(kNm)	NEd (kN)
Basement floor	Mo2	130.26	-118.61	2668.70
	Mo1	23.05	58.09	
Ground floor	Mo2	103.04	-105.82	2380.05
	Mo1	53.93	80.69	
1st floor	Mo2	106.26	116.16	2067.70
	Mo1	-101.25	-112.26	
2nd floor	Mo2	88.42	95.61	1781.68
	Mo1	-46.02	-69.64	
3rd floor	Mo2	77.14	81.29	1497.20
	Mo1	-40.37	-61.21	

4^{th} floor	Mo2	67.25	76.48	1213.37
	Mo1	-31.51	-53.74	
5^{th} floor	Mo2	49.52	-48.02	945.86
	Mo1	-21.88	37.00	

7.3 Effective length and radius of gyration

Effective length

The effective length, lo of a member is defined as the length of a pin-ended strut with constant normal force having the same cross-section and buckling load.

Sample calculation of effective length for basement floor column

For Braced members

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

Where l=2.2m, value of is for k1 and k2 can be calculated by the following formula.

K=$\frac{column\ stiffness}{\sum beam\ stiffness}$

In x direction

$$Kc=\frac{b^3h}{12L}==\frac{600^3*800}{12*2200}=6545454.55mm^3$$

$$Kb=\frac{bh^3}{12L}==2*\frac{400*500^3}{12*5400}+\frac{400*500^3}{12*4200}+\frac{400*500^3}{12*6300}=3196649mm^3$$

$$K1=\frac{6545454.55}{3196649}=2.05=k2$$

In y direction

$$Kc=\frac{bh^3}{12L}==\frac{600*800^3}{12*2200}=11636363.6mm^3$$

$$K1=\frac{11636363.6}{3196649}=3.64=k2$$

In x direction

$$lo=0.5*2.2\sqrt{\left(1+\frac{2.05}{0.45+2.05}\right)*\left(1+\frac{2.05}{0.45+2.05}\right)}=2.098m$$

In y direction

$$lo=0.5*2.2\sqrt{\left(1+\frac{3.64}{0.45+3.64}\right)*\left(1+\frac{3.64}{0.45+3.64}\right)}=2.14m$$

Radius of gyration

i = $\sqrt{\frac{Icolmn}{AC,column}}=\sqrt{\frac{0.0256m^4}{0.48m^2}}=231mm$

Slenderness ratio, slenderness limit and check for second order effect

Slenderness limit

Sample calculation for basement floor column

In the x direction

λlim=20$*A*B*C/\sqrt{}n$

Where A=0.7 ,=1.1 and $C$and n are calculated as followed

C=1.7$-rm$

rm= $\frac{Mo1}{Mo2}=\frac{-58.09}{118.61}=-0.49$

c =1.7-(-0.49)=2.19

n=$\frac{NEd}{AC*fcd}=\frac{2668.7}{0.8*0.6*11.33}=0.49$

λlim=20$*$0.7*1.1*2.19/$\sqrt{0.49}$=48.18

In the y direction

λlim=20$*A*B*C/\sqrt{}n$

Where A=0.7 ,=1.1 and $C$and n are calculated as followed

C=1.7$-rm$

rm= $\frac{Mo1}{Mo2}=\frac{23.05}{130.26}=0.177$

C =1.7-0.177=1.52

n=$\frac{NEd}{AC*fcd}=\frac{2668.7}{0.8*0.6*11.33}=0.49$

λlim=20$*$0.7*1.1*1.52/$\sqrt{0.49}$=47.86

Slenderness ratio

sample calculation for basement floor column

in the x direction

$\lambda=\frac{lo}{i}=\frac{2240.48}{231}$=9.7

in the y direction

$\lambda=\frac{lo}{i}=\frac{2235.73}{231}$=9.68

Check for second order effect

for both directions

Table 7. 2 Slenderness ratio, slenderness limit and check for second order effect

Column	lo(mm)		i(mm)	λ		λlim		Remark
	X	Y		X	Y	X	Y	
Basement	2098	2138	231	9.08	9.25	48.18	47.86	Ok
Ground	2079	2126	231	9	9.2	54.18	25.88	Ok
1st floor	2028	2090	231	8.78	9.05	58.66	58.36	Ok
2nd floor	2079	2126	231	9	9.2	53.43	48.85	Ok
3rd floor	2079	2126	231	9	9.2	53.97	48.91	Ok
4th floor	2079	2126	231	9	9.2	52.86	47.7	Ok
5th floor	2079	2126	231	9	9.2	54.35	47.12	Ok

$\lambda < \lambda lim \ldots\ldots\ldots no need of second order effect$

As we can see in the above Table 8-3 no need of second order effect for our columns.

Accidental eccentricity

Accidental eccentricity is the effects of cracking, creep, non-linear material properties and geometric imperfections, which normally means considering the structure being constructed ‘out of plumb’ (not vertical), which in isolated members is allowed for by introducing an additional eccentricity, of the axial load.

$ei = \frac{lo}{400}$ and the moment introduced is $Mi = ei * NEd$

Sample calculation for accidental eccentricity moment for basement floor column

In the x direction

$ei = \frac{lo}{400} = \frac{2098}{400} = 5.25mm = 0.00525m$

$Mi = 0.00525 * 2668.7 = 14.01KNm$

In the y direction

$ei = \frac{lo}{400} = \frac{2138}{400} = 5.35mm = 0.00535m$

$Mi = 0.00535 * 2668.7 = 14.27KNm$

Equivalent first order moment

According to ES EN 1992-1-1:2015 section 5.8.8.2 a reasonable estimate of the first-order moment near mid-height of the column is given by

$$Moe = max\begin{cases} 0.6 * Mo2 + 0.4 * Mo1 \\ 0.4 * Mo2 \end{cases}$$

But, their no need of calculation of equivalent moment in our building because our columns are short columns. And the design moment is the moment due to first order effects, MED, being numerically equal to the sum of the larger elastic end moment, $M02$, plus any moment due to geometric imperfection, Mi .(Concrete, steelwork, masonry and timber designs to British Standards and Euro codes)

According to ES EN 1992-1-1:2015 section 6.1(4) For reinforced concrete cross-sections subjected to a combination of bending moment and compression, the design value of the bending moment

should be at least $MED = e0*NED$ where eo= D/30 but not less than 20 mm where h is the depth of the section. column is given by:

$$M\text{Ed}=max\begin{cases} Mo2 + Mi \\ eo * NEd \end{cases}$$

Sample calculation of design moment for basement floor

In the x direction

$$M\text{Ed}=max\begin{cases} Mo2 + Mi \\ eo * NEd \end{cases}= max\begin{cases} 118.61 + 14.01 = 132.62KNm \\ \frac{800}{30} * 2668.7 = 71.165KNm \end{cases} =132.62\text{KNm}$$

In the y direction

$$M\text{Ed}=max\begin{cases} Mo2 + Mi \\ eo * NEd \end{cases}= max\begin{cases} 130.26 + 14.27 = 144.53KNM \\ \frac{800}{30} * 2668.7 = 71.165KNm \end{cases} =144.53\text{KNm}$$

Column	NED(kN)	Mo_2(KNm)		Mi(KNm)		$eo*NED$(KNm)	MED(KNm)	
		X	Y	X	Y		X	Y
Basement	2668.70	118.61	130.26	14.01	14.27	71.16	132.62	144.53
Ground	2380.05	105.82	103.04	13.99	12.65	63.47	119.81	142.91
1st floor	2067.70	116.16	106.26	10.74	10.8	55.14	126.9	113.84
2nd floor	1781.68	95.61	88.42	9.03	9.47	47.51	104.64	97.89
3rd floor	1497.20	81.29	77.14	7.78	7.96	33.93	89.07	85.1
4th floor	1213.37	76.48	67.25	6.306	6.45	32.36	82.78	37.7
5th floor	945.86	48.02	49.52	4.92	5.03	25.22	52.94	54.55

Calculating AS using MEd

Using interaction charts prepared for biaxial bending. The procedure involves:

1. Using $d2$ and $b2$ evaluate$\frac{d2}{h}, \frac{b2}{h}$ to choose appropriate chart.

2. Compute Normal force ratio: v=$\frac{\text{NEd}}{\text{fcd*b*h}}$, Moment ratios:=$\frac{\text{MEd}}{\text{fcd*b*h}^2}$

3. Enter the chart and pick ω (the mechanical steel ratio).

4. Compute ,=$\frac{\omega\text{*Ac*fcd}}{\text{fyd}}$

Sample calculation for basement floor column

d2 = cover+$\frac{Ølong}{2}$ + $Østr$ = 30+16+8 =54mm $and\ \frac{d2}{h} = \frac{54}{800}$=0.0675 ≅ 0.1

$\frac{b2}{h} = \frac{54}{800}$=0.0675 ≅ 0.1

NEd = $v = \frac{2668.7}{11.33*800*600} = 0.49$

MEd, x=132.62KNm, MEd,y=144.53KNm

μsdx $= \frac{\text{MEd}}{\text{fcd}*\text{b}*\text{h}^2} = \frac{132.62}{11.33*600*800^2} = 0.03$

μsdy$= \frac{\text{MEd}}{\text{fcd}*\text{b}*\text{h}^2} = \frac{144.53}{11.33*600*800^2} = 0.031$

Using the above data ω = 0.95

$As = \frac{\omega*\text{Ac}*\text{fcd}}{\text{fyd}} = \frac{0.95*800*600*11.33}{400}$=12916.2mm^2

According to ES EN 1992-1-1:2015 section 9.5.2(2) minimum longitudinal reinforcement is

AS, min = $\begin{cases} \frac{0.1NEd}{fyd} \\ 0.002AC \end{cases} = \begin{cases} \frac{0.1*2668.7}{400} = 667.175mm^2 \\ 0.002AC = 0.002*800*600 = 960mm^2 \end{cases}$ =960mm^2

AS, max = $\begin{cases} 0.04AC\text{............} out\ side\ lap\ location \\ 0.08AC\text{..................}.\ at\ laps\ location \end{cases}$

AS, max = $\begin{cases} 0.04*600*800 = 19200 \\ 0.08*600*800 = 38400 \end{cases}$

AS, min≤ AS ≤ AS, max…….ok

AS, provided = 12916.2mm^2

Column	NED(kN)	MEd(KNm)		Vsd	μsd,x	μsd,y	ω	As,prov
		X	Y		X	Y		
Basement	2668.70	132.6	144.53	0.49	0.03	0.031	0.95	12916
Ground	2380.05	119.81	142.91	0.44	0.014	0.02	0.8	10876
1st floor	2067.70	126.9	113.84	0.38	0.012	0.017	0.75	10197
2nd floor	1781.68	104.64	97.89	0.33	0.11	0.015	0.72	9789
3rd floor	1497.20	89.07	85.1	0.28	0.01	0.012	0.7	9512
4th floor	1213.37	82.78	37.7	0.22	0.007	0.01	0.55	7477
5th floor	945.86	52.94	54.55	0.17	0.006	0.007	0.4	5438

Number of bar, using ϕ32mm longitudinal

as = $\frac{3.14*32^2}{4} = 803.84mm^2$

$N=\frac{As}{as}=\frac{12916.2}{803.84}=16$

Table 7. 3 Number of bars for longitudinal reinforcement

Column	As prov(mm^2)	No. bar
Basement	12916	16ϕ32
Ground	10876	16ϕ32
1st floor	10197	16ϕ32
2nd floor	9789	14ϕ32
3rd floor	9512	12ϕ32
4th floor	7477	18ϕ24
5th floor	5438	12ϕ24

Transverse reinforcement

According to ES EN 1992-1-1:2015 section 9.5.3 The diameter of the transverse reinforcement (links, loops or helical spiral reinforcement) should not be less than 6 mm or one quarter of the maximum diameter of the longitudinal bars, whichever is the greater. The diameter of the wires of

welded mesh fabric for transverse reinforcement should not be less than 5 mm. The transverse reinforcement should be anchored adequately.

Check for diameter of transverse reinforcement

$$\phi t = \max\begin{cases} 6mm \\ \frac{\phi}{4}=\frac{32}{4}=8mm \end{cases} \quad \phi t = 8mm$$

The spacing of the transverse reinforcement along the column should not exceed Scl,.

Check for spacing

The value of Scl, for use in a Country may be found in its National Annex. The recommended value is the least of the following three distances:

- 20 times the minimum diameter of the longitudinal bars
- the lesser dimension of the column
- 400 mm

$$Scal,max = \begin{cases} 20 * 14 = 280mm \\ 500mm \\ 400mm \end{cases}$$

Scal,max = 280mm

Detailing

Lap length

According to ES EN 1992-1-1:2015 section 8.7.1 Forces are transmitted from one bar to another by: lapping of bars, with or without bends or hooks, welding and mechanical devices assuring load transfer in tension-compression or in compression only. The detailing of laps between bars shall be such that:

- the transmission of the forces from one bar to the next is assured;
- spalling of the concrete in the neighborhood of the joints does not occur;
- large cracks which affect the performance of the structure do not occur.

The design lap length is:

lo=$\alpha 1 * \alpha 2 * \alpha 3 * \alpha 5 * \alpha 6 * lb,req \geq lo,min$

$\alpha 1$, $\alpha 2$,3,$\alpha 5$=1.0&$\alpha 6$=1.5

lb,req =$\emptyset * \sigma sd/4 * fbd$ where, fbd=2.321Mpa,σsd=4000Mpa

fbd=2.25$n1n2fctd$

$$lo,min = \max\begin{cases} 0.3 * lb,req \\ 15 * \emptyset \\ 200mm \end{cases}$$

Table 7. 4 Summary lap length for column

Ø(mm)	lb,req (mm)	lo,min(mm)	lo (mm) provided lap length
32	1378.75	480	2330
24	1034.04	360	1745
14	603.19	210	1020

8 Lateral Load Analysis

8.1 Introduction

The most known lateral loads are wind load and earth quake loads. The knowledge on the analysis and impact of thus loads has a great role on structural analysis and design of civil engineering structures. Since our building is more than 15m long wind load is to be considered.

To be comparing the effect of EQ load and wind load, the effect of earth quake load is more, since to consider the effect of earth quake load according to EN-ES-8 2015

8.2 Wind load on structure

Previously, the analysis and design of roof for wind load is done.in this chapter the analysis and design of vertical walls of building for wind pressure will be done. Wind load depends on:

- ✓ Velocity of wind
- ✓ Shape of the building
- ✓ Height of the building
- ✓ Geographical location
- ✓ Stiffness of structure

$Cr(z)=Kr*\ln(\frac{z}{zo}), for Zmin \leq Z \leq Zmax$ or

$Cr(z)=Cr(Zmin)$ $for\ Z \leq Zmin$

Where, Kr is the terrain factor depending on roughness length Zo.

The terrain factor Kr should be determined using ES EN 1991-1-4:2015 expression (4.5)

$Kr=0.19(\frac{Z0}{Zo,II})^{0.07}$

Where, $Z_{0,II}$ 0.05 (for terrain category II, ES EN 1991-1-4:2015, Table 4.8),

Zmin is minimum height depend on terrain category, and

Zmax is maximum height is to be taken 200m unless otherwise defined

Zo is Roughness length depends on terrain category.

We have categorized Adama under terrain category IV in which at least 15% of the city surface is covered with buildings and their average height exceeds 15 m. According to ES EN 1991-1-4:2015, Table 4.1, values of (Zo) and (Zmin) are given as follow.

Zo = 1m $Z_{o,II}$ = 0.05m Zmin = 10m Z = 16.5m Zmax = 200m

Since, $Z_{min} \leq Z \leq Z_{max}$, $10 \leq 16.5 \leq 200$

$Kr = Kr*\ln(\frac{z}{zo}) = 0.19*(\frac{1}{0.05})^{0.07} = 0.234$

$Cr(z) = k_r*\ln(\frac{Z}{Z0}) = 0.234*\ln(\frac{16.5}{1}) = 0.656$

Vm(Z) = Cr(Z) *Co (Z) *Vb = 0.656*1*22 **=14.44m/s**

The total height of our building from the ground level to parafate wall is 16.5m. The height of roof and the height below the ground is not considered for wall pressure analysis due to wind.

I. Wind Turbulence

According to ES EN 1991-1-4:2015, section 4.4(11) is the turbulence intensity lv(z) at height Z is defined as the standard deviation of the turbulence divided by mean wind velocity. And it should be determined using ES EN 1991-1-4:2015 expression (4.7).

$lv(z)=\frac{\sigma v}{Vm(z)}=\frac{Kr}{Co(z)*\ln(\frac{Zo}{Z0,II})}$ for Zmin<Z<Zmax

$lv(z)= lv(Zmin)$ for Z<Zmin

Where, K_I is the turbulence factor its recommended value is 1,

Co(z) is orography factor ,

Zo is roughness length, and

σ_v is the standard deviation.

The Standard deviation is given by ES EN 1991-1-4:2015 expression 4.6. $\sigma v=Kr*Vb*K_l$

$\sigma v=0.234\times22m/sec\times1=5.148$

ThereforeWind Turbulence is $lv(16.5m)=\frac{\sigma v}{Vm(16.5)}=\frac{5.148}{14.44}=$ **0.357**

II. Peak Velocity Pressure,qp(z)

The peak velocity pressure qp(z) at height z, which includes mean and short term velocity fluctuations, should be determined by ES EN 1991-1-4:2015 Expression (4.8)

qp (z) $=[1+7*lv(z)]*\frac{1}{2}*\rho *V_m^2(z)$ or qp (z) $=Ce(z)*q_b$

Where ρdensity of air depends on attitude, temperature, barometric pressure of the region

Ce(z) is exposure factor determined by ES EN 1991-1-4:2015

qb is the basic velocity pressure, determined from ES EN 1991-1-4:2015 expression 4.9.

$qb=\frac{1}{2}*\rho*Vb^2=1/2*1.25*22^2=$ **302.5kg/ms²**

According to ES EN 1991-1-4:2015, the recommended value of air density is 1.25kg/m³.

$qp(16.5)=[1+7*lv(z)]*\frac{1}{2}*V_m^2(z)*\rho= (1+7*0.357*)*1/2*1.25*14.44^2=455.99N/m2=$ **0.456KN/m2**

IV. External wind pressure

The external wind pressure should be obtained, from ES EN 1991-1-4:2015 expression 5.1.

$We=qp(Ze)*Cpe$

Where, $qp\ (Ze)$ is the peak velocity pressure,

$Cpe$is the reference height for the external pressure, and

$Ze$is the pressure coefficient for the external pressure

The external pressure coefficients Cpe for buildings and parts of buildings depend on the size of the loaded area A. The external pressure coefficients are given for loaded areas **A** of 1**m²** and **10m²** in ES EN 1991-1-4:2015 table 7.4 for the appropriate building configurations as Cpe,1, for local coefficients & Cpe,10, for overall coefficients, respectively (ES EN 1991-1-4:2015 Section 7.2.1.

$Cpe=Cpe,1$ for $A \leq 1m^2$

$Cpe=Cpe,1-(Cpe,1-Cpe,10)\log_{10}A$ for $1m^2<A<10m^2$

$Cpe=Cpe,10$ for $A\geq 10m^2$

Where, $Cpe,1$ is external pressure coefficient for loaded area A of 1 m²

$Cpe,10$ is external pressure coefficient for loaded area A of 10 m², and

A is loaded area of roof zones

Vertical walls of rectangular plan buildings

The pressure distribution over a rectangular plan building based on its geometric proportion is given below, wherevelocity pressure should be assumed to be uniform over each horizontal strip and Reference height, Ze, depending on h and b, and corresponding velocity pressure.

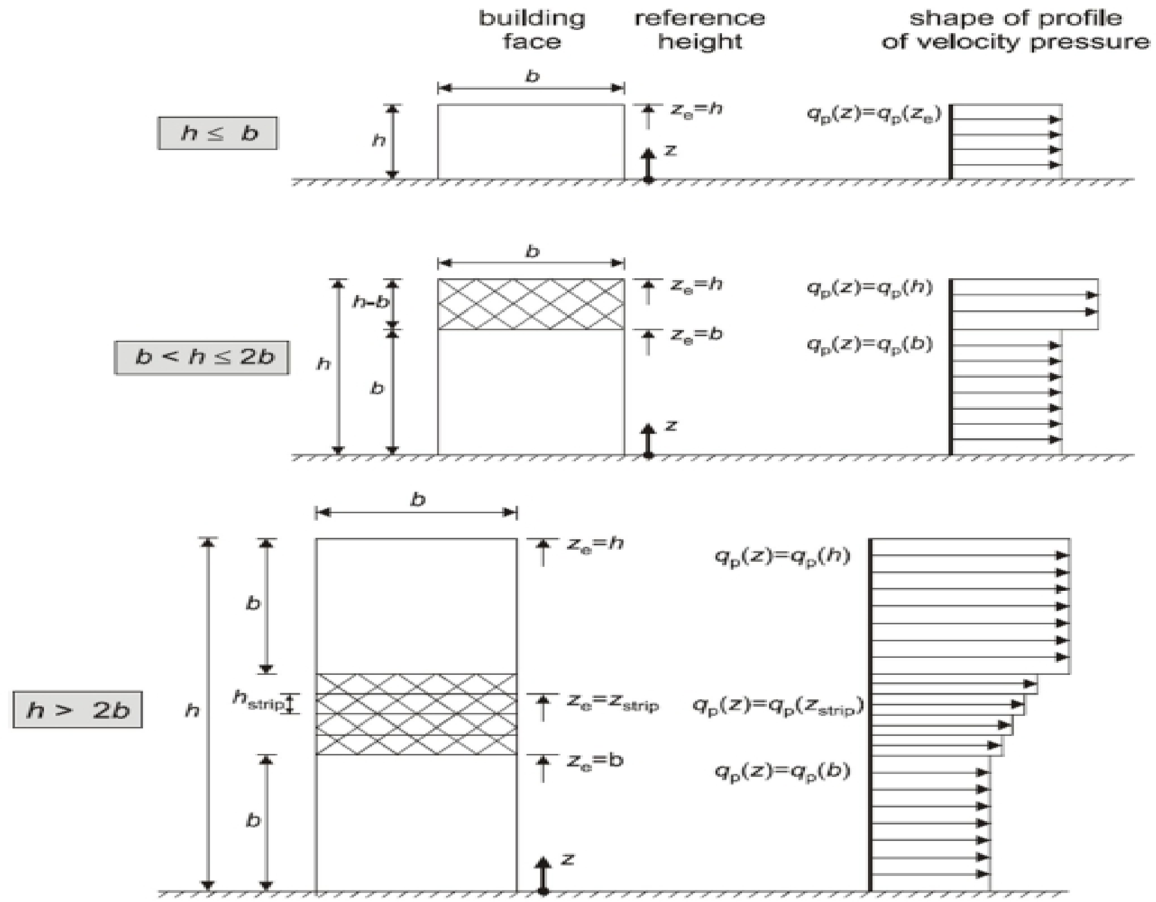


Figure 8. 1 Vertical walls of rectangular plan buildings

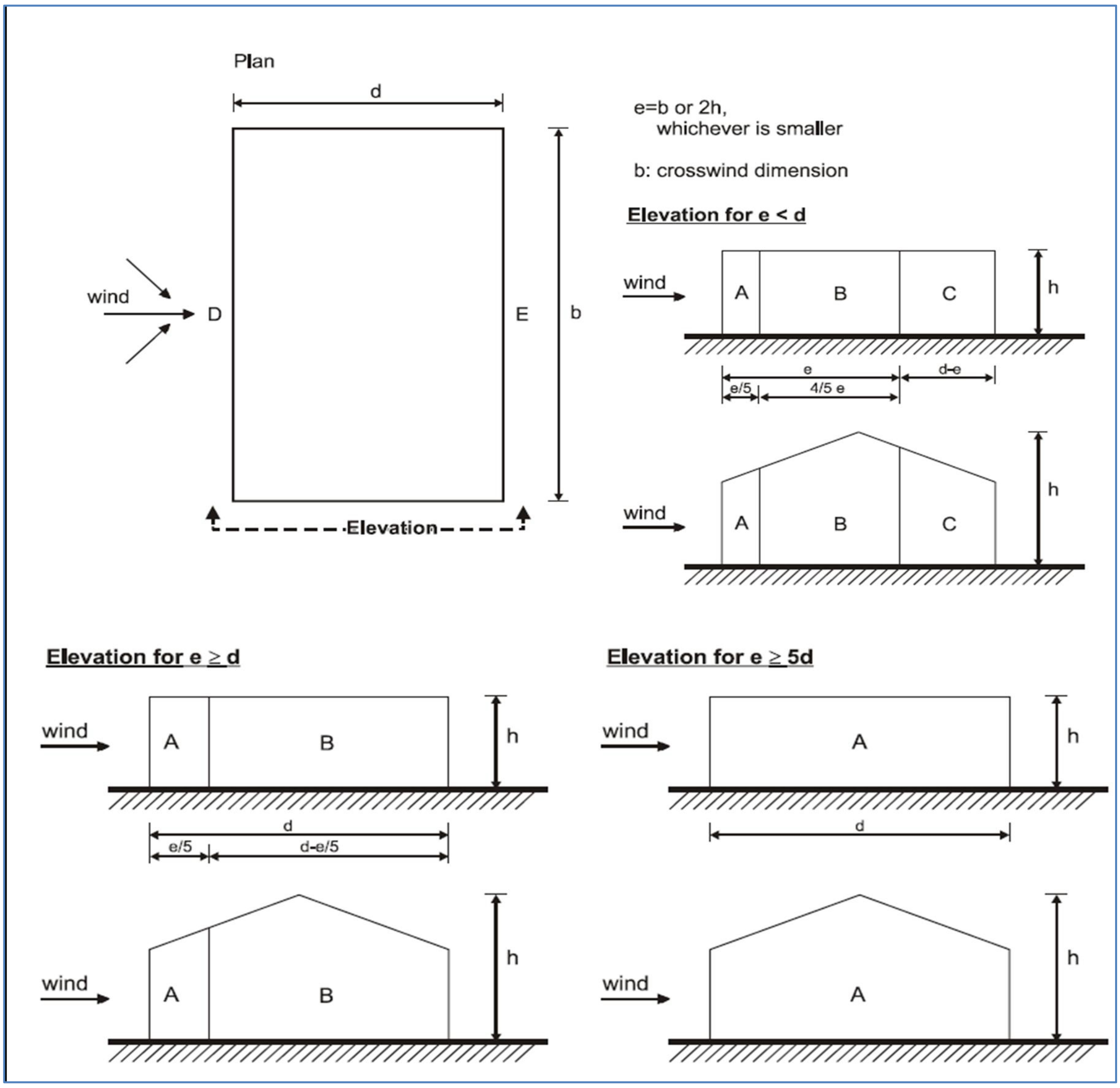
External Pressure Coefficient

The values of cpe,10 and cpe,1 may be given in the National Annex. The recommended values are given in Table 7.1, depending on the ratio h/d. For intermediate values of h/d, linear interpolation may be applied. The values of Table 7.1 also apply to walls of buildings with inclined roofs, such as duo pitch and mono-pitch roofs.

TTable 8. 1 Recommended values of Cpe for vertical walls of rectangular plan Building

Zone	A		B		C		D		E	
h/d	cpe,10	cpe,1	cpe,10	cpe,1	cpe,10	Cpe,1	cpe,10	cpe,1	cpe,10	cpe,1
5	-1.2	-1.4	-0.8	-1.1	-0.5		+0.8	+1.0	-0.7	
1	-1.2	-1.4	-0.8	-1.1	-0.5		+0.8	+1.0	-0.5	
≤ 0.25	-1.2	-1.4	-0.8	-1.1	-0.5		+0.7	+1.0	-0.3	

NOTE 2 For buildings with *h/d*> 5, the total wind loading may be based on the provisions given in Sections 7.6 to 7.8 and 7.9.2.



keys for vertical wall pressure surface division

III. Internal Pressure, Cpi

The internal pressure coefficient can be calculated using the following formula, if it is difficult to estimate μ accurately Cpi=+0.2 and Cpi=-0.3 is taken.

$$\mu = \frac{\sum \text{area of openings where } c_{pe} \text{ is negative or - 0.0}}{\sum \text{area of all openings}}$$

- ✓ Close to the openings at the leeward and parallel side (internal pressure), μ =0 and Cpi = **+0.2**
- ✓ Close all the openings at the windward direction (internal suction). μ = 1.0 and Cpi = **-0.3**

External wall surface zone in 3D mode

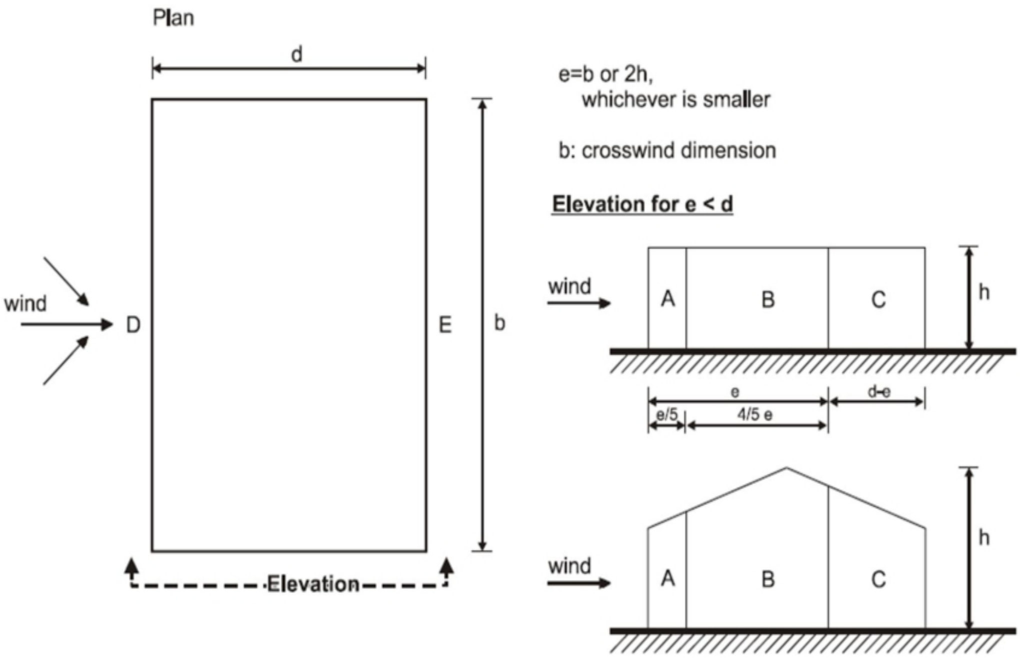


Figure 8. 2 The external pressure coefficients cpe,10 and cpe,1 for zone A, B, C, D and E

Wind Direction Ɵ=0°

h = 16.5 m

b = 24.3m, b is cross wind direction

d = 21.6m, d is along wind direction

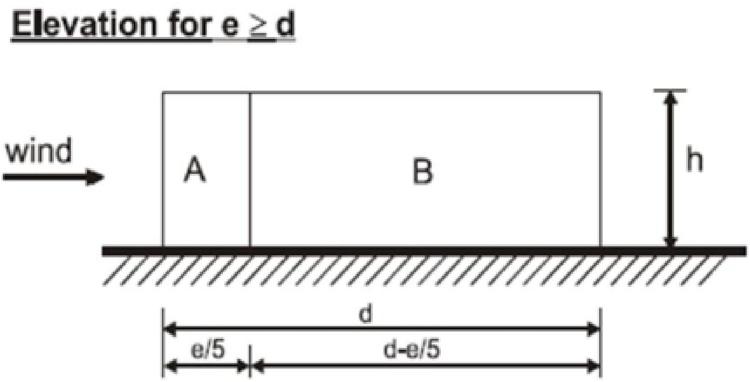
e = b or 2h, take whichever is smaller e = 24.3mm or 33m, there for e=24.3m

Condition 1: h≤b, 16.5m≤24.3m, satisfied

Condition 2: b<h≤2b,24.3m<16.5m<48.6m, not satisfied

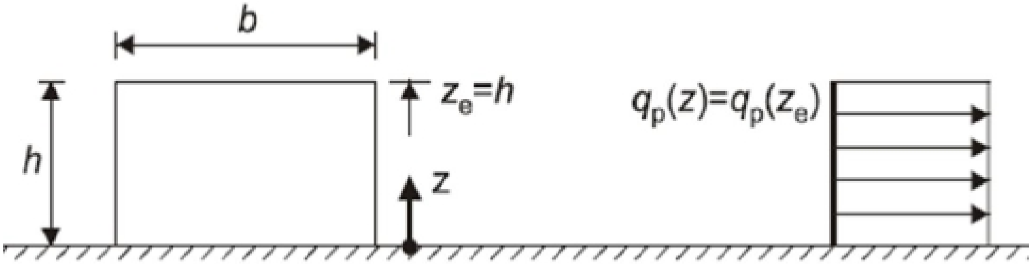
Condition 3: h>2b, 16.5m>48.6m, not satisfied

Since, e≥d, 24.3m≥21.6m, the following wall surface division is considered



Since, h≤b,16.5m≤24.3m, the building considered asone part with **case b** surface division.

e/5= 24.3/5 = 4.86m and d-e/5 = 21.6-4.86= 16.74m



- ✓ A=0.2e*h=4.86*16.5=80.19m^2
- ✓ B=(d-0.2e)*h=16.74*16.5=276.21m^2
- ✓ D=b*h=24.3*16.5=400.95m^2
- ✓ E=b*h=24.3*16.5=400.95m^2

  Since all areas of the surface are greater than 10m2 Cpe=Cpe,$_{10}$

Interpolation should be done between h/d 0.25 and h/d=1, to calculate the value of Cpe at h/d=16.5.5/21.6=**0.764**.

	A		B		D		E	
Area(m^2)	80.19		276.21		400.95		400.95	
h/d	Cpe 10	Cpe 1	Cpe 10	Cpe 1	Cpe 10	Cpe 1	Cpe 10	Cpe 1
0.25	-1.2	-1.4	-0.8	-1.1	0.7	1	0.3	
0.764	-1.2	-1.4	-0.8	-1.1	0.768	1	-0.248	
1	-1.2	-1.4	-0.8	-1.1	0.8	1	-0.5	

Figure4. 4 pressure distribution on the wall for e≥d and h≤b

The net wind surface pressure on each wall zone is calculated as follow, (Cpe-) is 0 here.

Wnet = qp(z)*[Cpe(-) – Cpi(+)]

Wnet = qp(z)*[Cpe(-) – Cpi(-)]

Wnet = qp(z)*[Cpe(+) – Cpi(+)]

Wnet = qp(z)*[Cpe(+) – Cpi(-)]

Wnet=qp(z)*(Cpe-Cpi) =0.456 *(Cpe-Cpi)KN/m^2

Zone	**A**	**B**	**D**	**E**
Area,m^2	80.19	276.21	400.95	400.95
Cpe	-1.2	-0.8	0.768	-0.248
Cpi,+	0.2	0.2	0.2	0.2

Cpi,-	-0.3	-0.3	-0.3	-0.3
Cpe- Cpi,+	-1.4	-1	0.568	-0.448
Cpe-Cpi,-	-0.9	-0.5	1.068	0.052
Wnet	-0.638	-0.456	0.259	-0.204
Wnet	-0.410	-0.228	0.487	0.024

Table 8. 2 calculation of external wind pressure when h ≤ b and wind direction ϴ=0°

Wind Direction ϴ=90°

h = 16.5 m

b= 21.6m, d is across wind direction

d = 24.3m, b is along wind direction

e = b or 2h, take whichever is smaller e = 21.6mm or 33m, therefor e=21.6m

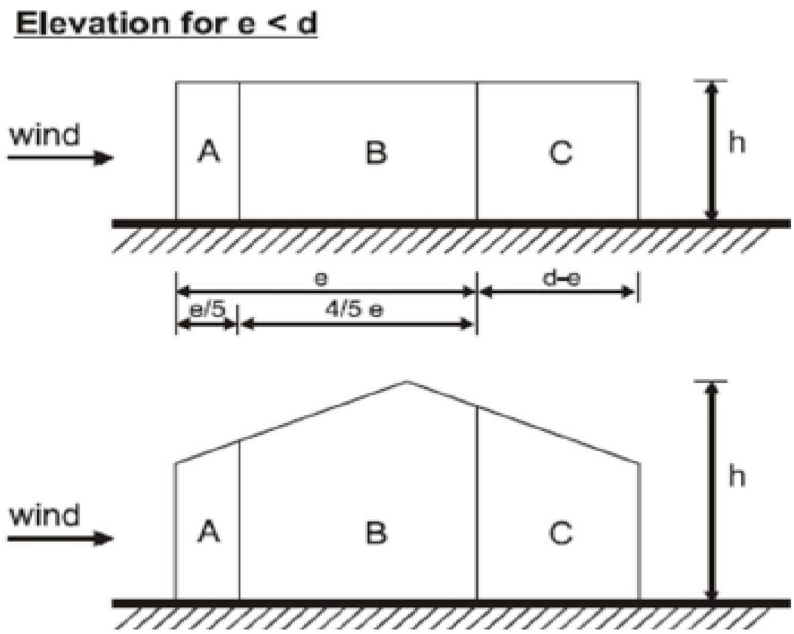
Condition 1: h≤b, 16.5m≤21.6m, satisfied

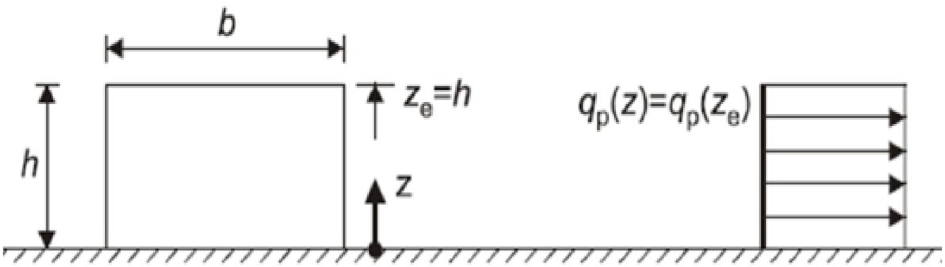
Condition 2: b<h≤2b, 21.6m<16.5m<43.2m, not satisfied

Condition 3: h>2b, 16.5m>43.2m, not satisfied

Since, e<d, 21.6m<24.3, andh≤b, 16.5m≤21.6m wall surface division Case a is used, and the whole height of the building has one part for analysis

e=21.6me/5= 4.32m 4e/5=17.28m and d-e = 24.3-21.6=2.7m





- ✓ A=0.2e*h=4.32*16.5=71.28m^2
- ✓ B=4e/5*h=17.28*16.5=285.12m^2
- ✓ C=(d-e)*h=2.7m*16.5m=44.5m^2
- ✓ D=b*h=21.6*16.5=356.4m^2
- ✓ E=b*h=21.6*16.5=356.4m^2

Since all areas of the surface are greater than 10m^2 Cpe=Cpe,$_{10}$

Interpolation should be done between h/d 0.25 and h/d=1, to calculate the value of Cpe at h/d=16.5.5/24.3=**0.679**.

	A		B		C		D		E	
Area,m^2	71.28		285.12		44.55		356.4		356.4	
h/d	Cpe 10	Cpe 1	Cpe 10	Cpe 1	Cpe 10	Cpe 1	Cpe 10	Cpe 1	Cpe 10	Cpe 1
0.25	-1.2	-1.4	-0.8	-1.1	-0.5		0.7	1	0.3	
0.679	-1.2	-1.4	-0.8	-1.1	-0.5		0.757	1	-0.1576	
1	-1.2	-1.4	-0.8	-1.1	-0.5		0.8	1	-0.5	

The net wind surface pressure on each wall zone is calculated as follow, (Cpe-) is 0 here.

Wnet = qp(z)*[Cpe(-) – Cpi(+)]

Wnet = qp(z)*[Cpe(-) – Cpi(-)]

Wnet = qp(z)*[Cpe(+) – Cpi(+)]

Wnet = qp(z)*[Cpe(+) – Cpi(-)]

Wnet=qp(z)*(Cpe-Cpi) =0.456 *(Cpe-Cpi)KN/m^2

Zone	**A**	**B**	**C**	**D**	**E**
Area,m^2	71.28	285.12	44.5	356.4	356.4
Cpe	-1.2	-0.8	-0.5	0.757	-0.158
Cpi,+	0.2	0.2	0.2	0.2	0.2
Cpi,-	-0.3	-0.3	-0.3	-0.3	-0.3
Cpe- Cpi,+	-1.4	-1	-0.7	0.557	-0.358
Cpe- Cpi,-	-0.9	-0.5	-0.2	1.057	0.142
Wnet(+)	-0.638	-0.456	-0.319	0.254	-0.163
Wnet(-)	-0.410	-0.228	-0.091	0.482	0.065

Table 8. 3 Maximum wind load lumped at each floor level

floor level	0degre	90degree	**Maximum,F,KN**
Roof	-5.301	-5.627	**-5.627**
5^{th}	-10.601	-11.254	**-11.254**
4^{th}	-10.601	-11.254	**-11.254**
3^{rd}	-10.601	-11.254	**-11.254**
2^{nd}	-10.601	-11.254	**-11.254**
1^{st}	-10.601	-11.254	**-11.254**
Ground	-5.301	-5.627	**-5.627**

8.3 Earth Quake Analysis

Introduction

Earth quack is a natural ground disturbance motion primary caused by the movement of molten magma under the ground due to temperature difference.

The building which are going to design are located at Adama city which has category-IV seismic characteristics according to new Ethiopian design code. If no clear geological investigation for seismic study is done assume ground type B and elastic spectrum type 2 for analysis and design.

Basic representation of seismic action (Page 9, Art. 1.4.2 – EBCS 8)

The earthquake motion at a given point of the surface is generally represented by an elastic ground acceleration spectrum, called "elastic response spectrum". Normalized elastic response spectra are shown in annex A – Figure A.1 of EBCS 8.

Seismic zone description of Adama city.	
Parameter	Description
Elastic response spectrum; *S*e(T)	Type 2
Category Importance factor for ordinary building	IV Importance class-II,Υ_1=1
Ground Type B	Vs,(30m/s)=360-800
Very dense sand and stiff clay	NSPT(blow/30cm)>50 Cu>250mpa Soil factor, S=1.2
For Ground Type B	T_B(s)=0.15

	$T_C(s)=0.5$ $T_D(s)=2$

Lateral load analysis

There are two methods of analysis

1) **Static analysis (or Seismic coefficient method),** ES EN 1998-1:2015,

This method is applicable when,

- Applied to buildings whose response is not significantly affected by higher modes of vibration.

  E.g. buildings which have regularity in plan and/or elevation
- For buildings that are regular in elevation and do have a fundamental period less than 2sec and 1four times the corner period Tc of the applicable design spectrum.
- Generally applicable to buildings up to 40 m in height

2) **Dynamic analysis (or Response spectrum method)**

Can be conducted for all types of buildings and especially for building subjected to higher mode of vibration and very tall building

Fundamental Requirements of Seismic Design

Structures in seismic region shall be designed and constructed in such a way the following requirements are met

- **No Collapse Requirement:**The structured may not undergo local and general collapse after seismic load, keeping structural integrity
- **Damage Limit Requirement:** The structure shall be designed & constructed to with stand a seismic load having a larger probability of occurrence than design seismic load without damage

Compliance Criteria

To satisfy functional requirements the following limit state should be checked

- **Ultimate Limit State:**Associated with collapse or other types of structural failure which endanger the safety of people
- **Serviceability Limit State:**Associated with damage occurrences corresponding which states beyond which specified service requirements are no longer met

Modal response spectrum analysis

ES EN 1998-1:2015 section 4.3.3.3.1 states that,

(1)P This type of analysis shall be applied to buildings which do not satisfy the conditions given ES EN 1998-1:2015 in 4.3.3.2.1(2) for applying the lateral force method of analysis.

(2)P The response of all modes of vibration contributing significantly to the global response shall be taken into account.

Non-linear methods

ES EN 1998-1:2015 section 4.3.3.4.1 states that,

(1)P The mathematical model used for elastic analysis shall be extended to include the strength of structural elements and their post-elastic behavior.

Selecting an appropriate type of analysis for our building

Check to use lateral force method of analysis

Requirements:

$T_1 \leq \begin{cases} 4Tc \\ 2.0sec \end{cases}$

Where

T_1 is fundamental period of vibration

Then according to ES EN 1998-1:2015 section 4.3.3.2.2 expression 4.6 for buildings with heights of up to 40 m the value of T1 (s) may be approximated by the following expression

$T_1 = CtH^{3/4}$

Where, Ct0.085 for moment resistant space steel frames,

0.075for moment resistant space concrete frames &for eccentrically braced steel frames

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.

Ct = 0.075

H = 19.95m+2.85m=22.8m including basement

$T_1 = 0.075*22.8^{3/4} = 0.783Sec$

As we have normal soil condition our site to be ground type A and we assumed type 2 spectrum

According to ES EN 1998-1:2015 section 3.2.2.2 table 3.3

$T_1 \leq \begin{cases} 4Tc \\ 2.0s \end{cases} = \begin{cases} 4 * 0.5 = 2Sec \\ 2Sec \end{cases}$

$0.783Sec \leq \begin{cases} 2Sec \\ 2Sec \end{cases}$ ……………..Ok

Check the criteria for regularity in elevation

According to ES EN 1998-1:2015 section 4.2.3.3 the criteria for regularity in elevation are:

- All lateral load resisting systems, such as cores, structural walls, or frames, shall run without interruption from their foundations to the top of the building or, if setbacks at different heights are present, to the top of the relevant zone of the building.
- Both the lateral stiffness and the mass of the individual story shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.

In our building no set backs are present as our building satisfy all above criteria. It is also regular

Therefore, static method of analysis is used for seismic design.

8.4 Mass and center of mass calculation for each member at each floor

Center of mass calculation for Basement Floor Column=2.85m/2=1.425m

Designation		H(m)	L(m)	B(m)	V(m³)	ϒc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	WY
1	C	1.425	0.6	0.6	0.513	25	12.825	3.8	-1.5	48.735	-19.2375
	E	1.425	0.6	0.6	0.513	25	12.825	15	-1.5	192.375	-19.2375
2	B	1.425	0.6	0.4	0.342	25	8.55	0	0	0	0
	C	1.425	0.6	0.4	0.342	25	8.55	3.8	0	32.49	0
	D	1.425	0.6	0.4	0.342	25	8.55	9.4	0	80.37	0
	E	1.425	0.6	0.4	0.342	25	8.55	15	0	128.25	0
	F	1.425	0.6	0.4	0.342	25	8.55	18.8	0	160.74	0
3	B	1.425	0.6	0.4	0.342	25	8.55	0	4.7	0	40.185
	C	1.425	0.6	0.4	0.342	25	8.55	3.8	4.7	32.49	40.185
	D	1.425	0.6	0.4	0.342	25	8.55	9.4	4.7	80.37	40.185
	E	1.425	0.6	0.4	0.342	25	8.55	15	4.7	128.25	40.185
	F	1.425	0.6	0.4	0.342	25	8.55	18.8	4.7	160.74	40.185
4	B	1.425	0.6	0.4	0.342	25	8.55	0	9.65	0	82.5075
	C	1.425	0.6	0.4	0.342	25	8.55	3.8	9.65	32.49	82.5075
	D	1.425	0.6	0.4	0.342	25	8.55	9.4	9.65	80.37	82.5075
	E	1.425	0.6	0.4	0.342	25	8.55	15	9.65	128.25	82.5075

	F	1.425	0.6	0.4	0.342	25	8.55	18.8	9.65	160.74	82.5075
5	B	1.425	0.6	0.4	0.342	25	8.55	0	15.1	0	129.105
	C	1.425	0.6	0.4	0.342	25	8.55	3.8	15.1	32.49	129.105
	D	1.425	0.6	0.4	0.342	25	8.55	9.4	15.1	80.37	129.105
	E	1.425	0.6	0.4	0.342	25	8.55	15	15.1	128.25	129.105
	F	1.425	0.6	0.4	0.342	25	8.55	18.8	15.1	160.74	129.105
6	B	1.425	0.6	0.4	0.342	25	8.55	0	20.2	0	172.71
	C	1.425	0.6	0.4	0.342	25	8.55	3.8	20.2	32.49	172.71
	D	1.425	0.6	0.4	0.342	25	8.55	9.4	20.2	80.37	172.71
	E	1.425	0.6	0.4	0.342	25	8.55	15	20.2	128.25	172.71
	F	1.425	0.6	0.4	0.342	25	8.55	18.8	20.2	160.74	172.71
						TOTAL	**213.75**			**2250.36**	**2084.063**

Center of mass calculation for slab on ground floor

slab number		W(m)	D(m)	V(m³)	ϒc	Weight	Xm(m)	Ym(m)	WX	WY
S1	5.1	3.8	0.25	4.845	24	116.28	1.9	17.65	220.93	2052.342
S2	5.6	5.1	0.25	7.14	24	171.36	6.6	17.65	1130.98	3024.5
S3	5.6	5.1	0.25	7.14	24	171.36	12.2	17.65	2090.59	3024.5
S4	5.1	3.8	0.25	4.845	24	116.28	16.9	17.65	1965.13	2052.3
S5	5.45	3.8	0.25	5.1775	24	124.26	1.9	12.375	236.09	1537.7
S6	5.6	5.45	0.25	7.63	24	183.12	6.6	12.375	1208.59	2266.1
S7	5.6	5.45	0.25	7.63	24	183.12	12.2	12.375	2234.06	2266.1
S8	5.4	3.8	0.25	5.1775	24	124.26	16.9	12.375	2099.9	1537.7

	5								9	
S9	5.6	4.95	0.25	6.93	24	166.32	6.6	7.175	1097.71	1193.3
S10	5.6	4.95	0.25	6.93	24	166.32	12.2	7.175	2029.10	1193.3
S11	4.7	3.8	0.25	4.465	24	107.16	1.9	2.35	203.60	251.8
S12	5.6	4.7	0.25	6.58	24	157.92	6.6	2.35	1042.27	371.1
S13	5.6	4.7	0.25	4.465	24	107.16	12.2	2.35	1307.35	251.8
S14	4.7	3.8	0.25	1.482	24	35.568	16.9	2.35	601.10	83.6
					Total	**2001.6**			**18136.2**	**21496.21**

Table 8. 4 Center of mass calculation for beam along number axis of grade beam

designation		L(m)	D(m)	W(m)	V(m³)	ϒc (KN/m3)	Weight	Xm (m)	Ym(m)	WX	WY
2	BC	3.8	0.6	0.4	0.912	25	22.8	1.9	-1.5	43.32	-34.2
	CD	5.6	0.6	0.4	1.344	25	33.6	6.6	-1.5	221.76	-50.4
	DE	5.6	0.6	0.4	1.344	25	33.6	12.2	-1.5	409.92	-50.4
	EF	3.8	0.6	0.4	0.912	25	22.8	17.6	-1.5	401.28	-34.2
3	BC	3.8	0.6	0.4	1.344	25	33.6	1.9	4.7	63.84	157.92
	CD	5.6	0.6	0.4	1.344	25	33.6	6.6	4.7	221.76	157.92
	DE	5.6	0.6	0.4	1.344	25	33.6	12.2	4.7	409.92	157.92
	EF	3.8	0.6	0.4	0.912	25	22.8	17.6	4.7	401.28	107.16
4	BC	3.8	0.6	0.4	0.912	25	22.8	1.9	9.65	43.32	220.02
	CD	5.6	0.6	0.4	1.344	25	33.6	6.6	9.65	221.76	324.24
	DE	5.6	0.6	0.4	1.344	25	33.6	12.2	9.65	409.92	324.24
	EF	3.8	0.6	0.4	0.912	25	22.8	17.6	9.65	401.28	220.02
5	BC	3.8	0.6	0.4	0.912	25	22.8	1.9	15.1	43.32	344.28
	CD	5.6	0.6	0.4	1.344	25	33.6	6.6	15.1	221.76	507.36

	DE	5.6	0.6	0.4	1.344	25	33.6	12.2	15.1	409.92	507.36
	EF	3.8	0.6	0.4	0.912	25	22.8	17.6	15.1	401.28	344.28
6	BC	3.8	0.6	0.4	0.912	25	22.8	1.9	20.2	43.32	460.56
	CD	5.6	0.6	0.4	1.344	25	33.6	6.6	20.2	221.76	678.72
	DE	5.6	0.6	0.4	1.344	25	33.6	12.2	20.2	409.92	678.72
	EF	3.8	0.6	0.4	0.912	25	22.8	17.6	20.2	401.28	460.56
						TOTAL	**574.8**			**5401.9**	**5482.1**

Center of mass calculation for Wall dead load on Ground floor

Designation		L,m	H,m	`W, m	V(m³)	ϒHCB	W	Xm,m	Ym, m	WX	WY
A	12	1.5	2	0.2	0.6	14	8.4	0.75	1	6.3	8.4
	23	4.7	2	0.2	1.88	14	26.32	3.85	1.575	101.332	41.454
	34	4.95	2	0.2	1.98	14	27.72	8.675	1.575	240.471	43.659
	45	5.45	2	0.2	2.18	14	30.52	13.88	1.575	423.465	48.069
	56	5.1	2	0.2	2.04	14	28.56	19.15	1.575	546.924	44.982
	67	1.35	2	0.2	0.54	14	7.56	22.38	1	169.155	7.56
B	12	1.5	2	0.2	0.6	14	8.4	0.75	1	6.3	8.4
	23	4.7	3.15	0.2	2.961	14	41.454	3.85	1.575	159.5979	65.29005
	34	4.95	3.15	0.2	3.1185	14	43.659	8.675	1.575	378.74183	68.76293
	45	5.45	3.15	0.2	3.4335	14	48.069	13.88	1.575	666.95738	75.70868
	56	5.1	3.15	0.2	3.213	14	44.982	19.15	1.575	861.4053	70.84665
	67	1.35	3.15	0.2	0.8505	14	11.907	22.38	1	266.41913	11.907
C	12	1.5	3.15	0.15	0.70875	14	9.9225	0.75	1	7.441875	9.9225
	23	4.7	3.15	0.15	2.22075	14	31.0905	3.85	1.575	119.69843	48.96754
	34	4.95	3.15	0.15	2.33888	14	32.74425	8.675	1.575	284.05637	51.57219
	45	5.45	3.15	0.15	2.57513	14	36.05175	13.88	1.575	500.2180	56.78151

										3	
	56	5.1	3.15	0.15	2.40975	14	33.7365	19.15	1.575	646.05398	53.13499
	67	1.35	3.15	0.15	0.63788	14	8.93025	22.38	1	199.81434	8.93025
D	12	1.5	3.15	0.15	0.70875	14	9.9225	0.75	1	7.441875	9.9225
	23	4.7	3.15	0.15	2.22075	14	31.0905	3.85	1.575	119.69843	48.96754
	34	4.95	3.15	0.15	2.33888	14	32.74425	8.675	1.575	284.05637	51.57219
	45	5.45	3.15	0.15	2.57513	14	36.05175	13.875	1.575	500.21803	56.78151
	56	5.1	3.15	0.15	2.40975	14	33.7365	19.15	1.575	646.05398	53.13499
	67	1.35	3.15	0.15	0.63788	14	8.93025	22.38	1	199.81434	8.93025
E	12	1.5	3.15	0.15	0.70875	14	9.9225	0.75	1	7.441875	9.9225
	23	4.7	3.15	0.15	2.22075	14	31.0905	3.85	1.575	119.6983	48.96754
	34	4.95	3.15	0.15	2.33888	14	32.74425	8.675	1.575	284.05637	51.57219
	45	5.45	3.15	0.15	2.57513	14	36.05175	13.875	1.575	500.21803	56.78151
	56	5.1	3.15	0.15	2.40975	14	33.7365	19.15	1.575	646.05398	53.13499
	67	1.35	3.15	0.15	0.63788	14	8.93025	22.375	1	199.81434	8.93025
F	12	1.5	3.15	0.2	0.945	14	13.23	0.75	1	9.9225	13.23
	23	4.7	3.15	0.2	2.961	14	41.454	3.85	1.575	159.5979	65.29005
	34	4.95	3.15	0.2	3.1185	14	43.659	8.675	1.575	378.74183	68.76293
	45	5.45	3.15	0.2	3.4335	14	48.069	13.875	1.575	666.95738	75.70868
	56	5.1	3.15	0.2	3.213	14	44.982	19.15	1.575	861.4053	70.84665
	67	1.35	3.15	0.2	0.8505	14	11.907	22.375	1	266.41913	11.907
G	12	1.5	2	0.2	0.6	14	8.4	0.75	1	6.3	8.4
	23	4.7	2	0.2	1.88	14	26.32	3.85	1.575	101.332	41.454

Table 8. 5 summary of weight and centroid of mass calculation for each floor

Floor Level	member	DL	LL	ΣWXi	ΣWYi	ΣWtot	XM	YM
Basement	column	213.75	0	2250.4	2084.1	213.8	10.53	9.75
	beam	1149.6	0	10744.6	11071.9	1149.6	9.35	9.63
	slab	2001.6	1772	34776.0	38279.6	2427.0	14.33	15.77
	stair	151.72	99.95	2364.2	1837.2	175.7	13.46	10.46
	wall	1918.9	0	20413.1	2885.1	1918.9	10.64	1.50
	∑	**5581**	**1872**	**72362.1**	**57854.5**	**6030.3**	**12.00**	**9.59**
Ground	column	450	0	4737.6	4387.5	450.0	10.53	9.75
	beam	1122	0	10370.0	10652.4	1122.0	9.24	9.49
	slab	2823.3	1772	42471.2	43888.7	3248.7	13.07	13.51
	stair	151.72	99.95	2364.2	1837.2	175.7	13.46	10.46
	wall	1918.9	0	20413.1	2885.1	1918.9	10.64	1.50
	∑	**6673.8**	**1872**	**82361.0**	**65526.1**	**7123.2**	**11.56**	**9.20**
first	column	236.25	0	2590.9	2286.9	236.3	10.97	9.68
	beam	1122	0	10370.0	10652.4	1122.0	9.24	9.49
	slab	2823.3	1772	42471.2	43888.7	3248.7	13.07	13.51
	stair	151.72	99.95	2364.2	1837.2	175.7	13.46	10.46
	wall	1918.9	0	20413.1	2885.1	1918.9	10.64	1.50
	∑	**6460.1**	**1872**	**80214.2**	**63425.5**	**6909.4**	**11.61**	**9.18**
second	column	236.25	0	2590.9	2286.9	236.3	10.97	9.68
	beam	1122	0	10370.0	10652.4	1122.0	9.24	9.49
	slab	2823.3	1772	42471.2	43888.7	3248.7	13.07	13.51
	stair	151.72	99.95	2364.2	1837.2	175.7	13.46	10.46
	wall	1918.9	0	20413.1	2885.1	1918.9	10.64	1.50
	∑	**6460.1**	**1872**	**80214.2**	**63425.5**	**6909.4**	**11.61**	**9.18**
third	column	236.25	0	2590.9	2286.9	236.3	10.97	9.68
	beam	1122	0	10370.0	10652.4	1122.0	9.24	9.49
	slab	2823.3	1772	42471.2	43888.7	3248.7	13.07	13.51
	stair	151.72	99.95	2364.2	1837.2	175.7	13.46	10.46
	wall	1918.9	0	20413.1	2885.1	1918.9	10.64	1.50
	∑	**6460.1**	**1872**	**80214.2**	**63425.5**	**6909.4**	**11.61**	**9.18**
fourth	column	236.25	0	2590.9	2286.9	236.3	10.967	9.680

	beam	336.6	0	3111.0	3195.7	336.6	9.242	9.494
	slab	2823.3	1772	42471.2	43888.7	3248.7	13.073	13.510
	stair	151.72	99.95	2364.2	1837.2	175.7	13.456	10.456
	wall	1918.9	0	20413.1	2885.1	1918.9	10.638	1.504
	∑	**5674.7**	**1872**	**72955.2**	**55968.9**	**6124.0**	**11.913**	**9.139**
fifth	column	27.675	0	260.1	198.6	27.7	9.400	7.175
	beam	55.875	0	592.0	54.3	55.9	10.595	0.972
	slab	346.5	221.8	3182.3	4077.3	399.7	7.961	10.200
	wall	172.31	0	1439.7	258.5	172.3	8.356	1.500
	∑	**602.36**	**221.8**	**5474.1**	**4588.6**	**655.6**	**8.350**	**6.999**
water tank floor	column	30.9	0	290.5	221.7	30.9	9.400	7.175
	beam	52.5	0	619.3	105.4	52.5	11.795	2.007
	slab	12.5	112.9	1178.4	224.6	39.6	29.767	5.673
	wall	209.86	0	1953.5	251.4	209.9	9.309	1.198
	∑	**305.76**	**112.9**	**4041.6**	**803.1**	**332.8**	**12.143**	**2.413**
roof	EGA sheet	13.076	8.329	218.7	108.5	15.1	14.505	7.200

Base shear force

According to ES EN 1998-1:2015 section 4.3.3.2.2,the seismic base shear forceFb, for each horizontal direction in which the building is analyzed, shall be determined using the expression:

$$Fb = Sd(T1)\cdot m\cdot\lambda$$

$S_d(T1)$ is the ordinate of the design spectrum at period T_1.

T1 is the fundamental period of vibration of the building for lateral motion in the direction considered.

m is the total mass of the building, above the foundation or above the top of a rigid basement,

λ is the correction factor, the value of which is equal to: $\lambda = 0.85$ if $T_1 < 2TC$ and the building has more than two stories or $\lambda = 1.0$ otherwise.

Design spectrum for elastic analysis

According to ES EN 1998-1:2015 section 3.2.2.5, the design spectrum for elastic analysis of the seismic action, Sd(T), is defined by the following expressions

$0 \le T \le TB$: $$Sd(T) = ag * s * \left[\frac{2}{3} + \frac{T}{TB}\right] * \left[\frac{2.5}{\eta} - \frac{2}{3}\right]$$

$TB \le T \le TC$: $$Sd(T) = ag * s * \frac{2.5}{q}$$

$TC \le T \le TD$: $$Sd(T) = max\begin{cases} ag * s * \frac{2.5}{q}\left(\frac{Tc}{T}\right) \\ \beta * ag \end{cases}$$ this condition is satisfied as 0.5≤ 0.783≤ 2

$$TD \leq T: \qquad Sd(T) = max\begin{cases} ag * s * \frac{2.5}{q}\left(\frac{Tc*TD}{T^2}\right) \\ \beta * ag \end{cases}$$

T is the vibration period of a linear single-degree-of-freedom system,

ag is the design ground acceleration on type A ground ($ag = \gamma I.ag_R$),

TB is the lower limit of the period of the constant spectral acceleration branch,

TC is the upper limit of the period of the constant spectral acceleration branch,

TD is the value defining the beginning of constant displacement response range of the spectrum,

S is the soil factor, and

$\beta$is the lower bound factor for the horizontal design spectrum. The recommended value is 0.2

values of the periods TB, TC and TD and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type.

According to ES EN 1998-1:2015 section 2.2.5 for ground type A and for type 2 elastic response spectra the values of S, TB, TC and TD taken from ES EN 1998-1:2015 table 3.3

Type 2 elastic response spectra for ground type B

Ground type	S	TB(s)	TC(s)	TD(s)
B	1.2	0.15	0.5	2.0

The ground acceleration,ag=$\gamma 1 * ag_R$, but ag_R=$a0$ from national annex of Ethiopia from zonation map Ethiopia for Adama (Zone -IV) is a_o/g =0.15

$\gamma_I$is importance factor and According to ES EN 1998-1:2015 section 4.2.5 table 4.3 for importance class II and Ordinary buildings shall be equal to 1.0.

ao=0.15*9.81=1.4715m/s^2

q The behavior factor, according to ES EN 1998-1:2015 section 5.3.3 a q of 1.5 may be used in deriving the seismic action for DCM regardless of structural system and regularity in elevation

ag=1*1.4715m =1.4715m/s^2

$$TC \leq T \leq TD: Sd(T) = max\begin{cases} ag * s * \frac{2.5}{q}\left(\frac{Tc}{T}\right) \\ \beta * ag \end{cases}$$

q=qo*kw$\geq$ 1.5

Where for structural type frame system, ductility class medium (DCM)

qo= 3.0*$\frac{\alpha u}{a1}$ on table5.1 of ES EN 8, 2015. where, $\alpha u \alpha 1$, is multiplication factor, for multi-story, multi bay frame or equivalent dual structure $\frac{\alpha u}{a1}$ = 1.3.

Kw = is the prevailing factor failure mode in structural system. It is equal to 1.0 for frame and frame equivalent system.

Then q = 3.0*1.3*1 = 3.9

$$Sd(T) = max\begin{cases} 1.4715 * 1.2 * \frac{2.5}{3.9}\left(\frac{0.5}{0.783}\right) = 0.723 \\ 0.2 * 1.4715 = 0.2943 \end{cases}$$

$Sd(T) = 0.723 m/s^2$

The total mass of the building

According to ES EN 1998-1:2015 section 3.2.4

The inertial effects of designing seismic action shall be evaluated by taking into account the presence of masses associated with all gravity loads appearing in the following action combination.

$\Sigma Gk, + \Sigma \psi E, i * Qk, i$

Where

ψE,i is the combination coefficient for variable action

Gk,j Characteristic value of permanent action ,j

Qki Characteristic value of loading variable action, i

The combination coefficient,,

$\psi E, = \varphi * \psi 2, i$

Where

φ Is 0.8 for story with correlated occupancies and for category A building according to ES EN 1998-1:2003 section 4.2.4 table 4.2, and

ψ2,i According to ES EN 1998-1:2003 section A1.2.2 table A1.1 for category A Building The recommended value is 0.3.

Then $\psi E,$=0.8∗0.3=0.24

Therefore $\Sigma Gk, + \Sigma 0.24 * Qk, i$

Weight of floor and its mass center

$Fb = Sd(T1) \cdot m \cdot \lambda = 0.723 m/s^2 * m * 0.85 = 0.723 * 4101.35 * 0.85 = 2520.5 KN$

Table 8. 6 weight summary according to storey level

Storey level	ΣWtot	Hi	WHi	WHi/ΣWHi	Fb, KN	Fi, KN	cumulative Fi
basement floor	6030.33	-2.850	-17186.45	-0.054	2520.50	-137.010	-137.01
ound floor	7123.18	3.150	22438.03	0.071	2520.50	178.875	41.865
first floor	6909.43	6.300	43529.43	0.138	2520.50	347.015	388.881
second floor	6909.43	9.450	65294.15	0.207	2520.50	520.523	909.403
third floor	6909.43	12.60	87058.86	0.275	2520.50	694.030	1603.434
fourth floor	6124.03	15.75	96453.53	0.305	2520.50	768.924	2372.358
WT floor	332.846	17.75	5908.03	0.019	2520.50	47.099	2419.456
fifth floor	655.577	18.75	12292.08	0.039	2520.50	97.992	2517.448
roof	19.187	19.95	382.787	0.001	2520.50	3.052	2520.500
Total			**316170.44**				

9 Design Of Shear Wall

9.1 Introduction

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

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

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

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

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

Where:

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

k=relative eccentricity ratio

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

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

9.1.1 Basement shear wall design

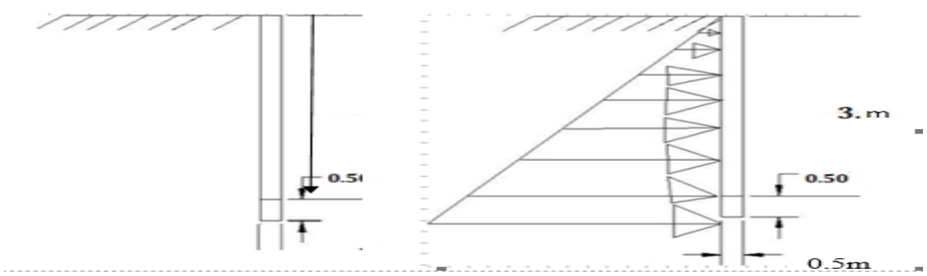


Figure 9. 1 basement shear wall

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

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

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

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

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

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

Slender wall: is a wall other than stocky wall.

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

9.1.2 Concrete Cover Determination

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

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

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

Where;

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

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

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

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

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

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

a) Cover Design for Bond

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

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

b) Cover Design for Corrosion/Durability

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

But based on the table Exposure class according to Table 4.1 the exposure class is Reduced 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

Cmin,dur=20mm

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

$Therefore$; $Cmin$= 20mm

c) ΔCdev (allowance in Design for Variation

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

9.2 Determination of depth for deflection

According to ES EN 1992:2015; the limit state of deformation may be checked by either: by limiting the span/depth ratio, according to 7.4.2 or by comparing a calculated deflection, according to 7.4.3, with a limit value

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

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

Where;

l/d - is the limit span/depth

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

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

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

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

F1=300/$\boldsymbol{\sigma s}$ =500/(fyk*As, req/As,pro)

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

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

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

K = 1.3 for end span

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

P_0=$\sqrt{fck * 10^{-3}}$=$\sqrt{20 * 10^{-3}}$=0.00447

For end span of one-way continuous slab

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

K = 1.3, $\frac{l}{d}$=20.27 because we used S460 multiply the value by $\frac{500}{fyk} = 1.25$

$\frac{l}{d}$=20.27*1.25=25.34 =l=lx= 3800mm d= 149.96mm

Using ϕ10 and cover 34mm H =149.96+ 30 +10/2= 184.96Use D = 200mm

d = 200-30-5 =165mm

Design of reinforcement

D = 200mm

d = 200 – 30 -10/ 2= 165mm

A-Geometry of the wall:

Height of the wall (H) =3.0m

Thickness of the wall (t) =200mm

Strap beam depth (D) =400mm

Strap beam width (B) =200mm

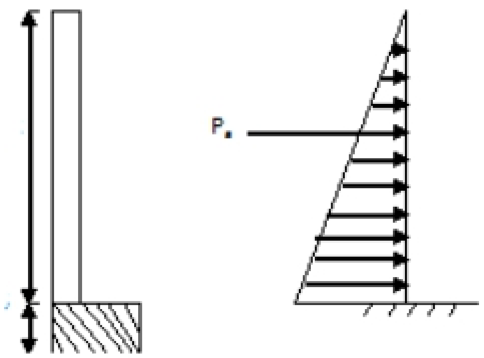
Total height of wall= 3+0.4=3.4m

Length of the wall (L) = 42.6m

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

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

Shear wall design for basement



Load calculation

Assume $\gamma soil = 18.5KN/m^3$, $\varphi = 30^0$

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

$Kp = 1/Ka = 3.03$

Active earth pressure

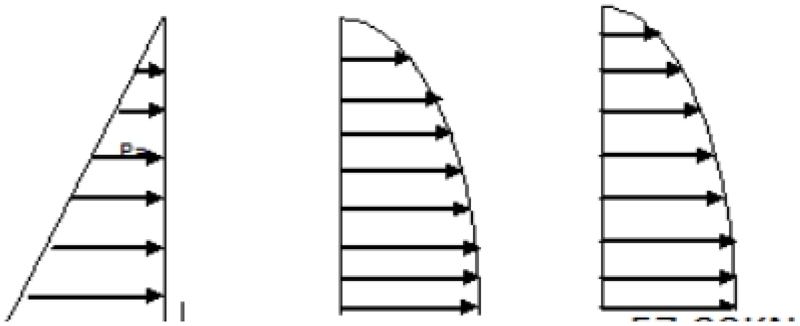
$Pa = 1/2Ka\gamma h^2= 27.47KN$

Factored Pa = 1.35Pa = 37.085KN

Determination of design shear and moment

V = 1/2pa*h= ½*37.085*3= 55.63KN

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



37.085KN/m 55.63 KN 55.63KNm

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

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

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

Etot = eo+e2+ ea

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

ea=Le/300≥20mm

Where, Le is effective buckling length of the wall

Assuming non-sway mode Le=0.7*L=0.7*3.8=2.66m.

ea=2660/300=8.87mm<20mm.

Therefore, take ea=20mm.

9.2.1 Determination of design eccentricity in out-plan direction

2-First order eccentricity:

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

3-second order eccentricity:

e2=0.4 $* h *(\frac{le}{10*h})^2$ where, h=length of the wall=42.6m.

= 0.4*42.6*(2.66/(10*42.6))2=0.87

Therefore, total eccentricity, etot=2660mm+0.87mm+20mm=2680.87mm

Relative eccentricity: According to EBCS 2-1995, section 6.2.2.1. The relative eccentricity is the ratio of total eccentricity to the column width of the same direction.

erel= etot/h=2.68/42.6=0.063

Determination of design eccentricity in-plan direction:

1-First order eccentricity:

No moment is carried out in this direction so:

eo=Md/Nsd=0

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

Therefore: etot=0.87mm+20mm=20.87mm.

From this erel=20.87/200=0.104 since D=200mm in this direction

Relative eccentricity ratio,k:

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

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

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

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

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

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

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

Check Depth for Flexure

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

Mrd= 84.18KNm/m>55.63KNm/m………………Ok

Reinforcement calculation

Longitudinal Reinforcement

At the Support

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

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

Ast, min=max$\begin{cases} \frac{0.26*2.2*1000*165}{460} = 205.75 \\ 0.0013*1000*165 = 214.5 \end{cases}$

Ast, min=205.75mm^2/m

Ast, max = 0.04Ac……………………EBCS EN1992: 2014 Art. 9.2.1.1(3)

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

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

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

Smax=400mm

Msd=55.63KNm/m

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

From General Design chart No 1,

Kz =0.884

As,cal=$\frac{Msd}{fydxdxKz}=\frac{55.63x10^6}{400x165x0.884}$

=1002.52mm^2/m it is between As, min and As,max……………OK

take As, cal=1002.52mm^2

Now determine the spacing of Φ10mm reinforcement bars by taking

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

S cal=$\frac{bxas}{As\ cal}$ =$\frac{1000x78.5}{1002.52}$=75.59mm<400mm (maximum spacing)……….OK

Provide ϕ10c/c spacing 70mm

On the Span

Msd= 55.63KNm/m

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

From General Design chart No 1,

Kz =0.951

As,cal=$\frac{Msd}{fydxdxKz}=\frac{55.63x10^6}{400x165x0.951}$ =982.27mm^2/m

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

Then, take As,cal =982.27mm^2/m

Now determine the spacing of Φ10mm reinforcement bars by taking

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

S cal=$\frac{bxas}{As\ cal}$=80.02mm<400mm (maximum spacing)………OK

Provide ϕ10c/c spacing 80mm

Transverse reinforcement

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

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

Smax=450mm

At the Support

As,t=0.2As=0.2*1002.52=200.4304mm^2/m

Now determine the spacing of Φ8 mm reinforcement bars by taking

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

S cal=$\frac{bxas}{As\ cal}$ =$\frac{1000*50.24}{200.304}$=219.09 mm<400mm (maximum spacing)………. OK

Provide ϕ8 c/c spacing 210mm

On the Span

As,t=0.2As=0.2*982.27=196.45mm^2/m

Now determine the spacing of Φ8 mm reinforcement bars by taking

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

S cal=$\frac{bxas}{As\ cal}$ =$\frac{1000*50.24}{196.45}$=255.73 mm< 450mm (minimum spacing)………. OK

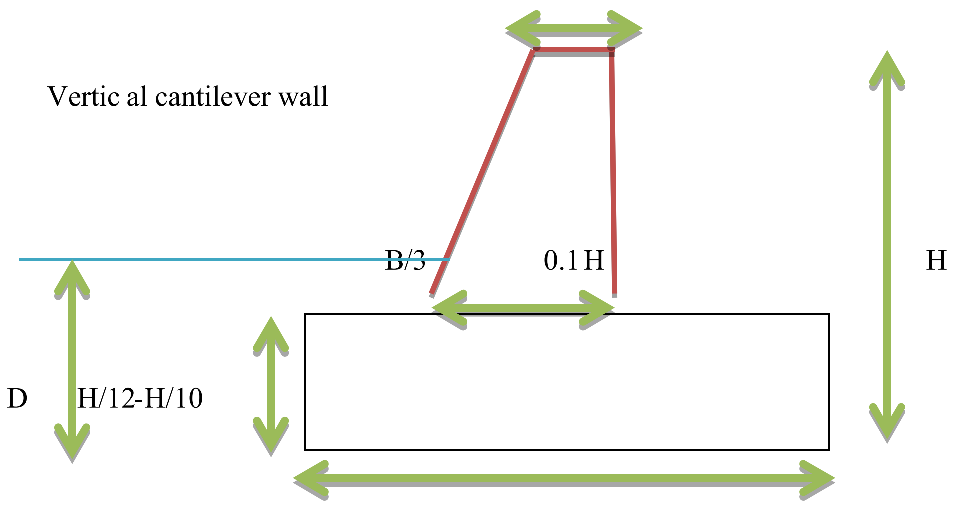
Provide ϕ8c/c spacing 250mm

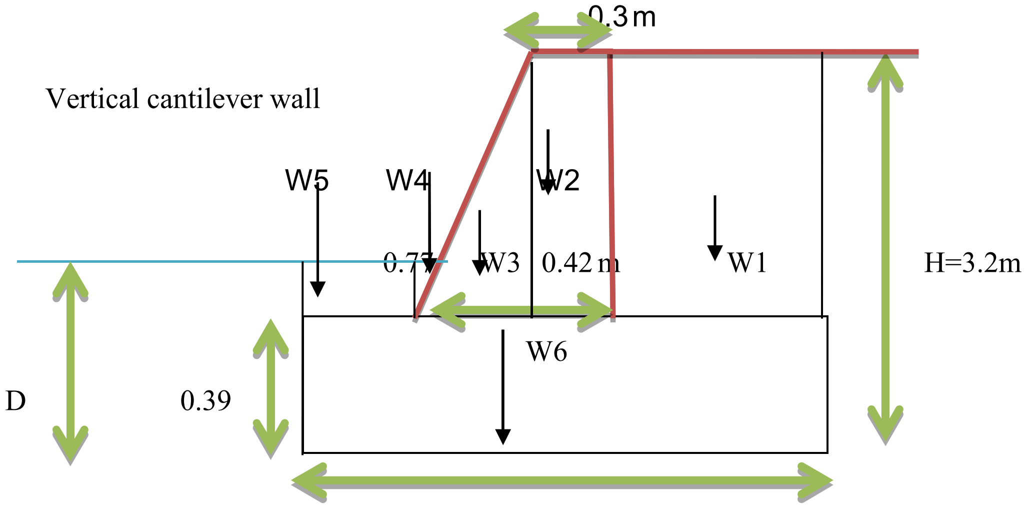
10 Design Of Retaining Wall

10.1 Introduction

- Retaining walls are structures used to provide stability of earth or other material where conditions prohibited the mass to assume its natural slope.

10.2 Proportioning of retaining wall





Calculation of load

For convenience the acting loads are segmented as shown above.

Weight =area *unit weight…..per unit width

W1=1.12m*2.81m*19KN/m^3=59.79 KN/m

W2=0.3m*2.81m*18KN/m^3=15.5KN/m

W3=0.5*2.81*0.12*18=1.08KN/m

W4=0.5*0.02*0.61*19=0.12KN/m

W5=0.75*0.61*19=8.7KN/m

W6=2.31*0.39*19=17.12KN/m

Ka= $\cos10*(\frac{cos10-\sqrt{\cos^2 \ 10-cos2\ 30}}{cos10+\sqrt{\cos^2 10}-cos2\ 30})$=0.34

Pa=0.5kaYH^2=0.5*0.34*19*4.4^2

=62.53KN/m

Pv = Pa*sin 10=10.86KN/m

Ph=Pa*cos10=61.58KN/m

Table 10.1 moments about the toe of the wall due to vertical and horizontal force

decryption	force		lever arm(m)	moment	
	vertical(KN)	horizontal(KN)		resisting(KNm)	over turning(KNm)
weight 1	59.79		1.75	141.925	
weight 2	15.5		1.04	21.32	
weight 3	1.08		0.85	3.485	
weight 4	0.12		0.78	0.0936	
weight 5	8.7		0.39	3.393	
weight 6	17.12		1.16	19.8592	
Pv	10.86		2.31	25.0866	
Ph		61.58	1.5	0	92.37

Sum vertical=142.5 sum horizontal=61.58 M resisting=215.2 M overturning=92.37

10.2.1 Stability of Retaining Wall

Check for Sliding Failure

Fs = resisting force /driving force

C=40Kpa δ=2/3

$\sum FR = W \times \tan\delta + \alpha \times c \times B$

B = bottom width of wall = 1.7m

W = summation of all vertical load on the wall

C = cohesion of the soil under base α = adhesion factor α = 0.55 for c/pa<1.5 δ = angle of wall friction ≈ Φ= 30^0

$\sum FR = W \times \tan\delta + \alpha \times c \times B = 142.4 \times \tan 20 + 0.55 \times 40 \times 2.31$

= 102.65KN/m

Fs=102.65/61.58=1.7≥1.5............ok

10.2.2 Check Overturning Stability

Taking moment about the toe of the wall and considering clockwise direction as positive moments. The overturning stability is checked as;

Fs=∑Mr/∑Mo=215.2/92.37=2.33≥2……………..safe

10.2.3 Check for bearing capacity failure

X=∑Mnet/∑V=215.2-92.37/142.5=0.86

e=B/2-X=2.31/2-0.86=0.295<B/6……….OK!!!!!!!!

B/6=0.385

The pressure distribution under the base slab can be expressed as

σmax=ΣV/(1+6*e/B)=142.5/2.31(1+6*0.295/2.31)=109KPa<250Kpa…….ok

σmin=ΣV/(1-6*e/B)=14.42KPa>0…..ok

Then determine the ultimate bearing capacity of the soil by terzhagi equation

Qult=1.3C*NC+r*D*Nq+0.4B*r*Nr

Using Φ= 30^0read the value of

Nq= 22.46

Nc= 37.16

Nr= 19.73

Qult = 1.3*40*37.16+19*22.46*1+0.4*2.31*19*19.73 = 2705Kpa

Fs= Qult/σmax=2705.44/109=24.8>3…………ok!!

11 Design 0f Shallow Foundation

11.1 Introduction

Foundation is that part of a structure which transmits the weight of the structure to the ground.

The main objectives of a foundation are the following;

- To distribute the weight of the structure over larger area so as to avoid over loading of the soil beneath.
- To load the sub structure evenly so as to avoid unequal settlement.
- To provide a level surface for building operations
- To take the sub structure deep into the ground and thus increase its stability and avoid overturning

The requirements in design of foundations are: [3]

- The pressure on the soil should not exceed the bearing capacity of the soil, or soil doesn't fail in shear.
- The settlement of the structure should be within the permissible limits.

11.1.1 Selection of foundation types

The choice of the appropriate type of foundation is affected by:

- Type of superstructure to be supported: function and load that it transfers to the foundation.
- Subsurface condition and/or type of soil.
- Cost of foundation type.

Soil Type

- Since there was no profile of the soil, we assumed the ground soil type as dense sand and gravel. Hence from EBCS-7, Table 6.3 we found the presumed design bearing resistance to be 350KPa. And the soil pressure distribution was assumed to be planar.

1. Design constants

Reinforcement data

Material Data

We used Concrete C-25/30	fyk(Mpa) = 460
Fck =25	fyd(Mpa) =400
Fcd(Mpa) = 14.167	gms = 1.15

Fctm(Mpa) = 2.2104 Es(Gpa) =200

gmc = 1.5 main bar(mm)=20mm

11.2 Design of isolated footing

Column	Point	FZ	MX	MY
C1	1	4789.90	35.91	9.07
C2	2	6961.17	4.96	9.56
C3	3	681.65	9.09	-1.56
C4	4	6296.95	4.58	-9.61
C5	5	4702.34	6.22	-9.26

Table: The values of axial load, Mx, My are obtained from ETABS 2016 analysis result from base reaction

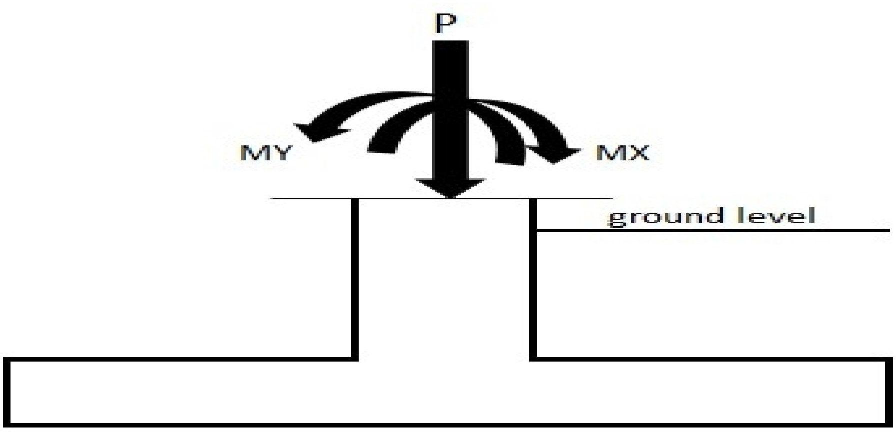


Figure 10. 1 Sample of isolated footing

Bearing capacity

The soil must be capable of carrying the loads from the structure placed up on it without a shear failure and with the resulting settlements being tolerable for the structure. The design bearing resistance can be taken from the presumed design bearing pressure for different soils according to EBCS-7, 1995 ART 6.10.2 Table 6.3 or can be calculated analytically according to EBCS-7, 1995 ART 6.5.22 for drained as well as un-drained conditions. We have assumed type of soil to be cohesive clay stiff soil. And thus we take the corresponding bearing capacity from the presumed design bearing pressure from EBCS-7, 1995 ART 6.10.2 Table 6.3 page 805 which is 350$\boldsymbol{kPa}$

Sample footing design of footing 1 at the axis A1

Dimension of column is 400x400mm

From bearing capacity of the soil using the theoretical method

$\sigma_{\text{Max}} \leq Qmax$

$$\sigma_{\text{Max/min}} = \frac{\text{p}}{\text{A}} * \left(1 \pm \frac{6\text{e}_\text{y}}{\text{B}} \pm \frac{6\text{e}_\text{x}}{\text{L}}\right) \dots \dots \dots \dots 1$$

Now finding the eccentricity of each loadings

Eccentricity for service load

$ex = \frac{MY}{FZ}, ey = \frac{MX}{FZ}$

Mx = 4.96 $ My= 9.56

$ey = \frac{4.96}{6961}$ =0.00071

$ex = \frac{9.56}{6961}$= 0.0013

Since our column is square assume square foundation B=L

Substituting these values in equation1, the dimensions of the footing is calculated as fallows

$$\sigma_{\text{All}} = \frac{\text{p}}{\text{A}} * \left(1 \pm \frac{6\text{e}_\text{y}}{\text{B}} \pm \frac{6\text{e}_\text{x}}{\text{B}}\right) \dots \dots \dots \dots 1$$

At limiting condition σ_{Max} = Qmax=350

350 = 6961/B*B*(1+6*0.00071/B+6*0.0013/B)

By rearranging the equation, we have the following simplified equation.

$350B^3$-1410.55B-51.623=0

By trial and error the value of B = 2.2m

$A=B^2=2.2*2.2=4.84m^2$

σmax=296.3Kpa, = 265.84Kpa

Since = 296.30< allowable(350Kpa)

So, the footing dimension is accepted

The stress at four corner of the footing

$$\sigma min = \frac{P}{A}\left(1 - 6 * \frac{ex}{B} - 6 * \frac{ey}{B}\right)$$

=265.84Kpa

$$\sigma int1 = \frac{P}{A}\left(1 + 6 * \frac{ex}{B} - 6 * \frac{ey}{B}\right)$$

=291.67Kpa

$$\sigma int2 = \frac{P}{A}\left(1 - 6 * \frac{ex}{B} + 6 * \frac{ey}{B}\right)$$

=291.20Kpa

$$\sigma max = \frac{P}{A}\left(1 - 6 * \frac{ex}{B} + 6 * \frac{ey}{B}\right)$$

=296.3Kpa

$\sigma aveg = (\sigma min + \sigma 1 + \sigma 2 + \sigma \max) /4$

=286.25Kpa

Depth determination

The critical section used for footing depth determination is similar to flat slab section.

These sections are

Wide beam shear section

Punching shear section

Bending moment section

Wide beam shear section

Critical section for shear is at distance d from the face of supports.

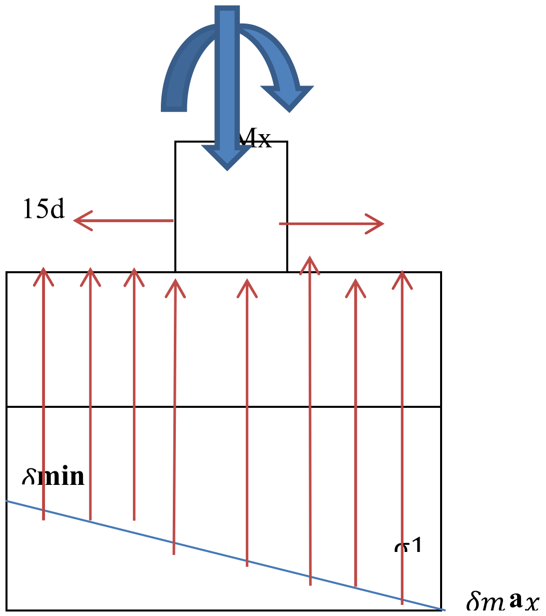


Figure 10. 2 wide beam

first assume the depth of footing pad to be

D=900mm Cover =50mm

Min φ=16

Depth for vrdc

The design shear at section Y-Y

$$Vyy = \sigma avge * B * (\frac{B}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000}) \text{.....................................Eqn 3}$$

$dvrc = D - cover - min\phi/2,$

=900-50-16/2 =842mm

$crdc = \frac{0.18}{\gamma}$ =0.18/1.5=0.12

$$k = \min of \{1 + \sqrt{\frac{200}{dvrc}} = 1 + \sqrt{\frac{200}{842}} = 1.48$$

K=1.48

$$\rho min = min\begin{cases}0.26\dfrac{ftcm}{400} = 0.26\dfrac{2.6}{400} = 0.001\\ 0.02\end{cases}$$

ρmin=0.001

$$Vyy = \sigma avge * B * (\frac{B}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000})$$

Vyy =286.25*2.2(2.2/2-0.4/2-842/1000) = 36.53

The design shear at section X-X

$$Vxx = \sigma avge * L * (\frac{L}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000})$$

Vxx =286.25*2.2(2.2/2-0.4/2-842/1000) = 36.53

$$Vsd = \max \begin{matrix}1. Vyy\\ 2.\ Vxx\end{matrix}$$

Vsd = 36.53

$$VR = Crdc * K * [100 * \rho min * FCK]^{\frac{1}{3}} * B * dvrc$$

Vrdc = 337.50

Vrdc>Vsd--------the depth is adequate for wide beam shear Depth for vrdc min

The design shear at section Y-Y

$$Vyy = \sigma avge * B * (\frac{B}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000})$$

Assume D =700mm

$dvrmin = D - cover - min\phi/2$

=700-50-16/2 = 642

$crdc = 0.18/_{\gamma}$

=0.18/1.5=0.12

$k = \min of\ \{1 + \sqrt{\frac{200}{dvrc}} = 1 + \sqrt{\frac{200}{642}} = 1.558$

K=1.55

$$\rho min = min \begin{cases} 0.26\frac{ftcm}{400} = 0.26\frac{2.6}{400} = 0.0014 \\ 0.02 \end{cases}$$

ρmin=0.0014

$$Vyy = \sigma avge * B * (\frac{B}{2} - \frac{col(H)}{2} - \frac{dvrmin}{1000})$$

Vyy = 286.25*2.2*(2.2/2-0.4/2-642/1000) = 162.48

The design shear at section X-X

$$Vxx = \sigma avge * L * (\frac{L}{2} - \frac{col(H)}{2} - \frac{dvrc}{1000})$$

VX-X = 162.48

$$Vsd = \max \begin{matrix} 1. Vyy \\ 2.\ Vxx \end{matrix}$$

Vsd= 162.48

$$VRDC = Crdc * K * [100 * \rho min * FCK]^{\frac{1}{3}} * B * dvrmin$$

Vrdc = 257.33

Vrdc>Vsd--------the depth is adequate for wide beam shear

The depth of footing for wide beam shear

$$d = \max \quad \begin{matrix} Dvrdc - cover - \frac{\phi}{2} \\ Dvmin \end{matrix}$$

Dmax from 842m $ 642m is……………….dmax = 842m

b. Punching shear

Critical section for shear is at distance 2d from the face of column

check punching shear at critical perimeter (column face)

$Ved, red = Fz - \sigma avg*$(H+B)

Ved, = 4789.90-286.25*(0.4+0.4) = 1302.744KN

θ = 0.383972435

Cot θ = 2.475

Tan θ = 0.404

Cw=1

$$V = 0.6 * [1 - \frac{FCK}{250}]$$

V=0.54*[1-25/250] =0.486

Z=0.9*d=0.9*842=757.8

U_0=perimeter of column

U_0 =2*((3*d/1000+H))+((3*d/1000+0.5))

= 2*((3*842/1000+0.5))+((3*842/1000+0.5))

=12.11

Vrd, max = cw* U_0*1000*z*Θ*fcd)/(cotΘ+tanΘ)/1000

= (1*12.11*1000*757.8**22*11.33/ (2.475+0.404

=1610.74

Ved, red<Vrd, max ---1610.74>1124.3--------punching shear at the face of the column is **safe**

check punching shear at critical perimeter (1.5d distance from the column face)

$$Ved, red = Fz - \sigma avg * \left[3 * \frac{d}{1000} + H\right] * [3 * \frac{d}{1000} + B]$$

Ved, red = 680.26

U_0=perimeter of column

Uo=2*[$Lpunching$+$Wpunching$]

d = 842

L Punching=[3*d/1000+H] ----[3*0.842+0.5] =3.026

W Punching=[3*d/100+H] ---[3*0.832+0.5] =3.026

Uo=2∗[3.026+3.026]]=12.11

$Vrdc = crdc * k * \frac{[100*\rho min*fck]1/3}{3} * Uo*d$

$Vrdc = 0.12 * 1 * \frac{[100*0.0014*25]1/3}{3} * 12.11*842$

$Vrdc$= 1850.04

$Vmin = 0.035 * k^{1.5} * Fck^{0.5} * Uo*d$

$Vmin = 0.035 * 1^{1.5} * 25^{0.5} * 12.11 * 642$

Vmin = 1360.56

Min of Vmin, Vrdc>>Ved, red----1850.04>680.26----- punching shear at the face of the column is safe check punching shear at critical perimeter (2d distance from the column face)

$$Ved,red = Fz - \sigma avg * \left[4 * \frac{d}{1000} + H\right] * [4 * \frac{d}{1000} + B]$$

Ved, = −2872.16

U0=perimeter of column

$Uo = 2 * [Lpunching + Wpunch]$

L Punching=[4*d/100+H] ----[4*0.842+0.5] =3.868

W Punching=[4*d/100+H] ---[4*0.842+0.5] =3.868

$Uo = 2 * [3.868 + 3.868]$ =15.472

$Vrdc = crdc * k * \frac{[100*\rho min*fck]1/3}{3} * Uo*d$

$VrdcVrdc = 0.12 * 1 * [100 * 0.0014 * 25](1/3) * 12.11 * 842$

Vrdc=1660.67

$Vmin = 0.035 * k^{1.5} * Fck^{0.5} * Uo * d$

$Vmin = 0.035 * 1^{1.5} * 25^{0.5} * 12.11 * 842$

Vmin=1784.40

Min of Vmin, Vrdc>>Ved, red----1660.67>-2872.16----- punching shear at the face of the column is safe

The depth of footing required for both wide beam and punching shear

D=d+cover/2=842+50+8=900

c. Reinforcement Calculation parallel to longer direction

$$Msdx = \sigma avg * B * [\left(\frac{L}{2} - \frac{col(H)}{2}\right)^2]/B$$

$Msdx$=286.25*2.2*[($\frac{2.2}{2}$ $\frac{col(0.4)2)}{2}$]/2.45 = 122.09

KZ= Msdx/fckbd^2*d(0.25-(k/1.134)) = 0.986

Msdx=122.09kNm

$AScalculated = msdx*$ 106/$(kz*d*Fyd)$

$AScalculated$ = 122.09* 10^6/ (0.986 * 842 * 400)

AS calculated = 422.79mm^2

$ASmin = 0.26 * fctm/fyk) * 1000 * d$

$ASmin$ = (0.26 *2.565/460) * 1000 * 842

Asmin = 1403.83mm^2

$Asmax = 0.04 * 1000 * d$

$ASmax$ = 0.04 * 1000 * 842

Asmax=33680mm^2

AS, provided = AS calculated <= Asmin <=ASmax

As provided=1403.83 mm^2

$AScalculated = msdx* 10^6/(kz*d*Fyd)$

Msdx = σavg * L *[($\frac{B}{2} - \frac{COL(H)}{2}$)2]/L

$Msdx$ = 286.25 ∗ 2.2∗ [(2.2/2−(0.5)/ 2) 2]/2.45

Msdx=185.5Knm

Sd=msd∗ 10^6/[Fcd∗ 1000 ∗d^2]

=185.50 ∗ 106/[11.33 ∗ 1000 ∗842^2]

Sd = 0.021

KZ=0.986

$AScalculated$ = $msdx$∗ 10^6/(kz∗d∗Fyd)

$AScalculated$ = 185.5.50 ∗ 10^6/(0.986 ∗ 842 ∗ 400)

AS calculated=642.40mm^2

$ASmin$ = 0.26 ∗$fctm/fyk$) ∗ 1000 ∗d

$ASmin$ = (0.26 ∗2.565/ 460) ∗ 1000 ∗ 842

Asmin=1403.82mm^2

$ASmax$ = 0.04 ∗ 1000 ∗d

$ASmax$ = 0.04 ∗ 1000 ∗ 842

ASmax=33680mm^2

AS, provided = AS calculated <= Asmin <=ASmax

AS, provided =1403.82 Spacing

Spacing= Min $\begin{cases} \frac{b*as}{AS} = \frac{1000*3.14*16^2}{1403.82} = 143.15mm \\ 3*d = 3*842 = 2526mm \end{cases}$

400mm So we take the minimum spacing =140mm

Provide φ16 C/C 140mm

Conclusion And Recommendation

Conclusion

To meet objectives of the design in achieving an acceptable probability that structures being designed can perform satisfactorily during their intended life we have seriously applied ES EN Standards. For designing and analysis of this B+G+5 Commercial building Euro Code was the main and fundamental reference for any relevant assumptions carried out in this project papers. On analysis of this project many tedious calculation works were compiled with ETABS software and the result was compared with few hand calculations even if the adequacy of each building elements was understandable by the software program and can generate the output results with acceptable sign convention.

For the design of members the initial assign of the load is basic for the whole design in selection of design critical members, frames, and load combination. The use of ETABs analysis to some extent recognized but it gives results only if the loads are inserted correctly otherwise error could occur in selection of design forces (members).

In accordance with our design and analysis, the structure is economical and safe against lateral and different load application, but for actual implementation as the soil bearing capacity for the foundation was taken from relevant assumption, it needs further soil exploration and modification of foundations accordingly.

Recommendation

As many times are elapsed in doing this project, there are some points that we recognized as problem, need to be corrected for the coming students who are going to do the same thing. Those problems we have seen include:

- Nowadays it is well known that analysis and design of any Engineering Structure is supported with Design Software, hence we recommend that it would have been better if Design Software Courses is given for Civil Engineering students as a subject in order to make the students familiar with different software.
- We recommend that, since design & analysis of a structure with the ES EN is new in our context, it demands patience and hard work and hence need to be thoroughly referred to get much out of it and some courses be given to have a good knowledge of the new code.

Reference[1] Ethiopian ministry of construction, *ES EN 2 1992-2015*. Ethiopia, 2015.

[2] *ES EN 1992-1-1:2015 Ethiopian Standard – Based on European Norm, Design of Concrete Structures, Part 1-1: General rules and rules for buildings*. Ethiopia: Ministry of construction, Federal Democratic Republic of Ethiopia, 2015.

[3] G. Mulu, “13) Mr. Mustefaa. 2021. Foundation design, Presented at the wolkite, Ethiopia,” Wolkite, Ethiopia, 2021.

[4] “ES- EN Actions on Structures 1991-1-4.” 2015.

Appendix A

A-1; Terrain categories and terrain parameters[4]

Table 4.1 — 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 height	0.05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0.3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1.0	10
The terrain categories are illustrated in Annex A.1.			

Table A- 1 Room Functions and their finishing materials

No	Functions	Finishing	Thickness	Unit wt (kN/m^3)
1	Kitchen	Ceramic	2 cm	21
2	Cafeteria	Terrazzo Tile	2 cm	23
3	Shop	Terrazzo Tile	2 cm	23
4	Corridor	Marble	3 cm	27
5	Retail	Terrazzo	2 cm	23

		Tile		
6	Balcony	Marble	3 cm	27
7	Toilet	Ceramic	2 cm	21
8	Shower	Ceramic	2 cm	21
9	Landing	Marble	3 cm	27
10	Internet Café	Terrazzo Tile	2 cm	23
11	Bed Room	PVC Tile	2 cm	16
12	Janitor Room	Terrazzo Tile	2 cm	23

A -2 Depth Determination for ground, first and fifth floor slab

Table A-2 1 Depth Determination for 1st floor slab

Panel	Support Condition	Lx	Ly	N	K	F1	F2&F3	Lx/d	d(mm)	D(mm)
S1	Interior	3800	5100	17.71	1.5	1.25	1	33.20	114.45	139.45
S2	Interior	5100	5600	17.71	1.5	1.25	1	33.20	153.6	178.6
S3	Interior	5100	5600	17.71	1.5	1.25	1	33.20	153.6	178.6
S4	Interior	3800	5100	17.71	1.5	1.25	1	33.20	114.45	139.45
S5	End span	3800	5450	17.71	1.3	1.25	1	28.78	132.04	157.04
S6	Interior	5450	5600	17.71	1.5	1.25	1	33.20	164.16	189.16
S7	Interior	5450	5600	17.71	1.5	1.25	1	33.20	164.16	189.16
S8	End span	3800	5450	17.71	1.3	1.25	1	28.78	132.04	157.04
S9	Interior	4950	5600	17.71	1.5	1.25	1	33.20	149.09	174.09
S10	Interior	4950	5600	17.71	1.5	1.25	1	33.20	149.09	174.09
S11	Interior	3800	4700	17.71	1.5	1.25	1	33.20	114.45	139.45
S12	Interior	4700	5600	17.71	1.5	1.25	1	33.20	141.56	166.56
S13	Interior	4700	5600	17.71	1.5	1.25	1	33.20	141.56	166.56

S14	Interior	3800	4700	17.71	1.5	1.25	1	33.20	114.45	139.45
C1	Cantilever	1400	1350	17.71	0.4	1.25	1	8.855	158.1	183.1
C2	Cantilever	1400	5100	17.71	0.4	1.25	1	8.855	158.1	183.1
C3	Cantilever	1400	5450	17.71	0.4	1.25	1	8.855	158.1	183.1
C4	Cantilever	1400	4700	17.71	0.4	1.25	1	8.855	158.1	183.1
C5	Cantilever	1400	1500	17.71	0.4	1.25	1	8.855	158.1	183.1
C6	Cantilever	1500	3800	17.71	0.4	1.25	1	8.855	169.39	194.39
C7	Cantilever	2700	5600	17.71	0.4	1.25	1	8.855	304.9	329.9
C8	Cantilever	2700	5600	17.71	0.4	1.25	1	8.855	304.9	329.9
C9	Cantilever	1500	3800	17.71	0.4	1.25	1	8.855	169.39	194.39
C10	Cantilever	1400	1500	17.71	0.4	1.25	1	8.855	158.1	183.1
C11	Cantilever	1400	4700	17.71	0.4	1.25	1	8.855	158.1	183.1
C12	Cantilever	1400	5450	17.71	0.4	1.25	1	8.855	158.1	183.1
C13	Cantilever	1400	5100	17.71	0.4	1.25	1	8.855	158.1	183.1
C14	Cantilever	1350	1400	17.71	0.4	1.25	1	8.855	152.45	177.45
C15	Cantilever	1350	3800	17.71	0.4	1.25	1	8.855	152.45	177.45
C16	Cantilever	1350	5600	17.71	0.4	1.25	1	8.855	152.45	177.45
C17	Cantilever	1350	5600	17.71	0.4	1.25	1	8.855	152.45	177.45
C18	Cantilever	1350	3800	17.71	0.4	1.25	1	8.855	152.45	177.45
								Dmax		329.9
								Dprovidede(mm)		330

A -3 Load determination for ground floor slab

Table A-3 1 self-weight of slab with different floor finish on the ground floor slab

Panel	Material	Thickness (m)	Unit Weight, r(KN/m2)	Dead load (KN/m^2)	Total Dead Load (KN/m2)
S1	Cement Screed	0.025	23	0.58	7.89
	RC Slab	0.28	24	6.72	
	Plastering & Painting		23	0.46	

		0.02			
S2	Cement Screed	0.025	23	0.58	7.67
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
S3	Cement Screed	0.025	23	0.58	9.02
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
	Marble	0.03	27	0.81	
S4	Cement Screed	0.025	23	0.58	**10.23**
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.69	
	Plastering wall	0.3	23	0.56	
S5	RC slab	0.28	24	6.72	**8.08**
	Ceramic	0.02	21	0.42	
	Ceramic screed	0.025	23	0.58	

	Plastering	0.02	23	0.46	
S6	Marble	0.03	27	0.81	**8.99**
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.85	
	Plastering	0.02	23	0.46	
S7	Cement Screed	0.025	23	0.58	**10.23**
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.69	
	Plastering wall	0.3	23	0.56	
S8	RC slab	0.28	24	6.72	**7.62**
	Ceramic	0.02	21	0.42	
	Ceramic screed	0.025	23	0.58	
S9	RC slab	0.28	24	6.72	8.08
	Ceramic	0.02	21	0.42	
	Ceramic screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	

S10	Ceramic	0.02	21	0.42	8.65
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
S11	Marble	0.03	27	0.81	8.88
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
S12	Ceramic	0.02	21	0.42	9.96
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
S13	RC slab	0.28	24	6.72	
	Ceramic	0.02	21	0.42	

	Ceramic screed	0.025	23	0.58	8.08
	Plastering	0.02	23	0.46	
S14	Cement Screed	0.025	23	0.58	10.45
	RC slab	0.28	24	6.72	
	Ceramic	0.02	21	0.42	
	Plastering wall	0.3	23	0.56	
C1	PVC	0.02	16	0.46	8.98
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Partition wall	0.05	14	0.69	
	Plastering wall	0.3	23	0.56	
C2	Cement Screed	0.025	23	0.58	9.11
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.69	
	Plastering wall	0.3	23	0.56	
	Cement Screed	0.025	23	0.58	

C3	RC Slab	0.28	24	6.72	8.45
	Plastering & Painting	0.02	23	0.45	
C4	Cement Screed	0.025	23	0.58	8.45
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C5	Marble	0.03	27	0.81	7.88
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
C6	Marble	0.03	27	0.81	8.85
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
	Cement Screed	0.025	23	0.58	7.88

C7	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
	Marble	0.03	27	0.81	
C8	Cement Screed	0.025	23	0.58	8.23
	RC Slab	0.28	24	6.72	
	Plastering &Painting	0.02	23	0.45	
	Marble	0.03	27	0.81	
C9	Cement Screed	0.025	23	0.58	7.64
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C10	Partition wall	0.05	14	0.69	8.87
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	

	Marble	0.03	27	0.81	
C11					8.45
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C12	Cement Screed	0.025	23	0.58	8.98
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
	Marble	0.03	27	0.81	

Panel	Dead Load(KN/m^2)	Live Load(KN/m^2)	Total Load(KN/m^2)
S1	7.89	3.5	15.9
S2	7.67	4.5	17.10
S3	9.02	2	15.177
S4	10.23	3.5	19.06
S5	8.08	3.5	15.98
S6	8.99	3.5	17.38
S7	10.23	4.5	20.56
S8	7.62	4.5	17.03
S9	8.08	2	13.90
S10	8.65	3.5	16.92
S11	8.88	3.5	17.23
S12	9.96	4.5	20.19
S13	10.45	4.5	20.85

S14	8.98	3.5	17.37
C1	9.11	3.5	17.54
C2	8.45	3.5	16.65
C3	8.45	4.5	18.15
C4	7.88	4.5	17.38
C5	8.85	4.5	18.02
C6	7.88	4.5	17.37
C7	8.23	3.5	16.36
C8	7.64	3.5	15.56
C9	8.87	4.5	18.60
C10	8.45	4.5	18.15
C11	8.45	3.5	16.65
C12	8.98	3.5	17.37

A-4 Moment Analysis

Table A-4 1 Moment Calculation for two way slab using coefficient method

Panel	Lx	Pd (KN/m)	$Lx^2(m^2)$	Bsx-	Bsx+	Bsy-	Bsy+	Mxs-	Msx+	Msy-	Msy+
S1	3800	15.9	14.44	0.042	0.0432	0.03	0.021	9.64	9.64	6.89	4.82
S2	5100	17.1	26.01	0.032	0.0332	0.03	0.021	14.23	14.23	13.34	9.34
S3	5100	15.177	26.01	0.061	0.0622	0.03	0.021	24.08	24.08	11.84	8.29
S4	3800	19.06	14.44	0.042	0.0432	0.04	0.021	11.56	11.56	11.01	5.78
S5	3800	15.98	14.44	0.051	0.0522	0.03	0.021	11.77	11.77	6.92	4.85
S6	5450	17.38	29.7025	0.042	0.0432	0.03	0.021	21.68	21.68	15.49	10.84
S7	5450	20.56	29.7025	0.051	0.0522	0.03	0.021	31.14	31.14	18.32	12.82
S8	3800	17.03	14.44	0.061	0.0622	0.03	0.021	15.00	15.00	7.38	5.16
S9	4950	13.9	24.5025	0.061	0.0622	0.03	0.021	20.78	20.78	10.22	7.15
S10	4950	16.92	24.5025	0.061	0.0622	0.03	0.021	25.29	25.29	12.44	8.71

S11	3800	17.23	14.44	0.061	0.0622	0.03	0.021	15.18	15.48	7.46	5.22
S12	4700	20.19	22.09	0.061	0.0622	0.03	0.021	27.21	27.74	13.38	9.37
S13	4700	20.85	22.09	0.05	0.0512	0.03	0.021	23.03	23.58	13.82	9.67
S14	3800	17.37	14.44	0.06	0.0612	0.03	0.021	15.05	15.35	7.52	5.27

Table A-4 2 moment cantilever slabs

Moment Calculation for cantilever slabs					
Panel	Type of supportig Condition	$D.L_{ex}$	Lx	pd’=1.35 D.L +1. 5 L.L	MXS
C1	Two Way Cantilever	9.11	1.4	17.54	18.82
C2	One Way Cantilever	8.45	1.4	16.65	16.55
C3	One Way Cantilever	8.45	1.4	18.15	16.55
C4	One Way Cantilever	7.88	1.4	17.38	15.32
C5	Two Way Cantilever	8.85	1.4	18.02	16.44
C6	One Way Cantilever	7.88	1.5	17.37	15.72
C7	One Way Cantilever	8.23	2.7	16.36	16.1
C8	One Way Cantilever	7.64	2.7	15.56	16
C9	One Way Cantilever	8.87	1.5	18.6	17.08
C10	Two Way Cantilever	8.45	1.4	18.15	15.72
C11	One Way Cantilever	8.45	1.4	16.65	17.03
C12	One Way Cantilever	8.98	1.4	17.37	19.55

A-5 Moment adjustment for ground floor slab

Table A-5 1 Support moment adjustment along x-x direction

Support	Lx	Ly	Df	Mxs1	Mxs2	ΔM	Madj.
S1&S2	3.8	5.1	0.43	9.64	6.89	2.75	8.265
S1&S5	5.1	5.6	0.57	14.23	13.34	0.89	13.785
S1&C18	5.1	5.6	0.74	24.08	11.84	12.24	17.96
S1&C3	3.8	5.1	0.46	11.56	11.01	0.55	11.285
S3&S4	3.8	5.45	0.54	11.77	6.92	4.85	9.345
S3&S8	5.45	5.6	0.66	21.68	15.49	6.19	18.585

S3&S7	5.45	5.6	0.54	31.14	18.32	12.82	24.73
S3&C16	3.8	5.45	0.36	15	7.38	7.62	11.19
S8&C12	4.95	5.6	0.46	20.78	10.22	10.56	15.5
S7&S10	4.95	5.6	0.56	25.29	12.44	12.85	18.865
S11&S12	3.8	5.1	0.45	15.18	7.46	7.72	11.32

Table A-5 2 Support moment adjustment along y-y direction

support	Lx	Ly	Df	Mys1	Mys2	ΔM	Madj.
S1&S5	3.8	5.1	0.51	11.77	6.92	4.85	9.345
S2&S6	5.1	5.6	0.54	21.68	15.49	6.19	18.585
S3&S7	5.1	5.6	0.7	31.14	18.32	12.82	24.73
S4&S8	3.8	5.1	0.7	15	7.38	7.62	11.19
S5&S11	3.8	5.45	0.7	20.78	10.22	10.56	15.5
S9&S12	5.45	5.6	0.7	25.29	12.44	12.85	18.865
S10&S13	5.45	5.6	0.66	15.48	7.46	8.02	11.47
S8&S14	3.8	5.45	0.55	27.74	13.38	14.36	20.56

Table A-5 3 Span moment adjustment

Panel	Unadjusted Mom		Adjusted Support moment				Max. d/f		Unadjusted Span moment		Factor For adjusted Span mom				Adjusted span moment	
	Mxs	Mys	Mdx1	Mdy1	Mdx2	Mdy2	Δmxs	Δmys	mxf	myf	Cx1	Cx2	Cy1	Cy2	mxf	Myf
S1	25.54	19.95	16.47	5.45	22.2	16.47	11.76	0.02	6.98	4.1	0.238	0.484	0.055	0.248	9.750	7.020
S2	28.58	19.44	24.15	22.2	22.2	24.15	11.76	0.22	21.5	16.65	0.3434	0.335	0.186	0.367	25.61	21.01

S3	23. 47	22. 73	17.5 1	5.4 5	10. 36	17.5 1	10. 94	0.0 2	6.9 8	4.1	0.2 38	0.4 84	0.0 55	0.2 48	9. 5 93	6.8 14
S4	22. 04	21. 95	16.4 4	21. 3	10. 36	16.4 4	10. 94	0.0 1	20. 65	15. 98	0.3 43 4	0.3 35	0.1 86	0.3 67	23 . 86	20. 00
S5	19. 58	19. 95	14.9 2	37.	0	14.9 2	0	11.	30.	28.	0.2	0.4	0.0	0.2	36 .	29.
S6	23. 47	17. 45	24.1 2	17 . 2	11.2 3	24.1 2		03	85	5	38	84	55	48	19	11
S7	22. 04	22. 45	21.2 3	4.7 7	10. 18	21.2 3	12. 02	0.0 2	7.1 6	3.5 8	0.2 38	0.4 84	0.0 55	0.2 48	10 . 03	6.5 62
S8	19. 58	20. 23	22.3 4	22. 23	10. 18	22.3 4	12. 02	0.6	21. 54	16. 67	0.3 43 4	0.3 35	0.1 86	0.3 67	25 . 87	21. 19

Table A-5 4 Reinforcement bar for ground floor slab

Panel		M(KNM)	Rebar
S1	Mxs-	9.64	Ø 10C/C450mm
	Mxs+	9.64	Ø10C/C450mm
	Mys-	6.89	Ø1OC/C52Omm
	Mys+	4.82	Ø10C/C56Omm
S2	Mxs-	14.23	Ø10C/C310
	Mxs+	14.23	Ø1OC/C31Omm
	Mys-	13.84	Ø1OC/C33Omm
	Mys+	8.29	Ø1OC/C480mm
S3	Mxs-	24.08	Ø1OC/C220mm
	Mxs+	24.08	Ø1OC/C220mm

	Mys-	11.84	Ø1OC/C400mm
	Mys+	8.29	Ø1OC/C480mm
S4	Mxs-	11.56	Ø1OC/C430mm
	Mxs+	11.56	Ø1OC/C430mm
	Mys-	11.01	Ø1OC/C440mm
	Mys+	5.78	Ø1OC/C530mm
S5	Mxs-	11.77	Ø1OC/C420mm
	Mxs+	11.77	Ø1OC/C420mm
	Mys-	6.92	Ø1OC/C52Omm
	Mys+	4.85	Ø10C/C56Omm
S6	Mxs-	21.68	Ø1OC/C24Omm
	Mxs+	21.68	Ø1OC/C24Omm
	Mys-	15.49	Ø10C/C30Omm
	Mys+	10.89	Ø10C/C440mm
S7	Mxs-	31.14	Ø10C/C16Omm
	Mxs+	26.14	Ø10C/C20Omm
	Mys-	18.2	Ø10C/C23Omm
	Mys+	12.82	Ø10C/C430mm
S8	Mxs-	15	Ø10C/C310mm
	Mxs+	21.3	Ø10C/C250mm
	Mys-	7.38	Ø10C/C49Omm
	Mys+	5.16	Ø10C/C54Omm
S9	Mxs-	20.78	Ø10C/C25Omm
	Mxs+	18.76	Ø10C/C26Omm
	Mys-	10.22	Ø10C/C45Omm
	Mys+	7.15	Ø10C/C49Omm
S10	Mxs-	29.29	Ø10C/C190mm
	Mxs+	23.92	Ø10C/C230mm
	Mys-	12.44	Ø10C/C41Omm
	Mys+	8.71	Ø10C/C46Omm
S11	Mxs-	15.18	Ø10C/C30Omm
	Mxs+	16.15	Ø10C/C290mm
	Mys-	7.46	Ø10C/C49Omm
	Mys+	5.22	Ø10C/C54Omm
S12	Mxs-	27.21	Ø10C/C190mm
	Mxs+	27.74	Ø10C/C190mm

	Mys- Mys+	13.38 9.37	Ø10C/C400mm Ø10C/C450mm

Table A-5 5 Reinforcement calculation for cantilever slab for ground floor

Panel	M(KNM)		Rebar
C1	Mxs	18.82	Ø10C/C210mm
C2	Mxs	16.55	Ø10 C/C270mm
C3	Mxs	16.55	Ø10 C/C2700mm
C4	Mxs	15.32	Ø10 C/C290mm
C5	Mxs	16.44	Ø10 C/C270mm
C6	Mxs	15.72	Ø10 C/C280mm
C7	Mxs	16.1	Ø10 C/C270mm
C8	Mxs	16	Ø10 C/C270mm
C9	Mxs	17.08	Ø10 C/C280mm
C10	Mxs	15.72	Ø10 C/C280mm
C11	Mxs	17.03	Ø10 C/C280mm
C12	Mxs	19.55	Ø10 C/C200mm

A-6 Load determination for 1st floor slab

Table A-6 1 self-weight of slab with different floor finish on the 1st floor

Panel	Material	Thickness (m)	Unit Weight, r(KN/m2)	Dead load (KN/m^2)	Total Dead Load (KN/m2)
	Ceramic	0.02	21	0.42	

S1	Cement Screed	0.025	23	0.58	8.18
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
S2	Marble	0.03	27	0.81	8.57
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
S3	PVC	0.02	16	0.46	8.135
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
S4	Cement Screed	0.025	23	0.58	**10.23**
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.69	

	Plastering wall	0.3	23	0.56	
S5	RC slab	0.28	24	6.72	**8.08**
	Ceramic	0.02	21	0.42	
	Ceramic screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
S6	Marble	0.03	27	0.81	**8.99**
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.85	
	Plastering	0.02	23	0.46	
S7	Cement Screed	0.025	23	0.58	**10.23**
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.69	
	Plastering wall	0.3	23	0.56	
S8	RC slab	0.28	24	6.72	**7.72**
	Ceramic	0.02	21	0.42	
	Ceramic screed	0.025	23	0.58	

S9	RC slab	0.28	24	6.72	8.08
	Ceramic	0.02	21	0.42	
	Ceramic screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
S10	Ceramic	0.02	21	0.42	8.18
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
S11	Marble	0.03	27	0.81	7.88
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
S12	Ceramic	0.02	21	0.42	9.96
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	

	Plastering & Painting	0.02	23	0.46	
S14	Cement Screed	0.025	23	0.58	10.15
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.69	
	Plastering wall	0.3	23	0.56	
C1	PVC	0.02	16	0.46	8.89
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
	Plastering wall	0.3	23	0.56	
C2	Cement Screed	0.025	23	0.58	10.11
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.69	
	Plastering wall	0.3	23	0.56	
	PVC	0.02	16	0.46	

C3	Cement Screed	0.025	23	0.58	8.45
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C4	PVC	0.02	16	0.46	8.88
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C5	Marble	0.03	27	0.81	7.88
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
C6	Marble	0.03	27	0.81	7.88
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering &	0.02	23	0.46	

	Painting				
C7	Cement Screed	0.025	23	0.58	7.88
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
	Marble	0.03	27	0.81	
C8	Cement Screed	0.025	23	0.58	8.23
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
	Marble	0.03	27	0.81	
C9	PVC	0.02	16	0.46	8.5
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C10	PVC	0.02	16	0.46	8.5
	Cement Screed	0.025	23	0.58	

	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C11	PVC	0.02	16	0.46	8.5
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C12	Cement Screed	0.025	23	0.58	7.72
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C13	PVC	0.02	16	0.46	8.08
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	

C14	PVC	0.02	16	0.46	8.89
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C15	PVC	0.02	16	0.46	8.89
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	
C16	Cement Screed	0.025	23	0.58	9.12
	RC slab	0.28	24	6.72	
	Partition wall	0.05	14	0.69	
	Plastering wall	0.3	23	0.56	
C17	PVC	0.02	16	0.46	8.89
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	

	Plastering & Painting	0.02	23	0.45	
C18	PVC	0.02	16	0.46	8.89
	Cement Screed	0.025	23	0.58	
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.45	

Panel	Dead Load(KN/m^2)	Live Load(KN/m^2)	Total Load(KN/m^2)
S1	8.18	4.5	17.79
S2	8.57	4.5	18.31
S3	8.13	2	13.97
S4	10.23	3.5	19.06
S5	8.08	3.5	16.78
S6	8.98	3.5	17.07
S7	10.23	4.5	19.06
S8	8.08	4.5	17.78
S9	7.72	3.5	15.79
S10	8.08	3.5	16.78
S11	8.18	3.5	15.35
S12	7.88	4.5	16.29
S13	9.96	4.5	20.20
S14	10.15	3.5	18.95
C1	8.89	3.5	17.25
C2	10.11	3.5	18.89
C3	8.45	4.5	18.15
C4	8.88	4.5	18.74
C5	7.88	4.5	17.38
C6	7.88	4.5	17.38
C7	7.88	4.5	17.38
C8	8.23	3.5	16.36
C9	8.51	3.5	16.73
C10	8.5	4.5	16.89

C11	8.5	4.5	16.89
C12	7.72	3.5	15.64
C13	8.08	3.5	16.78
C14	8.89	3.5	17.25
C15	8.89	4.5	17.67
C16	9.12	3.5	17.56
C17	8.89	4.5	18.75
C18	8.89	3.5	17.25

A-7 Moment Analysis

Table A-7 1 Moment Calculation for two way slab using coefficient method

Panel	Ly/Lx	Pd (KN/m)	$Lx^2(m^2)$	Bsx-	Bsx+	Bsy-	Bsy+	Mxs-	Msx+	Msy-	Msy+
S1	1.34	17.79	14.44	0.04	0.03	0.03	0.02	22.39	19.44	19.44	17.08
S2	1.1	18.31	26.01	0.03	0.02	0.03	0.02	22.33	19.55	22.73	19.55
S3	1.1	13.97	26.01	0.06	0.04	0.03	0.02	27.40	22.98	19.44	17.08
S4	1.34	19.06	14.44	0.04	0.03	0.04	0.03	25.52	21.94	14.72	11.14
S5	1.43	16.78	14.44	0.05	0.05	0.03	0.02	24.74	25.63	19.44	17.08
S6	1.03	17.07	29.70	0.04	0.03	0.03	0.02	26.71	22.73	22.73	19.55
S7	1.03	19.06	29.70	0.05	0.05	0.03	0.02	25.54	16.47	19.95	17.46
S8	1.43	17.78	14.44	0.06	0.05	0.03	0.02	28.58	24.15	19.44	17.08
S9	1.13	15.79	24.50	0.06	0.04	0.03	0.02	23.47	17.51	22.73	19.55

S10	1.13	16.78	24.50	0.06	0.04	0.03	0.02	22.04	16.44	21.95	18.96
S11	1.24	15.35	14.44	0.06	0.05	0.03	0.02	19.58	14.92	19.95	17.46
S12	1.19	16.29	22.09	0.06	0.04	0.03	0.02	23.47	24.12	17.45	15.35
S13	1.19	20.196	22.09	0.05	0.04	0.03	0.02	22.04	21.23	22.45	19.05
S14	1.24	18.95	14.44	0.06	0.05	0.03	0.02	19.58	22.34	20.23	18.4

Table A-7 2 moment cantilever slabs

Moment Calculation for cantilever slabs					
Panel	Type of supportig Condition	$D.L_{ex}$	Lx	pd'=1.35 D.L +1. 5 L.L	MXS
C1	Two Way Cantilever	8.89	1.4	17.25	16.905
C2	One Way Cantilever	10.11	1.4	18.89	18.51
C3	One Way Cantilever	8.45	1.4	18.15	17.78
C4	One Way Cantilever	8.88	1.4	18.74	18.86
C5	Two Way Cantilever	7.88	1.4	17.38	17.03
C6	One Way Cantilever	7.88	1.5	17.38	19.55
C7	One Way Cantilever	7.88	2.7	17.38	63.35

C8	One Way Cantilever	8.23	2.7	16.36	59.63
C9	One Way Cantilever	8.51	1.5	16.73	18.82
C10	Two Way Cantilever	8.5	1.4	16.89	16.55
C11	One Way Cantilever	8.5	1.4	16.89	16.55
C12	One Way Cantilever	7.72	1.4	15.64	15.32
C13	One Way Cantilever	8.08	1.4	16.78	16.44
C14	Two Way Cantilever	8.89	1.35	17.25	15.72
C15	One Way Cantilever	8.89	1.35	17.67	16.1
C16	One Way Cantilever	9.12	1.35	17.56	16.00
C17	One Way Cantilever	8.89	1.35	18.75	17.08
C18	One Way Cantilever	8.89	1.35	17.25	15.72

A-8 Moment adjustment for 1st floor

Table A-8 1 Support moment adjustment along x-x direction

Support	Lx	Ly	Df	Mxs1	Mxs2	ΔM	Madj.
S1&S2	3.8	5.1	0.53	22.39	19.44	2.95	21.40

S1&S5	5.1	5.6	0.67	22.33	19.55	2.78	20.94
S1&C18	5.1	5.6	0.54	27.40	22.98	4.42	25.19
S1&C3	3.8	5.1	0.66	25.52	21.94	3.58	23.73
S3&S4	3.8	5.45	0.54	24.74	25.63	1.58	24.65
S3&S8	5.45	5.6	0.66	26.71	22.73	3.98	24.72
S3&S7	5.45	5.6	0.54	25.54	16.47	9.07	22.87
S3&C16	3.8	5.45	0.66	28.58	24.15	3.75	26.34
S8&C12	4.95	5.6	0.46	23.47	17.51	5.96	20.56
S7&S10	4.95	5.6	0.66	22.04	16.44	5.76	18.24
S9&S11	3.8	4.7	0.54	19.58	14.92	4.46	16.23
S13&S14	4.7	5.6	0.56	23.47	24.12	0	23.79

Table A-8 2 Support moment adjustment along y-y direction

support	Lx	Ly	Df	Mys1	Mys2	ΔM	Madj.
S1&S5	3.8	5.1	0.51	23.86	25.34	1.48	24.6

S2&S6	5.1	5.6	0.54	25.34	29.23	3.89	27.3
S3&S7	5.1	5.6	0.7	4.8	20.97	16.17	9.65
S4&S8	3.8	5.1	0.7	5.45	22.2	16.8	10.44
S5&S11	3.8	5.45	0.7	5.45	21.3	18.85	10.36
S9&S12	5.45	5.6	0.7	4.77	22.23	17.46	10.18
S10&S13	5.45	5.6	0.66	5.83	4.77	1.08	5.3
S8&S14	3.8	5.45	0.55	24	25.1	1.1	24.55

Table A-8 3 Span moment adjustment

Panel	Unadjusted mom		Adjusted Support moment				Max. d/f		Unadjusted Span moment		Factor For adjusted Span mom				Adjusted span moment	
	Mxs	Mys	Mdx1	Mdy1	Mdx2	Mdy2	Δ mxs	Δmys	mxf	myf	Cx1	Cx2	Cy1	Cy2	mxf	Myf
S1	22.39	19.44	19.44	23.86	25.34	19.44	0.74	9.43	20.2	17.74	0.356	0.314	0.22	0.374	23.42	20.09
S2	22.33	22.73	19.55	25.34	27.3	19.55	2.7	4.33	22.1	18.84	0.365	0.304	0.238	0.376	24.40	20.89

S3	27.40	19.44	22.98	29.23	29.23	22.98	1.93	4.20	26.23	22.5	0.358	0.311	0.226	0.375	28.23	24.17
S4	25.52	14.72	21.94	40.37	0	21.94	0	9.43	33.5	30.9	0.368	0.295	0.25	0.377	36.28	33.26
S5	24.74	19.44	25.63	4.8	20.97	25.63	11.32	6.78	7.17	3.58	0.238	0.484	0.055	0.248	13.16	7.65
S6	26.71	22.73	22.73	20.97	9.65	22.73	11.32	3.41	20.3	15.7	0.3434	0.335	0.186	0.367	25.33	20.49
S7	25.54	19.95	16.47	5.45	22.2	16.47	11.76	0.02	6.98	4.1	0.238	0.484	0.055	0.248	9.750	7.020
S8	28.58	19.44	24.15	22.2	22.2	24.15	11.76	0.22	21.5	16.65	0.3434	0.335	0.186	0.367	25.61	21.01
S9	23.47	22.73	17.51	5.45	10.36	17.51	10.94	0.02	6.98	4.1	0.238	0.484	0.055	0.248	9.593	6.814
S10	22.04	21.95	16.44	21.3	10.36	16.44	10.94	0.01	20.65	15.98	0.3434	0.335	0.186	0.367	23.86	20.00
S11	19.58	19.95	14.92	37.	0	14.92	0	11.	30.	28.	0.2	0.4	0.0	0.2	36 .	29.
S12	23.47	17.45	24.12	17 . 2	11.23	24.12		03	85	5	38	84	55	48	19	11

S13	22.04	22.45	21.23	4.77	10.18	21.23	12.02	0.02	7.16	3.58	0.238	0.484	0.055	0.248	10.03	6.562
S14	19.58	20.23	22.34	22.23	10.18	22.34	12.02	0.6	21.54	16.67	0.3434	0.335	0.186	0.367	25.87	21.19

Table A-8 4 Reinforcement bar for 1st floor slab

Panel		M(KNM)	Rebar
S1	Mxs-	22.39	Ø10 C/C200mm
	Mxs+	19.44	Ø10 C/C220mm
	Mys-	19.44	Ø10 C/C220mm
	Mys+	17.08	Ø10C/C290mm
S2	Mxs-	22.33	Ø10 C/C200mm
	Mxs+	19.55	Ø10 C/C220mm
	Mys-	22.73	Ø10 C/C200mm
	Mys+	19.55	Ø10 C/C220mm
S3	Mxs-	27.4	Ø10 C/C180mm
	Mxs+	22.98	Ø10 C/C200mm
	Mys-	19.44	Ø10 C/C220mm
	Mys+	17.08	Ø10C/C290mm
S4	Mxs-	25.52	Ø10 C/C190mm
	Mxs+	21.94	Ø10 C/C200mm
	Mys-	14.17	Ø10 C/C330mm
	Mys+	11.14	Ø10 C/C360mm
S5	Mxs-	24.74	Ø10 C/C200mm
	Mxs+	25.63	Ø10 C/C190mm
	Mys-	19.44	Ø10 C/C220mm
	Mys+	17.08	Ø10C/C290mm
S6	Mxs-	26.71	Ø10 C/C180mm
	Mxs+	22.73	Ø10 C/C200mm

	Mys- Mys+	22.73 19.55	Ø10 C/C200mm Ø10 C/C220mm
S7	Mxs- Mxs+ Mys- Mys+	25.54 16.47 19.95 17.46	Ø10 C/C190mm Ø10C/C330mm Ø10 C/C220mm Ø10C/C290mm
S8	Mxs- Mxs+ Mys- Mys+	28.58 24.15 19.44 17.08	Ø10 C/C170mm Ø10 C/C200mm Ø10 C/C220mm Ø10C/C290mm
S9	Mxs- Mxs+ Mys- Mys+	23.47 17.51 22.73 19.55	Ø10 C/C200mm Ø10C/C290mm Ø10 C/C200mm Ø10 C/C220mm
S10	Mxs- Mxs+ Mys- Mys+	22.04 16.44 21.95 18.96	Ø10 C/C210mm Ø10C/C330mm Ø10 C/C210mm Ø10C/C230mm
S11	Mxs- Mxs+ Mys- Mys+	19.58 14.92 19.95 17.46	Ø10 C/C220mm Ø10 C/C330mm Ø10 C/C220mm Ø10C/C290mm
S12	Mxs- Mxs+ Mys- Mys+	23.47 24.12 17.45 15.35	Ø10 C/C200mm Ø10 C/C200mm Ø10C/C290mm Ø10 C/320mm
S13	Mxs- Mxs+ Mys- Mys+	22.04 21.23 22.45 19.05	Ø10 C/C200mm Ø10 C/C210mm Ø10 C/C200mm Ø10 C/C220mm
S14	Mxs- Mxs+ Mys- Mys+	19.58 22.34 20.23 18.4	Ø10 C/C220mm Ø10 C/C200mm Ø10 C/C230mm Ø10C/C270mm

Table A-8 5 Reinforcement calculation for cantilever slab 1st floor

Panel	M(KNM)		Rebar
C1	Mxs	16.905	Ø10C/C320mm
C2	Mxs	18.51	Ø10 C/C270mm
C3	Mxs	17.78	Ø10C/C290mm
C4	Mxs	18.86	Ø10 C/C270mm
C5	Mxs	17.03	Ø10C/C290mm
C6	Mxs	19.55	Ø10 C/C220mm
C7	Mxs	63.35	Ø10C/C140mm
C8	Mxs	59.63	Ø10C/C150mm
C9	Mxs	18.82	Ø10C/C270mm
C10	Mxs	16.55	Ø10C/C320mm
C11	Mxs	16.55	Ø10C/C320mm
C12	Mxs	15.32	Ø10 C/C330mm
C13	Mxs	16.44	Ø10C/C320mm
C14	Mxs	15.72	Ø10 C/C330mm
C15	Mxs	16.1	Ø10C/C320mm
C16	Mxs	16	Ø10C/C320mm
C17	Mxs	17.08	Ø10C/C290mm
C18	Mxs	15.72	Ø10 C/C330mm

A-9 Load determination for 5st floor slab

Table A-9 1 self-weight of slab with different floor finish on the 5st floor

Panel	Material	Thickness (m)	Unit Weight, r(KN/m2)	Dead load (KN/m^2)	Total Dead Load (KN/m2)
	Ceramic	0.02	21	0.42	

S1	Cement Screed	0.025	23	0.58	9.89
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	
	Marble	0.03	27	0.81	
S2	Cement Screed	0.025	23	0.58	7.57
	RC Slab	0.28	24	6.72	
	Plastering & Painting	0.02	23	0.46	

Panel	Dead Load(KN/m^2)	Live Load(KN/m^2)	Total Load(KN/m^2)
S1	9.98	3.5	18.72
S2	7.57	4.5	16.96

Moment Analysis

Table A-10 Moment Calculation for two way slab using coefficient method

Panel	Lx	Pd (KN/m)	Lx^2(m^2)	Bsx-	Bsx+	Bsy-	Bsy+	Mxs-	Msx+	Msy-	Msy+
S1	4.95	18.72	24.5	0.036	0.042	0.03	0.034	16.51	19.26	13.75	15.59
S2	4.95	16.96	24.5	0.041	0.032	0.03	0.021	21.31	16.63	15.59	10.91

A -10 Moment adjustment for 5th floor slab

Table A-10 1 Support moment adjustment along x-x direction

Support	Lx	Ly	Df	Mxs1	Mxs2	ΔM	Madj.
S1&S2	4.95	5.6	0.54	19.26	16.51	2.75	17.88

Table A-11 2 Reinforcement bar for ground floor slab

Panel		M(KNM)	Rebar
S1	Mxs-	16.51	Ø 10C/C320mm
	Mxs+	19.26	Ø10C/C220mm
	Mys-	13.75	Ø1OC/C43Omm
	Mys+	15.59	Ø10C/C33Omm
S2	Mxs-	21.38	Ø10C/C200mm
	Mxs+	16.63	Ø1OC/C32Omm
	Mys-	15.59	Ø1OC/C33Omm
	Mys+	10.91	Ø1OC/C480mm

A -11 Load transfer to supporting beam

Table A-11 3 Load transfer from ground floor slab to beam

Panel	Type	Lx	Ly	Ly/Lx	n(pd)	βvx	βvy	Vsx	Vsy
S1	Interior	3800	5100	1.34	15.9	0.34	0.42	20.54	25.38
S2	Interior	5100	5600	1.10	17.1	0.42	0.33	36.63	28.78
S3	Interior	5100	5600	1.10	15.177	0.32	0.44	24.77	34.06
S4	Interior	3800	5100	1.34	19.06	0.36	0.42	26.07	30.42
S5	End span	3800	5450	1.43	15.98	0.34	0.36	20.65	21.86
S6	Interior	5450	5600	1.03	17.38	0.45	0.36	42.62	34.10
S7	Interior	5450	5600	1.03	20.56	0.39	0.45	43.70	50.42
S8	End span	3800	5450	1.43	17.03	0.36	0.32	23.30	20.71
S9	Interior	4950	5600	1.13	13.9	0.45	0.34	30.96	23.39
S10	Interior	4950	5600	1.13	16.92	0.47	0.34	39.36	28.48
S11	Interior	3800	4700	1.24	17.23	0.33	0.34	21.61	22.26
S12	Interior	4700	5600	1.19	20.19	0.48	0.35	45.55	33.21

S13	Interior	4700	5600	1.19	20.85	0.34	0.35	33.32	34.30
S14	Interior	3800	4700	1.24	17.37	0.33	0.343	21.78	22.64

Panel	Type	Lx	Ly	Ly/Lx	n(pd)	WL(KN)
C1	Cantilever	1350	1400	1.04	17.54	23.679
C2	Cantilever	1400	5100	3.64	16.65	23.31
C3	Cantilever	1400	5450	3.89	18.15	25.41
C4	Cantilever	1400	4700	3.36	17.38	24.332
C5	Cantilever	1400	1500	1.07	18.02	25.228
C6	Cantilever	1500	3800	2.53	17.37	26.055
C7	Cantilever	2700	5600	2.07	16.36	44.172
C8	Cantilever	2700	5600	2.07	15.56	42.012
C9	Cantilever	1500	3800	2.53	18.6	27.9
C10	Cantilever	1400	1500	1.07	18.15	25.41
C11	Cantilever	1400	4700	3.36	16.65	23.31
C12	Cantilever	1400	5450	3.89	17.37	24.318

Table A-12 1 Load transfer from 1st floor slab to beam

Panel	Type	Lx	Ly	Ly/Lx	n(pd)	βvx	Bvy	Vsx	Vsy
S1	Interior	3800	5100	1.34	17.79	0.35	0.32	23.66	21.63
S2	Interior	5100	5600	1.10	18.31	0.47	0.33	43.89	30.82
S3	Interior	5100	5600	1.10	13.97	0.32	0.34	22.80	24.22
S4	Interior	3800	5100	1.34	19.06	0.38	0.32	27.52	23.18
S5	End span	3800	5450	1.43	16.78	0.34	0.36	21.68	22.96
S6	Interior	5450	5600	1.03	17.07	0.35	0.36	32.56	33.49
S7	Interior	5450	5600	1.03	19.06	0.39	0.35	40.51	36.36
S8	End span	3800	5450	1.43	17.78	0.46	0.32	31.08	21.62
S9	Interior	4950	5600	1.13	15.79	0.42	0.34	32.83	26.57
S10	Interior	4950	5600	1.13	16.78	0.37	0.35	30.73	29.07
S11	Interior	3800	4700	1.24	15.35	0.43	0.34	25.08	19.83
S12	Interior	4700	5600	1.19	16.29	0.38	0.35	29.09	26.80
S13	Interior	4700	5600	1.19	20.196	0.4	0.35	37.97	33.22

S14	Interior	3800	4700	1.24	18.95	0.43	0.33	30.96	23.76

Panel	Type	Lx	Ly	Ly/Lx	n(pd)	WL(KN)
C1	Cantilever	1350	1400	1.04	17.25	23.2875
C2	Cantilever	1400	5100	3.64	18.89	26.446
C3	Cantilever	1400	5450	3.89	18.15	25.41
C4	Cantilever	1400	4700	3.36	18.74	26.236
C5	Cantilever	1400	1500	1.07	17.38	24.332
C6	Cantilever	1500	3800	2.53	17.38	26.07
C7	Cantilever	2700	5600	2.07	17.38	46.926
C8	Cantilever	2700	5600	2.07	16.36	44.172
C9	Cantilever	1500	3800	2.53	16.73	25.095
C10	Cantilever	1400	1500	1.07	16.89	23.646
C11	Cantilever	1400	4700	3.36	16.89	23.646
C12	Cantilever	1400	5450	3.89	15.64	21.896
C13	Cantilever	1400	5100	3.64	16.78	23.492
C14	Cantilever	1350	1400	1.04	17.25	23.2875
C15	Cantilever	1350	3800	2.81	17.67	23.8545
C16	Cantilever	1350	5600	4.15	17.56	23.706
C17	Cantilever	1350	5600	4.15	18.75	25.3125
C18	Cantilever	1350	3800	2.81	17.25	23.2875

Table A-12 2 Load transfer from 5th floor slab to beam

Panel	Type	Lx	Ly	Ly/Lx	n(pd)	βvx	βvy	Vsx	Vsy
S1	End span	4.95	5.6	1.13	18.72	0.45	0.33	41.70	30.58
S2	End span	4.95	5.6	1.13	16.96	0.37	0.33	31.06	27.70

Appendix B - Beam

B -1 Reinforcement for beam

Beam design on axis 3 for all story

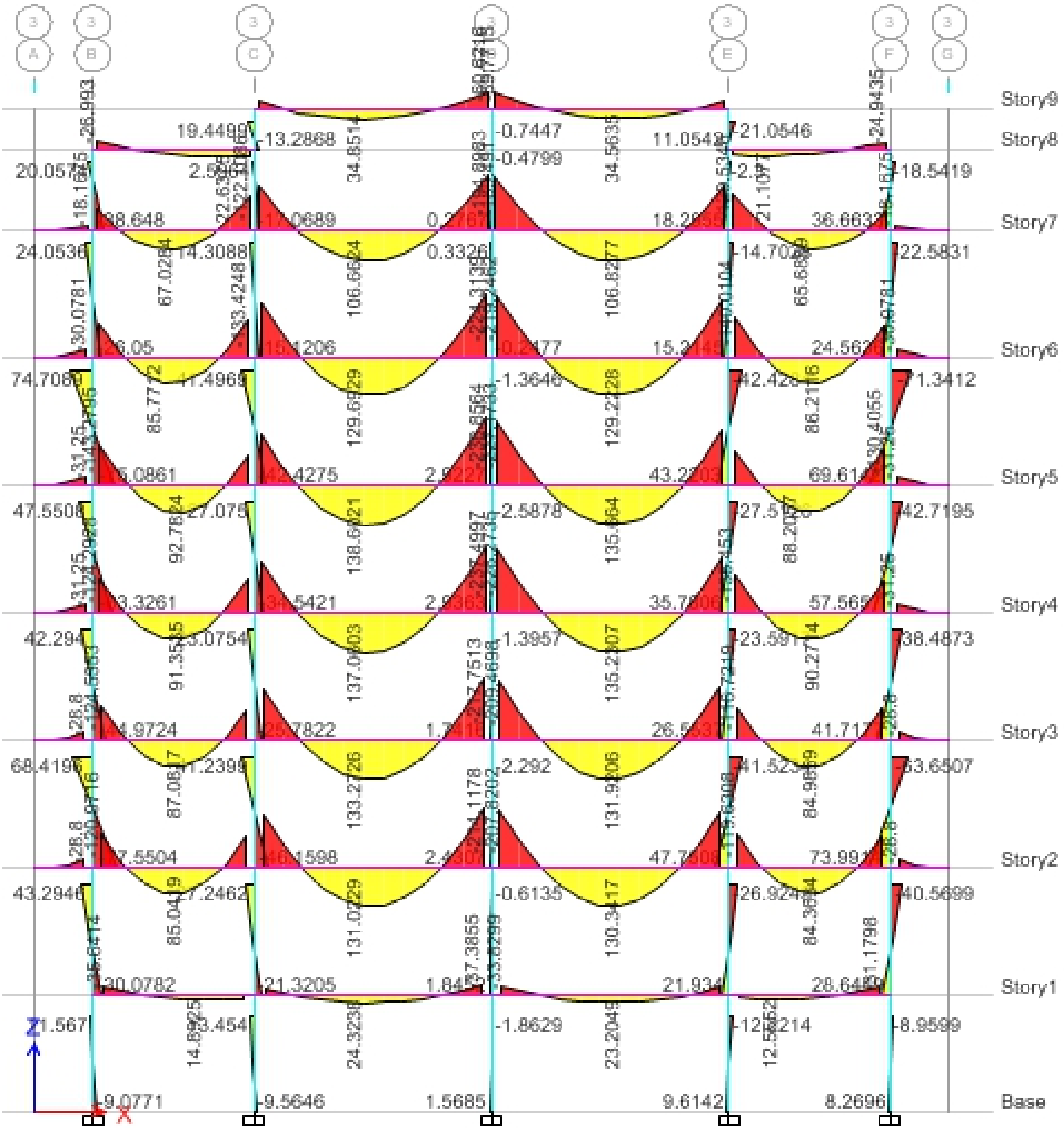


Figure B -1.1 bending moment diagram of beam on axis 3

Axis-3	Top Tied Beam (Roof)												
Type	Loc	Moment (KNm	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark

		)											
support	B	26.99	300	355	0.0357	343.44117	single	138.45	4260	196.467	196.4674	0.9771582	2∅16
Span	B-C	19.45	300	355	0.0257	346.74975	single	138.45	4260	140.231	140.2308	0.6974575	2∅16
support	C	13.28	300	355	0.0176	349.40981	single	138.45	4260	95.0174	138.45	0.4725821	2∅16
Span	C-D	34.85	300	355	0.0461	339.92043	single	138.45	4260	256.31	256.31	1.2747935	2∅16
support	D	50.67	300	355	0.0670	332.59203	single	138.45	4260	380.872	380.872	1.8943202	2∅16
Span	D-E	34.56	300	355	0.0457	340.05168	single	138.45	4260	254.079	254.079	1.2636975	2∅16
Support	E	11.05	300	355	0.0146	350.36116	single	138.45	4260	78.8472	138.45	0.3921576	2∅16
Span	E-F	21.05	300	355	0.0278	346.05309	single	138.45	4260	152.072	152.072	0.7563515	2∅16
Support	F	24.94	300	355	0.0330	344.34721	single	138.45	4260	181.067	181.0672	0.9005632	2∅16

Table B-1 1 Reinforcement for beam on axis -3

Axis-3	Fifth floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	B	18.16	300	355	0.0240	347.30935	single	138.45	4260	130.719	138.45	0.6501502	2∅16
span	B-C	67.02	300	355	0.0886	324.63506	single	138.45	4260	516.118	516.118	2.566985	3∅16
support	C	22.63	300	355	0.0299	345.36231	single	138.45	4260	163.813	163.8135	0.8147492	2∅16
span	C-D	106.66	300	355	0.1411	303.27099	single	138.45	4260	879.247	879.2466	4.373056	5∅16
support	D	194.8	300	355	0.2576	230.92674	double	138.45	4260	2108.89	2108.894	10.488879	11∅16
Span	D-E	106.82	300	355	0.1413	303.1774	single	138.45	4260	880.837	880.8374	4.380968	5∅16
Support	E	9.53	300	355	0.0126	351.00662	single	138.45	4260	67.8762	138.45	0.3375918	2∅16
Span	E-F	65.66	300	355	0.0868	325.31325	single	138.45	4260	504.591	504.5906	2.5096517	3∅16
Support	F	167.5	300	355	0.2215	260.38421	double	138.45	4260	1608.2	1608.2	7.9986097	8∅16

Axis-3	fourth floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	B	30.71	300	355	0.0406	341.78	single	138.45	4260	224.63	224.63	1.1172287	2∅16

Span	B-C	85.77	300	355	0.1134	314.94	single	138.45	4260	680.835	680.8349	3.3862273	4∅16
support	C	133.42	300	355	0.1764	286.51	double	138.45	4260	1164.2	1164.197	5.7902986	6∅16
Span	C-D	129.69	300	355	0.1715	288.99	double	138.45	4260	1121.91	1121.907	5.579963	6∅16
support	D	119.24	300	355	0.1577	295.69	single	138.45	4260	1008.16	1008.163	5.0142375	6∅16
Span	D-E	129.22	300	355	0.1709	289.30	double	138.45	4260	1116.65	1116.645	5.5537916	6∅16
Support	E	140.01	300	355	0.1852	281.97	double	138.45	4260	1241.37	1241.374	6.1741458	7∅16
Span	E-F	86.21	300	355	0.1140	314.71	single	138.45	4260	684.84	684.8399	3.406147	4∅16
Support	F	50.97	300	355	0.0674	332.45	single	138.45	4260	383.291	383.2911	1.9063516	2∅16

Axis-3	third floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	B	31.25	300	355	0.0413	341.54	single	138.45	4260	228.742	228.7418	1.1376791	2∅16
span	B-C	92.78	300	355	0.1227	311.14	single	138.45	4260	745.482	745.4825	3.7077612	4∅16
support	C	27.07	300	355	0.0358	343.41	single	138.45	4260	197.07	197.0701	0.9801557	2∅16
span	C-D	138.6	300	355	0.1833	282.95	double	138.45	4260	1224.58	1224.582	6.0906278	7∅16
support	D	165.34	300	355	0.2187	262.28	double	138.45	4260	1575.99	1575.993	7.8384216	8∅16
Span	D-E	135.66	300	355	0.1794	284.98	double	138.45	4260	1190.06	1190.065	5.9189525	6∅16
Support	E	43.33	300	355	0.0573	336.03	single	138.45	4260	322.363	322.3629	1.603317	2∅16
Span	E-F	88.2	300	355	0.1166	313.64	single	138.45	4260	703.04	703.0403	3.496669	4∅16
Support	F	130.4	300	355	0.1725	288.53	double	138.45	4260	1129.88	1129.884	5.6196368	6∅16

Axis-3	second floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	B	31.25	300	355	0.0413	341.54	single	138.45	4260	228.742	228.7418	1.1376791	2∅16
span	B-C	91.35	300	355	0.1208	311.93	single	138.45	4260	732.146	732.1462	3.6414315	4∅16
support	C	34.54	300	355	0.0457	340.06	single	138.45	4260	253.925	253.9252	1.2629326	2∅16
span	C-D	137.8	300	355	0.1822	283.51	double	138.45	4260	1215.12	1215.124	6.0435877	7∅16
support	D	197.45	300	355	0.2611	227.15	double	138.45	4260	2173.17	2173.169	10.808562	11∅16
Span	D-E	135.2	300	355	0.1788	285.30	double	138.45	4260	1184.72	1184.723	5.8923843	6∅16
Support	E	35.78	300	355	0.0473	339.50	single	138.45	4260	263.477	263.4766	1.3104379	2∅16

Span	E-F	90.27	300	355	0.1194	312.51	single	138.45	4260	722.125	722.1254	3.5915918	4∅16
Support	F	57.56	300	355	0.0761	329.29	single	138.45	4260	437.001	437.0011	2.1734862	3∅16

Axis-3	first floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	B	28.8	300	355	0.0381	342.64	single	138.45	4260	210.135	210.1349	1.0451352	2∅16
span	B-C	87.08	300	355	0.1152	314.24	single	138.45	4260	692.779	692.7793	3.4456347	4∅16
support	C	25.78	300	355	0.0341	343.98	single	138.45	4260	187.367	187.3674	0.9318981	2∅16
span	C-D	133.2	300	355	0.1762	286.65	double	138.45	4260	1161.68	1161.676	5.7777597	6∅16
support	D	214.1	300	355	0.2831	181.46	double	138.45	4260	2949.63	2949.631	14.670402	15∅16
Span	D-E	131.92	300	355	0.1745	287.51	double	138.45	4260	1147.08	1147.076	5.7051414	6∅16
Support	E	26.55	300	355	0.0351	343.64	single	138.45	4260	193.155	193.1549	0.9606831	2∅16
Span	E-F	84.95	300	355	0.1123	315.38	single	138.45	4260	673.389	673.3892	3.349195	4∅16
Support	F	41.71	300	355	0.0552	336.78	single	138.45	4260	309.62	309.6198	1.5399372	2∅16

Axis-3	ground floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	B	28.8	300	355	0.0381	342.64	single	138.45	4260	210.135	210.1349	1.0451352	2∅16
span	B-C	85.04	300	355	0.1125	315.33	single	138.45	4260	674.205	674.2052	3.3532538	4∅16
support	C	46.15	300	355	0.0610	334.72	single	138.45	4260	344.69	344.6903	1.7143656	2∅16
span	C-D	131.02	300	355	0.1733	288.11	double	138.45	4260	1136.88	1136.878	5.6544193	6∅16
support	D	214.11	300	355	0.2832	181.27	double	138.45	4260	2952.86	2952.862	14.686474	15∅16
Span	D-E	130.34	300	355	0.1724	288.56	double	138.45	4260	1129.21	1129.209	5.6162776	6∅16
Support	E	47.75	300	355	0.0631	333.97	single	138.45	4260	357.442	357.4417	1.7777864	2∅16
Span	E-F	84.36	300	355	0.1116	315.70	single	138.45	4260	668.046	668.0465	3.3226225	4∅16
Support	F	28.8	300	355	0.0381	342.64	single	138.45	4260	210.135	210.1349	1.0451352	2∅16

Axis-3	Basement												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	B	35.84	300	355	0.0474	339.47	single	138.45	4260	263.94	263.9396	1.3127407	2∅16
span	B-C	14.89	300	355	0.0197	348.72	single	138.45	4260	106.748	138.45	0.5309243	2∅16
support	C	21.32	300	355	0.0282	345.94	single	138.45	4260	154.075	154.0751	0.7663139	2∅16
span	C-D	24.32	300	355	0.0322	344.62	single	138.45	4260	176.426	176.4261	0.8774797	2∅16
support	D	37.38	300	355	0.0494	338.77	single	138.45	4260	275.85	275.8502	1.3719796	2∅16
Span	D-E	23.21	300	355	0.0307	345.11	single	138.45	4260	168.136	168.1358	0.8362467	2∅16
Support	E	21.93	300	355	0.0290	345.67	single	138.45	4260	158.606	158.6056	0.7888472	2∅16
Span	E-F	12.56	300	355	0.0166	349.72	single	138.45	4260	89.7867	138.45	0.4465669	2∅16
Support	F	40.56	300	355	0.0536	337.31	single	138.45	4260	300.61	300.61	1.4951259	2∅16

Beam design on axis 3 for all storey

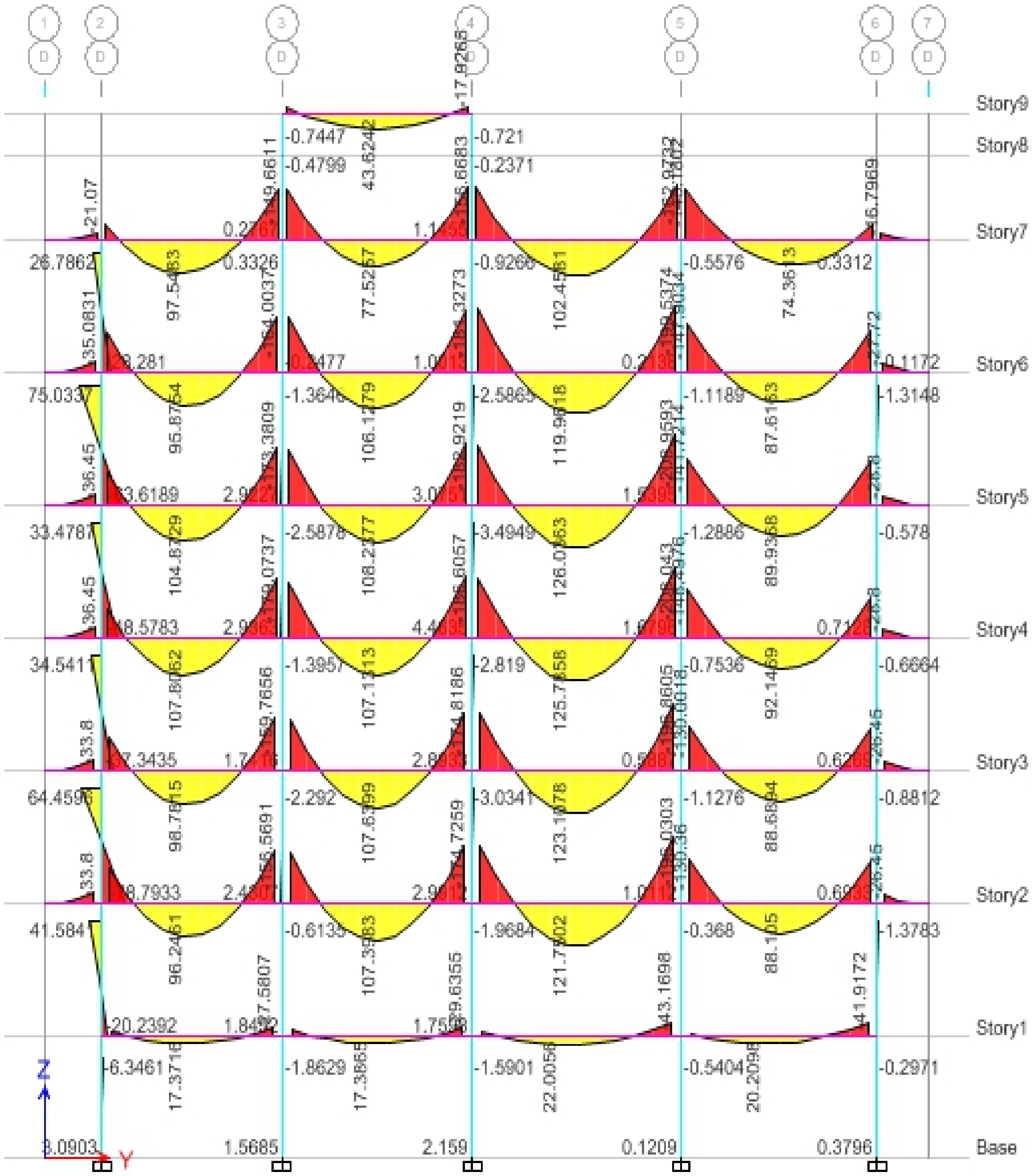


Figure B -1.2 bending moment diagram of beam on axis -D

Table B-1 2 Reinforcement for beam on axis -D

Axis-D	Top Tie beam												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	2	0	300	355	0.0000	355.00	single	138.45	4260	0	138.45	0	0
span	2-3	0	300	355	0.0000	355.00	single	138.45	4260	0	138.45	0	0
support	3	0.74	300	355	0.0010	354.69	single	138.45	4260	5.21578	138.45	0.0259414	2∅16
span	3-4	43.62	300	355	0.0577	335.90	single	138.45	4260	324.65	324.6505	1.6146944	2∅16
support	4	17.93	300	355	0.0237	347.41	single	138.45	4260	129.027	138.45	0.6417319	2∅16
Span	4-5	0	300	355	0.0000	355.00	single	138.45	4260	0	138.45	0	0
Support	5	0	300	355	0.0000	355.00	single	138.45	4260	0	138.45	0	0
Span	5-6	0	300	355	0.0000	355.00	single	138.45	4260	0	138.45	0	0
Support	6	0	300	355	0.0000	355.00	single	138.45	4260	0	138.45	0	0

Axis-D	Fifth floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	2	21.07	300	355	0.0279	346.04	single	138.45	4260	152.22	152.2204	0.7570893	2∅16
span	2-3	97.54	300	355	0.1290	308.50	single	138.45	4260	790.45	790.4499	3.931413	4∅16
support	3	164.03	300	355	0.2169	263.41	double	138.45	4260	1556.81	1556.806	7.7429915	8∅16
span	3-4	77.52	300	355	0.1025	319.29	single	138.45	4260	606.972	606.9721	3.0188604	4∅16
support	4	156.66	300	355	0.2072	269.50	double	138.45	4260	1453.24	1453.242	7.2279003	8∅16
Span	4-5	102.45	300	355	0.1355	305.71	single	138.45	4260	837.806	837.8064	4.1669471	5∅16
Support	5	162.97	300	355	0.2155	264.31	double	138.45	4260	1541.46	1541.463	7.6666816	8∅16
Span	5-6	74.36	300	355	0.0983	320.92	single	138.45	4260	579.273	579.2731	2.8810958	3∅16
Support	6	46.79	300	355	0.0619	334.42	single	138.45	4260	349.784	349.7836	1.7396975	2∅16

Axis-D	Fourth floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	2	35.08	300	355	0.0464	339.82	single	138.45	4260	258.081	258.0807	1.2836002	2∅16
span	2-3	95.87	300	355	0.1268	309.43	single	138.45	4260	774.571	774.5708	3.8524362	4∅16
support	3	164	300	355	0.2169	263.43	double	138.45	4260	1556.37	1556.369	7.7408208	8∅16
span	3-4	106.12	300	355	0.1403	303.59	single	138.45	4260	873.886	873.8865	4.3463964	5∅16
support	4	164.32	300	355	0.2173	263.16	double	138.45	4260	1561.03	1561.032	7.7640089	8∅16
Span	4-5	119.96	300	355	0.1586	295.24	single	138.45	4260	1015.79	1015.792	5.0521842	6∅16
Support	5	199.53	300	355	0.2639	223.96	double	138.45	4260	2227.28	2227.276	11.077667	12∅16

Span	5-6	87.61	300	355	0.1159	313.96	single	138.45	4260	697.629	697.6293	3.4697567	4∅16
Support	6	27.72	300	355	0.0367	343.12	single	138.45	4260	201.972	201.9717	1.0045346	2∅16

Axis-D	Thrid floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	2	36.45	300	355	0.0482	339.19	single	138.45	4260	268.651	268.6513	1.3361747	2∅16
span	2-3	104.87	300	355	0.1387	304.31	single	138.45	4260	861.53	861.5298	4.2849388	5∅16
support	3	173.38	300	355	0.2293	254.99	double	138.45	4260	1699.86	1699.86	8.4544922	9∅16
span	3-4	108.23	300	355	0.1431	302.35	single	138.45	4260	894.908	894.9078	4.4509488	5∅16
support	4	182.92	300	355	0.2419	245.34	double	138.45	4260	1863.97	1863.972	9.2707244	10∅16
Span	4-5	126.03	300	355	0.1667	291.38	single	138.45	4260	1081.31	1081.309	5.3780404	6∅16
Support	5	206.95	300	355	0.2737	210.17	double	138.45	4260	2461.69	2461.693	12.243575	13∅16
Span	5-6	89.93	300	355	0.1189	312.70	single	138.45	4260	718.98	718.9798	3.5759465	4∅16
Support	6	26.8	300	355	0.0354	343.53	single	138.45	4260	195.037	195.0366	0.9700415	2∅16

Axis-D	Second floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	2	34.54	300	355	0.0457	340.06	single	138.45	4260	253.925	253.9252	1.2629326	2∅16
span	2-3	107.8	300	355	0.1426	302.60	single	138.45	4260	890.607	890.6069	4.4295581	5∅16
support	3	179.07	300	355	0.2368	249.39	double	138.45	4260	1795.09	1795.087	8.9281147	9∅16
span	3-4	107.13	300	355	0.1417	303.00	single	138.45	4260	883.923	883.9229	4.3963143	5∅16
support	4	186.6	300	355	0.2468	241.22	double	138.45	4260	1933.9	1933.903	9.6185357	10∅16
Span	4-5	125.75	300	355	0.1663	291.56	single	138.45	4260	1078.24	1078.238	5.3627668	6∅16
Support	5	206.04	300	355	0.2725	212.16	double	138.45	4260	2427.9	2427.904	12.075518	13∅16
Span	5-6	92.14	300	355	0.1219	311.49	single	138.45	4260	739.504	739.5041	3.6780272	4∅16
Support	6	26.8	300	355	0.0354	343.53	single	138.45	4260	195.037	195.0366	0.9700415	2∅16

Axis-D	First floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	2	64.45	300	355	0.0852	325.91	single	138.45	4260	494.379	494.3788	2.4588621	3∅16
span	2-3	98.78	300	355	0.1306	307.80	single	138.45	4260	802.314	802.314	3.9904208	4∅16
support	3	159.76	300	355	0.2113	266.99	double	138.45	4260	1495.94	1495.944	7.4402855	8∅16

span	3-4	107.64	300	355	0.1424	302.70	single	138.45	4260	889.009	889.0089	4.4216098	5∅16
support	4	174.81	300	355	0.2312	253.62	double	138.45	4260	1723.13	1723.133	8.5702418	9∅16
Span	4-5	123.1	300	355	0.1628	293.26	single	138.45	4260	1049.41	1049.411	5.2193943	6∅16
Support	5	195.86	300	355	0.2590	229.45	double	138.45	4260	2134.04	2134.042	10.613958	11∅16
Span	5-6	88.68	300	355	0.1173	313.38	single	138.45	4260	707.452	707.4518	3.5186102	4∅16
Support	6	26.45	300	355	0.0350	343.68	single	138.45	4260	192.403	192.4026	0.9569414	2∅16

Axis-D	ground floor												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	2	64.45	300	355	0.0852	325.91	single	138.45	4260	494.379	494.3788	2.4588621	3∅16
span	2-3	96.24	300	355	0.1273	309.22	single	138.45	4260	778.079	778.0792	3.8698858	4∅16
support	3	156.56	300	355	0.2070	269.58	double	138.45	4260	1451.88	1451.884	7.2211456	8∅16
span	3-4	107.39	300	355	0.1420	302.84	single	138.45	4260	886.514	886.5143	4.4092025	5∅16
support	4	174.72	300	355	0.2311	253.71	double	138.45	4260	1721.66	1721.656	8.5628956	9∅16
Span	4-5	121.75	300	355	0.1610	294.11	single	138.45	4260	1034.89	1034.887	5.147157	6∅16
Support	5	196	300	355	0.2592	229.25	double	138.45	4260	2137.42	2137.418	10.630746	11∅16
Span	5-6	88.1	300	355	0.1165	313.69	single	138.45	4260	702.122	702.1223	3.4921031	4∅16
Support	6	26.45	300	355	0.0350	343.68	single	138.45	4260	192.403	192.4026	0.9569414	2∅16

Axis-D	Basement												
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of bar	Remark
support	2	41.58	300	355	0.0550	336.84	single	138.45	4260	308.6	308.5998	1.5348641	2∅16
span	2-3	17.37	300	355	0.0230	347.65	single	138.45	4260	124.91	138.45	0.6212558	2∅16
support	3	27.56	300	355	0.0364	343.19	single	138.45	4260	200.764	200.7644	0.9985297	2∅16
span	3-4	17.38	300	355	0.0230	347.65	single	138.45	4260	124.983	138.45	0.6216212	2∅16
support	4	1.75	300	355	0.0023	354.27	single	138.45	4260	12.3492	138.45	0.0614206	2∅16
Span	4-5	22	300	355	0.0291	345.64	single	138.45	4260	159.126	159.126	0.7914353	2∅16
Support	5	43.1	300	355	0.0570	336.14	single	138.45	4260	320.55	320.55	1.5943004	2∅16
Span	5-6	20.2	300	355	0.0267	346.42	single	138.45	4260	145.775	145.7753	0.7250339	2∅16
Support	6	41.91	300	355	0.0554	336.69	single	138.45	4260	311.19	311.1898	1.5477458	2∅16

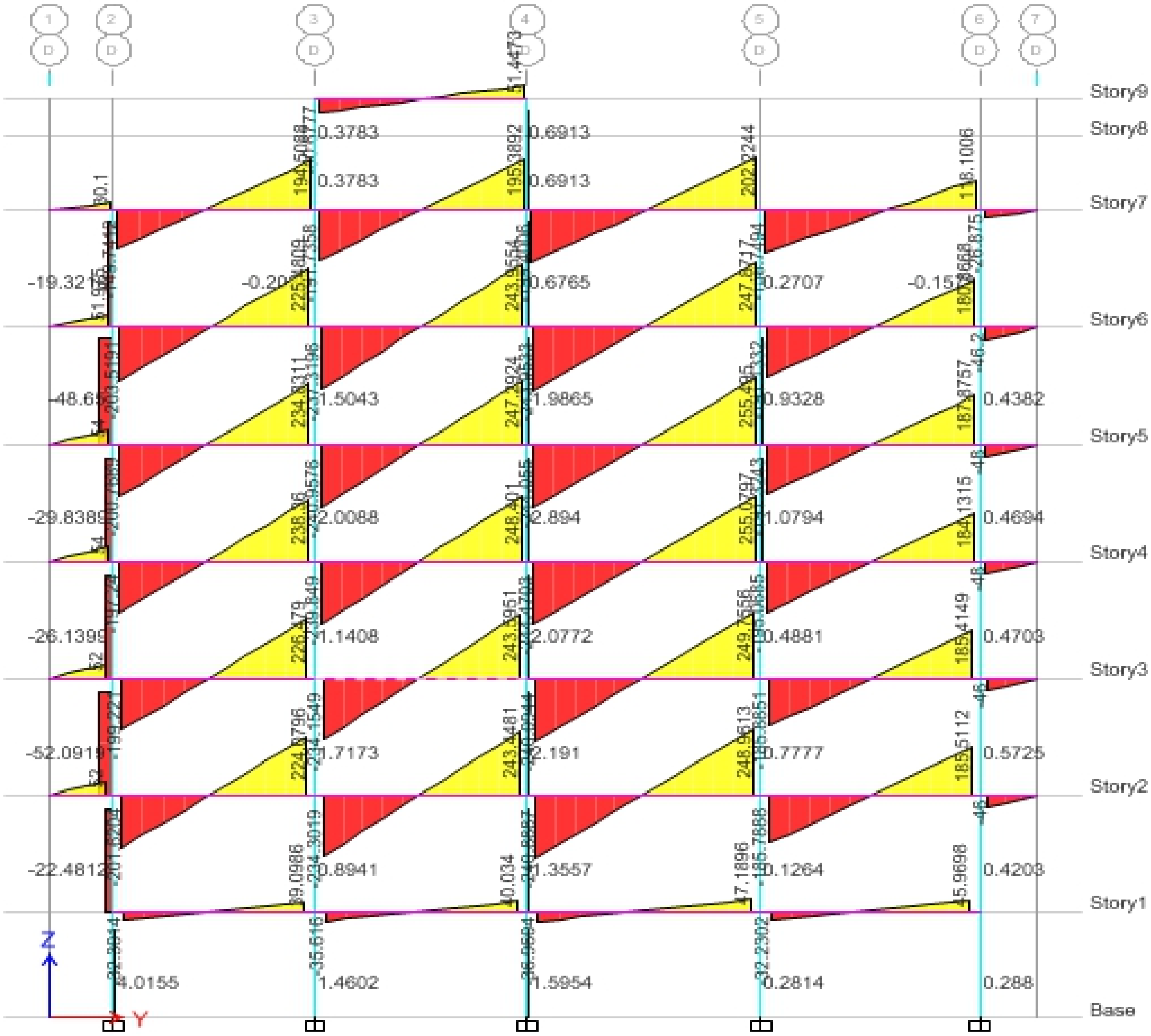


Fig shear force diagram for beam in axis D

Table B-1 3 shear reinforcement of beam on axis D

Fifth floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
2-3	194.5	119.45	266.25	231.4	292.6	231.4	∅8 c/c 230
3-4	195.38	138.53	266.25	199.6	292.6	199.6	∅8 c/c 190
4-5	202.2	88.68	266.25	311.7	292.6	266.3	∅8 c/c 310
5-6	118.1	104.54	266.25	241.8	292.6	245.8	∅8 c/c 240

Fourth floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark

2-3	225.4	120.88	266.25	228.7	292.6	228.7	∅8 c/c 220
3-4	243.95	146.14	266.25	189.2	292.6	189.2	∅8 c/c 180
4-5	247.87	80.13	266.25	345.0	292.6	266.3	∅8 c/c 330
5-7	180.38	127.77	266.25	250.46	292.6	243.67	∅8 c/c 240

Third Floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
2-3	234.83	118.36	266.25	233.6	292.6	233.6	∅8 c/c 230
3-4	247.29	145.94	266.25	189.4	292.6	189.4	∅8 c/c 180
4-5	255.49	178.55	266.25	351.9	292.6	266.3	∅8 c/c 330
5-6	187.87	112.2	266.25	212.32	292.6	234.13	∅8 c/c 230

	Second floor						
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
2-3	238.36	118.36	266.25	233.6	292.6	233.6	∅8 c/c 230
3-4	248.4	145.94	266.25	189.4	292.6	189.4	∅8 c/c 180
4-5	255.07	78.55	266.25	351.9	292.6	266.3	∅8 c/c 330
5-6	184.13	123.56	266.25	246.65	292.6	194.34	∅8 c/c 190

First Floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
2-3	226.47	26.86	266.25	1029.2	292.6	266.3	∅8 c/c 230
3-4	243.59	31.76	266.25	870.5	292.6	266.3	∅8 c/c 330
4-5	249.75	28.67	266.25	964.3	292.6	266.3	∅8 c/c 230
5-6	185.41	45.56	266.25	567.67	292.6	262.45	∅8 c/c 260

Ground Floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
2-3	224.07	124.44	266.25	222.2	292.6	222.2	∅8 c/c 220
3-4	243.44	98.78	266.25	279.9	292.6	266.3	∅8 c/c 220
4-5	248.95	89.96	266.25	307.3	292.6	266.3	∅8 c/c 220
5-6	185.51	134.5	266.25	256.67	2.92.6	254.43	∅8 c/c 220

Basement							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
2-3	39.09	23.67	266.25	1168.0	292.6	266.3	∅8 c/c 260
3-4	40.03	28.98	266.25	954.0	292.6	266.3	∅8 c/c 260
4-5	47.18	34.54	266.25	800.4	292.6	266.3	∅8 c/c 260
5-6	45.96	32.56	266.25	657.5	292.6	266.3	∅8 c/c 260

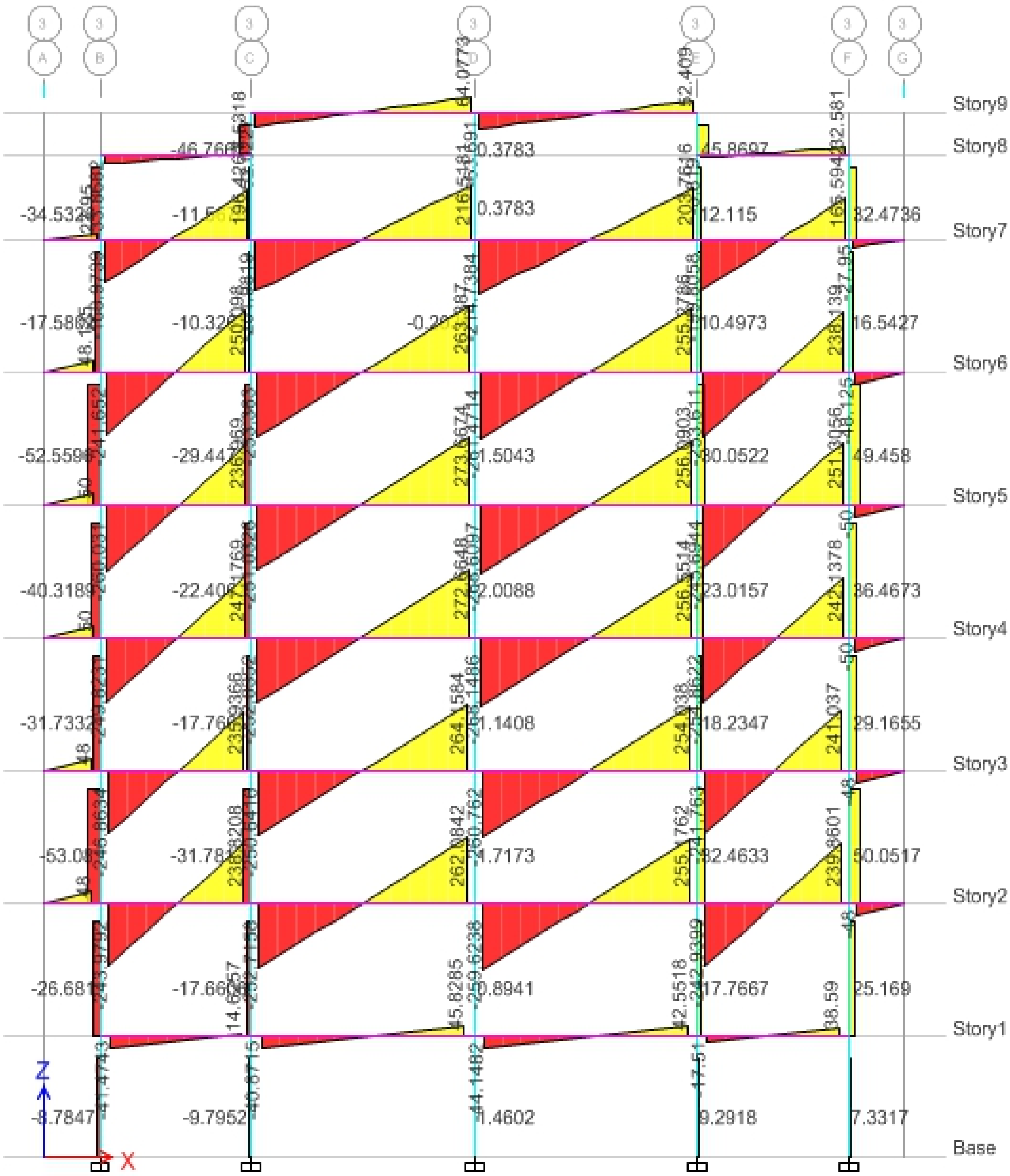


Figure B -1.4 shear force diagram for beam in axis 3

Table B-1 4 shear reinforcement of beam on axis 3

Top tie beam							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
B-C	46.76	35.65	266.25	775.5	292.6	266.3	∅8 C/C 220
C-D	64.07	45.98	266.25	601.3	292.6	266.3	∅8 C/C 220
D-E	52.4	42.87	266.25	644.9	292.6	266.3	∅8 C/C 220
E-F	32.58	28.32	266.25	625.4	292.6	266.3	∅8 C/C 220

Fifth Floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
B-C	196.42	102.3	266.25	270.2	292.6	266.3	∅8 C/C 250
C-D	216.51	162.43	266.25	170.2	292.6	170.2	∅8 C/C 170
D-E	203.7	144.43	266.25	191.4	292.6	191.4	∅8 C/C 190
E-F	165.59	122.73	266.25	162.65	292.6	283.54	∅8 C/C 280
Fourth floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
B-C	250.09	183.33	266.25	150.8	292.6	150.8	∅8 C/C 150
C-D	263.39	211.13	266.25	130.9	292.6	130.9	∅8 C/C 130
D-E	255.28	214.56	266.25	128.8	292.6	128.8	∅8 C/C 120
E-F	238.13	201.45	266.25	123.33	292.6	124.33	∅8 C/C 120

Third Floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
B-C	236.69	187.32	266.25	147.6	292.6	147.6	∅8 C/C 140
C-D	273.68	231.33	266.25	119.5	292.6	119.5	∅8 C/C 110
D-E	256.09	225.45	266.25	122.6	292.6	122.6	∅8 C/C 120
E-F	251.31	235.54	266.25	139.98	292.6	239	∅8 C/C 230

Second Floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
B-C	247.17	213.17	266.25	325.8	292.6	266.3	∅8 C/C 250

C-D	272.66	231.33	266.25	298.8	292.6	266.3	∅8 C/C 250
D-E	256.55	225.45	266.25	298.9	292.6	266.3	∅8 C/C 250
E-F	242.13	213.45	266.25	276.3	292.6	266.3	∅8 C/C 250

First Floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
B-C	235.9	222.23	266.25	124.4	292.6	292.5	∅8 C/C 290
C-D	264.51	198.44	266.25	139.3	292.6	291.2	∅8 C/C 290
D-E	254.04	213.34	266.25	129.6	292.6	290.6	∅8 C/C 290
E-F	241.03	213.34	266.25	145.8	292.6	292.7	∅8 C/C 290

Ground Floor							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
B-C	238.82	118.36	266.25	233.6	292.6	233.6	∅8 C/C 230
C-D	262.08	145.94	266.25	189.4	292.6	189.4	∅8 C/C 180
D-E	255.07	78.55	266.25	351.9	292.6	266.3	∅8 C/C 260
E-F	239.86	145.54	266.25	234.56	292.6	236.67	∅8 C/C 230

Basement							
span	Vmax	Ved	Sbmax	Scal	Smin	Spro	Remark
B-C	14.62	11.32	266.25	2442.2	292.6	266.3	∅8 C/C 260
C-D	45.82	34.54	266.25	800.4	292.6	266.3	∅8 C/C 260
D-E	44.14	34.67	266.25	797.4	292.6	266.3	∅8 C/C 260
E-F	38.59	29.76	266.25	645.54	292.6	266.3	∅8 C/C 260

Appendix C- Lateral load

C-1 Center mass of the floor

Table C-1 1 Center mass of the column for ground floor

ground floor column=(3.15+2.85)=3m

Designati	H(m)	L(m	B(m	V(m³	ϒc	Weig	Xm(	Ym(	WX	WY

on			)	)	)	(KN/m3)	ht	m)	m)		
	C	3	0.6	0.6	1.08	25	27	3.8	-1.5	102.6	-40.5
1	E	3	0.6	0.6	1.08	25	27	15	-1.5	405	-40.5
	B	3	0.6	0.4	0.72	25	18	0	0	0	0
	C	3	0.6	0.4	0.72	25	18	3.8	0	68.4	0
2	D	3	0.6	0.4	0.72	25	18	9.4	0	169.2	0
	E	3	0.6	0.4	0.72	25	18	15	0	270	0
	F	3	0.6	0.4	0.72	25	18	18.8	0	338.4	0
	B	3	0.6	0.4	0.72	25	18	0	4.7	0	84.6
	C	3	0.6	0.4	0.72	25	18	3.8	4.7	68.4	84.6
3	D	3	0.6	0.4	0.72	25	18	9.4	4.7	169.2	84.6
	E	3	0.6	0.4	0.72	25	18	15	4.7	270	84.6
	F	3	0.6	0.4	0.72	25	18	18.8	4.7	338.4	84.6
	B	3	0.6	0.4	0.72	25	18	0	9.65	0	173.7
	C	3	0.6	0.4	0.72	25	18	3.8	9.65	68.4	173.7
4	D	3	0.6	0.4	0.72	25	18	9.4	9.65	169.2	173.7
	E	3	0.6	0.4	0.72	25	18	15	9.65	270	173.7
	F	3	0.6	0.4	0.72	25	18	18.8	9.65	338.4	173.7
	B	3	0.6	0.4	0.72	25	18	0	15.1	0	271.8
	C	3	0.6	0.4	0.72	25	18	3.8	15.1	68.4	271.8
5	D	3	0.6	0.4	0.72	25	18	9.4	15.1	169.2	271.8
	E	3	0.6	0.4	0.72	25	18	15	15.1	270	271.8
	F	3	0.6	0.4	0.72	25	18	18.8	15.1	338.4	271.8
	B	3	0.6	0.4	0.72	25	18	0	20.2	0	363.6
	C	3	0.6	0.4	0.72	25	18	3.8	20.2	68.4	363.6
6	D	3	0.6	0.4	0.72	25	18	9.4	20.2	169.2	363.6
	E	3	0.6	0.4	0.72	25	18	15	20.2	270	363.6
	F	3	0.6	0.4	0.72	25	18	18.8	20.2	338.4	363.6
						TOTAL	**450**			**4737.6**	**4387.5**

Table C-1 2 Centre of mass calculation for First floor to fourth column typical, 4 in number

Designation		H(m)	L(m)	B(m)	V(m³)	ϒc (KN/m³)	Weight	Xm (m)	Ym(m)	WX	WY
1	C	3.15	0.5	0.5	0.7875	25	19.688	3.8	-1.5	74.813	-29.53

	E	3.15	0.5	0.5	0.7875	25	19.688	15	-1.5	295.31	-29.53
2	B	3.15	0.4	0.3	0.378	25	9.45	0	0	0	0
	C	3.15	0.4	0.3	0.378	25	9.45	3.8	0	35.91	0
	D	3.15	0.4	0.3	0.378	25	9.45	9.4	0	88.83	0
	E	3.15	0.4	0.3	0.378	25	9.45	15	0	141.75	0
	F	3.15	0.4	0.3	0.378	25	9.45	18.8	0	177.66	0
3	B	3.15	0.4	0.3	0.378	25	9.45	0	4.7	0	44.415
	C	3.15	0.4	0.3	0.378	25	9.45	3.8	4.7	35.91	44.415
	D	3.15	0.4	0.3	0.378	25	9.45	9.4	4.7	88.83	44.415
	E	3.15	0.4	0.3	0.378	25	9.45	15	4.7	141.75	44.415
	F	3.15	0.4	0.3	0.378	25	9.45	18.8	4.7	177.66	44.415
4	B	3.15	0.4	0.3	0.378	25	9.45	0	9.65	0	91.193
	C	3.15	0.4	0.3	0.378	25	9.45	3.8	9.65	35.91	91.193
	D	3.15	0.4	0.3	0.378	25	9.45	9.4	9.65	88.83	91.193
	E	3.15	0.4	0.3	0.378	25	9.45	15	9.65	141.75	91.193
	F	3.15	0.4	0.3	0.378	25	9.45	18.8	9.65	177.66	91.193
5	B	3.15	0.4	0.3	0.378	25	9.45	0	15.1	0	142.7
	C	3.15	0.4	0.3	0.378	25	9.45	3.8	15.1	35.91	142.7
	D	3.15	0.4	0.3	0.378	25	9.45	9.4	15.1	88.83	142.7
	E	3.15	0.4	0.3	0.378	25	9.45	15	15.1	141.75	142.7
	F	3.15	0.4	0.3	0.378	25	9.45	18.8	15.1	177.66	142.7
6	B	3.15	0.4	0.3	0.378	25	9.45	0	20.2	0	190.89
	C	3.15	0.4	0.3	0.378	25	9.45	3.8	20.2	35.91	190.89
	D	3.15	0.4	0.3	0.378	25	9.45	9.4	20.2	88.83	190.89
	E	3.15	0.4	0.3	0.378	25	9.45	15	20.2	141.75	190.89
	F	3.15	0.4	0.3	0.378	25	9.45	18.8	20.2	177.66	190.89
						TOTAL	**236.25**			**2590.9**	**2286.9**

Table C-1 3 Center of mass calculation for fifth floor column H=(3.15+3)/2=3.075

Designation		H(m)	L(m)	B(m)	V(m³)	ϒc(KN/m³)	Weight	Xm(m)	Ym(m)	WX	WY
3	C	3.075	0.3	0.2	0.1845	25	4.6125	3.8	4.7	17.5275	21.6788

	D	3.075	0.3	0.2	0.1845	25	4.6125	9.4	4.7	43.3575	21.6788
	E	3.075	0.3	0.2	0.1845	25	4.6125	15	4.7	69.1875	21.6788
4	C	3.075	0.3	0.2	0.1845	25	4.6125	3.8	9.65	17.5275	44.5106
	D	3.075	0.3	0.2	0.1845	25	4.6125	9.4	9.65	43.3575	44.5106
	E	3.075	0.3	0.2	0.1845	25	4.6125	15	9.65	69.1875	44.5106
						TOTAL	**27.675**			**260.145**	**198.568**

TTable C-1 4 Center of mass calculation for water tank column H=(2+3.15)/2=2.575m

Designation		H(m)	L(m)	B(m)	V(m³)	ϒc(KN/m3)	Weight	Xm(m)	Ym(m)	WX	WY
3	B	2.575	0.3	0.2	0.1545	25	3.8625	0	4.7	0	18.1538
	C	2.575	0.3	0.2	0.1545	25	3.8625	3.8	4.7	14.6775	18.1538
	E	2.575	0.3	0.2	0.1545	25	3.8625	15	4.7	57.9375	18.1538
	F	2.575	0.3	0.2	0.1545	25	3.8625	18.8	4.7	72.615	18.1538
4	B	2.575	0.3	0.2	0.1545	25	3.8625	0	9.65	0	37.2731
	C	2.575	0.3	0.2	0.1545	25	3.8625	3.8	9.65	14.6775	37.2731
	E	2.575	0.3	0.2	0.1545	25	3.8625	15	9.65	57.9375	37.2731
	F	2.575	0.3	0.2	0.1545	25	3.8625	18.8	9.65	72.615	37.2731
						TOTAL	**30.9**			**290.46**	**221.708**

Table C-2 1 Center of mass calculation for beam along letter axis of grade beam

designation		L(m)	D(m)	W(m)	V(m³)	ϒc (KN/m3)	Weight	Xm (m)	Ym(m)	WX	WY
B	2-3	3.8	0.6	0.4	0.912	25	22.8	0	2.35	0	53.58
	3-4	5.6	0.6	0.4	1.344	25	33.6	0	7.175	0	241.08
	4-5	5.6	0.6	0.4	1.344	25	33.6	0	12.375	0	415.8
	5-6	3.8	0.6	0.4	0.912	25	22.8	0	17.65	0	402.42
C	2-3	3.8	0.6	0.4	1.344	25	33.6	3.8	2.35	127.68	78.96
	3-4	5.6	0.6	0.4	1.344	25	33.6	3.8	7.175	127.68	241.08
	4-5	5.6	0.6	0.4	1.344	25	33.6	3.8	12.375	127.68	415.8
	56	3.8	0.6	0.4	0.912	25	22.8	3.8	17.65	86.64	402.42
D	2-3	3.8	0.6	0.4	0.912	25	22.8	9.4	2.35	214.32	53.58
	3-4	5.6	0.6	0.4	1.344	25	33.6	9.4	7.175	315.84	241.08
	4-5	5.6	0.6	0.4	1.344	25	33.6	9.4	12.375	315.8	415.8

										4	
	5-6	3.8	0.6	0.4	0.912	25	22.8	9.4	17.65	214.32	402.42
E	2-3	3.8	0.6	0.4	0.912	25	22.8	15	2.35	342	53.58
	3-4	5.6	0.6	0.4	1.344	25	33.6	15	7.175	504	241.08
	4-5	5.6	0.6	0.4	1.344	25	33.6	15	12.375	504	415.8
	5-6	3.8	0.6	0.4	0.912	25	22.8	15	17.65	342	402.42
F	2-3	3.8	0.6	0.4	0.912	25	22.8	18.8	2.35	428.64	53.58
	3-4	5.6	0.6	0.4	1.344	25	33.6	18.8	7.175	631.68	241.08
	4-5	5.6	0.6	0.4	1.344	25	33.6	18.8	12.375	631.68	415.8
	5-6	3.8	0.6	0.4	0.912	25	22.8	18.8	17.65	428.64	402.42
						TOTAL	**574.8**			**5342.6**	**5589.8**

Table C-2 2 center of mass for beam along number axis 1st upto 3rd floor

designation		L(m)	D(m)	B(m)	V(m³)	ϒc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	WY
2	AB	1.4	0.5	0.4	0.28	25	7	-0.7	-1.5	-4.9	-10.5
	BC	3.8	0.5	0.4	0.76	25	19	1.9	-1.5	36.1	-28.5
	CD	5.6	0.5	0.4	1.12	25	28	6.6	-1.5	184.8	-42
	DE	5.6	0.5	0.4	1.12	25	28	12.2	-1.5	341.6	-42
	EF	3.8	0.5	0.4	0.76	25	19	17.6	-1.5	334.4	-28.5
	FG	1.4	0.5	0.4	0.28	25	7	21.3	-1.5	149.1	-10.5
3	AB	1.4	0.5	0.4	0.76	25	19	-0.7	4.7	-13.3	89.3
	BC	3.8	0.5	0.4	1.12	25	28	1.9	4.7	53.2	131.6
	CD	5.6	0.5	0.4	1.12	25	28	6.6	4.7	184.8	131.6
	DE	5.6	0.5	0.4	1.12	25	28	12.2	4.7	341.6	131.6
	EF	3.8	0.5	0.4	0.76	25	19	17.6	4.7	334.4	89.3
	FG	1.4	0.5	0.4	0.28	25	7	21.3	4.7	149.1	32.9
4	AB	1.4	0.5	0.4	0.28	25	7	-0.7	9.65	-4.9	67.55
	BC	3.8	0.5	0.4	0.76	25	19	1.9	9.65	36.1	183.35
	CD	5.6	0.5	0.4	1.12	25	28	6.6	9.65	184.8	270.2
	DE	5.6	0.5	0.4	1.12	25	28	12.2	9.65	341.6	270.2
	EF	3.8	0.5	0.4	0.76	25	19	17.6	9.65	334.4	183.35
	FG	1.4	0.5	0.4	0.28	25	7	21.3	9.65	149.1	67.55

5	AB	1.4	0.5	0.4	0.28	25	7	-0.7	15.1	-4.9	105.7
	BC	3.8	0.5	0.4	0.76	25	19	1.9	15.1	36.1	286.9
	CD	5.6	0.5	0.4	1.12	25	28	6.6	15.1	184.8	422.8
	DE	5.6	0.5	0.4	1.12	25	28	12.2	15.1	341.6	422.8
	EF	3.8	0.5	0.4	0.76	25	19	17.6	15.1	334.4	286.9
	FG	1.4	0.5	0.4	0.28	25	7	21.3	15.1	149.1	105.7
6	AB	1.4	0.5	0.4	0.28	25	7	-0.7	20.2	-4.9	141.4
	BC	3.8	0.5	0.4	0.76	25	19	1.9	20.2	36.1	383.8
	CD	5.6	0.5	0.4	1.12	25	28	6.6	20.2	184.8	565.6
	DE	5.6	0.5	0.4	1.12	25	28	12.2	20.2	341.6	565.6
	EF	3.8	0.5	0.4	0.76	25	19	17.6	20.2	334.4	383.8
	FG	1.4	0.5	0.4	0.28	25	7	21.3	20.2	149.1	141.4
						TOTAL	**561**			**5214.2**	**5298.9**

Table C-2 3 center of mass calculation for beam along letter axis, 1st upto 3rd floor

designation		L(m)	D(m)	W(m)	V(m³)	ϒc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	WY
B	12	1.4	0.5	0.4	0.28	25	7	0	-0.75	0	-5.25
	23	3.8	0.5	0.4	0.76	25	19	0	2.35	0	44.65
	34	5.6	0.5	0.4	1.12	25	28	0	7.175	0	200.9
	45	5.6	0.5	0.4	1.12	25	28	0	12.375	0	346.5
	56	3.8	0.5	0.4	0.76	25	19	0	17.65	0	335.35
	67	1.4	0.5	0.4	0.28	25	7	0	20.875	0	146.125
C	12	1.4	0.5	0.4	0.76	25	19	3.8	-0.75	72.2	-14.25
	23	3.8	0.5	0.4	1.12	25	28	3.8	2.35	106.4	65.8
	34	5.6	0.5	0.4	1.12	25	28	3.8	7.175	106.4	200.9
	45	5.6	0.5	0.4	1.12	25	28	3.8	12.375	106.4	346.5
	56	3.8	0.5	0.4	0.76	25	19	3.8	17.65	72.2	335.35
	67	1.4	0.5	0.4	0.28	25	7	3.8	20.875	26.6	146.125
D	12	1.4	0.5	0.4	0.28	25	7	9.4	-0.75	65.8	-5.25
	23	3.8	0.5	0.4	0.76	25	19	9.4	2.35	178.6	44.65
	34	5.6	0.5	0.4	1.12	25	28	9.4	7.175	263.2	200.9
	45	5.6	0.5	0.4	1.12	25	28	9.4	12.375	263.2	346.5
	56	3.8	0.5	0.4	0.76	25	19	9.4	17.65	178.6	335.35
	67	1.4	0.5	0.4	0.28	25	7	9.4	20.875	65.8	146.125
E	12	1.4	0.5	0.4	0.28	25	7	15	-0.75	105	-5.25
	23	3.8	0.5	0.4	0.76	25	19	15	2.35	285	44.65
	34	5.6	0.5	0.4	1.12	25	28	15	7.175	420	200.9

	45	5.6	0.5	0.4	1.12	25	28	15	12.375	420	346.5
	56	3.8	0.5	0.4	0.76	25	19	15	17.65	285	335.35
	67	1.4	0.5	0.4	0.28	25	7	15	20.875	105	146.125
F	12	1.4	0.5	0.4	0.28	25	7	18.8	-0.75	131.6	-5.25
	23	3.8	0.5	0.4	0.76	25	19	18.8	2.35	357.2	44.65
	34	5.6	0.5	0.4	1.12	25	28	18.8	7.175	526.4	200.9
	45	5.6	0.5	0.4	1.12	25	28	18.8	12.375	526.4	346.5
	56	3.8	0.5	0.4	0.76	25	19	18.8	17.65	357.2	335.35
	67	1.4	0.5	0.4	0.28	25	7	18.8	20.875	131.6	146.125
						TOTAL	**561**			**5155.8**	**5353.525**

Table C-2 4 Center of mass calculation for beam along number axis 4th floor

designation		L(m)	D(m)	W(m)	V(m³)	ϒc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	WY
2	AB	1.4	0.3	0.2	0.084	25	2.1	-0.7	-1.5	-1.47	-3.15
	BC	3.8	0.3	0.2	0.228	25	5.7	1.9	-1.5	10.83	-8.55
	CD	5.6	0.3	0.2	0.336	25	8.4	6.6	-1.5	55.44	-12.6
	DE	5.6	0.3	0.2	0.336	25	8.4	12.2	-1.5	102.48	-12.6
	EF	3.8	0.3	0.2	0.228	25	5.7	17.6	-1.5	100.32	-8.55
	FG	1.4	0.3	0.2	0.084	25	2.1	21.3	-1.5	44.73	-3.15
3	AB	1.4	0.3	0.2	0.228	25	5.7	-0.7	4.7	-3.99	26.79
	BC	3.8	0.3	0.2	0.336	25	8.4	1.9	4.7	15.96	39.48
	CD	5.6	0.3	0.2	0.336	25	8.4	6.6	4.7	55.44	39.48
	DE	5.6	0.3	0.2	0.336	25	8.4	12.2	4.7	102.48	39.48
	EF	3.8	0.3	0.2	0.228	25	5.7	17.6	4.7	100.32	26.79
	FG	1.4	0.3	0.2	0.084	25	2.1	21.3	4.7	44.73	9.87
4	AB	1.4	0.3	0.2	0.084	25	2.1	-0.7	9.65	-1.47	20.265
	BC	3.8	0.3	0.2	0.228	25	5.7	1.9	9.65	10.83	55.005
	CD	5.6	0.3	0.2	0.336	25	8.4	6.6	9.65	55.44	81.06
	DE	5.6	0.3	0.2	0.336	25	8.4	12.2	9.65	102.48	81.06
	EF	3.8	0.3	0.2	0.228	25	5.7	17.6	9.65	100.32	55.005
	FG	1.4	0.3	0.2	0.084	25	2.1	21.3	9.65	44.73	20.265
5	AB	1.4	0.3	0.2	0.084	25	2.1	-0.7	15.1	-1.47	31.71
	BC	3.8	0.3	0.2	0.228	25	5.7	1.9	15.1	10.83	86.07
	CD	5.6	0.3	0.2	0.336	25	8.4	6.6	15.1	55.44	126.84
	DE	5.6	0.3	0.2	0.336	25	8.4	12.2	15.1	102.48	126.84
	EF	3.8	0.3	0.2	0.228	25	5.7	17.6	15.1	100.32	86.07

	FG	1.4	0.3	0.2	0.084	25	2.1	21.3	15.1	44.73	31.71
6	AB	1.4	0.3	0.2	0.084	25	2.1	-0.7	20.2	-1.47	42.42
	BC	3.8	0.3	0.2	0.228	25	5.7	1.9	20.2	10.83	115.14
	CD	5.6	0.3	0.2	0.336	25	8.4	6.6	20.2	55.44	169.68
	DE	5.6	0.3	0.2	0.336	25	8.4	12.2	20.2	102.48	169.68
	EF	3.8	0.3	0.2	0.228	25	5.7	17.6	20.2	100.32	115.14
	FG	1.4	0.3	0.2	0.084	25	2.1	21.3	20.2	44.73	42.42
						TOTAL	**168.3**			**1564.26**	**1589.67**

Table C-2 5 Center of mass calculation for beam along letter axis, 4th floor

designation		L(m)	D(m)	W(m)	V(m³)	ϒc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	WY
B	12	1.4	0.3	0.2	0.084	25	2.1	0	-0.75	0	-1.575
	23	3.8	0.3	0.2	0.228	25	5.7	0	2.35	0	13.395
	34	5.6	0.3	0.2	0.336	25	8.4	0	7.175	0	60.27
	45	5.6	0.3	0.2	0.336	25	8.4	0	12.375	0	103.95
	56	3.8	0.3	0.2	0.228	25	5.7	0	17.65	0	100.605
	67	1.4	0.3	0.2	0.084	25	2.1	0	20.875	0	43.8375
C	12	1.4	0.3	0.2	0.228	25	5.7	3.8	-0.75	21.66	-4.275
	23	3.8	0.3	0.2	0.336	25	8.4	3.8	2.35	31.92	19.74
	34	5.6	0.3	0.2	0.336	25	8.4	3.8	7.175	31.92	60.27
	45	5.6	0.3	0.2	0.336	25	8.4	3.8	12.375	31.92	103.95
	56	3.8	0.3	0.2	0.228	25	5.7	3.8	17.65	21.66	100.605
	67	1.4	0.3	0.2	0.084	25	2.1	3.8	20.875	7.98	43.8375
D	12	1.4	0.3	0.2	0.084	25	2.1	9.4	-0.75	19.74	-1.575
	23	3.8	0.3	0.2	0.228	25	5.7	9.4	2.35	53.58	13.395
	34	5.6	0.3	0.2	0.336	25	8.4	9.4	7.175	78.96	60.27
	45	5.6	0.3	0.2	0.336	25	8.4	9.4	12.375	78.96	103.95
	56	3.8	0.3	0.2	0.228	25	5.7	9.4	17.65	53.58	100.605
	67	1.4	0.3	0.2	0.084	25	2.1	9.4	20.875	19.74	43.8375
E	12	1.4	0.3	0.2	0.084	25	2.1	15	-0.75	31.5	-1.575
	23	3.8	0.3	0.2	0.228	25	5.7	15	2.35	85.5	13.395
	34	5.6	0.3	0.2	0.336	25	8.4	15	7.175	126	60.27
	45	5.6	0.3	0.2	0.336	25	8.4	15	12.375	126	103.95
	56	3.8	0.3	0.2	0.228	25	5.7	15	17.65	85.5	100.605
	67	1.4	0.3	0.2	0.084	25	2.1	15	20.875	31.5	43.8375
F	12	1.4	0.3	0.2	0.084	25	2.1	18.8	-0.75	39.48	-1.575

	23	3.8	0.3	0.2	0.228	25	5.7	18.8	2.35	107.16	13.395
	34	5.6	0.3	0.2	0.336	25	8.4	18.8	7.175	157.92	60.27
	45	5.6	0.3	0.2	0.336	25	8.4	18.8	12.375	157.92	103.95
	56	3.8	0.3	0.2	0.228	25	5.7	18.8	17.65	107.16	100.605
	67	1.4	0.3	0.2	0.084	25	2.1	18.8	20.875	39.48	43.8375
						TOTAL	**168.3**			**1546.74**	**1606.058**

Table C-2 6 Center of mass calculation for beam along number axis of 5th floor

designation		L(m)	D(m)	W(m)	V(m³)	ϒc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	WY
3	CD	5.6	0.3	0.2	0.336	25	8.4	1.9	-1.5	15.96	-12.6
	DE	5.6	0.3	0.2	0.336	25	8.4	6.6	-1.5	55.44	-12.6
4	CD	5.6	0.3	0.2	0.336	25	8.4	12.2	-1.5	102.48	-12.6
	DE	5.6	0.3	0.2	0.336	25	8.4	17.6	-1.5	147.84	-12.6
						TOTAL	**33.6**			**321.72**	**-50.4**

Table C-2 7 Center of mass for beam along number axis of 5th floor

designation		L(m)	D(m)	W(m)	V(m³)	ϒc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	W
C	34	4.95	0.3	0.2	0.297	25	7.425	6.6	4.7	49.005	34
D	34	4.95	0.3	0.2	0.297	25	7.425	12.2	4.7	90.585	34
E	34	4.95	0.3	0.2	0.297	25	7.425	17.6	4.7	130.68	34
						TOTAL	**22.28**			**270.27**	**10**

Table C-2 8 Center of mass for beam along number axis of water tank floor

designation		L(m)	D(m)	W(m)	V(m³)	ϒc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	W
3	BC	3.8	0.3	0.2	0.228	25	5.7	1.9	-1.5	10.83	-8
	EF	3.8	0.3	0.2	0.228	25	5.7	6.6	-1.5	37.62	-8
4	BC	3.8	0.3	0.2	0.228	25	5.7	12.2	-1.5	69.54	-8
	EF	3.8	0.3	0.2	0.228	25	5.7	17.6	-1.5	100.32	-8
						TOTAL	**22.8**			**218.31**	**-3**

Table C-2 9 Center of mass beam along number axis of water tank floor

designation		L(m)	D(m)	W(m)	V(m³)	ϒc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	W
B	34	4.95	0.3	0.2	0.297	25	7.425	6.6	4.7	49.005	34
C	34	4.95	0.3	0.2	0.297	25	7.425	12.2	4.7	90.585	34
E	34	4.95	0.3	0.2	0.297	25	7.425	17.6	4.7	130.68	34
F	34	4.95	0.3	0.2	0.297	25	7.425	17.6	4.7	130.68	34
						TOTAL	**29.7**			**400.95**	**13**

Table C-2 10 Center of mass for slab from 1st to 4th floor

slab number	L(m)	W(m)	D(m)	V(m³)	ϒc,KN/m3	Weight	Xm(m)	Ym(m)	WX	WY
S1	5.1	3.8	0.25	4.845	25	121.1	1.9	17.65	230.14	2137.9
S2	5.6	5.1	0.25	7.14	25	178.5	6.6	17.65	1178.1	3150.5
S3	5.6	5.1	0.25	7.14	25	178.5	12.2	17.65	2177.7	3150.5
S4	5.1	3.8	0.25	4.845	25	121.1	16.9	17.65	2047	2137.9
S5	5.45	3.8	0.25	5.1775	25	129.4	1.9	12.375	245.93	1601.8
S6	5.6	5.45	0.25	7.63	25	190.8	6.6	12.375	1259	2360.5
S7	5.6	5.45	0.25	7.63	25	190.8	12.2	12.375	2327.2	2360.5
S8	5.45	3.8	0.25	5.1775	25	129.4	16.9	12.375	2187.5	1601.8
S9	5.6	4.95	0.25	6.93	25	173.3	6.6	7.175	1143.5	1243.1
S10	5.6	4.95	0.25	6.93	25	173.3	12.2	7.175	2113.7	1243.1
S11	4.7	3.8	0.25	4.465	25	111.6	1.9	2.35	212.09	262.32
S12	5.6	4.7	0.25	6.58	25	164.5	6.6	2.35	1085.7	386.58
S13	5.6	4.7	0.25	4.465	25	111.6	12.2	2.35	1361.8	262.32
S14	4.7	3.8	0.25	1.482	25	37.05	16.9	2.35	626.15	87.068
C1	3.8	1.56	0.25	1.482	25	37.05	1.9	5.48	70.395	203.03
C2	3.8	1.56	0.25	1.482	25	37.05	16.9	5.48	626.15	203.03
C3	1.5	1.4	0.25	0.525	25	13.13	-0.7	-0.75	-9.188	-9.8438
C4	3.8	1.5	0.25	1.425	25	35.63	1.9	-0.75	67.688	-26.719
C5	5.6	1.5	0.25	2.1	25	52.5	6.6	-0.75	346.5	-39.375
C6	5.6	1.5	0.25	2.1	25	52.5	12.2	-0.75	640.5	-39.375
C7	3.8	1.5	0.25	1.425	25	35.63	16.9	-0.75	602.06	-26.719

C8	1.5	1.4	0.25	0.525	25	13.13	19.5	-0.75	255.94	-9.8438
C9	4.7	1.4	0.25	1.645	25	41.13	19.5	2.35	801.94	96.644
C10	4.95	1.4	0.25	1.7325	25	43.31	19.5	4.825	844.59	208.98
C11	5.45	1.4	0.25	1.9075	25	47.69	19.5	7.55	929.91	360.04
C12	5.1	1.4	0.25	1.785	25	44.63	19.5	10.1	870.19	450.71
C13	1.4	1.35	0.25	0.4725	25	11.81	19.5	10.775	230.34	127.28
C14	3.8	1.35	0.25	1.2825	25	32.06	16.9	10.775	541.86	345.47
C15	5.6	1.35	0.25	1.89	25	47.25	12.2	10.775	576.45	509.12
C16	5.6	1.35	0.25	1.89	25	47.25	6.6	10.775	311.85	509.12
C17	3.8	1.35	0.25	1.2825	25	32.06	1.9	10.775	60.919	345.47
C18	1.4	1.35	0.25	0.4725	25	11.81	-0.7	10.775	-8.269	127.28
					Total	**2823**			**25831**	**27105**

Table C-2 11 Center of mass calculation for slab on fifth floor

slab number	L(m)	W(m)	D(m)	V(m³)	Υc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	WY
S1	5.6	4.95	0.25	6.93	25	173.25	2.8	7.175	485.1	1243.069
S2	5.6	4.95	0.25	6.93	25	173.25	8.4	7.175	1455.3	1243.069
					TOTAL	**346.5**			**1940.4**	**2486.138**

Table C-2 12 Center of mass calculation for slab on water tank floor

slab number	L(m)	W(m)	D(m)	V(m³)	Υc (KN/m3)	Weight	Xm(m)	Ym(m)	WX	WY
S1	4.95	3.8	0.25	0.25	25	6.25	1.9	7.175	11.875	44.84375
S2	4.95	3.8	0.25	0.25	25	6.25	16.9	7.175	105.625	44.84375
					TOTAL	**12.5**			**117.5**	**89.6875**

Table C-2 13 Center of mass for Wall from first floor to fourth floor along number axis similar for four storey

Designation	L,m	H,m	W,m	V(m³)	Υ_{HCB}	W	Xm,m	Ym,m	WX	WY

1	A-B	1.4	3.15	0.15	0.66	14	9.26	-0.7	1	-6.488	9.26
	B-C	3.8	3.15	0.15	1.79	14	21.28	1.9	1.575	40.432	33.516
	C-D	5.6	3.15	0.15	2.64	14	31.36	6.6	1.575	206.976	49.392
	D-E	5.6	3.15	0.15	2.24	14	31.36	12.2	1.575	382.592	49.392
	E-F	3.8	3.15	0.15	1.79	14	9.26	17.6	1.575	374.528	33.516
	F-G	1.4	3.15	0.15	0.56	14	9.26	21.3	1	166.992	7.84
2	A-B	1.4	3.15	0.15	0.882	14	12.348	-0.7	1	-8.6436	12.348
	B-C	3.8	3.15	0.15	1.79	14	33.516	1.9	1.575	63.6804	52.7877
	C-D	5.6	3.15	0.15	3.528	14	49.392	6.6	1.575	325.9872	77.7924
	D-E	5.6	3.15	0.15	3.528	14	49.392	12.2	1.575	602.5824	77.7924
	E-F	3.8	3.15	0.15	1.79	14	33.516	17.6	1.575	589.8816	52.7877
	F-G	1.4	3.15	0.15	0.882	14	9.26	21.3	1	263.0124	12.348
3	A-B	1.4	3.15	0.15	0.6615	14	9.261	-0.7	1	-6.4827	9.261
	B-C	3.8	3.15	0.15	1.7955	14	25.137	1.9	1.575	47.7603	39.59078
	C-D	5.6	3.15	0.15	2.646	14	37.044	6.6	1.575	244.4904	58.3443
	D-E	5.6	3.15	0.15	2.646	14	37.044	12.2	1.575	451.9368	58.3443
	E-F	3.8	3.15	0.15	1.7955	14	25.137	17.6	1.575	442.4112	39.59078
	F-G	1.4	3.15	0.15	0.6615	14	9.261	21.3	1	197.2593	9.261
4	A-B	1.4	3.15	0.15	0.6615	14	9.261	-0.7	1	-6.4827	9.261
	B-C	3.8	3.15	0.15	1.7955	14	25.137	1.9	1.575	47.7603	39.59078
	C-D	5.6	3.15	0.15	2.646	14	37.044	6.6	1.575	244.4904	58.3443
	D-E	5.6	3.15	0.15	2.646	14	37.044	12.2	1.575	451.9368	58.3443
	E-F	3.8	3.15	0.15	1.7955	14	25.137	17.6	1.575	442.4112	39.59078
	F-G	1.4	3.15	0.15	0.6615	14	9.261	21.3	1	197.2593	9.261
5	A-B	1.4	3.15	0.15	0.6615	14	9.261	-0.7	1	-6.4827	9.261
	B-C	3.8	3.15	0.15	1.7955	14	25.137	1.9	1.575	47.7603	39.59078
	C-D	5.6	3.15	0.15	2.646	14	37.044	6.6	1.575	244.4904	58.3443
	D-E	5.6	3.15	0.15	2.646	14	37.044	12.2	1.575	451.9368	58.3443
	E-F	3.8	3.15	0.15	1.7955	14	25.137	17.6	1.575	442.4112	39.59078
	F-G	1.4	3.15	0.15	0.6615	14	9.261	21.3	1	197.2593	9.261
6	A-B	1.4	3.15		0.882	14	12.348	-0.7	1	-8.6436	12.348
	B-C	3.8	3.15	0.15	1.79	14	33.516	1.9	1.575	63.6804	47.49
	C-D	5.6	3.15	0.15	2.64	14	49.392	6.6	1.575	325.9872	58.34
	D-E	5.6	3.15	0.15	2.64	14	49.392	12.2	1.575	602.5824	58.34
	E-F	3.8	3.15	0.15	1.79	14	33.516	17.6	1.575	589.8816	47.49

Table C-2 14 Center of mass for Wall from first floor to fourth floor along letter axis similar for all four storey

Designation		L,m	H,m	W,m	V(m³)	ϒHCB	W	Xm,m	Ym, m	WX	WY
A	1-2	1.5	2	0.2	0.6	14	8.4	0.75	1	6.3	8.4
	2-3	4.7	2	0.2	1.88	14	26.32	3.85	1.575	101.332	41.454
	3-4	4.95	2	0.2	1.98	14	27.72	8.675	1.575	240.471	43.659
	4-5	5.45	2	0.2	2.18	14	30.52	13.88	1.575	423.465	48.069
	5-6	5.1	2	0.2	2.04	14	28.56	19.15	1.575	546.924	44.982
	6-7	1.35	2	0.2	0.54	14	7.56	22.38	1	169.155	7.56
B	1-2	1.5	2	0.2	0.6	14	8.4	0.75	1	6.3	8.4
	2-3	4.7	3.15	0.2	2.961	14	41.454	3.85	1.575	159.5979	65.29005
	3-4	4.95	3.15	0.2	3.1185	14	43.659	8.675	1.575	378.74183	68.76293
	4-5	5.45	3.15	0.2	3.4335	14	48.069	13.88	1.575	666.95738	75.70868
	5-6	5.1	3.15	0.2	3.213	14	44.982	19.15	1.575	861.4053	70.84665
	6-7	1.35	3.15	0.2	0.8505	14	11.907	22.38	1	266.41913	11.907
C	1-2	1.5	3.15	0.15	0.70875	14	9.9225	0.75	1	7.441875	9.9225
	2-3	4.7	3.15	0.15	2.22075	14	31.0905	3.85	1.575	119.69843	48.96754
	3-4	4.95	3.15	0.15	2.33888	14	32.74425	8.675	1.575	284.05637	51.57219
	4-5	5.45	3.15	0.15	2.57513	14	36.05175	13.88	1.575	500.21803	56.78151
	5-6	5.1	3.15	0.15	2.40975	14	33.7365	19.15	1.575	646.05398	53.13499
	6-7	1.35	3.15	0.15	0.63788	14	8.93025	22.38	1	199.81434	8.93025
D	1-2	1.5	3.15	0.15	0.70875	14	9.9225	0.75	1	7.441875	9.9225
	2-3	4.7	3.15	0.15	2.22075	14	31.0905	3.85	1.575	119.69843	48.96754
	3-4	4.95	3.15	0.15	2.33888	14	32.74425	8.675	1.575	284.05637	51.57219
	4-5	5.45	3.15	0.15	2.57513	14	36.05175	13.875	1.575	500.21803	56.78151
	5-6	5.1	3.15	0.15	2.40975	14	33.7365	19.15	1.575	646.05398	53.13499
	6-7	1.35	3.15	0.15	0.63788	14	8.93025	22.38	1	199.81434	8.93025

E	1-2	1.5	3.15	0.15	0.70875	14	9.9225	0.75	1	7.441875	9.9225
	2-3	4.7	3.15	0.15	2.22075	14	31.0905	3.85	1.575	119.6983	48.96754
	3-4	4.95	3.15	0.15	2.33888	14	32.74425	8.675	1.575	284.0563 7	51.57219
	4-5	5.45	3.15	0.15	2.57513	14	36.05175	13.87 5	1.575	500.2180 3	56.78151
	5-6	5.1	3.15	0.15	2.40975	14	33.7365	19.15	1.575	646.0539 8	53.13499
	6-7	1.35	3.15	0.15	0.63788	14	8.93025	22.37 5	1	199.8143 4	8.93025
F	1-2	1.5	3.15	0.2	0.945	14	13.23	0.75	1	9.9225	13.23
	2-3	4.7	3.15	0.2	2.961	14	41.454	3.85	1.575	159.5979	65.29005
	3-4	4.95	3.15	0.2	3.1185	14	43.659	8.675	1.575	378.7418 3	68.76293
	4-5	5.45	3.15	0.2	3.4335	14	48.069	13.87 5	1.575	666.9573 8	75.70868
	5-6	5.1	3.15	0.2	3.213	14	44.982	19.15	1.575	861.4053	70.84665
	6-7	1.35	3.15	0.2	0.8505	14	11.907	22.37 5	1	266.4191 3	11.907
G	1-2	1.5	2	0.2	0.6	14	8.4	0.75	1	6.3	8.4
	2-3	4.7	2	0.2	1.88	14	26.32	3.85	1.575	101.332	41.454
	3-4	4.95	2	0.2	1.98	14	27.72	8.675	1.575	240.471	43.659
	4-5	5.45	2	0.2	2.18	14	30.52	13.87 5	1.575	423.465	48.069
	5-6	5.1	2	0.2	2.04	14	28.56	19.15	1.575	546.924	44.982
	6-7	1.35	2	0.2	0.54	14	7.56	22.37 5	1	169.155	7.56
						TOTAL	**988.2793**			**11441.962**	**1488.712**

Table C-2 15 Center of mass for Wall of fifth floor along both axis

designation		L,m	H,m	W,m	V(m³)	Υ_{HCB}	W	Xm,m	Ym,m	WX	WY
3	C-D	5.6	3	0.15	2.52	14	35.28	6.6	1.5	52.92	52.92
	D-E	5.6	3	0.15	2.52	14	35.28	12.2	1.5	430.42	52.92
4	C-D	5.6	3	0.15	2.52	14	35.28	6.6	1.5	232.85	52.92
	D-E	5.6	3	0.15	2.52	14	35.28	12.2	1.5	430.42	52.92
D	3-4	4.95	3	0.15	2.2275	14	31.185	9.4	1.5	293.14	46.78
						TOTAL	**172.305**			**1439.74**	**258.46**