STRUCTURAL DESIGN

CHAPTER ONE

1. INTRODUCTION

Now a day's construction industry plays a great role for the development of a nation in all aspects. As we all agree, behind every construction activity there must have a structural analysis and design, from this consideration directly or indirectly structural analysis and design have a huge application in the development of a nation. A structural design is executed in such a way that the building will remain fit with appropriate degrees of reliability and in an economic way. It should sustain all the actions and influences during execution and use. Therefore, structural design focuses on structural safety and serviceability with due durability. It must also optimize the cost expended in building the structure and maintenance. The project will contain detail structural analysis of the building, structural design, detail working structural drawings & statistical report.

This project deals about the structural analysis and design of a G+6 urban building considering all the external effects according to EBCS, 1995. It has six chapters. The contents and duties accomplished in each chapter are explained below.

The **first** chapter deals with the introduction part of this project. The introduction part includes: objectives of the project, scope, specification &code, and general design data& material properties.

The **second chapter** deals about the wind load analysis and design on roofs and roof slabs. The external wind pressure coming from different directions were collected and transferred to frames according to EBCS, 1995.We divided the roof of the building into three parts(Duo-pitched, Mono-pitched and roof slab) and each of its truss members made of steel were designed to resisting axial forces.

The **third** chapter focuses on the analysis and design of slabs and staircases. Most of the slabs are ribbed slabs and some solid slabs. The depths of all the ribbed slabs are made the same for construction simplicity and reinforcement of each is determined using EBCS2, 1995.

The **Fourth** chapter is about the calculation of lateral forces particularly earthquakes loads.

The weight of the building was computed by considering all elements from small to large. The center of mass and center of stiffness were computed by assuming preliminary sections. The lateral forces were distributed to each floor and subsequently to frame joints according to their stiffness.

The **fifth** chapter deals on the analysis and design of the frame of the structures and shear wall design. It was analyzed using **ETABS** as space frame taking combinations for the existing lateral and vertical loads. Therefore, the beams and columns were designed using the loads obtained from analysis by taking the worst effect.

Finally, the last chapter focuses on foundation design of the structure. After calculating the bearing capacity of the soil an isolated square footing is considered by taking two worst load combinations to support and safely distribute all the actions coming from the super structure.

1.1 OBJECTIVES

The prime objective of structural design is structural safety and serviceability. In case the structure fails, it must be in such a way it will minimize risks and casualty.
The specific objectives of this part of our senior project are:

- ✓ To integrate different disciplines such as Urban Engineering, Architecture, Civil engineering, Construction technology & management & likes.
- ✓ To get basic structural design knowledge & minimize the cost of building through structural design.
- ✓ To relate theoretical knowledge with the actual site condition
- ✓ To develop the skills of applying different architectural and structural soft wares

- ✓To Use the courses that we took for required purpose
- ✓To achieve an acceptable probability that structures being designed will perform satisfactorily during their intended life.

1.2 Scope of the project

The scope of the project is to design a G+6 of urban building typology. It includes the activities like: roof design, slab design, stair design, beam design, column design, shear wall design & footing design. And also includes the details of each designs.

1.3 Specification and code

This structural design of our project is executed based on the Ethiopian Building Code of Standard (EBCS) prepared in 1995 E.C. This code follows the Limit State design approach. Limit state is a state beyond which the structure no longer satisfies the design performance requirements. It consists of two states namely Ultimate Limit and serviceability Limit states. Ultimate Limit states are conditions related with collapse or states prior to structural failure. Its main concern is the safety of structure and people. Serviceability Limit states are those associated to conditions beyond which a structure does not accomplish specified service requirements. It is mainly concerned about the function of construction works, comfort of people, and appearance.

The aim of design is to achieve acceptable probabilities that the structure will not became unfit for the use for which it is intended, that is, it will not reach a limit state

1.4 General design data and material properties

Purpose: Residential urban building G+6 Building in reinforced concrete ribbed slab.

Location: Addis Ababa, Zone 2

Material

Concrete C – 25 and Steel S – 300

Steel RHS for roof truss and purloin

EGA-500 for roof cover is used

Loading

Since the site is located in Addis Ababa, which is in the area of seismic zone **2**, according to EBCS-8, 1995 in addition to vertical loading Earthquake and wind loading was considered. The combination is done based on EBCS-2; 1995 section 3.6.

Combo 1=1.3DL+1.6LL

Combo 2, 3=0.75(1.3DL+1.6LL)$\pm$ EQ_x

Combo 4, 5=0.75(1.3DL+1.6LL)$\pm$ EQ_y

Combo 6, 7=0.75(1.3DL+1.6LL)$\pm$ EQ_x $\pm$5%Eccentricity in y direction

Combo 8, 9=0.75(1.3DL+1.6LL)$\pm$ EQ_y $\pm$ 5%Eccentricity in x direction

Combo 10, 11 =0.75(1.3DL+1.6LL) $\pm$ wind load $\pm$5%Eccentricity

Combo 12 =Envelope

Codes and References

EBCS -1995 and Euro Code 2-1992 (as used by the software), almost similar to EBCS-2-1995

Partial safety factors – concrete γ_c=1.5(ordinary loading) [EBCS-2, 1995 table 3.1]

Steel γ_s=1.15

Unit weight of concrete γ_c=25KN/m3

Supporting ground condition = allowable bearing capacity of 200KPa

Design constants

F_c = 25Mpa

$f_{ck}=f_c/1.25=25/1.25=20$ Mpa

f_{yk} =300Mpa

γ_c (partial safety factor for concrete)= 1.5

γ_s (partial safety factor for steel)=1.15

$f_{cd} = 0.8*f_{ck}/\gamma_c = 0.8*20/1.5=11.33$Mpa

$fyd=f_{yk}/\gamma_s=300/1.15=260.87$Mpa

$m=f_{yd}/0.8*f_{cd}=260.87/0.8*11.33= 28.77$

$C_1= 2.5/m=2.5/28.77= 0.087$

$C_2 =0.32*f_{cd}*m^2 =0.32*11.33*28.77^2 = 3002.34$

$$\rho b = \frac{0.8\varepsilon c * fcd}{(\varepsilon c + \varepsilon s) * fyd}$$

E_s (modulus elasticity of steel) =200Gpa=200000Mpa

ε_c (Strain of concrete)=0.0035

$\varepsilon_s = f_{yd}/Es=260.87/200000= 0.0013$

$$\rho b = \frac{0.8\varepsilon c * fcd}{(\varepsilon c + \varepsilon s) * fyd} = \frac{0.8 * 0.0035 * 11.33}{(0.0035 + 0.0013) * 260.87} = 0.025$$

$\rho_{max}=0.75*\rho_b=0.75*0.025= 0.0189$

$\rho_{min=}0.5/f_{yk}=0.5/300= 0.0017$

$K_1=1.6-d \geq 1$ $K_1=1.6-0.178=1.422>1$ OK!

K_2= 1+50ρmin≤2 K_2= 1+50*0.0017=1.083<2 OK!

$$fctd = \frac{0.21 * fck^{2/3}}{\gamma_c} = \frac{0.21 * 20^{2/3}}{1.5} = 1.03$$

Design loads

Pd= γ_f*F_k

Where F_k = characteristics loads

γ_f = partial safety factor for loads

= 1.3 for dead loads

= 1.6 for live loads [EBCS-2, 1995 table 3.3]

Non structural material

HCB wall(ϒh)	14 KN/m³	
terrazo tile	23 KN/m³	
mortar(plastering)	23 KN/m³	[EBCS-1, 1995 table 2.1 & 2
ceramic tile	27 KN/m³	
PVC	16 KN/m³	

CHAPTER TWO

2. Wind Load Analysis and Design

2.1 Roof Analysis and Design

Wind is a moving air which in turn possesses energy and this kinetic energy should be resisted by using appropriate deign for different kinds of structural elements like roofs ,walls. The action of wind can be of the type of suction or pressure to our structures both externally or internally. However these effects are more magnified for structures with more openings and large surface areas. Therefore our focus is on the most sensitive part of the building that is the roof.

2.1.1 Method of Analysis

Even though there are two methods for wind load analysis ,namely Quasi static method and dynamic analysis we prefer Quasi static since our structure is assumed to be less susceptible to dynamic excitation and from EBCS-1,1995 section 3.9.3 a building which satisfies the criterion:

(ForC_d <1.2and building height less than 200m) can be analyzed using quasi static method of analysis

2.1.1 Design Information

For our case the building variables are:

- Height of the building =22.5m
- Width of the building= 20m
- Coefficient of dynamic=0.98 ……….(from fig 3.7,Ebcs-1,1995)
- Building is going to be built in seismic region-2. From EBCS -8,1995,table 1.3 we select Addis Ababa

According to EBCS -1 table 3.2 Addis Ababa is categorized as **category-2**

Category-2 is classified as farmland with boundary hedges, occasional small farm structure, houses or trees. The variables for terrain category-2 are:

- Terrain factor K_T=0.19
- Minimum height (Z_{min}) =4m

- Roughness length (Z_o) =0.05

Elevation of Addis Ababa is between 2000 and 3000 from which its air density (ρ) =0.94Kg/m^3Table 3.1, EBCS-1, 1995)

The types of roofs we used are:-

1. Duo-pitched
2. Mono-pitched
3. Roof slab

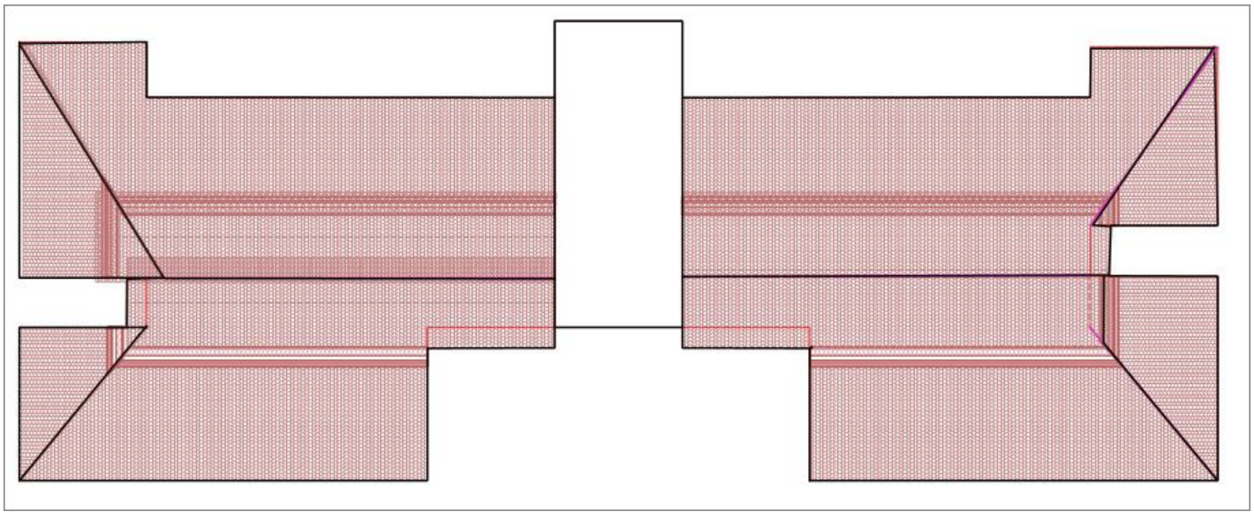


Fig.1 roof lay out

Analysis and Design of Duo-Pitched Roof

Wind action acts on the roof in two directions

- Wind perpendicular to the ridge (ø=0^0)
- Wind parallel to the ridge (ø=90^0)

A) Wind Perpendicular to the Ridge (ø=0^0)

There are two types of wind pressures acting on the roof

- External wind pressure
- Internal wind pressure

I. External Wind Pressure

The external wind pressure that acts on the roof is obtained using the following formula

$$w_e = q_{ref} c_e(z_e) c_{pe}$$

Where:-

- w_e:- External wind pressure
- q_{ref}:- Reference mean wind pressure
- $c_e(z_e)$:- Exposure coefficient that takes into account the influence of terrain roughness
- c_{pe}:- External wind pressure coefficient

$$q_{ref} = \frac{\rho}{2} v_{ref}^2$$

Where:-

- ρ: - Air density which is a function of altitude. For a site located at an altitude greater than 2000m above sea level $\rho = 0.94 Kg/m^3$
- v_{ref}:- Reference wind velocity

$$v_{ref} = c_{DIR} c_{TEM} c_{ALT} v_{ref,0}$$

Where:-

- c_{DIR}:- Is the direction factor to be taken as 1.0
- c_{TEM}:-Is the temporary (seasonal) factor to be taken as 1.0
- c_{ALT}:-Is the altitude factor as 1.0
- $v_{ref,0}$:- The basic volume of the wind velocity to be taken as 22m/s

  $v_{ref} = 1 \times 1 \times 1 \times 22\, m/s$

  $v_{ref} = 22\, m/s$

$$q_{ref} = \frac{\rho}{2} v_{ref}^2$$

$q_{ref} = \frac{0.94}{2} 22^2$, $q_{ref} = 227.48\, N/m^2$

Pressure Coefficients

According to EBCS-1 figure A.6. , the pressure coefficients for duo-pitched are given as follow:-

- Wind direction perpendicular to the ridge ($ø=0^0$)

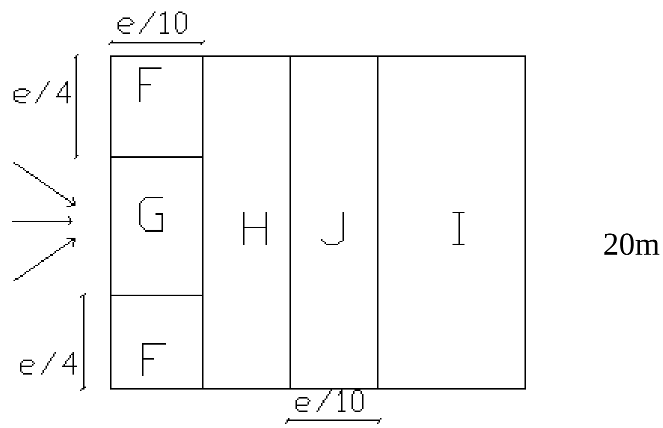


Figure 1 Perpendicular Wind Direction

$c_e(z) =$ - L $c_r(Z)c_t(Z)]$

Where:-

- $c_e(z)$:- is the exposure coefficient as defined in section 3.8.5 of EBCS - 1
- $c_r(z)$:- is the roughness coefficient as defined in section 3.8.3 of EBCS-1
- $c_t(z)$:- is the topographic coefficient as defined in section 3.8.4 of EBCS-1

For our building no escarpments or hills are located around and therefore $c_t(z)$ =1.00. The roughness coefficient at a height Z is defined by

$$c_r(z) = K_T l_n(z/z_0), \text{ For } z_{min} \leq z \leq 200m$$

$$c_r(z) = c_r(z_{min}), \text{For } z \leq z_{min}$$

From EBCS-1, table 3.2,

$K_T = 0.19$, $Z_O = 0.05$, $Zmin_{.} = 4m$

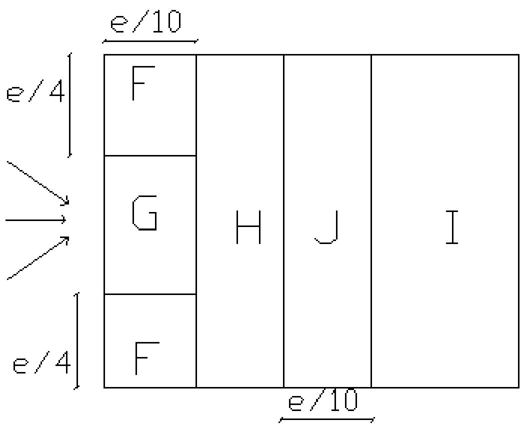
Since Z is 22.5m>Zmin =4m , $c_r(z)$ is given by:-

$$C_{r(z)} = K_T * \ln(Z/Z_o) = \mathbf{1.16}$$

Therefore,

$C_e(z) = C_r^2(z)C_t^2(z)[1+ 7K_T/C_{r(z)}C_{t(z)}]$ =**2.89**

- **External Pressure Coefficient**



2000cm

Figure 2 External Pressure Coefficient

e =min (b or 2h),
=min (20 or 2*22.5) =**20m**

Zones	Area of each zone (m^2)	Remark
F	2*5 = 10	
G	2*10 = 20	
H	6.5*20 = 130	
J	2*20 = 40	
I	6.5*20 = 130	

Table 1 Zone Area for Duo-Pitched Roof

From EBCS -1 table A-4 two cases where the roof is subjected to wind actions

- **Case -1**

When F, G, H subjected to **suction** according to EBCS -1 table A.4

The $c_{pe,1}$ and $c_{pe,10}$ values for **15**^{0} inclinations are:

Zones										
Pitch Angle	**F**		**G**		**H**		**I**		**J**	
15^0	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$
	-2	-0.9	-1.5	-0.8	-0.3	-0.3	-0.4	-0.4	-1.5	-1

Table 1 External Pressure Coefficients (Duo-Pitched) EBCS – 1, 1995

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

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

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

Zones	Area (m^2)	c_{pe}
F	10	-0.9
G	20	-0.8
H	130	-0.3
I	130	-0.4
J	40	-1

Table 2 External Pressure Coefficients Used

External Wind Pressure (w_e)

$$w_e = q_{ref} c_e(z_e) c_{pe}$$

$$= 227.48*2.890*c_{pe}$$

$= \underline{\mathbf{0.657}}\ c_{pe}\ kN/m^2$

II. **Internal Wind Pressure**

$$w_i = q_{ref} c_e(z_e) c_{pi}$$

$$= 227.48*2.890*c_{pi}$$

$$= \underline{\mathbf{0.657}}\ c_{pi}\ kN/m^2$$

According to EBCS -1, 1995 A.2.9 for closed building with internal partition and opening windows the extreme values are

- $c_{pi} = 0.8$ …………………………….. (For pressure)
- $c_{pi} = -0.5$ …………………………….. (For suction)

Therefore, $w_{net} = w_e \pm w_i$

- **Case 1**

The unfavorable condition is when the external wind action is pressure and internal is suction

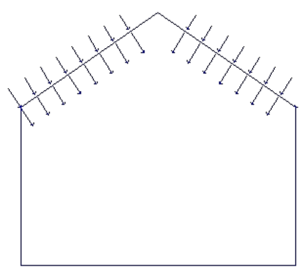


Figure 3 External Pressure and Internal Suction

$w_{net} =$ **0.657***$(c_{pe} - c_{pi})$

	F	G	H	I	J
c_{pe}	0.2	0.2	0.2	-0.4	-1.0
c_{pi}	-0.5	-0.5	-0.5	-0.5	-0.5

$c_{pe} - c_{pi}$	0.7	0.7	0.7	0.1	0.4
$w_{net}(\ kN/m^2)$	0.460	0.460	0.460	0.0657	0.263

Table 3 Wnet for Duo-Pitched Roof, Case 1

w_{net}= **0.460** (kN/m^2)

- **Case 2**

The unfavorable condition is when the external wind action is suction and internal is pressure.

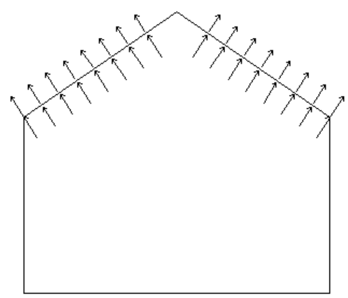


Figure 4 External Suction and Internal Pressure

w_{net}= **0.657** $*(c_{pe} - c_{pi})$

	F	G	H	I	J
c_{pe}	-0.9	-0.8	-0.3	-0.4	-1
c_{pi}	0.8	0.8	0.8	0.8	0.8
$c_{pe} - c_{pi}$	-1.70	-1.60	-1.1	-1.2	-1.8
$w_{net}(\ kN/m^2)$	-1.117	-1.051	-0.723	-0.788	-1.183

Table 4 W_{net} for Duo-Pitched Roof, Case 2

W_{net} = -1.183 KN/m

Therefore, when wind is perpendicular to the ridge ($\theta = 0^0$):

- the largest suction **= -1.183 KN/m**
- the largest pressure **= 0.460 KN/m**

B) Wind Parallel to the Ridge (ø=90^0)

There are also two types of wind pressures acting on the roof:

a) External wind pressure

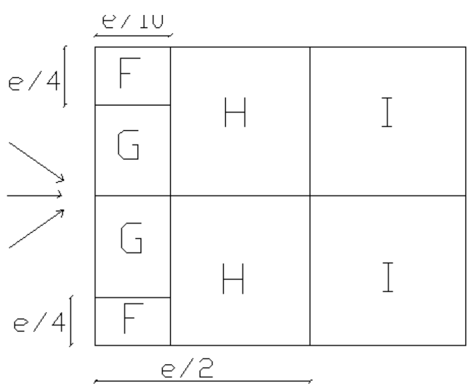
b) Internal wind pressure

a)External wind pressure: The external wind pressure that acts on the roof is obtained as follows:

$w_e = q_{ref} c_e(z_e) c_{pe}$

- **Pressure Coefficients:**

According to EBCS-1 figure A.6. , the pressure coefficients for duo-pitched are given as follow**:**



1700=b

Figure 5 Wind Parallel to the Ridge

$e = \min(b \; or \; 2 \times h)$

= min (b or 2h) = (17 or 2*22.5)

e = 17m

Zones	Area of each zone (m^2)	Remark
F	1.7*4.25 = 7.225	
G	1.7*4.25 = 7.225	
H	6.50*8.50 = 55.25	
I	11.50*8.50 = 97.75	

Table 5 Zone Areas for Wind Parallel to the Ridge

The unfavorable condition is only when the external wind action is **suction**. According to EBCS – 1, table A-4, the $c_{pe,1}$ and$c_{pe,10}$ values for 15^0 inclinations are:-

F		G		H		I	
$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$
-2	-1.3	-2	-1.3	-1.2	-0.6	-0.5	-0.5

Table 6 External Pressure Coefficients for 15^0, EBCS-1, 1995

The external pressure coefficient c_{pe} is given by:

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

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

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

Zones	Area (m^2)	c_{pe}
F	7.225	-1.399
G	7.225	-1.399
H	55.25	-0.6
I	97.75	-0.5

Table 7 External Pressure Coefficients

I. External Wind Pressure (w_e)

$$w_e = q_{ref} c_e(z_e) c_{pe}$$

$$= 227.48*2.890*c_{pe}$$

$$= \mathbf{0.657}\ c_{pe}\ kN/m^2$$

II. Internal Wind Pressure (w_i)

$$w_i = q_{ref} c_e(z_e) c_{pi}$$

$$= 227.48*2.890*c_{pi}$$

$$= \mathbf{0.657}\ c_{pi}\ kN/m^2$$

According to EBCS -1, 1995 A.2.9 for closed building with internal partition and opening windows the extreme values are:

c_{pi} = 0.8 …………………………….. (For pressure)

c_{pi} = -0.5 …………………………….. (For suction

Therefore, $w_{net}=w_e \pm w_i$

The unfavorable condition is when the external wind action is suction and the internal is pressure

	F	**G**	**H**	**I**
c_{pe}	-1.399	-1.399	-0.6	-0.5
c_{pi}	0.8	0.8	0.8	0.8
$c_{pe} - c_{pi}$	-2.199	-2.199	-1.4	-1.3
w_{net}(kN/m^2)	-1.450	-1.450	-0.9198	-0.854

Table 8 W_{net} When the External Wind Action is Suction

$$w_{net} = \mathbf{-1.450}\ (\ kN/m^2)$$

Therefore, for the duo-pitched roof, when wind direction is parallel to the roof ridge (**ø=90^0**) and perpendicular to the roof ridge **($\theta = 0^0$)** the maximum suction and pressure will be:

- maximum suction = **-1.450** kN/m^2
- maximum pressure = **0.460**kN/m^2

2.2 Analysis and Design of Purlin

2.2.1 Design Information

- **Component Material Data**
  - G-28 CIS
  - Weight of G-28 CIS = 0.04KN/m^2
  - Purlin spacing =1.1m
  - From ASTM manual $50 \times 50 \times 3\ RHS$ purlin section is taken

– $b = 50mm$ - $I_x = I_y = 20.5cm^4$

– $h = 50mm$ - $r_x = r_y = 1.91cm$

– $t = 3.0mm$ - $W_{el} = 8.2cm^3$

– $A = 5.6mm^2$ - $W_{pl} = 9.3cm^3$

– $weight = 4.39Kg/m$

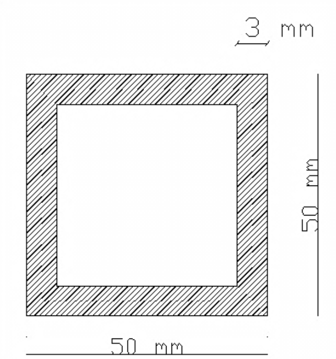


Figure 6 Purlin section

2.2.2 Load Cases

The three significant loads which act on purlin are:

A. Dead load
B. Live load
C. Wind load

A. Dead load

- Dead load self-weight of the purlin
  $= 4.39 \times 9.81 \times 10^{-3}$

$= 0.043KN/m$

- Dead load from G-28 CIS
  $= 0.04 \times 1.1$

  $= 0.044KN/m$

- Applying 50% increase for overlapping and fastening for G-28 CIS
  $= 0.044 + (0.5 \times 0.044)$
  $= 0.066KN/m$

- Total Dead load $= D_{l\ from\,self\,weight} + D_{l\ from\,G-28\,CIS}$
  $= 0.043 + 0.066$

  $= 0.109\ KN/m$

B. Live load: according to EBCS-1, 1995 slopping roofs are under category H. The characteristics values of Q_k and q_k are given in table 2.13 and 2.14 in EBCS -1, 1995.

- The distributed live load $q_k = 0.25kN/m^2$
- The concentrated live load $Q_k = 1.00kN$

Therefore the uniformly distributed and concentrated live loads on our roof truss are:

- Uniformly distributed live load
  $= q_k \times c/c\ distance\ of\ successive\ purlin$
  $= 0.25 \times (\frac{1.1 + 1.1}{2})$
  $= 0.275KN/m$

- Concentrated live load
  $Q_k = 1\ KN$

C. Wind load

There are two critical wind loads

- $W_{net\ (suction)} = -1.645 \times 1.1$
  $= -1.8095KN/m$

- $W_{net\ (pressure)} = 0.376 \times 1.1$
  $= 0.4136\ KN/m$

Note that the wind load acts perpendicular to the rafter, however the LL and DL acts at angle $\theta = 15^0$ for Duo-pitched. Therefore DL and LL are resolved into parallel and perpendicular to the rafter.

2.2.2.1 Duo-Pitched Roof

Loads parallel to the rafter

- Dead load $= 0.109 \times \sin 15^0$ = **0.03KN/m**
- Live load
  - Uniformly distributed live load
    $= 0.275 \times \sin 15^0$ = **0.0712KN/m**
  - Concentrated live load
    $= 1 \times \sin 15^0$ = **0.2588KN**

Loads perpendicular to the rafter

- Dead load $= 0.109 \times \cos 15^0$ = **0.1053KN**/m
- Live load
  - Uniformly distributed live load
    $= 0.275 \times \cos 15^0$ = **0.266KN/m**
  - Concentrated live load
    $= 1 \times \cos 15^0$ = **0.966KN**

2.3 Load Combination

According to EBCS-1, 1995 section 2.3

I. Dead load + live load

- Load parallel to the rafter

$$P_d = 1.3DL + 1.6LL(UDL)$$

= 1.3(0.03) +1.6(0.0712) KN/m

= **<u>0.153 KN/m</u>**

Or

$$P_d = 1.3DL + 1.6LL(concentrated)$$

= 1.3(0.03) KN/m + 1.6(0.2588) KN

= **<u>0.039 KN/m + 0.414 KN</u>**

- Loads perpendicular to the rafter

$$P_d = 1.3DL + 1.6LL(UDL)$$

= 1.3(0.1053) KN/m + 1.6(0.266) KN/m

= **0.563 KN/m**

Or

$P_d = 1.3DL + 1.6LL(concentrated)$

= 1.3(0.1053) KN/m + 1.6(0.966) KN

= 0.137 KN/m + 1.546 KN

II. Dead load + Wind load

- Load parallel to the rafter

$P_d = 0.9DL + 1.3WL$

= 0.9(0.03) KN/m + 1.3(0)

= **0.027 KN/m**

- Loads perpendicular to the rafter

$P_d = 0.9DL + 1.3WL(Suction)$

= 0.9(0.1053KN/m + 1.3(-1.450) KN/m

= -1.980 KN/m

Or

$P_d = 0.9DL + 1.3WL(pressure)$

= 0.9(0.1053KN/m + 1.3(0.460) KN/m

= 0.693 KN/m

III. Dead load + Live load + Wind load

- **Loads parallel to the rafter**

$P_d = 0.8(\ 1.3DL + 1.6LL_{conc} + 1.6WL)$

= 0.8(1.3*0.03 + 1.6*0.2588 + 1.6*0

= 0.0312 KN/m +0.414 KN

Or

$P_d = 0.8(\ 1.3DL + 1.6LL_{UDL} + 1.6WL)$

= 0.8(1.3*0.03 KN/m + 1.6*0.0712KN/m

= 0.122 KN/m

- **Loads perpendicular to the rafter**

$P_d = 0.8(\,1.3DL + 1.6LL_{conc} + 1.6WL_{suction})$

= 0.8(1.3*0.03 KN/m + 1.6*0.2588 KN + 1.6*-1.450 KN/m

= -1.825 KN/m + 0.331 KN

Or

$P_d = 0.8(\,1.3DL + 1.6LL_{conc} + 1.6WL_{pressure})$

= 0.8(1.3*0.03 KN/m + 1.6*0.2588 KN + 1.6*0.460 KN/m

= 0.620 KN/m + 0.414 KN

Or

$P_d = 0.8(\,1.3DL + 1.6LL_{UDL} + 1.6WL_{suction})$

= 0.8(1.3*0.03 KN/m + 1.6*0.266 KN/m + 1.6*-1.450 KN/m

= -1.484 KN/m

Or

$P_d = 0.8(\,1.3DL + 1.6LL_{UDL} + 1.6WL_{pressure})$

= 0.8(1.3*0.03 KN/m + 1.6*0.266 KN/m + 1.6*0.460 KN/m

= 0.968 KN/m

Then, we are going to select the most critical load combination.

- For critical load combination will be:
  - Loads parallel to the rafter

    = 0.039 KN/m + 0.414 KN
  - For perpendicular to the rafter

    = **-1.980 KN/m**

2.4 Determination of Maximum Moment and Shear force

I. Loads Parallel to the Rafter

W = 0.039 KN/m, P = 0.414 KN and L = 2.5m

$$M_{max} = \frac{wl^2}{8} + \frac{pl}{4}$$

$= 0.039*2.5^2/8 + 0.414*2.5/4$

= **0.289 KN.m**

V_{max} = WL/2 +P/2 = (0.039*2.5)/2 + (0.414/2)

= 0.256 KN

II. Loads Perpendicular to the Rafter

W = -1.980 KN/m

$M_{max} = wl^2/8 = (-1.980*2.5^2)/8$

= **-1.55 KN.m**

V_{max} = WL/2 = (-1.98*2.5)/2

= -2.475 KN

2.5 Check for Adequacy of Section

According to EBCS-3, 1995 ordinary **hot rolled steel** with grade of **Fe_{360}** is taken.

Hence, f_y = 235Mpa

F_u= 360Mpa

I. Section classification according to EBCS-3,1995

Depth = h - 3

= 50-3

= 41mm

C= b – 3t

= 50 – 3*3

= 41mm

$$\epsilon = \sqrt{235}/fy \quad = \sqrt{235/235} = 1.0$$

<u>Consider the Web part</u>

d/t = 41/3= 13.67mm < 72*1 = 72

So, the web part is classified as **class-1**

<u>Again consider the Flange</u>

c/t_f = 41/3 =13.67mm < 72*1= 72, flange is **class-1**

Therefore the cross section of the material is classified as **class-<u>I</u>**

II. Flexural resistance

$$M_{pl,rd} = W_{pl} \times f_y /(\gamma_{m1})$$
$$= 9.83 \times 235 \times 10^{-3} /(1.1)$$
$$= 2.1\ KN/m$$

- For load parallel to the rafter

$M_{pl,rd} = 2.1\frac{KN}{m} > M_{max} = (0.289KN.m)$ …………………………..OK!

- For loads perpendicular to the rafter

$M_{pl,rd}$ =2.1$\frac{KN}{m} > M_{max}\,(1.55KN.m)$ …………………………..OK

III. Shear resistance

- **Shear buckling**

  If d/$t_w < 69 \in$ that is

  $=41/3 < 69$

  $=13.667 < 69$

I.e. shear buckling can be ignored

- **Plastic shear resistance**

$V_{pl,rd=}\ A_v \times \frac{f_y}{\sqrt{3}}/\gamma_{mo}$ Where: $A_v = h \times t_w$

$= 50 \times 3 \times 1.092$

$= 163.8mm^2$

$V_{pl,rd=}\ 163.8\ \times 235\sqrt{3}/1.1$

= 20.203KN

But $V_{max} = \sqrt{(V_{II}{}^2 + V_{\perp}{}^2)}$

$= \sqrt{(0.29^2+2.55^2)}$

$=$ 2.56KN

$V_{pl,rd=}20.203 > 2.56$ …………………………………….Ok!

2.6 Analysis and Design of Roof Truss

2.6.1 Duo-Pitched Roof

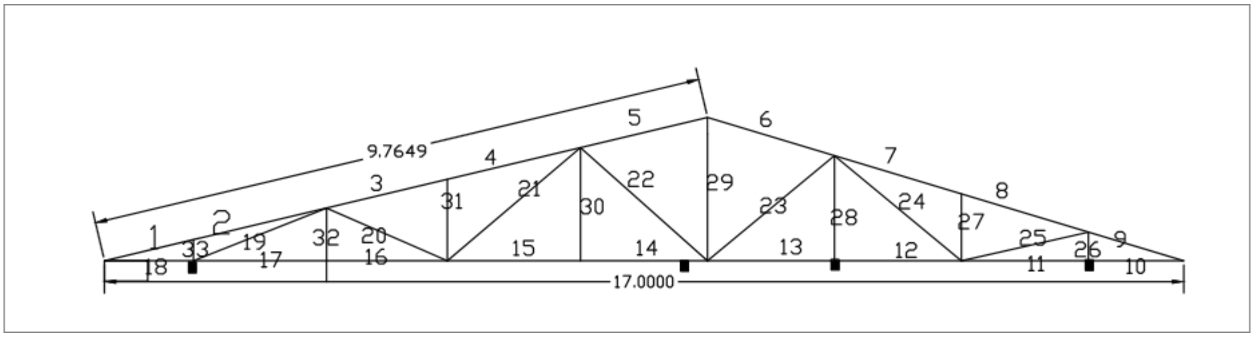


Figure 11 Duo-Pitched Roof Truss

I. Section of Truss Member

From ASTM standard RHS $(60 \times 60 \times 3)$ is chosen for horizontal, vertical and top (rafter) members.

Properties

$A = 6.8cm^2 r = 2.32cm$

$I = 36.6\ cm^4 I_z = 56.9cm^4$

$w_{pl} = 14.5\ cm^3 C_{t=}17.7cm^3$

$w_{el} = 12.2cm^2$ Weight $= 5.34kg/m$

✓ For Diagonal member RHS $(50 \times 50 \times 3)$

$A = 5.6cm^2 r = 1.11cm$

$I = 20.5\ cm^4 I_z = 32cm^4$

$w_{pl} = 9.88\ cm^3 C_{t=}11.8cm^3$

$w_{el} = 8.2cm^2$ Weight $= 4.39kg/m$

II. Loading Cases in Truss

- Dead load from weight of purlin

= 4.39*9.81 = 43.10N/m= **0.0431 KN/m**

- Own weight of truss member (weight of all member)

✓ $weight\ of horizontal + vertical + top\ members$

$$5.34 \times 9.81 \times 29.162 = 1527.66N = 1.52766KN$$

✓ $weight\ of\ diagonal\ members$

$$4.39 \times 11.748 \times 9.81 = 505.938N = 0.505938KN$$

✓ $total\ DL\ from\ own\ weight\ is\ given\ by: -$

$$1.52766 + 0.505938 = 2.033.598KN$$

– DL from weight of CIS

$$0.04\left(\frac{(2.5+2.5)}{2}\right) = 0.1\ KN/m$$

- Live Load

According to EBCS-1, 1995, the distributes live load is given by $0.25\ KN/m^2$
The distributed live load per meter length equals

$$= 0.25\ KN/m^2 \times \left(\frac{(2.5+2.5)}{2}\right) = 0.625\ KN/m$$

- Wind Load
  - Case 1 – positive wind load (Pressure)

$$pressure = 0.3760.25\,KN/m^2 \times \left(\frac{(2.5+2.5)}{2}\right) = 0.94\ KN/m$$

$$horizontal\,pressure = 0.94 \times \sin 15 = 0.243\ KN/m$$

$$vertical\,pressure = 0.94 \times \cos 15 = 0.91\ KN/m$$

  - Case 2 – negative wind load (suction)

$$suction = -1.645 \times \left(\frac{(2.5+2.5)}{2}\right) = -4.11\,KN/m$$

$$horizontal\ suction = -4.11 \times \sin 15 = -1.06\,KN/m$$

$$vertical\ suction = -4.11 \times \cos 15 = -3.969\ KN/m$$

III. Load Combinations for Limit State Design According to EBCS-1, 1995

- Combination 1= 1.3DL + 1.6LL
- Combination 2 = 0.9DL + 1.3$WL_{pressure}$
- Combination 3 = 0.9DL + 1.3$WL_{suction}$
- Combination 4 = 0.8(1.3DL + 1.6LL + 1.6$WL_{pressure}$
- Combination 5 = 0.8(1.3DL + 1.6LL + 1.6$WL_{suction}$

CHAPTER THREE

3. ANALYSIS AND DESIGN OF RIBBED SLAB

Ribbed slab is the slab made of pre cast or caste in situ beam system that is used together with hollow concrete blocks. The pre cast beam is spaced at an interval of 550mm and the toping is a one way slab that is supported on the pre cast beam. These are economical for building where there are long spans and light and moderate live loads such as in hospitals or apartment buildings.

3.1 DETAILING PROVISIONS FOR RIBBED SLABS AS PER EBCS-2, 1995

A) Sizes:

i. Ribs shall not be less than 70mm in width;

ii. The depth of ribs should not exceed four times the minimum width of the rib. Excluding any topping.

iii. The rib spacing shall not exceed 1.0m, unless calculation requires for rib spacing larger than 1m.

iv. Thickness of topping shall not be less than 40mm, or less than 1/10 the clear distance between ribs.

B) Minimum Reinforcement

i. Unless calculation requires, minimum reinforcement to be provided for joist includes two bars, where one in bent near the support and the other straight.

ii. The topping (slab) shall be provided with a reinforcement mesh providing in both directions for temperature and shrinkage problems a cross sectional area not less than 0.001of the section of the rib ($0.001*b*t_f$) or $0.008b*t_f$ at right angle to the joist.

iii. If the rib spacing exceeds 1.0m, the toping shall be designed as a slab resting on ribs, considering load concentrations, if any.

C) Transverse ribs

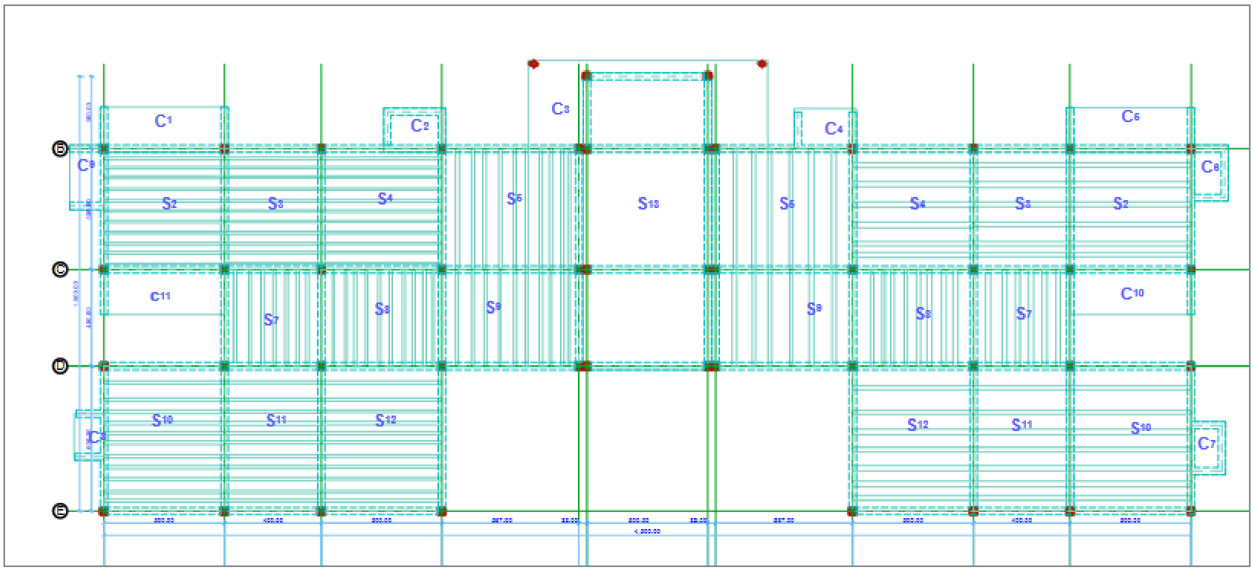
i. Transverse ribs shall be provided if the span of the ribbed slab exceeds 6m.

ii. When transverse ribs are provided, the center to center distance shall not exceed 20times the overall depth of the ribbed slab.

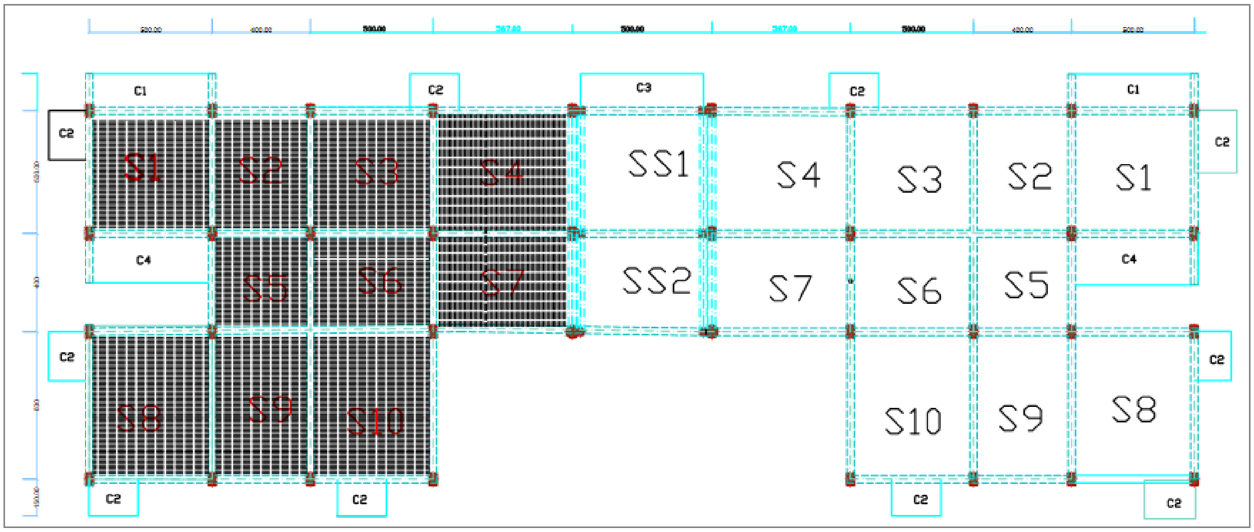
iii. The transverse rib shall be designed for at least half the values of maximum moments and shear force in the longitudinal direction spanning ribs.

Construction method

An advantage of such construction systems is either effectiveness in spanning longer openings and in reducing the dead loads by essentially eliminating concrete in tension in the space between the ribs below the neutral axis. Near the supports the full depth is retained (the slab is made solid) to achieve greater shear strength.



Typical 1-6 Ribbed Slab Floor Plan



Typical 1-6 Ribbed Slab Floor Plan

3.2 General procedures for the design of ribbed slab

Step-1: Determining minimum dimension

1.1 Width of joists and spacing

Assume that b_w= 120mm > 70 mm ………………………OK! And consider 510 mm HCB for filling the void.

- ✓ Spacing = 510mm + 120 mm = 630mm
- ✓ Spacing between joists should be less than 1000mm.
- ✓ Number of joists = Girder length/Spacing

1.2 Minimum depth of serviceability of the joist

$$\mathbf{d \geq [0.4 + 0.6 \frac{\mathit{Fyk}}{400}] \, Le/\beta a}$$

d > 0.85 Le/βa

d - The minimum depth required for deflection

f_{yk} - characteristic strength of the reinforcing bar

L_e - effective length

β_a - factor depend on support condition

[EBCS-2, 1995, Table 5.1]

Member	Simply	End	Interior	Cantilever
(β_a)	20	24	28	10

Joist depth, D_j for each slab

The depth of joists for each slab or panel can be calculated using the formula:

$$d \geq [0.4 + 0.6 \frac{Fyk}{400}] Le/\beta a$$

Effective depth for critical panels

Panel	Width(mm)	Length(mm)	Span type	βa	d_j(mm)	Area
S_1	500	500	End span	24	177.08	25
S_2	500	400	End span	24	141.67	20
S_3	500	500	End span	24	177.08	25
S_4	567	500	End span	24	177.08	28.35
S_5	400	400	Interior	28	121.43	16
S_6	500	400	Interior	28	121.43	20
S_7	567	400	End span	24	141.67	22.68
S_8	600	500	End span	24	177.08	30

S_9	600	400	End span	24	141.67	24
S_{10}	600	500	End span	24	177.08	30
SS_1	500	500	End span	40	106.25	25
SS_2	500	400	End span	37.5	90.70	20
C_1	500	150	Cantilever	10	127.50	7.5
C_2	200	150	Cantilever	10	127.50	3
C_3	200	150	Cantilever	10	127.50	3
C_4	200	150	Cantilever	10	127.50	3
Maximum depth					**177.08**	

From the above table, the governing depth (**d_j**) will be the one with maximum depth. That is:

d_j = 177.08 mm

- ***Consider the reinforcements:***
  - ✓ Ǿ 14mm deformed bar
  - ✓ Ǿ 6mm stirrup
  - ✓ 15mm concrete cover

Since the governing depth (d_j) is = 177.08mm, then

$D_j = d_j +15 + 7 + 6$

$= 177.08 +15 +7 + 6$

$D_j = 205.08$ mm

To be more sure and safe, **D_j = 220 mm**

Again, check for **EBCS-2** requirement

$$D_j < 4b_j$$

$$220mm < 4(120)$$

$$220\ mm < 480\ mm \ldots\ldots\ldots\ldots OK$$

1.3 Toping as per EBCS-2

$$h_t = \min \begin{cases} \text{1/10 of clear distance b/n joist} \\ \text{1/10(510) = 51 mm} \\ \text{40mm} \end{cases}$$

Therefore, h_t =40mm and total depth of our ribbed slab will be

$$D = h_t + D_j$$

$$D = 80mm + 220mm$$

$$\underline{\mathbf{D = 300\ mm}}$$

- Thickness of topping shall not less than 40mm, or not less than $\frac{1}{10}$ of the clear distance between ribs

  Assume thickness of toping is 80mm > 40 mm……………OK!

1.4 Traverse requirements

If the rib span is less than 6m, then we do not need any traverse ribs. Here in our case, each span is less than 6m.

Step -2. LOAD COMPUTATION

Loads may be computed on the bases of rib geometry.

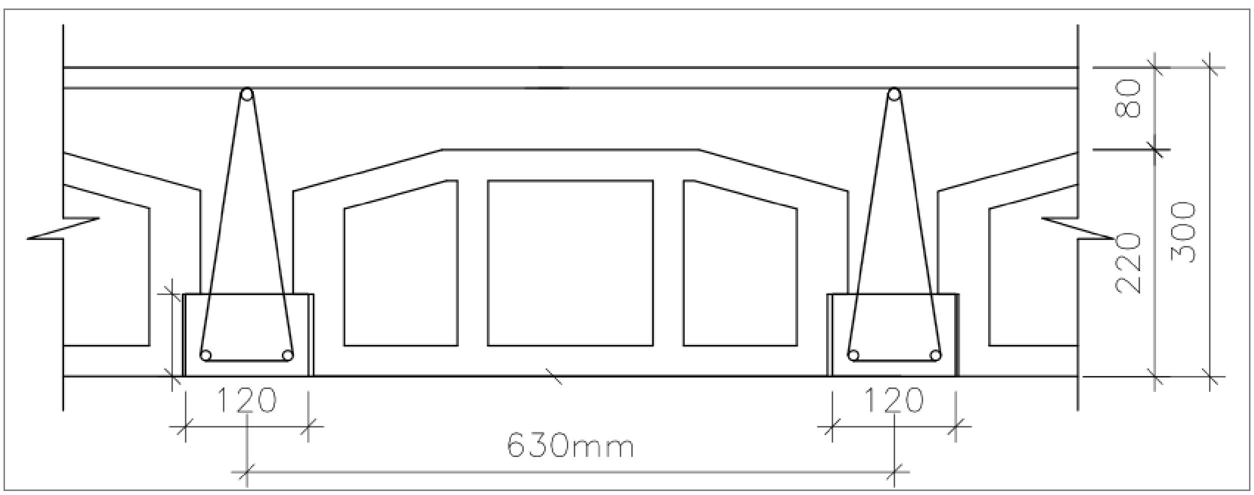
2.1. DESIGN LOAD

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 overall depth time's unit weight of concrete.

2.2. LIVE LOAD

Since the building is residential building we assume the live load to be 2KN/m2 (EBCS-2-1995 sec.2.6.3 table 2.9). The design load is the combination of the live load, the dead load from the partition walls and finishing. That is area for domestic and residential activities are categorized under category-A.

- ✓ Resident, kitchen, toilet and soon = 2 KN/m^2
- ✓Stair case = 3 KN/m
- ✓Balcony = 4 KN/m^2



Dead weight

- 20mm thick Terrazzo tile floor finish ………………= 0.63*0.02*23 = 0.0126KN/m
- 30mm thick cement screed ………………………..= 0.63*0.03x24 = 0.454KN/m
- Hollow concrete block(HCB) ……={ [(0.63*0.3)-[(0.12*0.28)+(0.06*0.63)]}*14= 1.65 KN/m
- Finishing and plastering ...……………………….=(0.63*0.025*23) = 0.362 KN/m

Total self weight = 2.4786 KN/m

Dead load due to part ion walls

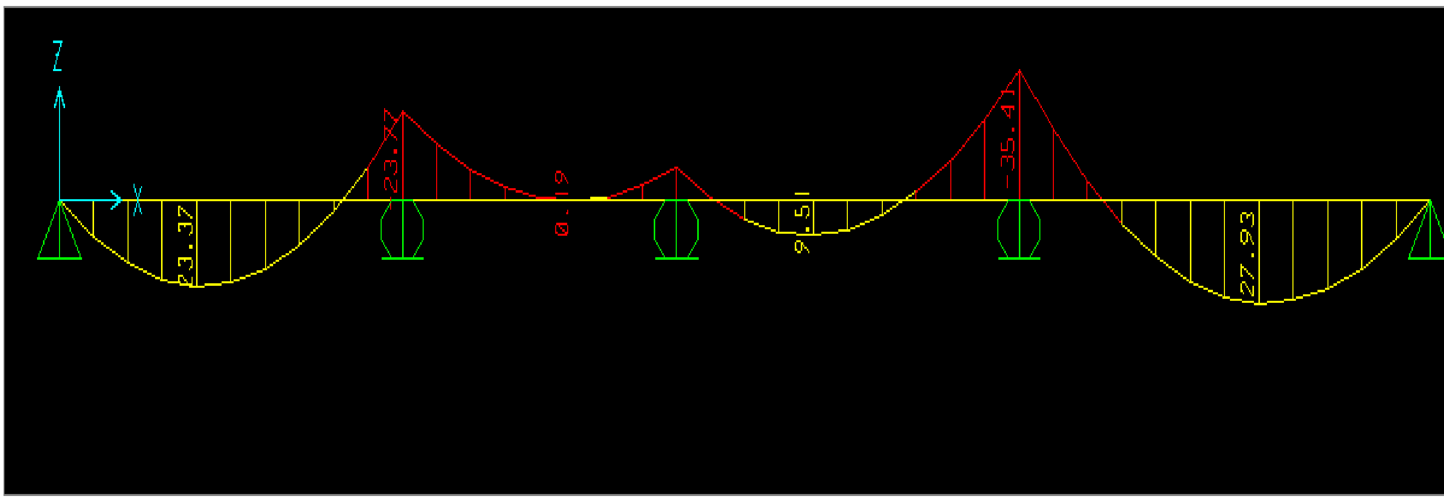
Dead loads can be also added to each panels due to some part ions available.

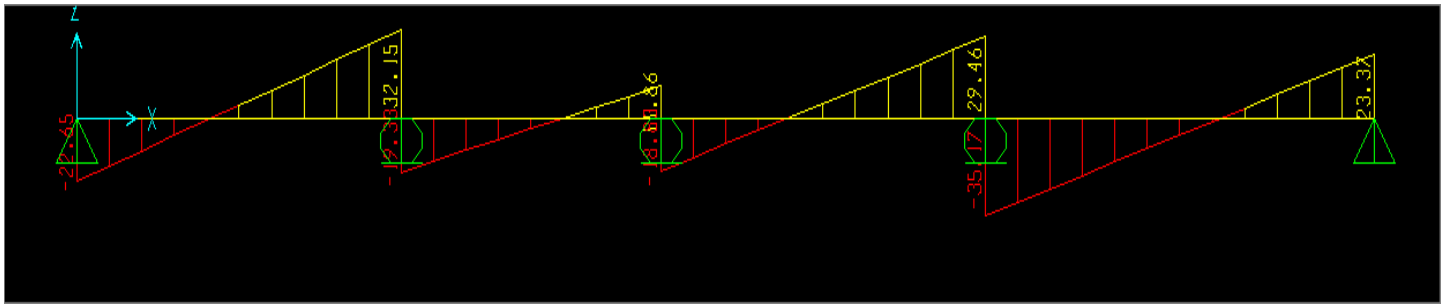
Panels	Live Load(KN/m^2)	Part ion wall load	Self weight	Total dead load	Total design load(factored)
S_1	2*0.63=1.26	1.729	2.4786	4.210	**7.489**
S_2	2*0.63=1.26	0.00	2.4786	2.4786	**5.240**
S_3	2*0.63=1.26	1.022	2.4786	3.500	**6.566**
S_4	2*0.63=1.26	1.075	2.4786	3.554	**6.636**
S_5	2*0.63=1.26	1.272	2.4786	3.7500	**6.891**
S_6	2*0.63=1.26	1.172	2.4786	3.650	**6.761**
S_7	2*0.63=1.26	1.320	2.4786	3.798	**6.953**
S_8	2*0.63=1.26	1.163	2.4786	3.640	**6.748**
S_9	2*0.63=1.26	0.952	2.4786	3.430	**6.475**
S_{10}	2*0.63=1.26	1.614	2.4786	4.090	**7.333**

SS_1	3.00	0.00	3.5986	3.5986	**9.478**
SS_2	3.00	0.00	3.5986	3.5986	**9.480**
C_1	3.00	0.00	3.5986	3.5986	**9.480**
C_2	4.00	0.00	3.5986	3.5986	**11.08**
C_3	3.00	0.00	3.5986	3.5986	**9.480**
C_4	3.00	2.413	3.5986	6.010	**12.620**

✓ **Adjusted Moment and Shear Diagram (SAP- OUTPUT)**

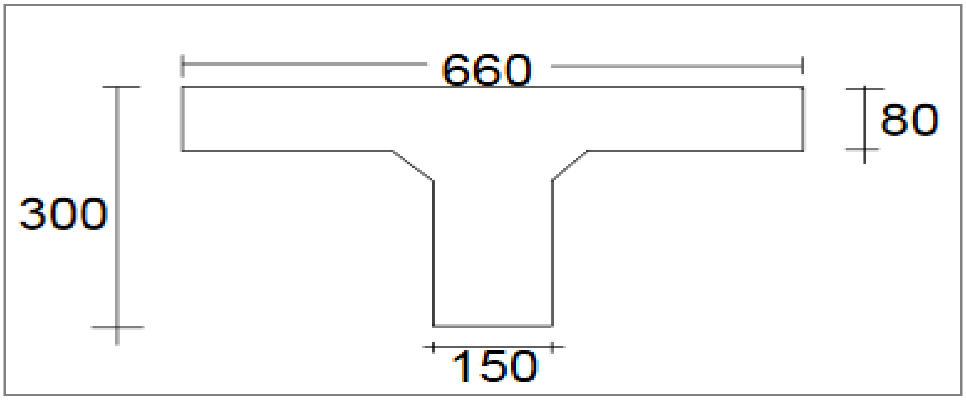
- **Section F-F and panels (S_1, S_2, S_3, & S_4)**





Step-3: **Flexural design of joists**

Generally, Ribs (joists**)** are designed as regular T- beam sections supported by girders.



Typical T- beam section

- ✓ **Dimension:**

  $d = D – 15- dia.14/2 = 300 -15 -7$

  $d = 278mm$

- ✓ **Checking depth of the section**

  d $\geq \sqrt{M/0.2952f_{cd}b_w} = \sqrt{(35.41*10^6)/0.2952*11.33*150}$

  $d \geq 265.70mm$ So, depth is safe which is $d = 278mm$

- ✓ **Checking the moment capacity(Limiting moment)**

  Limiting moment for 15% moment redistribution

  $X/d =0.328$, $x = 0.328d = 0.328*278$

  $X = 91.18mm$

  - **For positive moment**

    $\mathbf{M}_{lim} = 0.8x*f_{cd}*[d – 0.4x]*b_e = 0.8*91.18*11.33*[278 - (0.4*91.18)]*660$

    $M_{lim} =131.74$ KNm

But, the maximum applied moment at the upper section is **35.41KNm** < **M_{lim} =131.74 KNm.** So, the section is safe.

- **For Negative moment**

  $\mathbf{M}_{lim} = 0.8x*f_{cd}*[d – 0.4x]*b_j = 0.8*91.18*11.33*[278 - (0.4*91.18)]*150$

  $\mathbf{M_{lim} = 29.94\ KNm}$

But, again the maximum applied moment at the bottom section is **27.93KNm** < $\mathbf{M_{lim} = 29.94}$ **KNm.** So, the section is safe.

✓ **Checking Neutral Axis position (section F-F)**

Assume the Neutral axis in the topping, $x < h_t = 80mm$

- **For the negative moment($\mathbf{M_{app}}$ = 35.41KNm)**

$M_{app} = 0.8x*b_e*f_{cd}*(d – 0.4x)$, $0.8*x*660*11.33*[278 – (0.4*x)] = 35.41*10^6$

Rearranging this: $x^2 - 695x + 14798 = 0$**,**

x = 22mm. So, the assumption is correct.

✓ **<u>Re-enforcement for joists</u>**

- **For Negative moment(M=23.77KNm @support(F,B)**

  $\rho = \{1 –[1 -2M/(b_e d^2 f_{cd})]^{0.5}\}fcd/f_{yd} =$

  $= \{1 –[1-2*23.77*10^6/(660*278^2*11.33)]^{0.5} 11.33/260.87$

  ρ = 0.0018,   **But,** $\rho_{min} = 0.6/f_{yk} = 0.6/300 = \mathbf{0.002}$

Therefore, we use $\boldsymbol{\rho_{min}} = \mathbf{0.002}$ to calculate the area of re-enforcements

. $A_s = \rho bd = 0.002*150*278$

$\mathbf{= 85mm^2}$

Therefore, Provide **2 Ø10** at the support (F, B)

- **For Negative moment(M=35.41KNm @support(F,D)**

$\rho = \{1 - [1 - 2M/(b_e d^2 f_{cd})]^{0.5}\} fcd/f_{yd}$

$= \{1 - [1 - 2*35.41*10^6/(660*278^2*11.33)]^{0.5} 11.33/260.87$

$\boldsymbol{\rho = 0.003}$, But, $\rho_{min} = 0.6/f_{yk} = 0.6/300 = \mathbf{0.002}$

Therefore, we use **ρ=0.003** to calculate the area of re-enforcements

. $A_s = \rho bd = 0.003*150*278$

$= 126mm^2$

Therefore, Provide **2 Ø12** at the support (F, D)

- **For Positive moment(M=27.93KNm @span**

By considering the maximum moment at span we can design the four panels **(s_1, s_2 s_3 &s_4)** in this section.

$\rho = \{1 - [1 - 2M/(b_e d^2 f_{cd})]^{0.5}\} fcd/f_{yd}$

$= \{1 - [1 - 2*27.93*10^6/(660*278^2*11.33)]^{0.5} 11.33/260.87$

$\boldsymbol{\rho = 0.0022}$, But, $\rho_{min} = 0.6/f_{yk} = 0.6/300 = \mathbf{0.002}$

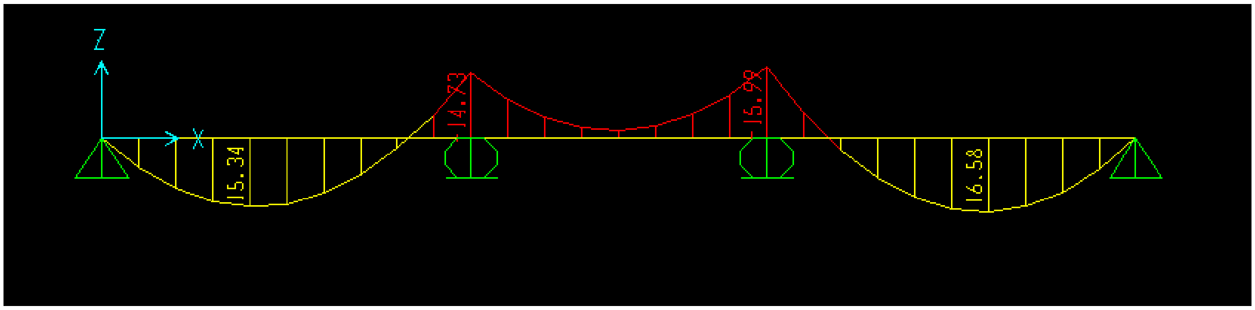
Therefore, we use **ρ=0.0022** to calculate the area of re-enforcements

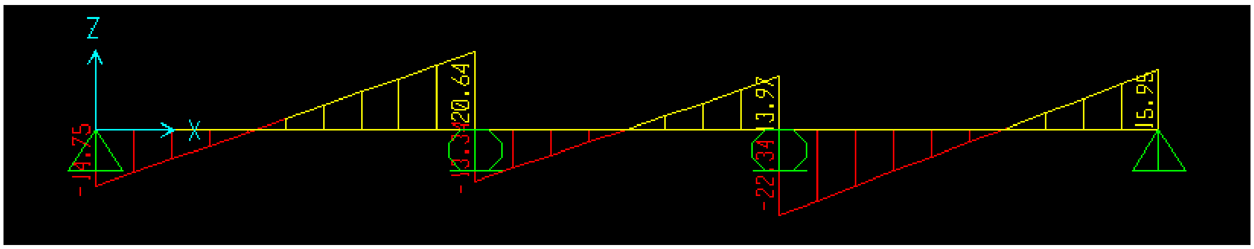
. $A_s = \rho bd = 0.0022*150*278$

$= 92mm^2$

Therefore, Provide **2 Ø10** at each spans)

Section H-H and panels (S_8 ,S_9 ,& S_{10})





Typical T- beam section

- ✓ **Dimension:**

  d = D – 15- dia.14/2 = 300 -15 -7

  d = 278mm

- ✓ **Checking depth of the section**

  d $\geq \sqrt{M/0.2952 f_{cd} b_w} = \sqrt{(16.58*10^6)/0.2952*11.33*150}$

  d ≥ 182mm So, depth is safe which is d = 278mm

- ✓ **Checking the moment capacity(Limiting moment)**

  Limiting moment for 15% moment redistribution

  X/d =0.328,   x = 0.328d = 0.328*278

  X = 91.18mm

- **For positive moment**

  $\mathbf{M}_{lim} = 0.8x*f_{cd}*[d - 0.4x]*b_e = 0.8*91.18*11.33*[278 - (0.4*91.18)]*660$

  $\mathbf{M_{lim} = 131.74\ KNm}$

But, the maximum applied moment at the upper section is **16.58KNm** < $\mathbf{M_{lim}}$ **=131.74 KNm.** So, the section is safe.

- **For Negative moment**

  $\mathbf{M}_{lim} = 0.8x*f_{cd}*[d - 0.4x]*b_j = 0.8*91.18*11.33*[278 - (0.4*91.18)]*150$

  $\mathbf{M_{lim} = 29.94\ KNm}$

But, again the maximum applied moment at the bottom section is **16.58KNm** < $\mathbf{M_{lim}}$ **=29.94 KNm.** So, the section is safe.

✓ **Checking Neutral Axis position (section H-H)**

Assume the Neutral axis in the topping, $x < h_t = 80mm$

- **For the P osetive moment($\mathbf{M_{app}}$ =16.58KNm)**

$M_{app} = 0.8x*b_e*f_{cd}*(d - 0.4x)$, $0.8*x*660*11.33*[278 - (0.4*x)] = 16.58*10^6$

Rearranging this: $x^2 - 695x + 6929 = 0$,

x =12mm. So, the assumption is correct.

✓ Re-enforcement for joists

- **For Negative moment(M=15.99KNm**

  $\rho = \{1 - [1 - 2M/(b_e d^2 f_{cd})]^{0.5}\} fcd/f_{yd} =$

  $= \{1 - [1-2*15.99*10^6/(660*278^2*11.33)]^{0.5} 11.33/260.87$

  ρ = 0.0012, **But,** $\rho_{min} = 0.6/f_{yk} = 0.6/300 = \mathbf{0.002}$

Therefore, we use $\boldsymbol{\rho_{min}}$ **=0.002** to calculate the area of re-enforcements

. $A_s = \rho bd = 0.002*150*278$

$\mathbf{= 85mm^2}$

Therefore, Provide **2 Ø10** at the support.

- **For Positive moment(M=16.58KNm @span**

By considering the maximum moment at span we can design the four panels **(S_8,S_9 & S_{10})** in this section.

$$\rho = \{1 - [1 - 2M/(b_e d^2 f_{cd})]^{0.5}\} fcd/f_{yd}$$

$$= \{1 - [1 - 2*16.58*10^6/(660*278^2*11.33)]^{0.5} 11.33/260.87$$

ρ = 0.0013, But, $\rho_{min} = 0.6/f_{yk} = 0.6/300 =$ **0.002**

Therefore, we use **ρ=0.002** to calculate the area of re-enforcements

. $A_s = \rho bd = 0.002*150*278$

$= 85mm^2$

Therefore, Provide **2 Ø10** at each spans)

3.5 Solid Slab Analysis and Design

3.5.1 Design Procedure

There are two types of slabs based on the load transferring mechanisms. These are one way and two way slabs.

One-way slabs transmit their load in one direction while two way slabs resist applied load in two directions. These types of slabs are composed of rectangular panels supported at all four edges by walls or beams stiff enough to be treated as unyielding. In our case most of the slabs are two way and need to be analyzed based on the principle of two way actions.

Depth for Deflection

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

$$\left(0.4 + 0.6\frac{f_{yk}}{400}\right) \times \frac{L_e}{\beta_a}$$

Where: Le is effective length of the slab

f_{yk} *is the characterstic tensile strength of reinforcement*

β_a *is the appropriate constant from the following table*

Member	Simply Supported	End Span	Interior Span	Cantilevers
Beams	20	24	28	10
Slabs				
Span ratio=2:1	25	30	35	12
Span ratio=1:1	35	40	45	10
Flat slabs(based on longer span)	24			

Table15 values from EBCS

Note that: For slabs with intermediate span ratio, linear interpolation can be used.

Loading

Dead and live loads are calculated depending on the service of the slabs and self-weight. Ignoring any localized effects caused by concentrated load, the partition loads are distributed over the area of the slab. The design loads are factored according to the following formula.

$$P_d = 1.3G_k + 1.6Q_k$$

Where

$P_d = total\ factored\ design\ load$

$G_k = total\ dead\ load\ on\ slab$

$Q_k = total\ live\ load\ on\ slab$

Analysis

The analysis of slab moments of two way slabs is accomplished by the formula:

$$m_i = \alpha_i P_d {L_x}^2$$

Where $m_i = the\ design\ moment\ per\ unit\ width\ at\ the\ point\ of\ reference$

$\alpha_i = the\ coefficient\ given\ in\ Table\ A-1\ in\ EBCS2-1995.$

$P_d = the\ design\ load$

$L_x = the\ length\ of\ the\ shorter\ span\ of\ the\ of\ the\ panel$

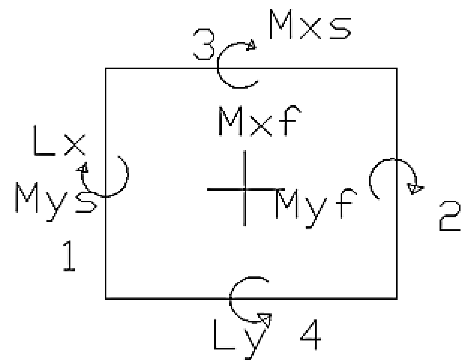
In the following diagram the symbols stand for:

s = support

f = span

x = direction of shorter span

y = direction of longer span



Designation for Slab Moment

Moment Adjustments

- **Support adjustment**

For a continuous support there will be two supports which are different in magnitude. These moments are usually different in magnitude and must be adjusted to come up with one design moment. Therefore, the difference is distributed on either side of the support to equalize the different moments.

There are two cases: -

A. If $\Delta M < 20\%$ of the larger moment, the design moment is the average of the two or the larger moment.
B. If $\Delta M \geq 20\%$, the unbalanced moment is distributed based on the stiffness without any carry over.

Let: $M_L = moment\ in\ the\ left$

$M_R = moment\ in\ the\ right$

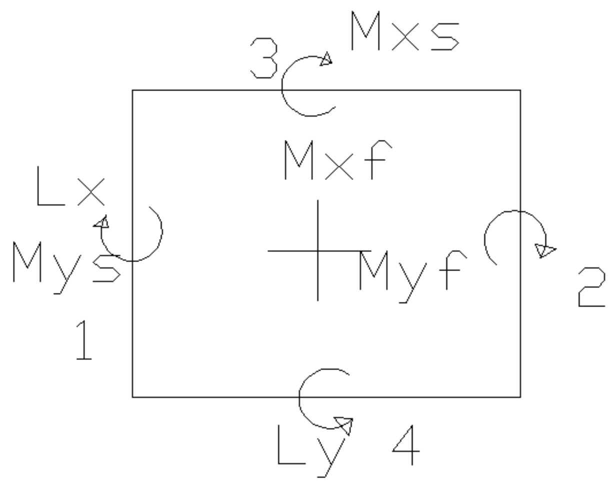
$K = the\ stiffness\ of\ the\ slab$

Therefore the design moment,M_d cab be calculated in eitherof the following formula

$$M_d = M_R - \frac{K_R}{K_R + K_L} \times \Delta M, taking\ the\ right, or$$

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

$$where\ K = \frac{I}{L_x}$$



Span adjustment

If the moment in the adjusted support decreases, the span moment are increased to compensate for the changes in the support moments. The design moments for the spans are calculated.

$$M_{x,rev} = M_{x,feild} + C_x \times \Delta M$$

$$M_{y,rev} = M_{y,feild} + C_y \times \Delta M$$

Where: - $\Delta M = the\ change\ in\ moment\ in\ all\ supports.$

$C_x and C_y are\ coefficients\ for\ adjusting\ span\ moments\ in\ EBCS - 2, Table\ A5.$

Load transfer to frames

Finally loads are transferred to beams as shear. The shear is calculated using the formula (EBCS-2, 1995).

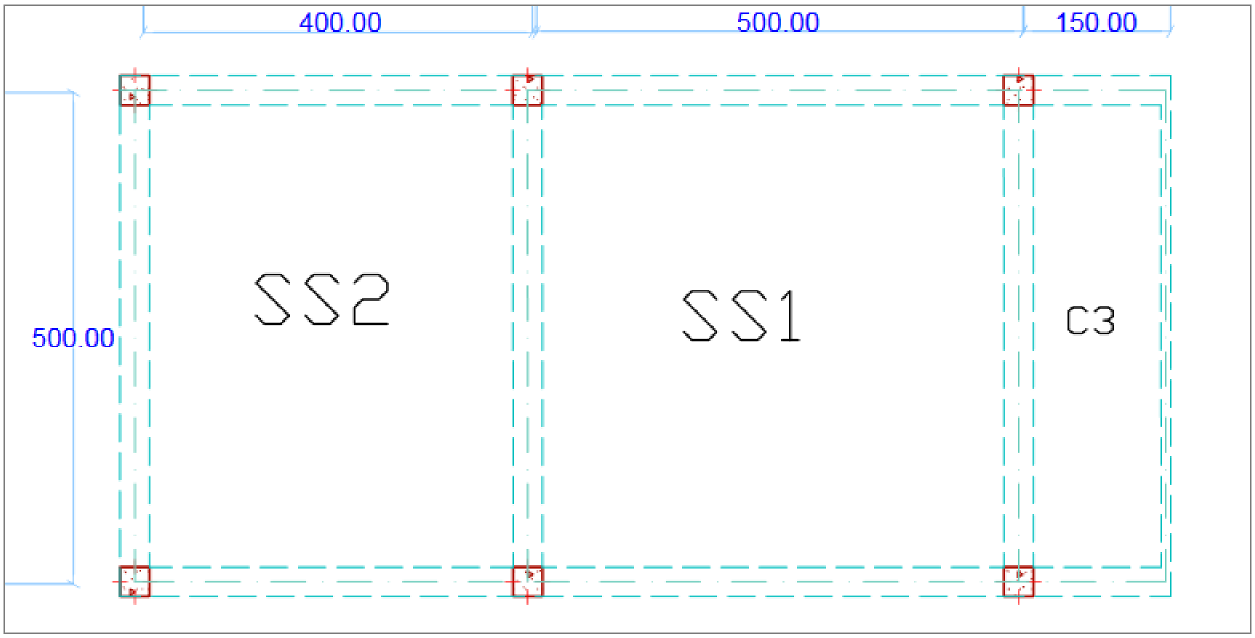
$$V_x = \beta_{vx} \times P_d \times L_x$$

$$V_y = \beta_{vy} \times P_d \times L_x$$

The load transfer coefficients are read from EBCS-2, 1995 of Table A-3.

The design load on a beam determined in the above may be taken as the maximum shear in the slab at the support which will be distributed on 75% of the span of the beam. For the sake of simplicity the load is uniformly distributed throughout the length of the beam by multiplying the existing shear by 0.92.

Analysis and Design of Floor Slab



Typical floor plan lay out

Depth Determination

$d \geq \left(0.4 + 0.6\frac{f_{yk}}{400}\right) \times \frac{L_e}{\beta_a}$ $\quad$ *for C_{25} concrete and S_{300} reinforcement,*

$d \geq 0.85 \times \frac{L_e}{\beta_a}$

Panel	Type	Le(mm)	β_a	d(mm)
SS_1	End panel	5000	40	106.25
SS_2	Intermediate panel	4000	37.5	90.67
C_3	Cantilever	1600	12	113.33

Therefore, depth is governed by the maximum that is 113.33mm

Over all depth (D) = 113.33 + 7+15 = **135.33 mm**

For safety against any unforeseen considerations, we can consider 150mm overall depth of slab.

Hence, d = 150 -15-7 = **128 mm**

Loading

The slab is loaded with both dead load and live load. The dead load comes from the slab self-weight, floor finish, cement screed, plastering and partition wall, if any is present.

Panel SS_2 and C_3

- ✓ $15cm\ thick\ RC\ slab = 0.15 \times 25 = 3.75\,KN/m^2$
- ✓ $2.5cm\ plastering = 0.025 \times 23 = 0.575\ KN/m^2$
- ✓ $20mm\ PVC\ floor\ finish = 0.02 \times 16 = 0.32\ KN/m^2$
- ✓ $30mm\ cement\ screed = 0.03 \times 23 = 0.69\,KN/m^2$

Total dead load = 3.75 +0.575 +0.32 +0.69 =**5.335 KN/m^2**

Live Load = **2 KN/m^2**

Therefore, design load will be:

Pd = 1.3(Dead load) + 1.6(Live load)

Pd = 1.3(5.335) + 1.6(2)

Pd = 6.9355 +3.20 = 10.1355 KN/m^2

Analysis of Individual Panel Moments

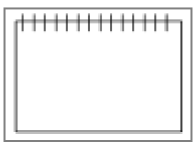
- ✓ $For\ two\ way\ slab,\ m_i = \alpha_i P_d {L_x}^2$
- ✓ For cantilever, $M = \frac{P_{dd}{L_x}^2}{2} + P_{dc} L_x$

Where,

P_{dd} is distributed load and

P_{dc} is concentrated load

Using the above formulas, all the panel moments were computed and are tabulated in the table below.

Panel	Type	P_d (KN/m^2)	L_x (m)	$\frac{L_y}{L_x}$	α_i				$m_i(KNm)$			
					α_{xs}	α_{xf}	α_{ys}	α_{yf}	m_{xs}	m_{xf}	m_{ys}	m_{yf}
SS_2		10.14	4.00	1.3	0.076	0.057	-	0.044	12.33	9.25	0.00	7.14
C_1	Cantilever	10.14	1.50	3.3	-	-	-	-	-	-	-	11.41
C_2	Cantilever	10.14	1.50	2.0	-	-	-	-	-	-	-	11.41
C_3	Cantilever	10.14	1.50	3.1	-	-	-	-	-	-	-	11.41
C_4	Cantilever	10.14	2.00	2.5	-	-	-	-	-	-	-	20.28

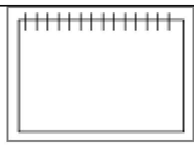
Unadjusted Moment for Floor Slab

Moment Adjustment

Since panel C_3 is cantilever, we can't adjust the support moment.

Thus, **m_{yf} = 11.41KN.m**

Load Transfer to Beams

Panel	Type	P_d (KN/m^2)	L_x (m)	$\frac{L_y}{L_x}$	β_{vi}				$V_i(KN)$			
					β_{vcx}	β_{vdx}	β_{vcy}	β_{vdx}	V_{cx}	V_{dx}	V_{cy}	V_{dy}
SS_2		10.14	4.0	1.3	0.53	0.35	-	0.3	21.5	19.2	-	12.2
C_1	Cantilever	10.14	1.50	3.3	-	-	-	-	11.41	-	-	-
C_2	Cantilever	10.14	1.50	2.0	-	-	-	-	11.41	-	-	-
C_3	Cantilever	10.14	1.50	3.1	-	-	-	-	11.41	-	-	-
C_4	Cantilever	10.14	2.00	2.5	-	-	-	-	20.28	-	-	-

$V_x = \beta_{vx} \times P_d \times L_x$

$V_y = \beta_{vy} \times P_d \times L_x$

Table Load Transfer to Beams from Floor Slabs

Reinforcement Bars for Slabs

The reinforcements are provided for the design moments based on the formula below

$A_s = \rho bd, EBCS - 2,1995$

$$\rho = \frac{1}{2}\left[c_1 \pm \sqrt{c_1{}^2 - \frac{4M}{bd^2 c_2}}\right]$$

$$where: c_1 = \frac{2.5}{m}, \quad c_2 = 0.32m^2 f_{cd}, \quad m = \frac{f_{yd}}{0.8f_{cd}}$$

The geometrical main reinforcement ration ρ at any section of a slab where positive reinforcement is required by analysis shall not be less than that given by:

$$\rho_{min} = \frac{0.5}{f_{yk}} \quad where\ f_{yk}\ is\ in\ MP_a$$

$The\ minimum\ slab\ reinforcement(A_{s,min}) = \rho_{min}bd$

$$Maximum\ spacing(S_{max}) \leq \begin{cases} \frac{f_{yk} a_s}{0.5d} \\ 2t_s \\ 350mm \end{cases}$$

$where: a_s\ is\ area\ of\ a\ single\ bar\ and\ d\ is\ the\ effective\ depth\ of\ the\ slab$

The required spacing (S_{req}) is calculated as:

$$S_{req} = \frac{a_s \times b}{A_s} \leq S_{max}$$

$b\ is\ taken\ as\ to\ be\ a\ 1000mm\ strip\ of\ the\ slab$

The table below shows the reinforcements provides for each slab.

Position	Moment	Depth	Steel Ration	A_s Calculated	S Required	ϕ of Bar
	KN m	(mm)	(ρ)	(mm^2)	(mm)	(mm)
SS_2	12.33	128	0.0029	382.00	280.00	3Φ12
C_1	11.41	128	0.0028	353.00	300.00	4Φ12
C_2	11.41	128	0.0028	353.00	300.00	4Φ12
C^3	11.41	128	0.0028	353.00	300.00	4Φ12
C_4	20.28	128	0.0051	645.00	175.00	6Φ12

CHAPTER FOUR

4. DESIGN OF STAIR CASE

The purpose of stairs is to provide pedestrian access to different levels within the building. The geometric form of the stair case depending on individual circumstances involved. These are two main components of stair case. Stair and landing slab. The flight and landing can arrange in different forms to get different types of stair cases. Rise and going are two terms associated with a stair. Risers refer to vertical height of a step and going represents the horizontal dimension.

Stair case analysis and design is similar to slabs. It involves the analysis steps followed for slabs. The inclined configuration is analyzed by projecting the loads on a horizontal plane. The stair contains **three flights** with the same configuration.

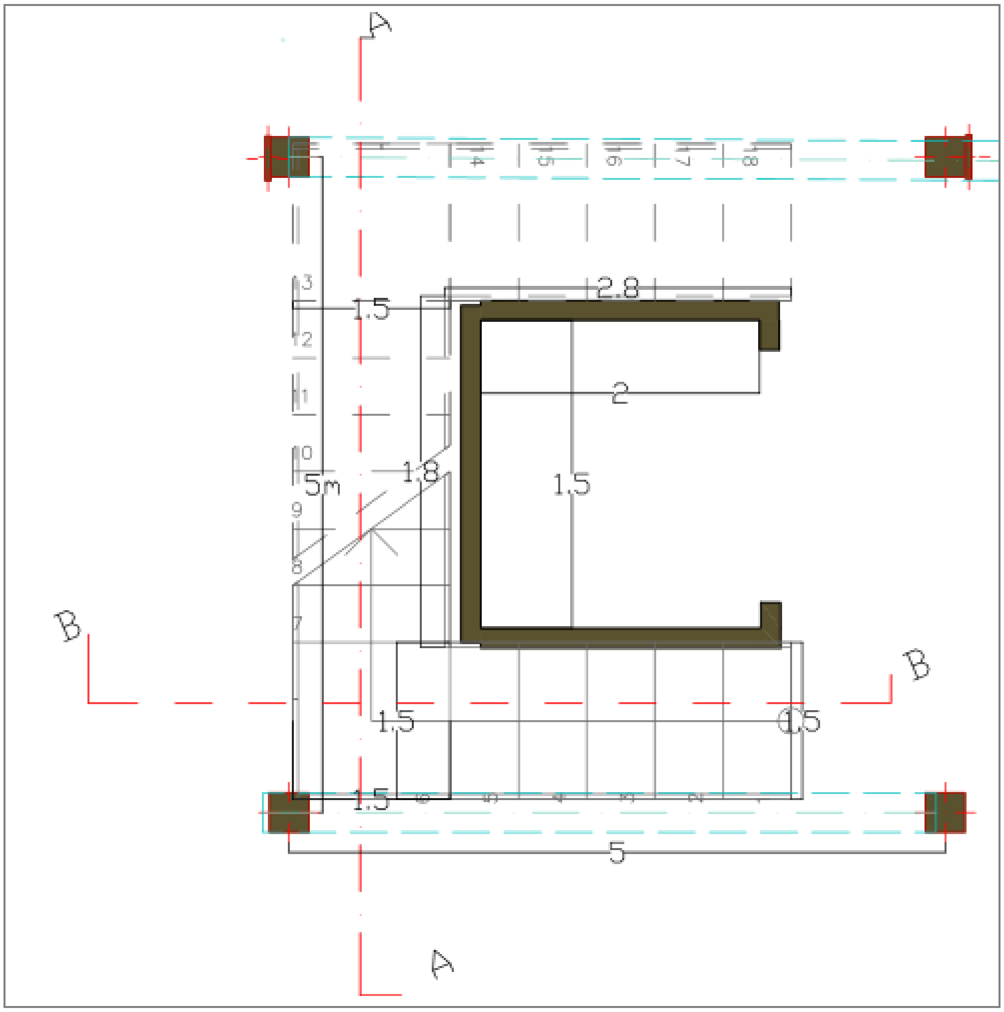


Figure: Stair Lay out

4.1. DESIGN PROCEDURE

- **Determination of depth for deflection**: This is a function of design tensile strength of steel, effective span length of the shortest span in which more load is expected to transfer and support condition.
  - ✓ **Loading**: - This determines the total load in the stair and landing
  - ✓ **Analysis**: - determines moment and shear forces based on the analyzed moment

- ✓ **Check depth for flexure**: - this step helps to cross check the design depth as it is safe for flexure or not, if not revise the depth determined in step 1 and also the loads.
- ✓ **Reinforcement provision:** using the computed moments, number and area of reinforcement bars determined.
- ✓ **Detailing**: the arrangement of reinforcement bars and their length are determined and drawn.

Material properties

- Steel grade f_{yk}= 300MPa
- Concrete class f_{ck} = 25MPa
- Class one work is used.

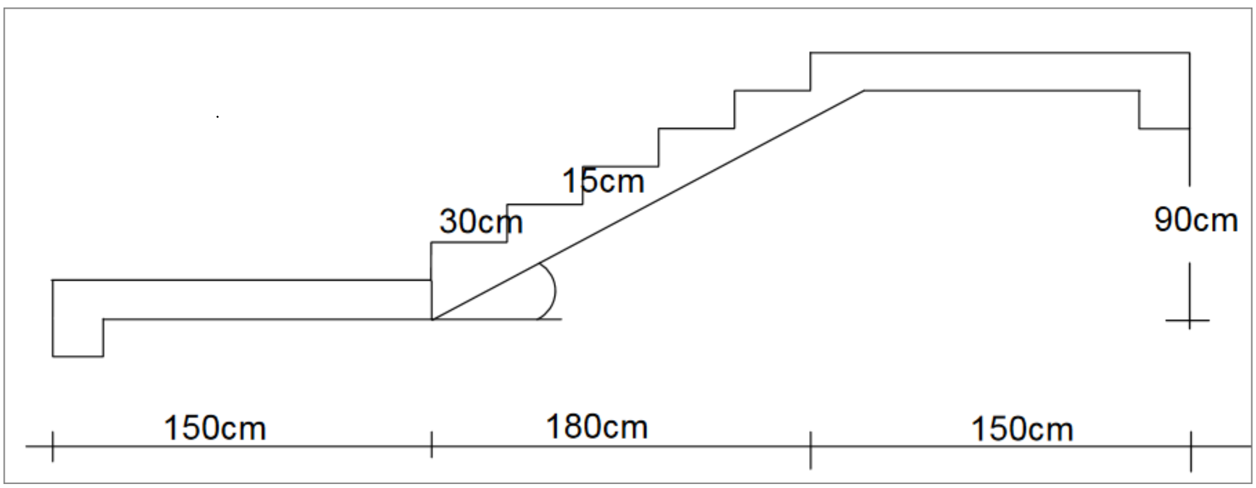


Figure Section view of stair A-A

The following information are taken from the above section of stair case

- ✓ Effective length L_e = 4.8m
- ✓ β_a= for simply supported 24
- ✓ Number of riser = 6
- ✓ Number of tread = 5

- ✓ Width of tread = 30cm
- ✓ Height of riser = 15cm

Check whether the load is transferred to one way or in two ways

L_y/L_x = 4.8/1.5 = 3.2 >2 which is **one way**

Determination of depth for deflection (Section A-A)

$d \geq \left(0.4 + 0.6\frac{f_{yk}}{400}\right) \times \frac{L_e}{\beta_a}$ ≥ (0.4 + 0.6*300/400)*4800/24 = **170 mm**

To provide same depth for all types of stairs compare overall depth and take maximum of all sections.

Determination of depth for deflection (Section B-B)

$d \geq \left(0.4 + 0.6\frac{f_{yk}}{400}\right) \times \frac{L_e}{\beta_a}$ ≥ 0.85* 4800/24 = **170 mm**

To determine the overall depth of the stair slab, we consider the following:

- ✓ Assume ϕ14
- ✓ Concrete cover = 25mm

Overall depth = d +$\frac{}{2}$ + concrete cover = 170 + 7 + 25 = **202mm**

Use D = **220mm**

d = 220 – 7 – 25 = **188mm**

Determination of angle of inclination

The angle of inclination for sections is the same.

Angle of section A-A = Angle of section B-B

$$\theta = tan^{-1}\frac{.90}{1.80.} = \mathbf{29.62}$$

Determination of Dead loads of the stair case

✓ Dead load at Steps(Inclined part**)**

- Own weight of the slab =1.5*0.22*25 =8.27KN/m
- 30 mm thick cement screed =1.5*0.03*23 = 1.035 KN/m
- 20mm thick floor finish = 1.5*0.02*27 = 0.81 KN/m
- Dead load from steps = 0.5*1.5*0.15*25 =2.81 KN/m

Total = 12.92 KN/m

✓ Total live load on the landing = 3 KN/m^2

Design load (P_d) **=** 1.3(Dead load) + 1.6(Live load**)**

= 1.3(12.92) + 1.6(3.0)

<u>Design load (P_d = 21.596 KN/m</u>

✓ Dead load at Landing

- Own weight of slab=1.5*0.22*25 = 8.25 KN/m
- 30mm thick cement screed=1.5*0.03*23 = 1.035 KN/m
- 20mm thick of terrazzo finish=1.5*0.02*27 = 0.81 KN/m

Total dead load on the landing **= 10.095 KN/m**

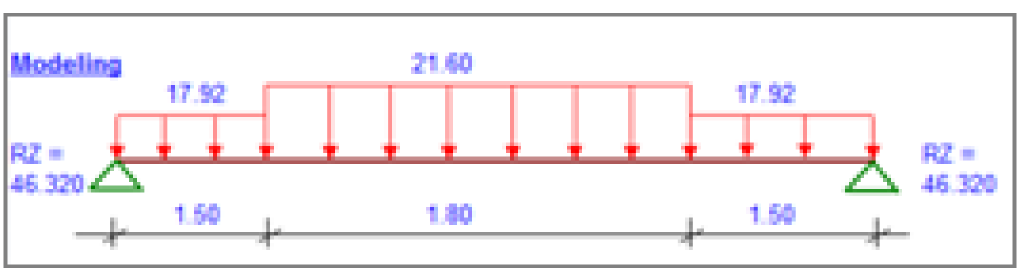
✓ Total live load on the landing = 3 KN/m^2

Design load (P_d) **=** 1.3(Dead load) + 1.6 (Live load)

= 1.3(7.475) + 1.6(3)

<u>Design load (P_d) = 17.92 KN/m(Landing)</u>

➢ **Modeling stair case**



$\sum Fy = 0$ And $\sum Mo = 0$ where clockwise moment is positive

And there are two reactions, R_1 and R_2 at the supports and $R_1 = R_2$

$R_1 + R_2$ = (1.5*17.92)*2 + (21.6*1.80) = 92.64 KN

R_1 = 46.32 KN and R2 = 46.32 KN

Using design templates we can calculate the design actions.

- **Design actions:**
- Design moment = **55.142KNm**
- Design shear = **46.32KN**

- **Checking for deflection**

Since W= 21.6KN/m , $\Delta_{max} = \frac{5wL^4}{384EI}$ =14.64 mm

L= 4.80m

E = 29Gpa

I = 0.0004m^4

But, according to EBCS-2, section 5.2.2, the final deflection shall not exceed the value

$\delta = \frac{L_e}{200}$ = 4.8/200 = **24 mm**

Since the actual deflection is less than the recommended value (14.64mm < 24 mm), then it is **safe.**

- **Check for Shear Capacity**

Before we go to reinforcement calculation, it is better to check the shear capacity of the section.

$V_c = 0.25 f_{ctd} k_1 k_2 b_w d$ Where, f_{ctd}=1000Kpa, $k_1 = 1 + 50\rho$, k_2 = 1.6-d ,b=1.5 and d=0.188

$$V_c = 0.25 f_{ctd} k_1 k_2 b_w d$$

= 0.25*1000*1.085*1.412*1.5*0.188

= **108 KN > 46.32 KN**............OK! that is no shear reinforcement is required. But it is recommended to use ϕ8c/c300mm to hold the main reinforcement.

- **Check depth for flexure**

Considering maximum moment: M = 55.142KNm

$$d = \sqrt{\frac{55.142*10^6}{.2952*1500*11.33}} = 104.84\text{mm}$$

Then, D = d + 7 +25 = 104.84+7+25

=**136.84 mm < 220mm**..............OK!

Reinforcement Calculation

To calculate the reinforcements first we have to calculate steel reinforcement ratio

$$\rho = \frac{1}{m}\left[1 - \sqrt{1 - \frac{2mR_n}{f_{yd}}}\right]$$

Where, $R_n = M_u/bd^2 = 55.142/1.5*0.188^2 = 1.04N/mm^2$

$m = f_{yd}/f_{cd} = 260.87/11.33 = 23.03$

= 1/23.03[1-√1-2*23.03*1040/260.87] = **0.0042**

Therefore, the longitudinal reinforcement can be calculated using the calculated raw.

$A_s = \rho bd$ = 0.0042*1500*188

= **<u>1185 mm²</u>**

But, $A_{smin} = \frac{0.5bd}{fyk}$ where fyk = characteristic yield strength

b = width

d = depth

$= \frac{(0.5*1500*188)}{300} = 470\text{mm}^2$ Since $A_s > A_{min}$ then it is OK

- **Spacing:** **S** = $1000a_s/A_s$ =(1000*3.14*49)/1185 = 129.84 mm = **125mm**

$S_{max} = \min\begin{cases} 2D \\ 350 \end{cases}$ = 2*220=440mm or 350mm, then Spacing will be **S = 125 mm**

Therefore, provide ϕ14 c/c 125mm

- **Minimum reinforcement**

$A_{smin} = \rho_{min}.bd$

$\rho_{min} = \frac{0.4}{fyk}$ =0.4/300 =0.00133

A_{smin} = 0.00133*1500*188 =375.06mm^2 and S = 1000*3.14*36/375.06 = 301.44mm

Therefore, provide ϕ12 c/c 300mm

- **Transverse reinforcement**

According to EBCS 2, Sec7.2.2.1 the ratio of secondary reinforcement to main reinforcement shall be at least equal to 20%.

That is 20 % (1185) =237mm^2 and use diameter 10mm steel rebar

- ✓ **Spacing** = 1000 *as/As

  = (1000*3.14*25) / 237 = 331mm

Therefore, provide ϕ10 c/c 330mm

CHAPTER FIVE

5.0 Earth Quake Analysis

Base Shear Determination

The base shear is given by the following formula

- ✓ $f_b = S_d(T_1)$ W

Where,

f_b=base shear

W=Total weight of the building

$S_d(T_1)$=ordinate of the design spectrum at period T_1 which is given by

- ✓ $S_d(T_1)=\alpha\beta\gamma$

Where, α= ratio of the design bed rock acceleration to the acceleration of gravity, g. It is given by:

- ✓ $\alpha=\alpha_0 I$

Where, α_0= the bed rock acceleration ratio for the site and depends on the seismic zone.

α_0=0.1 for earth quake zone-2 (From EBCS-1995, table 1-1)

I=Importance factor of the structure

I=1.2 (EBCS-8, 1995 table 2-4), thus **α=0.1*1.2=0.12**

$\beta =$ Is the design response factor for the site and is given by

- ✓ $\beta = 1.2S/T^{\left(\frac{2}{3}\right)} \leq 2.5$

Where, S=site coefficient of soil characteristics

S=1.2 (for sub soil class B, EBCS-8, 1995, table1-2)

T_1= the fundamental period of vibration of the structure (in seconds) for translational motion in the direction of the motion. For structures up to 80m height, the value of T_1 may be approximated by

- ✓ $T_1=C_1 H^{3/4}$

Where, H=Height of the structure above the base in meter=22.5m

C_1=0.075 for RC moment resisting frames and eccentrically braced steel frame.

T_1=0.775sec

β=1.2*1.2/ ($0.775^{(2/3)}$) =1.71, but β ≤2.5 hence take β**=1.71**

γ is the behavior factor to account for energy dissipation capacity

- ✓ $\gamma=\gamma_0 \times k_D \times k_r \times k_w \leq 0.7$

Where,

γ_0=basic type of behavior factor, dependent on the structural type (EBCS-8, 1995, table 3-2) =0.2 (For frame system)

K_D=factor reflecting the ductility class

=2 (lower ductility, page 38)

K_R=factor reflecting the structural regularity in elevation

=1 (frame system)

k_w=factor reflecting the prevailing failure mode in the structural system with wall

=1 (frame system)

γ = 0.2*2*1*1 = 0.4

Therefore, $S_d(T_1)$ = 0.12*1.71*0.4 = **0.082**

✓ f_b=0.082W, which is 8% of the total weight, is acting as a horizontal force.

3.1. Story Shear Determination

The base shear force shall be distributed over the height of a structure concentrated at each floor level as

$$F_i = \frac{(F_b - F_t)W_i h_i}{\sum_{i=0}^{n} W_i h_i}$$

Where,

n= number of stories

F_i=is the concentrated lateral force acting at floor i,

F_t= is the a concentrated extra force (in addition to F_n) at the top of the structure accounting whiplash for slender structure, which is given by

$$F_t = \begin{cases} 0 \\ 0.07\, T_1 F_b \le 0.25\, f_b \text{ for } T > 0.7 \end{cases}$$

since T_1=0.775sec

$F_t = 0.07T_1 f_b \le 0.25 f_b$

= 0.07*T* f_b = 0.07*0.775* F_b

= $0.05425 f_b$

5.1 Calculation of Weights

The following tables give the mass of each element (column, beam, slab, partition walls, roof, windows and doors etc.). They also give the x and y – coordinates of each elements

The lumped mass at each floor level was calculated by taking half portion from above and half from below of the floor.

1. Ground Floor											
Designation	**Unit Weight/PD**	**Height or Width**	**Length**		**Area**	**Volume**	**Weight**	**Moment Arm**		**Moment**	
	(KN/m^{3})	**(m)**	**Lx**	**Ly**	**(m^{2})**	**(m^{3})**	**(KN)**	**Xm**	**Ym**	**Mx(KN m)**	**My(KN m)**
COLUMN											
C1	25	3.00	0.4	0.4	0.16	0.48	**12.00**	0.00	0.00	**0.00**	**0.00**
C2	25	3.00	0.4	0.4	0.16	0.48	**12.00**	5.00	0.00	**0.00**	**60.00**
C3	25	3.00	0.4	0.4	0.16	0.48	**12.00**	9.00	0.00	**0.00**	**108.00**
C4	25	3.00	0.4	0.4	0.16	0.48	**12.00**	14.0	0.00	**0.00**	**168.00**
C5	25	3.00	0.4	0.4	0.16	0.48	**12.00**	0.00	6.00	**72.00**	**0.00**
C6	25	3.00	0.4	0.4	0.16	0.48	**12.00**	5.00	6.00	**72.00**	**60.00**
C7	25	3.00	0.4	0.4	0.16	0.48	**12.00**	9.00	6.00	**72.00**	**108.00**
C8	25	3.00	0.4	0.4	0.16	0.48	**12.00**	14.00	6.00	**72.00**	**168.00**
C9	25	3.00	0.4	0.4	0.16	0.48	**12.00**	20.00	6.00	**72.00**	**240.00**
C10	25	3.00	0.4	0.4	0.16	0.48	**12.00**	0.00	10.00	**120.00**	**0.00**
C11	25	3.00	0.4	0.4	0.16	0.48	**12.00**	5.00	10.00	**120.00**	**60.00**
C12	25	3.00	0.4	0.4	0.16	0.48	**12.00**	9.00	10.00	**120.00**	**108.00**
C13	25	3.00	0.4	0.4	0.16	0.48	**12.00**	14.00	10.00	**120.00**	**168.00**
C14	25	3.00	0.4	0.4	0.16	0.48	**12.00**	20.00	10.00	**120.00**	**240.00**
C15	25	3.00	0.4	0.4	0.16	0.48	**12.00**	0.00	15.00	**180.00**	**0.00**
C16	25	3.00	0.4	0.4	0.16	0.48	**12.00**	5.00	15.00	**180.00**	**60.00**

C17	25	3.00	0.4	0.4	0.16	0.48	**12.00**	9.00	15.00	**180.00**	**108.00**
C18	25	3.00	0.4	0.4	0.16	0.48	**12.00**	14.00	15.00	**180.00**	**168.00**
C19	25	3.00	0.4	0.4	0.16	0.48	**12.00**	20.00	15.00	**180.00**	**240.00**
SUM							**228.0**			**1860.0**	**2064.0**

GROUND BEAMS											
1-1	25	20.00	0.30	0.40	0.12	2.40	**60.00**	10.00	15.00	**900.00**	**600.00**
2-2	25	20.00	0.30	0.40	0.12	2.40	**60.00**	10.00	10.00	**600.00**	**600.00**
3-3	25	20.00	0.30	0.40	0.12	2.40	**60.00**	10.00	6.00	**360.00**	**600.00**
4-4	25	14.00	0.30	0.40	0.12	1.68	**42.00**	7.000	0.00	**0.00**	**294.00**
A-A	25	16.50	0.30	0.40	0.12	1.98	**49.50**	0.00	10.00	**495.00**	**0.00**
B-B	25	16.50	0.30	0.40	0.12	1.98	**49.50**	5.00	10.00	**495.00**	**247.50**
C-C	25	15.00	0.30	0.40	0.12	1.80	**45.00**	9.00	7.50	**337.50**	**405.00**
D-D	25	15.00	0.30	0.40	0.12	1.80	**45.00**	14.00	7.50	**337.50**	**630.00**
E-E	25	10.50	0.30	0.40	0.12	1.26	**31.50**	20.00	7.50	**236.25**	**630.00**
GROUND SLAB											
S1	20.00	0.12	5.00	5.00	25.00	3.00	**60.00**	2.50	16.50	**990.00**	**150.00**
S2	20.00	0.12	4.00	5.00	20.00	2.40	**48.00**	7.00	16.50	**792.00**	**336.00**
S3	20.00	0.12	5.00	5.00	25.00	3.00	**60.00**	11.50	16.50	**990.00**	**690.00**
S4	20.00	0.12	6.00	5.00	30.00	3.60	**72.00**	17.00	16.50	**1188.00**	**1224.00**
S5	20.00	0.12	4.00	4.00	16.00	1.92	**38.40**	7.00	8.00	**307.20**	**268.80**
S6	20.00	0.12	5.00	4.00	20.00	2.40	**48.00**	11.50	8.00	**384.00**	**552.00**

S7	20.00	0.12	6.00	4.00	24.00	2.88	**57.60**	17.00	8.00	**460.80**	**979.20**
S8	20.00	0.12	5.00	6.00	30.00	3.60	**72.00**	2.50	3.00	**216.00**	**180.00**
S9	20.00	0.12	4.00	6.00	24.00	2.88	**57.60**	7.00	3.00	**172.80**	**403.20**
S10	20.00	0.12	5.00	6.00	30.00	3.60	**72.00**	11.50	3.00	**216.00**	**828.00**
PARTITION WALL											
W1	14	1.40	0.2	9.95	13.93	2.79	**39.06**	0.00	8.25	**322.25**	**0.00**
W2	10	1.40	0.10	3.80	5.32	0.53	**5.30**	3.33	1.90	**10.07**	**17.65**
W3	10	1.40	0.20	5.75	8.05	1.61	**16.10**	7.00	0.00	**0.00**	**112.70**
W4	10	1.40	0.10	2.00	2.80	0.28	**2.80**	4.00	2.50	**7.00**	**11.20**
W5	10	1.40	0.10	3.80	5.32	0.53	**5.30**	5.00	1.90	**10.07**	**26.50**
W6	10	1.40	0.10	3.80	5.32	0.53	**5.30**	6.50	1.90	**10.07**	**34.45**
W7	10	1.40	0.10	1.40	1.96	0.20	**2.00**	7.60	2.50	**5.00**	**14.90**
W8	10	1.40	0.10	1.40	1.96	0.20	**2.00**	8.00	2.50	**5.00**	**15.68**
W9	10	1.40	0.10	3.80	5.32	0.53	**5.32**	13.00	1.90	**10.11**	**69.16**
W10	10	1.40	0.10	3.80	5.32	0.53	**5.32**	15.00	1.90	**10.11**	**79.80**
W11	14	1.40	0.20	5.60	7.84	1.57	**21.95**	14.00	2.80	**61.46**	**307.30**
W12	10	1.40	0.10	2.26	3.16	0.32	**3.16**	1.13	3.76	**11.88**	**3.57**
W13	10	1.40	0.10	2.00	2.80	0.28	**2.80**	3.33	4.76	**13.33**	**9.32**
W14	14	1.40	0.20	17.20	24.08	4.82	**67.42**	6.00	8.60	**579.8**	**404.52**
W15	14	1.40	0.20	16.24	22.74	4.55	**63.66**	8.12	8.00	**509.3**	**516.92**
W16	14	1.40	0.20	2.00	2.80	0.56	**7.84**	5.00	7.00	**54.88**	**39.20**

W17	10	1.40	0.10	2.00	2.80	0.28	**2.80**	3.70	9.00	**25.20**	**10.36**
W18	10	1.40	0.10	1.60	2.24	0.22	**2.24**	5.00	9.20	**20.61**	**11.20**
W19	10	1.40	0.10	2.97	4.16	0.42	**4.16**	1.49	10.00	**41.60**	**6.20**
W20	14	1.40	0.20	2.10	2.94	0.59	**8.26**	3.00	11.95	**98.71**	**24.78**
W21	10	1.40	0.10	3.95	5.53	0.553	**5.53**	1.96	13.00	**71.89**	**10.84**
W22	14	1.40	0.20	5.20	7.28	1.46	**20.38**	2.60	16.50	**336.3**	**52.99**
W_{23}	14	1.40	0.20	1.50	2.10	0.42	**5.88**	5.00	15.75	**92.61**	**29.40**
W_{24}	10	1.40	0.10	2.00	2.80	0.28	**2.80**	11.38	9.00	**25.20**	**31.86**
W_{25}	10	1.40	0.10	1.60	2.24	0.22	**2.24**	14.00	8.80	**19.71**	**31.36**
W_{26}	10	1.40	0.10	1.10	1.54	0.15	**1.54**	12.55	10.00	**15.40**	**19.33**
W_{27}	14	1.40	0.20	8.20	11.48	2.29	**32.14**	15.90	12.10	**388.9**	**511.03**
W_{28}	10	1.40	0.10	3.00	4.20	0.42	**4.20**	18.10	9.50	**39.90**	**76.02**
W_{29}	10	1.40	0.10	5.10	7.14	0.71	**7.14**	15.60	11.00	**78.54**	**111.38**
W_{30}	10	1.40	0.10	2.78	3.89	0.39	**3.89**	13.18	13.61	**52.94**	**51.27**
W_{31}	14	1.40	0.20	5.50	7.70	1.54	**21.56**	10.00	15.00	**323.4**	**215.60**
SUM							**1408.2**			**12729.3**	**12442.8**

Table Mass Calculation for Ground Floor

First floor to Sixth floor											
Designation	Unit Weight/PD	Height or Width	Length		Area	Volume	Weight	Moment Arm		Moment	
	(KN/m^3)	(m)	Lx	Ly	(m^2)	(m^3)	(KN)	Xm	Ym	Mx(KN m)	My(KN m)
COLUMN											
C1	25	3.00	0.4	0.4	0.16	0.48	**12.00**	0.00	0.00	**0.00**	**0.00**
C2	25	3.00	0.4	0.4	0.16	0.48	**12.00**	5.00	0.00	**0.00**	**60.00**
C3	25	3.00	0.4	0.4	0.16	0.48	**12.00**	9.00	0.00	**0.00**	**108.00**
C4	25	3.00	0.4	0.4	0.16	0.48	**12.00**	14.0	0.00	**0.00**	**168.00**
C5	25	3.00	0.4	0.4	0.16	0.48	**12.00**	0.00	6.00	**72.00**	**0.00**
C6	25	3.00	0.4	0.4	0.16	0.48	**12.00**	5.00	6.00	**72.00**	**60.00**
C7	25	3.00	0.4	0.4	0.16	0.48	**12.00**	9.00	6.00	**72.00**	**108.00**
C8	25	3.00	0.4	0.4	0.16	0.48	**12.00**	14.00	6.00	**72.00**	**168.00**
C9	25	3.00	0.4	0.4	0.16	0.48	**12.00**	20.00	6.00	**72.00**	**240.00**
C10	25	3.00	0.4	0.4	0.16	0.48	**12.00**	0.00	10.00	**120.00**	**0.00**
C11	25	3.00	0.4	0.4	0.16	0.48	**12.00**	5.00	10.00	**120.00**	**60.00**
C12	25	3.00	0.4	0.4	0.16	0.48	**12.00**	9.00	10.00	**120.00**	**108.00**
C13	25	3.00	0.4	0.4	0.16	0.48	**12.00**	14.00	10.00	**120.00**	**168.00**
C14	25	3.00	0.4	0.4	0.16	0.48	**12.00**	20.00	10.00	**120.00**	**240.00**
C15	25	3.00	0.4	0.4	0.16	0.48	**12.00**	0.00	15.00	**180.00**	**0.00**
C16	25	3.00	0.4	0.4	0.16	0.48	**12.00**	5.00	15.00	**180.00**	**60.00**
C17	25	3.00	0.4	0.4	0.16	0.48	**12.00**	9.00	15.00	**180.00**	**108.00**
C18	25	3.00	0.4	0.4	0.16	0.48	**12.00**	14.00	15.00	**180.00**	**168.00**
C19	25	3.00	0.4	0.4	0.16	0.48	**12.00**	20.00	15.00	**180.00**	**240.00**
SUM							**228.0**			**1320.0**	**1896.0**

GIRDER BEAMS											
1-1	25	20.00	0.30	0.40	0.12	2.40	**60.00**	10.00	15.00	**900.00**	**600.00**
2-2	25	20.00	0.30	0.40	0.12	2.40	**60.00**	10.00	10.00	**600.00**	**600.00**
3-3	25	20.00	0.30	0.40	0.12	2.40	**60.00**	10.00	6.00	**360.00**	**600.00**
4-4	25	14.00	0.30	0.40	0.12	1.68	**42.00**	7.000	0.00	**0.00**	**294.00**
A-A	25	16.50	0.30	0.40	0.12	1.98	**49.50**	0.00	10.00	**495.00**	**0.00**
B-B	25	16.50	0.30	0.40	0.12	1.98	**49.50**	5.00	10.00	**495.00**	**247.50**
C-C	25	15.00	0.30	0.40	0.12	1.80	**45.00**	9.00	7.50	**337.50**	**405.00**
D-D	25	15.00	0.30	0.40	0.12	1.80	**45.00**	14.00	7.50	**337.50**	**630.00**
E-E	25	10.50	0.30	0.40	0.12	1.26	**31.50**	20.00	7.50	**236.25**	**630.00**
SUM							**442.5**			**3761.3**	**4006.5**

RIBBED SLAB											
S1	20.00	0.22	5.00	5.00	25.00	5.50	**110.00**	2.50	16.50	**1815.00**	**275.00**
S2	20.00	0.22	4.00	5.00	20.00	4.40	**88.00**	7.00	16.50	**1452.00**	**616.00**
S3	20.00	0.22	5.00	5.00	25.00	5.50	**110.00**	11.50	16.50	**1815.00**	**1265.00**
S4	20.00	0.22	6.00	5.00	30.00	6.60	**132.00**	17.00	16.50	**2178.00**	**2244.00**
S5	20.00	0.22	4.00	4.00	16.00	3.52	**70.40**	7.00	8.00	**563.20**	**492.80**
S6	20.00	0.22	5.00	4.00	20.00	4.40	**88.00**	11.50	8.00	**704.00**	**1012.00**
S7	20.00	0.22	6.00	4.00	24.00	5.28	**105.60**	17.00	8.00	**844.80**	**1795.20**
S8	20.00	0.22	5.00	6.00	30.00	6.60	**132.00**	2.50	3.00	**396.00**	**330.00**
S9	20.00	0.22	4.00	6.00	24.00	5.28	**105.60**	7.00	3.00	**316.80**	**739.20**
S10	20.00	0.22	5.00	6.00	30.00	6.60	**132.00**	11.50	3.00	**396.00**	**1518.00**
C_1	25.00	0.15	5.00	1.50	7.50	1.125	**28.125**	2.50	15.75	**442.97**	**70.31**
C_2	25.00	0.15	2.00	1.50	3.00	0.45	**11.25**	14.00	15.75	**177.19**	**157.50**

C_2	25.00	0.15	2.00	1.50	3.00	0.45	**11.25**	-0.75	14.00	**157.50**	**-8.44**
C_2	25.00	0.15	2.00	1.50	3.00	0.45	**11.25**	-0.75	5.00	**56.25**	**-8.44**
C_2	25.00	0.15	2.00	1.50	3.00	0.45	**11.25**	1.00	-0.75	**-8.44**	**11.25**
C_2	25.00	0.15	2.00	1.50	3.00	0.45	**11.25**	11.00	-0.75	**-8.44**	**123.75**
C_4	25.00	0.15	5.00	2.00	10.00	1.50	**37.50**	9.00	2.50	**93.75**	**337.50**
SUM							**1195.5**			**11391.58**	**10695.63**

PARTITION WALL											
W1	14	2.80	0.2	9.95	27.86	5.57	**77.98**	0.00	8.25	**643.34**	**0.00**
W2	10	2.80	0.10	3.80	10.64	1.06	**10.60**	3.33	1.90	**20.14**	**35.30**
W3	10	2.80	0.20	5.75	16.10	3.22	**32.20**	7.00	0.00	**0.00**	**225.40**
W4	10	2.80	0.10	2.00	5.60	0.56	**5.60**	4.00	2.50	**14.00**	**22.40**
W5	10	2.80	0.10	3.80	10.64	1.06	**10.60**	5.00	1.90	**20.14**	**53.00**
W6	10	2.80	0.10	3.80	10.64	1.06	**10.60**	6.50	1.90	**20.14**	**68.90**
W7	10	2.80	0.10	1.40	3.92	0.40	**4.00**	7.60	2.50	**10.00**	**29.80**
W8	10	2.80	0.10	1.40	3.92	0.40	**4.00**	8.00	2.50	**10.00**	**31.36**
W9	10	2.80	0.10	3.80	10.64	1.06	**10.64**	13.00	1.90	**20.22**	**138.32**
W10	10	2.80	0.10	3.80	10.64	1.06	**10.64**	15.00	1.90	**20.22**	**159.60**
W11	14	2.80	0.20	5.60	15.68	3.14	**43.90**	14.00	2.80	**122.9**	**614.60**
W12	10	2.80	0.10	2.26	6.32	0.64	**6.32**	1.13	3.76	**23.76**	**7.14**
W13	10	2.80	0.10	2.00	5.60	0.56	**5.60**	3.33	4.76	**26.66**	**18.65**
W14	14	2.80	0.20	17.20	48.16	9.68	**134.84**	6.00	8.60	**809.0**	**809.04**
W15	14	2.80	0.20	16.24	45.48	9.10	**127.32**	8.12	8.00	**1018**	**1033.8**
W16	14	2.80	0.20	2.00	5.60	1.12	**15.68**	5.00	7.00	**109.8**	**78.40**
W17	10	2.80	0.10	2.00	5.60	0.56	**5.60**	3.70	9.00	**50.40**	**20.72**

W18	10	2.80	0.10	1.60	4.48	0.44	**4.48**	5.00	9.20	**41.22**	**22.40**
W19	10	2.80	0.10	2.97	8.32	0.84	**8.32**	1.49	10.00	**83.20**	**12.40**
W20	14	2.80	0.20	2.10	5.88	1.18	**16.52**	3.00	11.95	**197.4**	**49.56**
W21	10	2.80	0.10	3.95	11.06	1.11	**11.06**	1.96	13.00	**143.8**	**21.68**
W22	14	2.80	0.20	5.20	14.56	2.92	**40.76**	2.60	16.50	**672.6**	**105.98**
W_{23}	14	2.80	0.20	1.50	4.20	0.84	**11.76**	5.00	15.75	**185.2**	**58.80**
W_{24}	10	2.80	0.10	2.00	5.60	0.56	**5.60**	11.38	9.00	**50.40**	**63.72**
W_{25}	10	2.80	0.10	1.60	4.48	0.44	**4.48**	14.00	8.80	**39.42**	**62.72**
W_{26}	10	2.80	0.10	1.10	3.08	0.30	**3.08**	12.55	10.00	**30.80**	**38.66**
W_{27}	14	2.80	0.20	8.20	22.96	4.58	**64.28**	15.90	12.10	**777.8**	**1022.1**
W_{28}	10	2.80	0.10	3.00	8.40	0.84	**8.40**	18.10	9.50	**79.80**	**152.04**
W_{29}	10	2.80	0.10	5.10	14.28	1.42	**14.28**	15.60	11.00	**157.1**	**222.76**
W_{30}	10	2.80	0.10	2.78	7.78	0.78	**7.78**	13.18	13.61	**105.9**	**102.54**
W_{31}	14	2.80	0.20	5.50	15.40	3.08	**43.12**	10.00	15.00	**646.8**	**431.20**
W_{32}	10	2.80	0.10	2.00	5.60	0.56	**5.60**	18.28	9.00	**50.40**	**102.37**
SUM							**765.64**			**6200.6**	**5796.8**

Roof									
Description	**Unit Weight**	**Length or Width**	**Area**	**Volume**	**Weight**	**Xm**	**Ym**	**Mx**	**My**
	(Dead load)	**(m)**	**(m^2)**	**(m^3)**	**(KN)**	**(m)**	**(m)**	**(KNm)**	**(KNm)**
DUO-PITCHED									
TRUSS	0.0524KN/m	415.66	-	-	**21.78**	10.00	7.50	**163.35**	**217.80**
PURLINE	0.043KN/m	360.00	-	-	**15.48**	10.00	7.50	**116.10**	**154.80**
G-28CIS	0.04KN/m^2	-	300.00	-	**12.00**	7.50	7.50	**90.00**	**90.00**
MONO PITCHED									
TRUSS	0.0524KN/m	98.91	-	-	**5.14**	2.50	7.50	**38.55**	**12.85**
PURLINE	0.043KN/m	56.00	-	-	**2.41**	2.50	7.50	**18.08**	**6.03**
G-28CIS	0.04KN/m^2	-	47.40	-	**1.896**	2.50	7.50	**14.22**	**4.74**
SUM					**58.71**			**440.3**	**486.22**

TOP TIE BEAMS											
1-1	25	20.00	0.20	0.20	0.04	0.80	**20.00**	10.00	15.00	**300.00**	**200.00**
2-2	25	20.00	0.20	0.20	0.04	0.80	**20.00**	10.00	10.00	**200.00**	**200.00**
3-3	25	20.00	0.20	0.20	0.04	0.80	**20.00**	10.00	6.00	**120.00**	**200.00**
4-4	25	14.00	0.20	0.20	0.04	0.56	**14.00**	7.000	0.00	**0.00**	**98.00**
A-A	25	16.50	0.20	0.20	0.04	0.66	**16.50**	0.00	10.00	**165.00**	**0.00**
B-B	25	16.50	0.20	0.20	0.04	0.68	**17.00**	5.00	10.00	**170.00**	**85.00**
C-C	25	15.00	0.20	0.20	0.04	0.60	**15.00**	9.00	7.50	**112.50**	**135.00**
D-D	25	15.00	0.20	0.20	0.04	0.60	**15.00**	14.00	7.50	**112.50**	**210.00**
E-E	25	10.50	0.20	0.20	0.04	0.42	**10.50**	20.00	7.50	**78.75**	**210.00**
SUM							**148.0**			**1258.75**	**1338.00**

COLUMN											
C1	25	1.50	0.20	0.30	0.06	0.09	**2.25**	0.00	0.00	**0.00**	**0.00**
C2	25	1.50	0.20	0.30	0.06	0.09	**2.25**	5.00	0.00	**0.00**	**11.25**
C3	25	1.50	0.20	0.30	0.06	0.09	**2.25**	9.00	0.00	**0.00**	**20.25**
C4	25	1.50	0.20	0.30	0.06	0.09	**2.25**	14.0	0.00	**0.00**	**31.50**
C5	25	1.50	0.20	0.30	0.06	0.09	**2.25**	0.00	6.00	**13.50**	**0.00**
C6	25	1.50	0.20	0.30	0.06	0.09	**2.25**	5.00	6.00	**13.50**	**11.25**
C7	25	1.50	0.20	0.30	0.06	0.09	**2.25**	9.00	6.00	**13.50**	**20.25**
C8	25	1.50	0.20	0.30	0.06	0.09	**2.25**	14.00	6.00	**13.50**	**31.50**
C9	25	1.50	0.20	0.30	0.06	0.09	**2.25**	20.00	6.00	**13.50**	**45.00**
C10	25	1.50	0.20	0.30	0.06	0.09	**2.25**	0.00	10.00	**22.50**	**0.00**
C11	25	1.50	0.20	0.30	0.06	0.09	**2.25**	5.00	10.00	**22.50**	**11.25**
C12	25	1.50	0.20	0.30	0.06	0.09	**2.25**	9.00	10.00	**22.50**	**20.25**
C13	25	1.50	0.20	0.30	0.06	0.09	**2.25**	14.00	10.00	**22.50**	**31.50**
C14	25	1.50	0.20	0.30	0.06	0.09	**2.25**	20.00	10.00	**22.50**	**45.00**
C15	25	1.50	0.20	0.30	0.06	0.09	**2.25**	0.00	15.00	**33.75**	**0.00**
C16	25	1.50	0.20	0.30	0.06	0.09	**2.25**	5.00	15.00	**33.75**	**11.25**
C17	25	1.50	0.20	0.30	0.06	0.09	**2.25**	9.00	15.00	**33.75**	**20.25**
C18	25	1.50	0.20	0.30	0.06	0.09	**2.25**	14.00	15.00	**33.75**	**31.50**
C19	25	1.50	0.20	0.30	0.06	0.09	**2.25**	20.00	15.00	**33.75**	**45.00**
SUM							**42.75**			**348.75**	**387.50**

Table Mass Calculation for Roof Level

FOUNDATION											
Designation	**Unit Weight/PD**	**Height or Width**	**Length**		**Area**	**Volume**	**Weight**	**Moment Arm**		**Moment**	
	(KN/m^3)	**(m)**	**Lx**	**Ly**	**(m^2)**	**(m^3)**	**(KN)**	**Xm**	**Ym**	**Mx(KN m)**	**My(KN m)**
COLUMN											
C1	25	1.80	0.4	0.4	0.16	0.288	**7.20**	0.00	0.00	**0.00**	**0.00**
C2	25	1.80	0.4	0.4	0.16	0.288	**7.20**	5.00	0.00	**0.00**	**36.00**
C3	25	1.80	0.4	0.4	0.16	0.288	**7.20**	9.00	0.00	**0.00**	**64.80**
C4	25	1.80	0.4	0.4	0.16	0.288	**7.20**	14.0	0.00	**0.00**	**100.80**
C5	25	1.80	0.4	0.4	0.16	0.288	**7.20**	0.00	6.00	**43.20**	**0.00**
C6	25	1.80	0.4	0.4	0.16	0.288	**7.20**	5.00	6.00	**43.20**	**36.00**
C7	25	1.80	0.4	0.4	0.16	0.288	**7.20**	9.00	6.00	**43.20**	**64.80**
C8	25	1.80	0.4	0.4	0.16	0.288	**7.20**	14.00	6.00	**43.20**	**100.80**
C9	25	1.80	0.4	0.4	0.16	0.288	**7.20**	20.00	6.00	**43.20**	**144.00**
C10	25	1.80	0.4	0.4	0.16	0.288	**7.20**	0.00	10.00	**72.00**	**0.00**
C11	25	1.80	0.4	0.4	0.16	0.288	**7.20**	5.00	10.00	**72.00**	**36.00**
C12	25	1.80	0.4	0.4	0.16	0.288	**7.20**	9.00	10.00	**72.00**	**64.80**
C13	25	1.80	0.4	0.4	0.16	0.288	**7.20**	14.00	10.00	**72.00**	**100.80**
C14	25	1.80	0.4	0.4	0.16	0.288	**7.20**	20.00	10.00	**72.00**	**144.00**
C15	25	1.80	0.4	0.4	0.16	0.288	**7.20**	0.00	15.00	**108.00**	**0.00**
C16	25	1.80	0.4	0.4	0.16	0.288	**7.20**	5.00	15.00	**108.00**	**36.00**
C17	25	1.80	0.4	0.4	0.16	0.288	**7.20**	9.00	15.00	**108.00**	**64.80**
C18	25	1.80	0.4	0.4	0.16	0.288	**7.20**	14.00	15.00	**108.00**	**100.80**
C19	25	1.80	0.4	0.4	0.16	0.288	**7.20**	20.00	15.00	**108.00**	**144.00**
SUM							**136.8**			**1116.0**	**1238.4**

- ✓ The lumped/ total mass at each floor level will be:
  - Ground **= 1636.20KN**
  - First to Sixth Floor = **2631.94 KN**
  - Roof = **249.46 KN**
  - Foundation = **136.80KN**
- ✓ Total weight = W_T = 1632.20 +(2631.94*6) +249.46 +136.80

  = <u>17810.10KN</u>

From the weight calculated above the value of f_b and F_t are

W_T = 17810.10KN

f_b=0.082W

= 0.082*17810.10

f_b**= 1460.42 KN**

F_t=0.05425f_b

$=0.07 \times T_1 \times F_b \leq 0.25 f_b$

= 0.07*0.775*f_b ≤ 0.25f_b = 0.07*0.775*1460.42 ≤ 0.25*712.404

= 79.23 ≤ 178.10

Therefore, we take **F**t **=79.23 KN**

5.2 Story Shear

5.2.1 Story Shear from Earth Quake

	f_b- F_t	h_i	w_i	$w_i h_i$	f_i
Foundation	1381.20	1.80	136.80	246.24	**1.33**
Ground	1381.20	4.80	1636.20	7853.76	**42.35**
1st	1381.20	7.80	2631.94	20529.132	**110.70**
2nd	1381.20	10.80	2631.94	28424.952	**153.30**
3rd	1381.20	13.80	2631.94	36320.772	**195.85**
4th	1381.20	16.80	2631.94	44216.592	**238.40**
5th	1381.20	19.80	2631.94	52112.412	**281.00**
6th	1381.20	22.80	2631.94	60008.232	**323.60**
Roof	1381.20	25.80	249.46	6436.068	**34.70**
sum			**17814.10**	**256,148.16**	

Table Story Shear Force for Earth Quake

5.3 Center of Mass

The center mass of the plan can be determined using the following formula:

$X_m = \sum w_i * X_i / (\sum w)$

$Y_m = \sum w_i * Y_i / (\sum w)$

Therefore the following table gives X_m and Y_m of each floor

	$\sum W$	$\sum wx = M_y$	$\sum wy = M_x$	X_m	Y_m
Foundation	136.80	1238.40	1116.00	9.05	8.16
Ground	1636.20	14506.80	14589.30	8.87	8.92
1st	2631.94	22394.93	22673.48	8.51	8.62
2nd	2631.94	22394.93	22673.48	8.51	8.62
3rd	2631.94	22394.93	22673.48	8.51	8.62
4th	2631.94	22394.93	22673.48	8.51	8.62
5th	2631.94	22394.93	22673.48	8.51	8.62
6th	2631.94	22394.93	22673.48	8.51	8.62
Roof	249.46	2211.72	2048.80	8.87	8.21

Table Calculation of Center of Mass

DESIGN OF SHEAR WALL FOR BUILDING ELEVATOR

Dan Techno Craft Technical Specification

Standard lift shaft and car sizes for center opening electrical traction

Load (kg)	**persons**	**Shaft width(mm**		**Shaft depth (mm)**		**Car width (mm)**	**Car Depth (mm)**	**Clear opening (mm)**	**Pit depth (mm)**	**Over head height**
		min	**max**	**min**	**max**					
320	4	1500	1650	1500	1700	1000	900	700X2000	1500	3600
480	**6**	**1700**	**1850**	**1700**	**1800**	**1050**	**1200**	**800X2000**	**1500**	**3600**
640	8	1700	1850	1780	1950	1100	1400	800X2000	1500	3600

Table Dan Techno Craft Shear Wall Technical Specification

From the above table data, we have just selected the following dimensions.

Capacity of lift =480 Kg

Nº of person to accommodate =6

Shaft width =1700mm

Shaft depth =1700mm

Car width =1200mm

*Clear opening =800*2000*

Pit depth =1500mm

Overhead height =3600mm

Design of Shear Wall

The wall is designed as isolated sway elements of a frame using the second –order theory of columns. /refer: EBCS-2, 1995.section 4.4 /

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 determined.

Lateral Load Determination

Mostly earth quake is the governing lateral load for frame analysis. In the case of frame system lateral forces are resisted by frame action of beams, columns and the rigid joints. While in the shear walls the lateral force is resisted by the wall itself in its major axes.

Base Shear Determination

The base shear is given by the following formula:

$f_b = S_d(T_1)\ W$

Where:

f_b=base shear

W=Total weight of the shear wall

$S_d(T_1)$=ordinate of the design spectrum at period T_1 which is given by

$S_d(T_1)=\alpha\beta\gamma$

Where: α= ratio of the design bed rock acceleration to the acceleration of gravity, which is given by: $\alpha = \propto_0 I$

Where:

$\propto_0$= The bed rock acceleration ratio for the site and depends on the seismic zone.

$\propto_0$=0.1 for earth quake zone-2 (From EBCS-1995, table 1-1)

I=Importance factor of the structure

I=1 (EBCS-8, 1995 table 2-4), thus α=0.1*1=0.1

β = Is the design response factor for the site and is given by

β =1.2S/$T^{(\frac{2}{3})}$ ≤ 2.5

Where:

S=site coefficient of soil characteristics

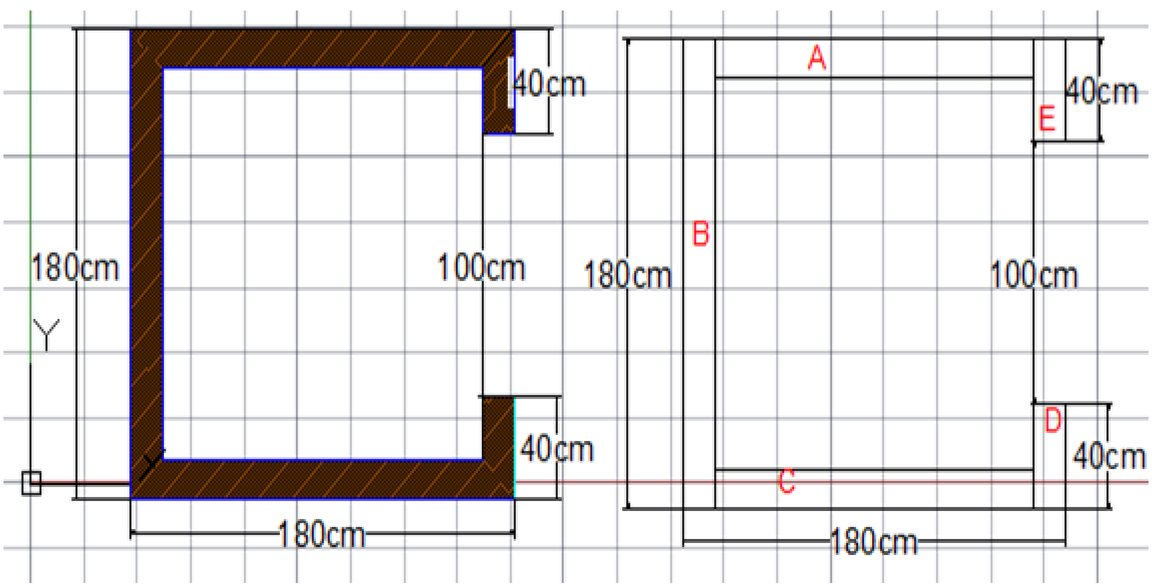
S=1.2 (for sub soil class B, EBCS-8, 1995, table1-2)

T_1= the fundamental period of vibration of the structure (in seconds) for translational motion in the direction of the motion. For structures up to 80m height, the value of T_1 may be approximated by:

T_1=$C_1 H^{3/4}$

Where:

H=Height of the structure above the base in meter=**25m**



b=0.15m

for structures with concrete or masonry shear walls, the value of C_1 may be taken as:

$$C_1 = \frac{0.075}{\sqrt{A_c}}$$

Where:$A_c = \sum(A_i\left(0.2 + (\frac{L_{wi}}{H})^2\right))$

A_c=combined effective area of the shear in the first story of the building, in m^2

Ai=X-sectional area of the shear wall-I in story of the building, in m^2

L_{wi}=length of the shear wall in the first story in the direction parallel to the applied forces, in meters with the restriction that L_{wi}/H shall not exceed 0.9. L_{wi}=2m

Case-1: When the lateral force acting in the Y-axis:

L_{wi} = 1.80m

A_B= 0.15*1.8 = 0.270m^2

A_D= A_E = 0.4*0.15 = 0.06 m^2

$A_c = \sum(A_i\left(0.2 + (\frac{L_{wi}}{H})^2\right))$ = 0.27*[0.2+ $(1.8/3)^2$] + 2*0.06*[0.2+ $(1.8/3)^2$]

A_c = 0.218m^2

Then C1 = 0.075/√A_c = 0.075/√0.218 = **C1 = 0.161**

T_1= $C_1 H^{3/4}$ = 0.161*$(25)^{0.75}$ = **1.794sec.**

$\beta = 1.2S/T^{(\frac{2}{3})} \leq 2.5$ = 1.2*1.2/ $(1.794)^{0.67}$ ≤ 2.5

=0.9733 ≤ 2.5 …… Ok!

Case-1: When the lateral force acting in the X-axis

L_{wi} = 1.80m

A_A = A_C = 0.15*1.5 = 0.225m^2

$A_c = \sum(A_i\left(0.2 + (\frac{L_{wi}}{H})^2\right))$ = 2*0.225*[0.2+ $(1.8/3)^2$] = 0.336 m^2

Ac=0.252 m^2

Again, C1=0.075/ $(0.252)^{0.5}$= 0.1494

T_1= 0.1494*$(25)^{0.75}$ = **1.670sec.**

β =1.2*1.2/ $(1.670)^{0.67} \leq 2.5$

= 1.022 ≤ 2.5………Ok!

To be conservative the maximum value from the two cases is taken

β = 1.022 and T_1= 1.794sec.

γ is the behavior factor to account for energy dissipation capacity

$\gamma = \gamma_0 \times k_D \times k_r \times k_w \leq 0.7$

Where:

γ_0=basic type of behavior factor, dependent on the structural type (EBCS-8, 1995, table 3-2)

= **0.2** (For core system)

K_D=factor reflecting the ductility class =1

K_R=factor reflecting the structural regularity in elevation =1 (frame system)

k_w =factor reflecting the prevailing failure mode in the structural system with wall= 1

$\gamma = \gamma_0 \times k_D \times k_r \times k_w \leq 0.7$ = 0.2*1*1*1 ≤ 0.7

γ = 0.2 ≤ 0.7………Ok!

Therefore, $S_d(T_1)$ =αβγ

= 0.1*1.022*0.2= 0.0204

And, Fb = **$S_d(T_1)$*W = 0.0204W**

W=seismic DL, obtained as the total permanent load plus 25% of the floor live load, for storage and ware house occupancies. In other occupancies, no allowance for live loads need be made. In our case we assumed that the elevator car is being there throughout & it always serves, hence it can be considered as storage occupancies to account the 25% allowance for live loads.

$$F_t = \begin{cases} 0 \text{ for } T<0.7 \\ 0.07\, T_1 F_b \leq 0.25\, f_b \text{ for } T > 0.7 \end{cases}$$

Since T_1=1.554sec

$F_t=0.07\ T_1\ f_b \leq 0.25 f_b$

$= 0.07*1.794*f_b \leq 0.25$

$= 0.126*f_b$

Permanent load calculation for shear walls

FOUNDATION

designation	Unit Wt.	height	t	b	area	volume	weight	x	y	M_x	M_y
SW_1(A)	25	1.80	0.15	1.50	0.23	0.54	13.5	0.90	1.725	23.30	12.15
SW_2(B)	25	1.80	0.15	1.80	0.27	0.65	16.3	0.075	0.90	14.67	14.67
SW_3(C)	25	1.80	0.15	1.50	0.23	0.50	13.5	0.825	0.075	1.01	1.01
SW_4(D)	25	1.80	0.15	0.40	0.06	0.144	3.60	1.725	0.20	0.72	6.21
SW_5(E)	25	1.80	0.15	0.40	0.06	0.144	3.60	1.725	0.02	0.72	6.21
	Total						**50.5**			**40.4**	**40.3**

GROUND FLOOR

designation	Unit Wt.	height	t	b	area	volume	weight	x	y	M_x	M_y
SW_1(A)	25	3.00	0.15	1.50	0.23	0.69	17.3	0.90	1.725	29.84	15.57
SW_2(B)	25	3.00	0.15	1.80	0.27	0.81	20.3	0.075	0.90	18.27	1.53
SW_3(C)	25	3.00	0.15	1.50	0.23	0.69	17.3	0.825	0.075	1.30	14.27
SW_4(D)	25	3.00	0.15	0.40	0.06	0.18	4.50	1.725	0.20	0.90	7.76
SW_5(E)	25	3.00	0.15	0.40	0.06	0.18	4.50	1.725	0.02	0.90	7.76
	Total						**63.9**			**51.21**	**46.89**

FIRST TO Sixth FLOOR

designation	Unit Wt.	height	t	b	area	volume	weight	x	y	M_x	M_y
SW_1(A)	25	3.00	0.15	1.50	0.23	0.69	17.3	0.90	1.725	29.84	15.57
SW_2(B)	25	3.00	0.15	1.80	0.27	0.81	20.3	0.075	0.90	18.27	1.53
SW_3(C)	25	3.00	0.15	1.50	0.23	0.69	17.3	0.825	0.075	1.30	14.27
SW_4(D)	25	3.00	0.15	0.40	0.06	0.18	4.50	1.725	0.20	0.90	7.76
SW_5(E)	25	3.00	0.15	0.40	0.06	0.18	4.50	1.725	0.02	0.90	7.76
	Total						**63.9**			**51.21**	**46.89**

ROOF FLOOR

designation	Unit Wt.	height	t	b	area	volume	weight	x	y	M_x	M_y
SW_1(A)	25	2.2	0.15	1.50	0.23	0.51	12.75	0.90	1.725	22.00	11.48
SW_2(B)	25	2.2	0.15	1.80	0.27	0.59	14.75	0.075	0.90	13.28	11.06
SW_3(C)	25	2.2	0.15	1.50	0.23	0.51	12.75	0.825	0.075	0.96	10.52
SW_4(D)	25	2.2	0.15	0.40	0.06	0.13	3.25	1.725	0.20	0.65	5.61
SW_5(E)	25	2.2	0.15	0.40	0.06	0.13	3.25	1.725	0.02	0.65	5.61
	Total						**46.75**			**37.54**	**44.27**

Therefore, total weight will be:

W = Total permanent load + 25% of floor live load

Live load for storage = 5KN/m^2

$$W = 544.35KN + 5KN/m^2*(1.5m*1.5m)*25\%$$

W = 547.16KN

And base shear will be:

$F_b = S_d(T_1)*W = 0.0204W$

$= 0.0204*547.16 =$ **11.162KN**

$F_t=0.07\ T_1 f_b \leq 0.25 f_b$

$= 0.07*1.794*f_b \leq 0.25$

$= 0.126*f_b = 0.126*11.162 =$ **1.41 KN**

- **Distribution of horizontal seismic forces to each story**

The base shear force is distributed over the height of the structure at each floor level according to the following formula:

$$F_i = \frac{(F_b - F_t)W_i h_i}{\sum_{i=0}^{n} W_i h_i}$$

Story Shear

	f_b - F_t	h_i	w_i	$w_i h_i$	f_i
Foundation	9.752	1.80	50.50	90.90	**0.12**
Ground	9.752	4.80	63.90	306.72	**0.41**
1st	9.752	7.80	63.90	498.42	**0.66**
2nd	9.752	10.80	63.90	690.12	**0.91**
3rd	9.752	13.80	63.90	881.82	**1.16**
4th	9.752	16.80	63.90	1073.52	**1.41**
5th	9.752	19.80	63.90	1265.22	**1.66**
6th	9.752	22.80	63.90	1456.92	**1.91**
Roof	9.752	25.00	46.75	1168.75	**1.53**
sum			**544.55**	**7432.39**	

Table: Story Shear Force

- **Determination of vertical loads**

Shaft roof slab design

$$d = \left(0.4 + 0.6\frac{f_{yk}}{400}\right) \times \frac{L_e}{\beta_a}, \quad \text{where Le = 1500mm}$$

β_a = 33

d = 38.64mm

Therefore, D will be:

D = d + concrete cover + half dia. Of reinforcement

D =38.64 +15 + 7 = 60.64mm

Use D = 100mm to be certain due to un foreseen reasons.

- ✓ **Effective depth off the shorter direction for normal slab**

  d_{used} =D- 7-15 = 100-7-15 = 78mm

- ✓ **Effective depth in the longer direction for normal slab**

  d_{used} = 100-14-7-15 = 64mm

- **Load calculation**

Dead load

- ✓ $15cm\ thick\ RC\ slab = 0.15 \times 25 = 3.75\,KN/m^2$
- ✓ $2.5cm\ plastering = 0.025 \times 23 = 0.575\,KN/m^2$
- ✓ $20mm\ PVC\ floor\ finish = 0.02 \times 16 = 0.32\,KN/m^2$
- ✓ $30mm\ cement\ screed = 0.03 \times 23 = 0.69\,KN/m^2$
- ✓ From self-weight of elevator car =$(2*320*9.81*10^{-3})/(1.8*1.8)$ =1.938KN/m^2

Total dead load excluding self weight of the elevator car = 3.75 +0.575 +0.32 +0.69 =**5.335** KN/m^2

P_d = 1.3*5.335 + 1.6*5 = 14.94 KN/m^2(excluding elevator car weight)

20% of p_d will be: 0.2*14.94 = 2.988 KN/m^2

And factored dead load for elevator will be:

= 1.3*1.938 = 2.519 KN/m^2 $\leq$ 20% Pd = 2.988 KN/m^2

Therefore, we simply distribute the load from the elevator car on the area of the slab

Total dead load = 5.335KN/m^2 + 1.938 KN/m^2 = 7.273 KN/m^2

Live load for storage = 5.0 KN/m^2

- **Design load and load combination**

We have only dead load and live load to be combined

Pd = 1.3DL + 1.6 LL = 1.3*7.273 + 1.6*5.0 = 17.50 KN/m^2

- **Analysis of the slab**

The slab is two way slab (L_y/L_x = 1.8/1.8 = 1.0

The analysis of slab moments of two way slabs is accomplished by the formula

$$\boldsymbol{m_i = \alpha_i P_d L_x{}^2}$$

Where $m_i = the\ design\ moment\ per\ unit\ width\ at\ the\ point\ of\ reference$

$\alpha_i = the\ coefficient\ given\ in\ Table\ A-1\ in\ EBCS2-1995.$

$P_d = the\ design\ load$

$L_x = the\ length\ of\ the\ shorter\ span\ of\ the\ of\ the\ panel$

And for slabs simply supported in all four sides (PCT = 1) and with span ratio of 1.0 and the value of moment coefficients will be as follows:

- ✓ α_{xf} = 0.024
- ✓ α_{yf} = 0.024

And the moments will be:

$$m_i = \alpha_i P_d L_x^{2}$$

m_{xf} = 0.024*17.50*1.8^2 = **1.40KN.m**

m_{yf} = 0.024*17.50*1.8^2 = **1.40KN.m**

- **Check depth for flexure**

Depth requirement for ultimate flexural strength of concrete compression stress capacity

$\mathbf{d} \geq \sqrt{}M/0.2952f_{cd}b_w = \sqrt{}(1.40*10^6)/0.2952*11.33*1000$

$d \geq 20.5$ mm but d_{used} = 78mm ≥ 20.5mm

- **Flexural reinforcement design**

$\rho = \{1 - [1 - 2M/(bd^2f_{cd})]^{0.5}\}fcd/f_{yd}$

$= \{1 - [1 - 2*1.4*10^6/(1000*78^2*11.33)]^{0.5}$ = **0.0009**

But, $\rho_{min} = 0.50/f_{yk}$ =0.5/300 = **0.0017**

Therefore, $\alpha A_s = \rho bd$ = 0.0017*1000*78

= 132.60mm^2

Assume dia. 10 mm deformed bar

- **Spacing:** **S** = 1000a_s/A_s =(1000*3.14*25)/133 = 590.22 mm = 590mm
- $S_{max} = \min\begin{cases} 2D \\ 350 \end{cases}$ = 2*100=200mm or 350mm, then Spacing will be **S = 200 mm**

Therefore, provide Ø10c/c200mm the same in both directions

- **Load transfer to the wall**

Based on the coefficient method for two way solid slabs the values of shear distribution factors are as given below:

$V_x = \beta_{vx} \times P_d \times L_x$= 0.33*17.50*1.80 = 10.395KN/m

$V_y = \beta_{vy} \times P_d \times L_x$ = 0.33*17.50*1.80 = 10.395KN/m

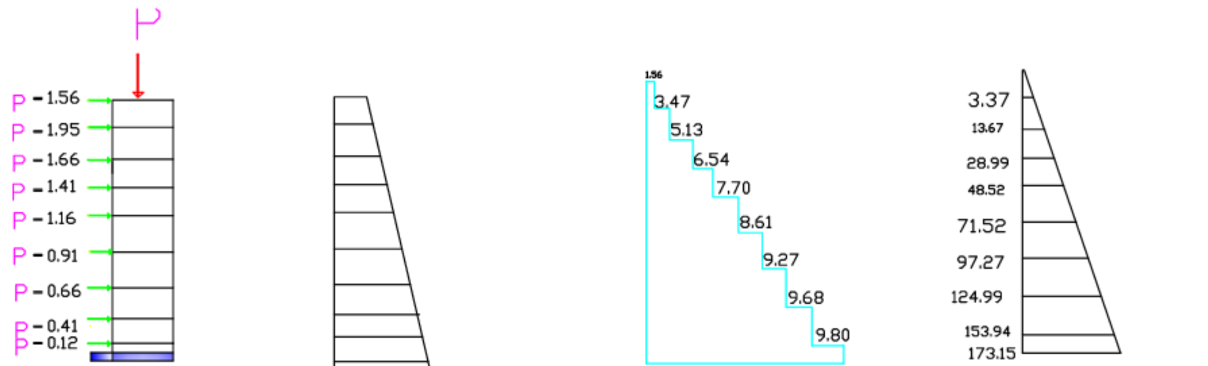
Hence the total vertical loads at the bottom of shear walls become:

$N_{sd} = P_d$ + wall weight at the bottom

And Pd for each wall is determined as follows:

- ✓ Wall-A : Pd = 10.395KN/m*1.5m = 15.59KN
- ✓ Wall-B: Pd = 10.395KN/m*1.8m = 18.71KN
- ✓ Wall-C: Pd = 10.395KN/m*1.5m = 15.59KN
- ✓ Wall-D: Pd = 10.395KN/m*0.4m = 4.16KN
- ✓ Wall-E: Pd = 10.395KN/m* 0.4m = 4.16KN

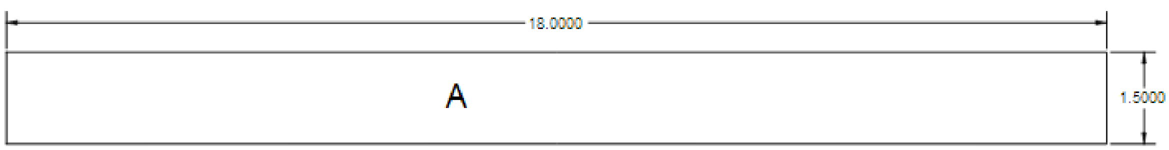
designation	**Area**	**Volume**	**Weight**	**P_d**	**N_{sd}**
A	0.225	5.625	140.625	15.59	**156.215**
B	0.270	6.750	168.750	18.71	**187.460**
C	0.225	5.625	140.625	15.59	**156.215**
D	0.06	1.500	37.500	4.16	**41.660**
E	0.06	1.500	37.500	4.16	**41.660**



shear wall diagram AXial force diagram shear force diagram moment diagram

- **Design of individual wall section**

✓ Design Wall-A



b = 0.15 and h = 1.80

Determination of design eccentricity in both directions

$$e_{tot} = e_a + e_o + e_2$$

Accidental (additional) eccentricity due to various imperfections

$e_a = L_e/300 \geq 20$mm …….EBCS-2/1995 Section 4.4.3

Where: L_e = is effective buckling length of the wall and assuming the top end of the shear wall to be simply supported.

$L_e = 0.7L$ where L is the wall height

$= 0.7*25 = 17.50$m

Then, $e_a = L_e/300 = 17.50/300 = 0.058*1000 = 58.33$mm

- **Determination of design eccentricity in H- direction**

- ✓ First order eccentricity:

$$e_o = M_d/ \mathbf{N_{sd}} = 173.15KN.m/156.215KN = \mathbf{1.11m}$$

- ✓ Second order eccentricity:

$$e_2 = 0.4h\,(L_e/10h)^2 = 0.4*1.8(17.5/10*1.8)^2 = \mathbf{0.681m}$$

- ✓ Total eccentricity:

$$e_{tot} = e_a + e_o + e_2 = 58.33 + 1110 + 681 = 1849.33mm = 1.85m$$

- ✓ Relative eccentricity: the relative eccentricity for the given direction is the ratio of the total eccentricity to the column width in the same direction

$$e_{rel} = e_{tot}/h = 1.85m/1.80 = 1.03m$$

- **Determination of design eccentricity in B- direction**

- ✓ First order eccentricity: no moment is carried in this direction as it is carried by the perpendicular walls, $M_d = 0$

$$e_o = M_d/ \mathbf{N_{sd} = 0}$$

- ✓ Second order eccentricity:

$$e_2 = 0.4h\,(L_e/10h)^2 = 0.4*1.8(17.5/10*1.8)^2 = 0.681m$$

- ✓ Total eccentricity:

$$e_{tot} = e_a + e_o + e_2 = 58.33 + 0 + 681 = 739.33mm = 0.739m$$

- ✓ Relative eccentricity:

$$e_{rel} = e_{tot}/b = 0.739m/0.15 = 4.93m$$

- ✓ Relative eccentricity ratio(k)

$$K = \text{Small } e_{rel}/\text{large } e_{rel} = 1.03/4.93 = 0.21$$

Equivalent eccentricity:

$$e_{equ} = e_{tot}(1+k\alpha) =$$

- ✓ Relative Normal force, $V = N_{sd}/f_{cd}*A_w = 156.215/(11.33*1800*150)$

V = 0.051

V	0	0.2	0.4	0.6	0.8	≥1.0
α	0.6	0.8	0.9	0.7	0.6	0.5

Then by interpolating for α = 0.65

$e_{equ} = e_{tot}(1+k\alpha) = 1.85(1+0.21*0.65) = 2.10m$

- **Design moment calculation**

Design moment, $M_{sd} = e_{equ} * N_{sd} = 2.10*156.215 =$ **328.50KN.m**

✓ **Design of vertical reinforcement**

$\mu = M_{sd} \div (f_{cd}*A*h) = 328.50*10^3.m \div (11.33N/mm^2*1800mm*150mm*1.80m)$

= **0.06**

ϖ = is the reinforcement ratio from chart using v and μ values.

= **0.3** ……….from chart biaxial No.43

✓ **Area of reinforcement**:

$A_{min} = 0.004A_c = 0.004*1800*150 = 1080mm^2$

$A_{max} = 0.04A_c = 0.04*1800*150 = 10800mm^2$

But, $A_s = (\varpi*f_{cd}*A_c) \div f_{yd} = (0.3*11.33*1800*150) \div 260.87 = 3518mm^2 \leq A_{max} = 10800mm^2$

Therefore, using two rows of bars (That is providing reinforcement at each face or internal and external face) of the wall. The area of steel reinforcement on each side wall will be:

$\mathbf{A_s = 3518mm^2/2 = 1759mm^2}$

✓ **Spacing of the vertical bars**

- The diameter of the vertical bars should not be less than 8mm
- The spacing of vertical bars should not exceed twice the wall thickness nor 300mm

$S = (b*a_s) \div A_s = (1800*3.14*49) \div 1759 = 157.44 = 155mm$

Therefore, provide **two Φ14 C/C 150mm** vertical reinforcement bars on each face of the wall.

- ✓ **Design of shear reinforcement**
- ✓ Check the diagonal compression failure of concrete

Section resistance, $V_{rd} = 0.25f_{cd}b_w d \geq V_d = 0.25*11.33*150*(1800\text{-}180) = 688.30KN$

$=688.30\ KN \geq V_d = 9.80KN$………..OK!

- ✓ Check the section capacity, V_c

$$V_C = 0.25f_{ctd}\ k_1\ k_2\ b_w\ d + V_{cn}$$

Where, $V_{cn} = (0.10b_w\ d^*\ N_{sd})/A_c = [0.10*150*(1800\text{-}180)*156.215] \div (1800*150)$

$= 14.06KN$

$K_1 = 1.6\text{-}\ d = 1.6\text{-}\ 0.18 = 1.42m$

$K_2 = 1 + 50\rho \leq 2.0$ but $\rho = A_s \div (b_w\ d) = 3518 \div (150(1800\text{-}180)) = 0.015$

$= 1 + 50*0.015$

$= 1.75$

Then $V_c = 0.25*1.03*1.42*1.75*150(1800\text{-}180) + 14.06$

$V_C = 169.60KN \geq 9.80KN$………OK

- ✓ **Area of shear reinforcement**

According to EBCS-2 section 6.2.1.2 the area of horizontal reinforcement shall not be less than one-half of the vertical reinforcement

Therefore, provide two stirrups and the area of shear reinforcement will be

$$A_s = 3518mm^2 \div 4 = 879.50mm^2$$

The diameter of the horizontal bars shall not be less than one quarter of the vertical reinforcement bars.

- ✓ **Spacing of shear reinforcement**

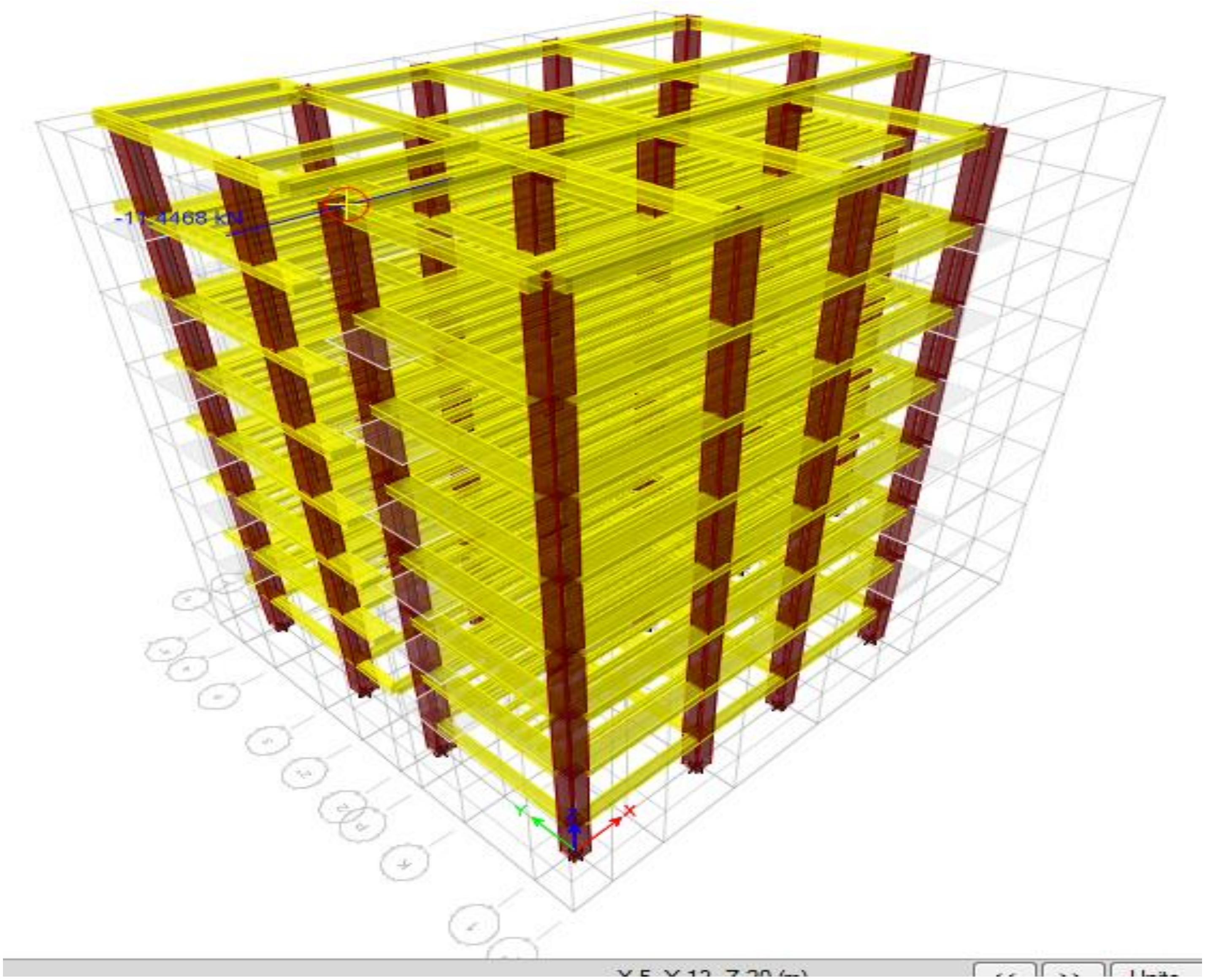
S =($a_{s*}\ b) \div A_s = (1800*3.14*36) \div 879.50 = 213.50mm^2$

Therefore, provide Φ12 C/C 210mm

<u>**NOTE**</u>**: The design of the other shear wall sides will be done as the above procedures.**

Chapter 6

6.1 Beam design and analysis



Beams are flexural members which are used to transfer the loads from slab to columns. Basically beams should be designed for flexure (moment). Furthermore it is essential to check and design the beam sections for torsion and shear..

In our case a particular beam axis is selected for design. The following output data is taken from 3-D analysis of the frame using ETABS.

The final design of a structure must be for the most unfavorable combination of loads that the structure is to support. However, some judgment is necessary in selecting loading conditions that can reasonably be combined. The load combinations that must be considered are normally specified in codes and standards(Ethiopian building code)

The output is shown only for severe maximum and severe minimum, which were used as basis for design of the frame elements. Hence output shows the envelope for the desired action. For sake of clarity and readability the loading and analysis results are presented in sample graphicall

Material Property Data - General

Name	Type	Dir/Plane	Modulus of Elasticity	Poisson's Ratio	Thermal Coefficient	Shear Modulus
C25	I so	All	29000000.000	0.2000	9.9000E-06	12083333.333

Material Property Data - General

Material Property Data - Mass & Weight

Material Property Data - Mass & Weight

Name	Mass per Unit Volume	Weight per Unit Volume
C25	2.5000E-09	2.5000E-05

Material Property Data - Concrete Design

Material Property Data - Concrete Design

Name	Lightweight Concrete	Concrete f_c	Rebar f_y	Rebar f_{ys}	Lightweight Reduc. Factor
C25	No	20.000	300.000	300.000	N/A

Frame Section Property Data - Concrete Columns

Frame Section Name	Material Name	Column Depth	Column Width	Rebar Pattern	Concrete Cover	Bar Size	Corner Bar Size
C60X60	C25	200.000	400.000	RR-4-2	25.000	20d	20d
C30X30	C25	400.000	200.000	RR-2-4	25.000	20d	20d

Frame Section Property Data - Concrete Beams

Frame Section Name	Material Name	Beam Depth	Beam Width	Top Cover	Bottom Cover
TTB30X40	C25	400.000	300.000	25.000	25.000
IMB50X40	C25	500.000	400.000	25.000	25.000
GB40X30	C25	400.000	300.000	25.000	25.000
RIB22X15	C25	220.000	150.000	15.000	15.000

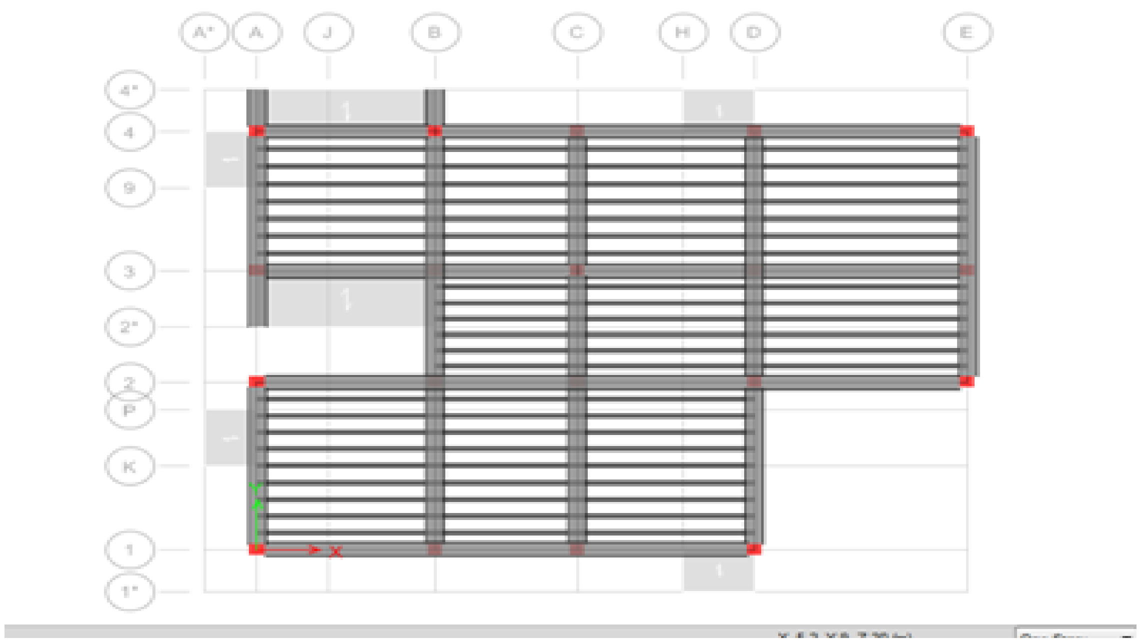
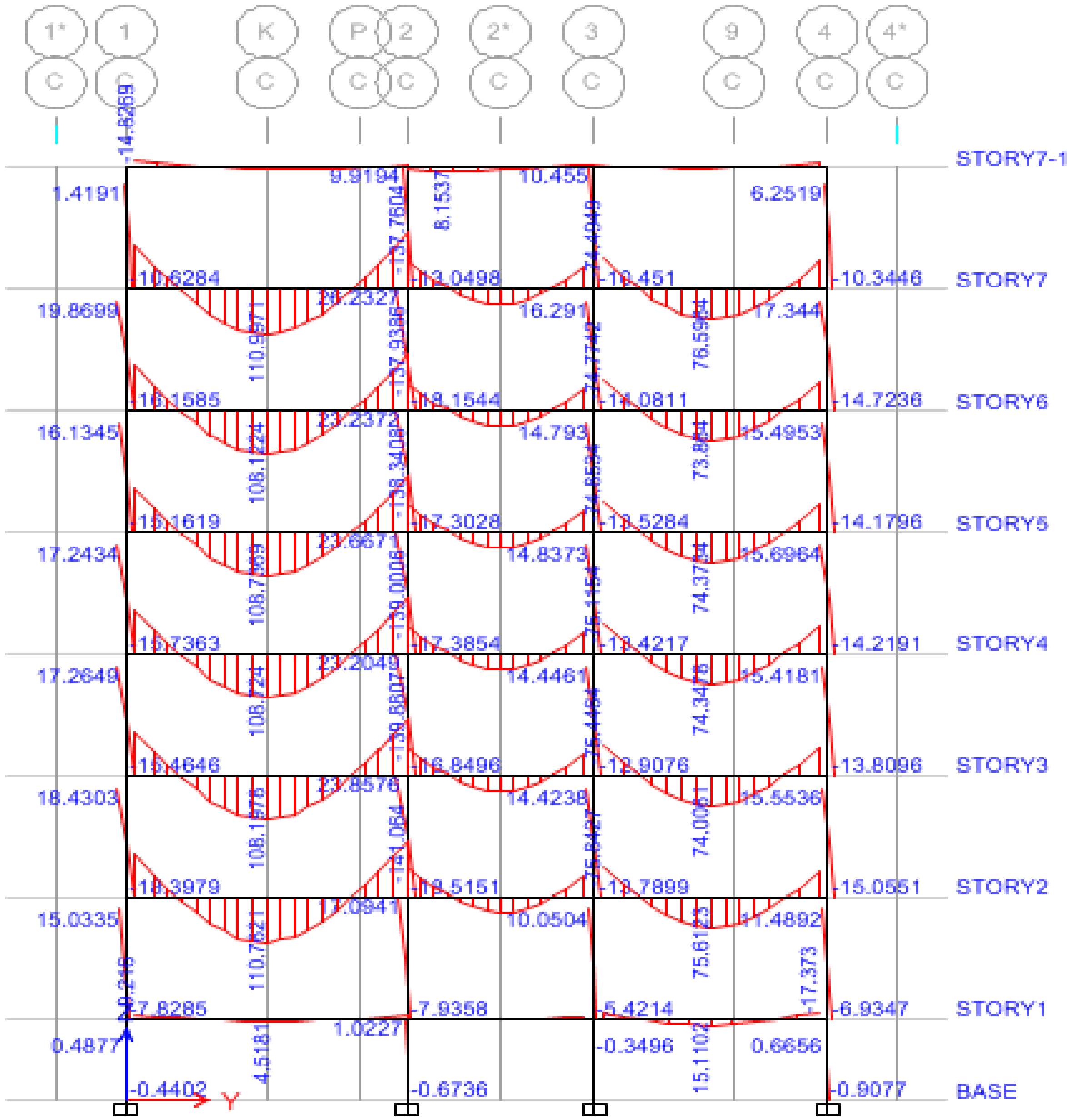
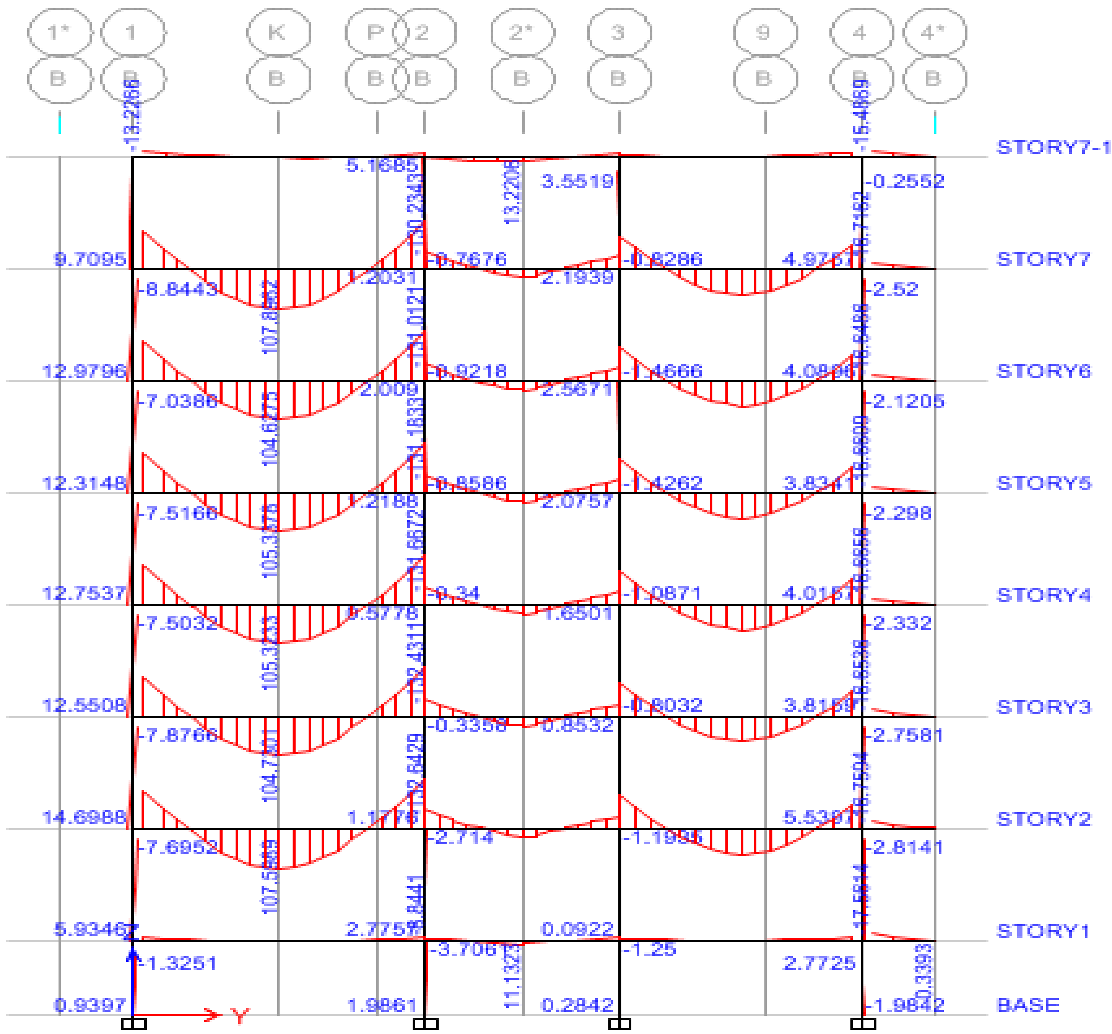


Fig. Typical Floor Layout

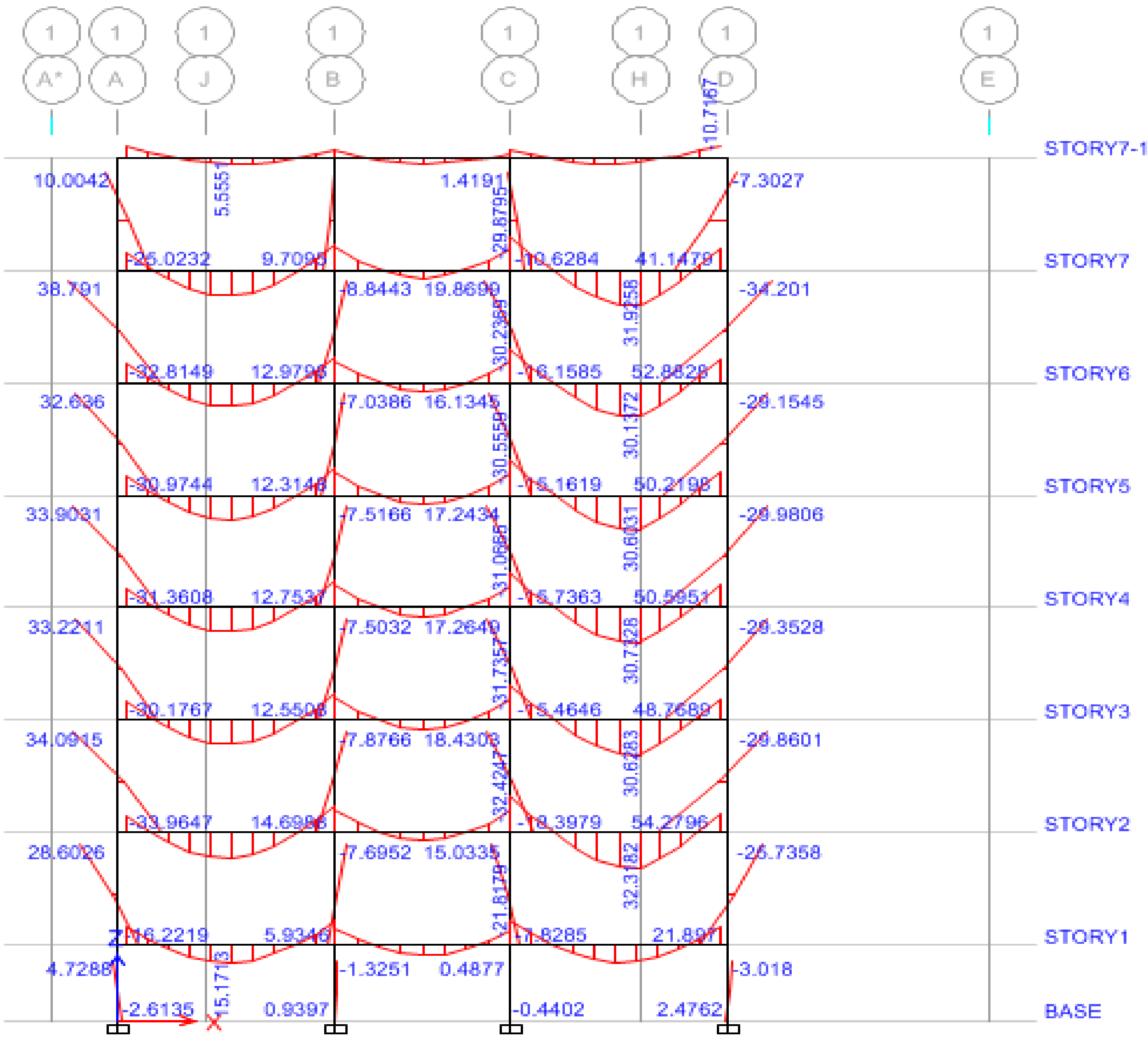
Axis -C Moment 3-3 Diagram in Y- direction



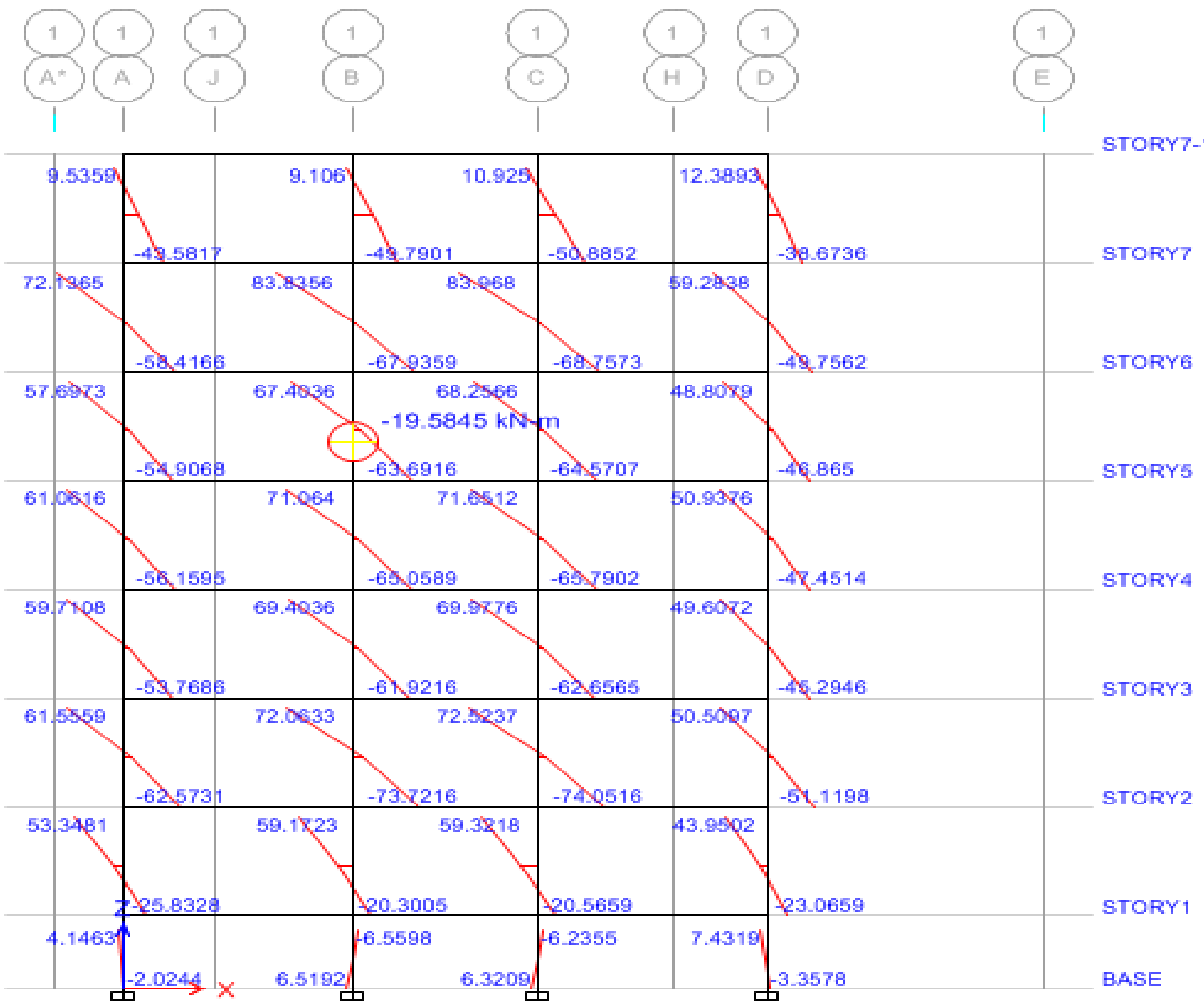
Axis -B Moment 3-3 Diagram in Y- direction



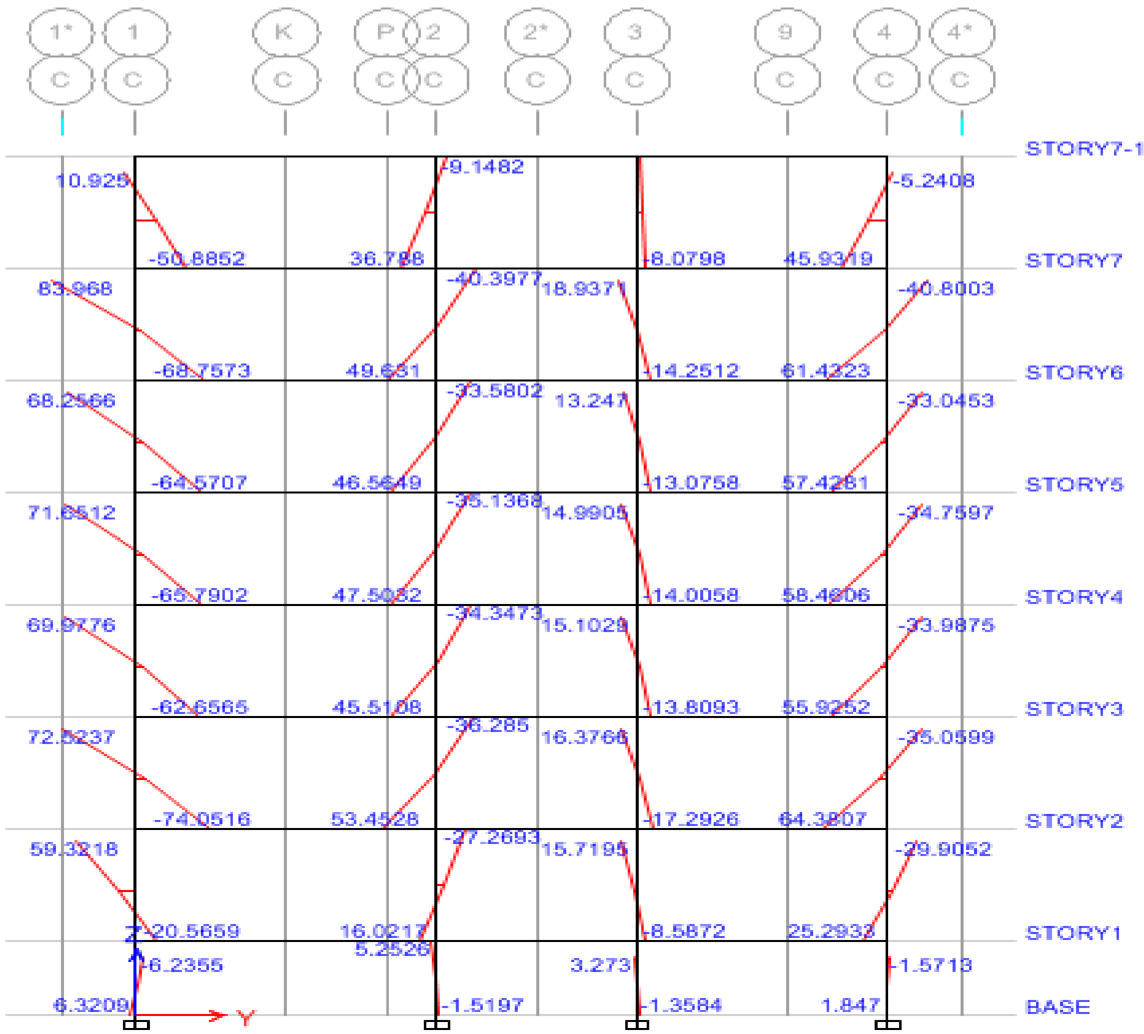
Axis -1 Moment 3-3 Diagram in X- direction



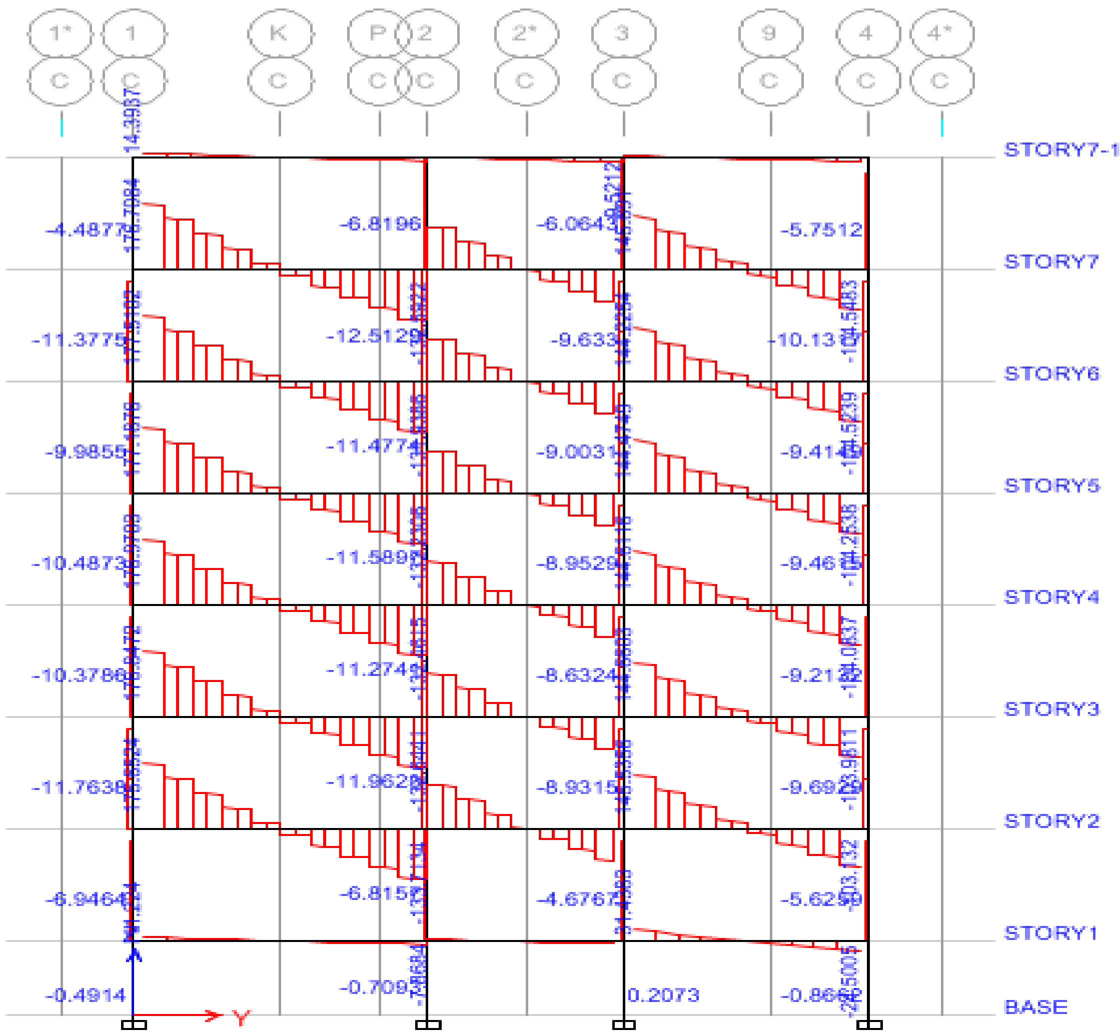
Axis -1 Moment 2-2 Diagram in X- direction



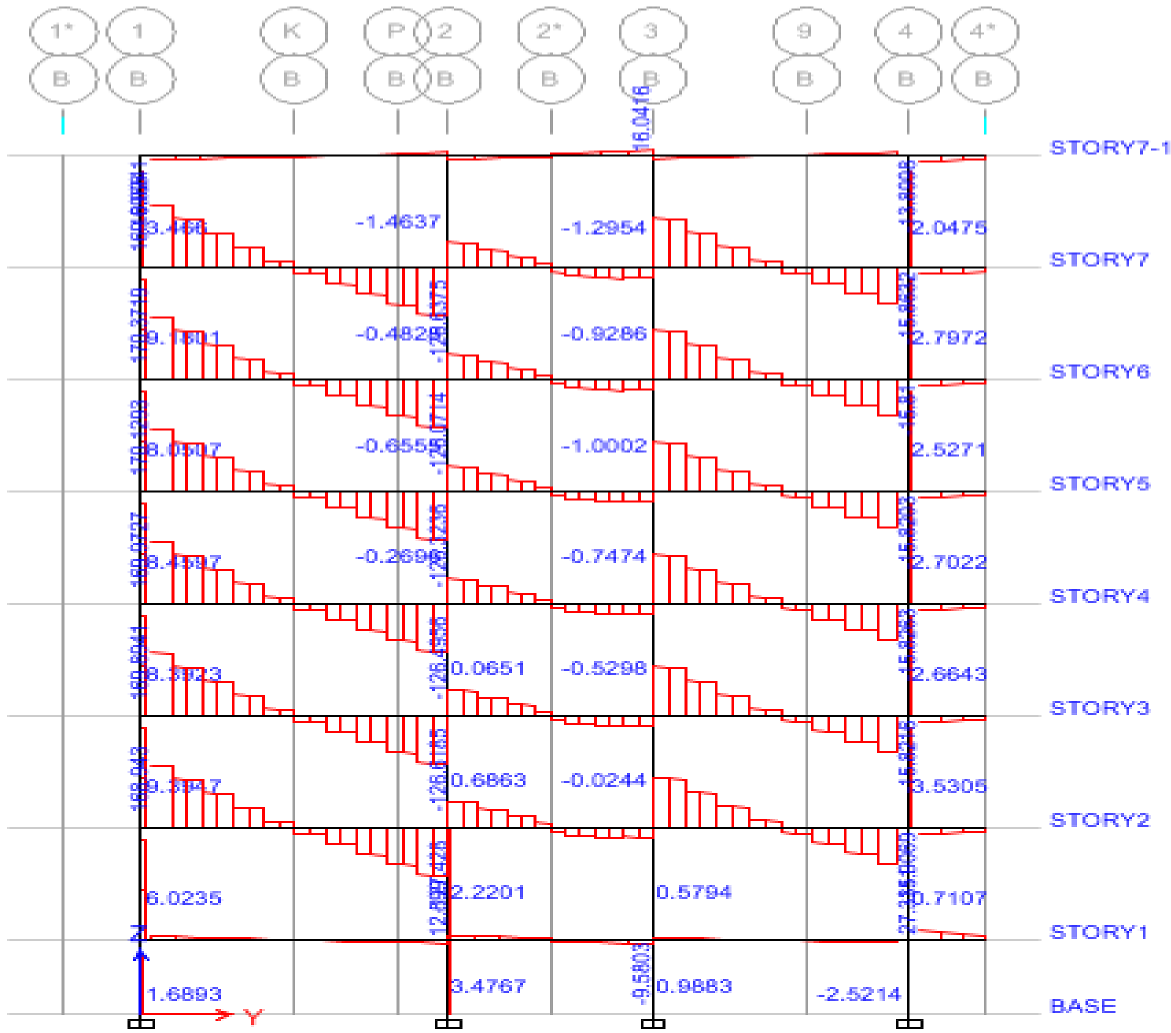
Axis -A Moment 2-2 Diagram in Y- direction



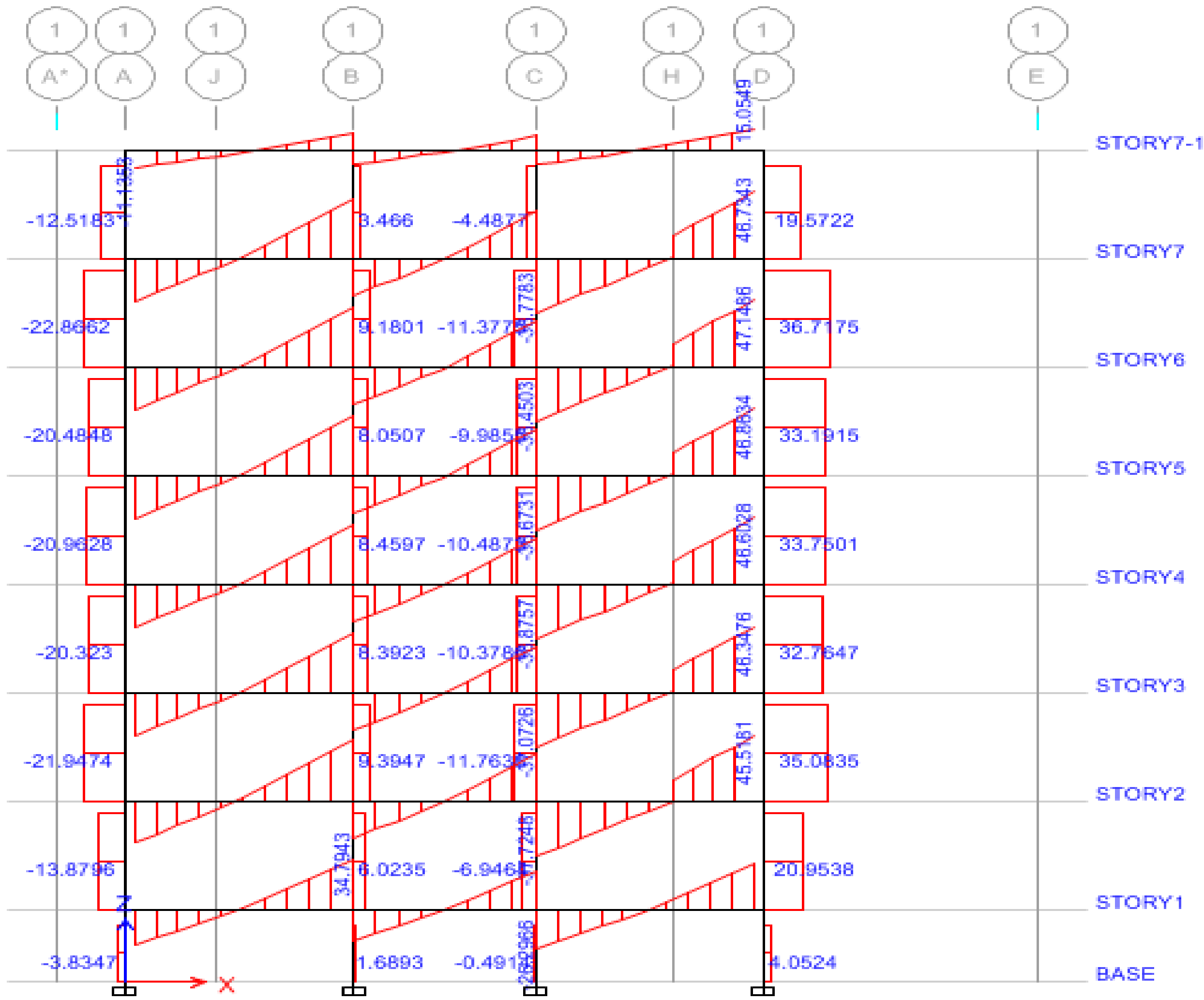
Axis -C Shear force 2-2 Diagram in Y- direction



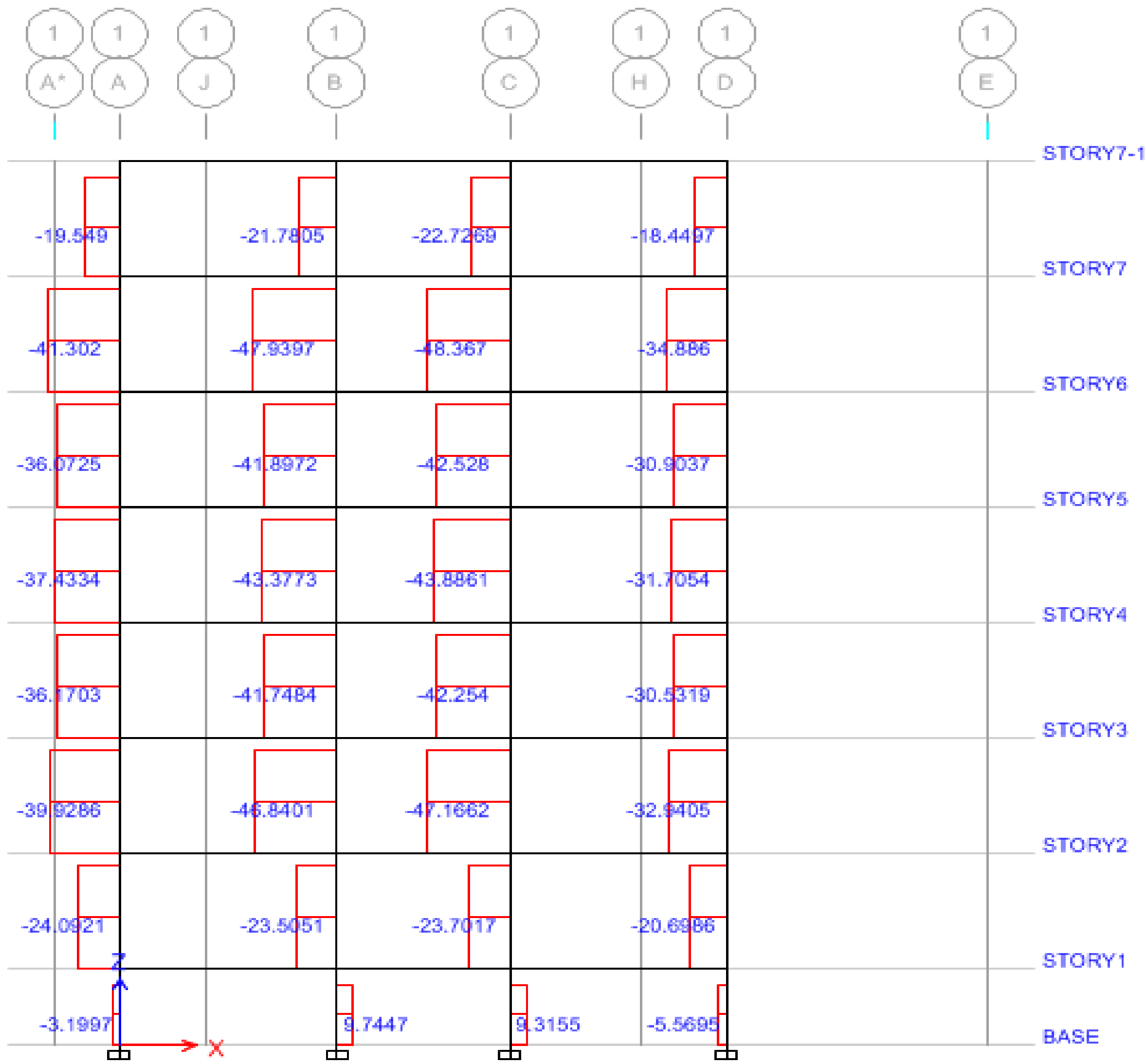
Axis -B Shear force 2-2 Diagram in X- direction



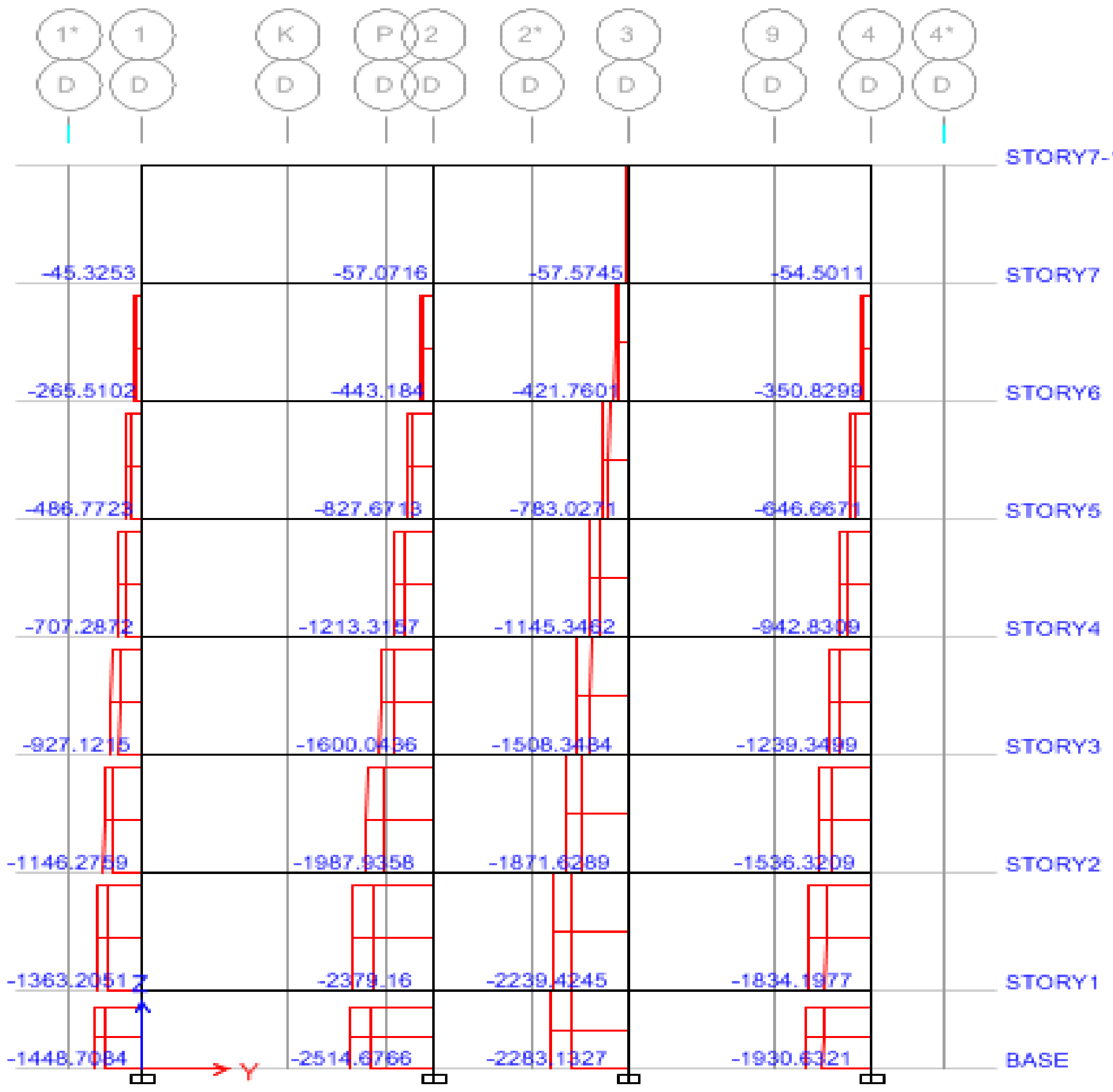
Axis -1 Shear force 2-2 Diagram in Y- direction



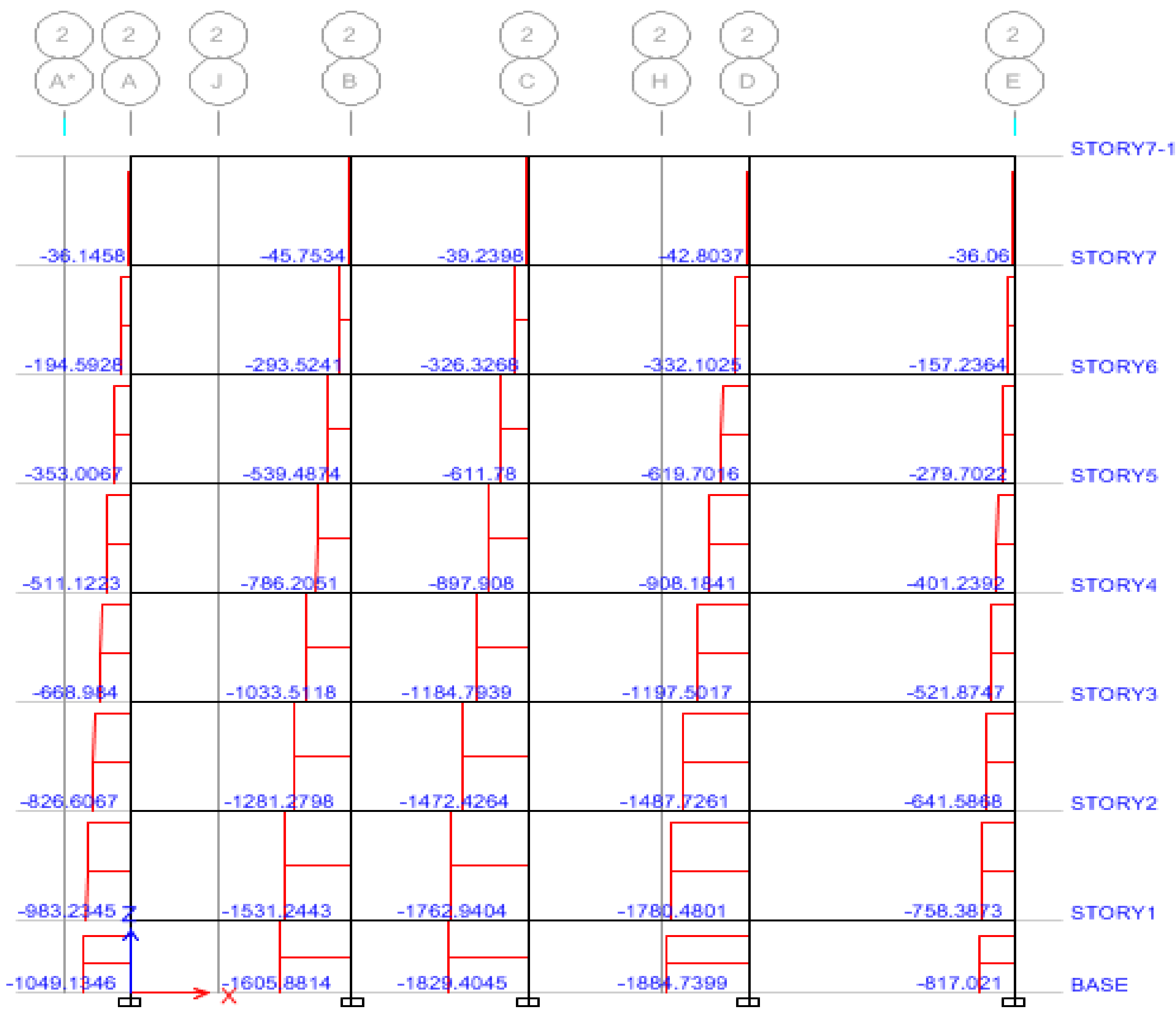
Axis -1 Shear force 3-3 Diagram in X- direction



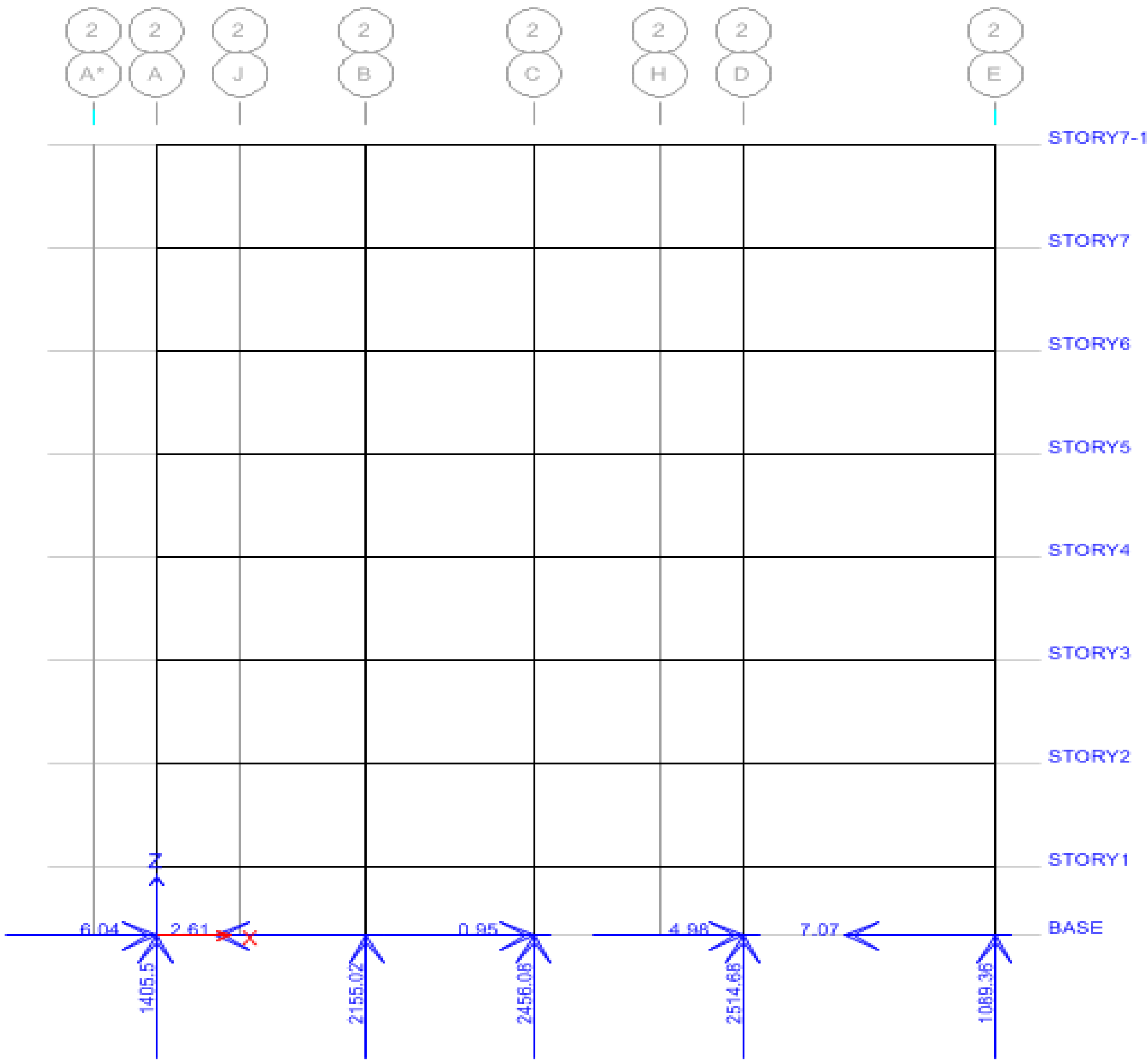
Axis -D Axial force Diagram in Y- direction



Axis -2 Axial force Diagram in X- direction



Axis-2 Restraint reactions



SUMMARY OF SAMPLE BEAM ANALYSIS RESULT FOR CRITICAL SECTIONS ALONG Y- DIRECTION

Story	Axis	Support	Span beteween	Max.support moment	Max.span moment	Shear force
3.0	C (Y-direction)	1	1&2	-99.95	108.2	176.84
		2	2&3	-139.9	37.7	132.64
		3	3&4	-75.50	74.00	144.66
		4	-	-69.4	-	103.98
7.0	C (Y-direction)	1	1&2	-106.00	111.00	85.16
		2	2&3	-137.8	35.50	115.50
		3	3&4	-74.5	76.60	176.7
		4	-	-67.70	-	145.00

SUMMARY OF SAMPLE BEAM ANALYSIS RESULT FOR CRITICAL SECTIONS <u>ALONG X-DIRECTION</u>

Story	Axis	Support	Span between	Max.support moment	Max.span moment	Shear force
3.0	1 (X-direction)	A	A&B	16.60	21.30	28.60
		B	B&C	22.50	7.90	42.08
		C	C&D	31.50	30.40	36.50
		D	D&E	19.00	-	46.06

6.1.1 Design of flexural & Shear Reinforcement

Based on EBCS-2, 1995 Art 7.2.1.1

- The geometric ratio of reinforcement ρ at any section of a beam where positive reinforcement is required by analysis shall not be less than.

$$\rho_{min} = \frac{0.6}{fyk} = 0.6/300 = 0.002$$

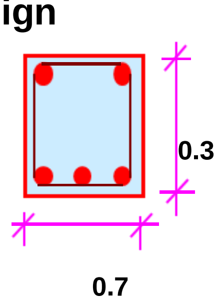
$$f_{cd} = \frac{0.85\,fck}{rc},\ fck = \frac{fcu}{1.25} = \frac{25}{1.25} = 20\text{MPa}$$

Using f_{yk} = 300MPa f_{yd}= 260.87Mpa

- ✓ The maximum reinforcement ratio ρ_{max} for either tension or compression reinforcement shall be 0.04

  ρmax=0.04
- ✓ In T- beam joints where the web in tension the ratio ℓ shall be computed for this purpose using width of Web.
  - **3rd and 7th Floor Main Girder Beam on axis-C (the same on both stories)**

Floor Beam Design



d = 0.265m m

Negative Moment	**99.95**	**KN-m**
Positive Moment	**108.2**	**KN-m**
Shear	**176**	**KN**

Calculate Reinforcement

-

Concrete - Grade C-25					Steel		
Fck	20,000	kpa	[EBCS 2 Table 2.3]		Fyk	300,000	kpa
Fctk	1,500	kpa	[EBCS 2 Table 2.4]		PS	1.15	
PS Factor	1.5		[EBCS 2 Table 3.1]		Fyd	260,870	kpa
Fcd	11333.33	kpa			Es	200,000	kpa
Fctd	1,000	kpa			ρ (min) =	0.002	
Ecm	29,000	kpa	[EBCS 2 Table 2.5]		ρ (max) =	0.0165	

$$\rho_{req} = \frac{1}{m}\left[1 - \sqrt{1 - \frac{2mR_n}{f_{yd}}}\right] \qquad m = \frac{f_{yd}}{f_{cd}} \qquad R_n = \frac{M_u}{bd^2}$$

Negative Reinforcement

m = 23.02
Rn = 2033.2604
ρ = 0.0087
As = 0.00161
As prvd = 0.0004021
Capacity = 27.11
Reserve = -72.88%

Positive Reinforcement

m = 23.02
Rn = 2201.08
ρ = 0.00947
As = 0.00176
As prvd = 0.0003
Capacity = 20.88
Reserve = -80.71%

Shear Capacity of Concrete

$$V_c = 0.25\, f_{ctd} k_1 k_2 b_w d$$

k_1 = 1.108

k_2 = 1.335

V_c = **68.62 KN**

Design Shear Reinforcement

$$s = \frac{A_v d f_{yd}}{V_s}$$

V_s = 107.38

Use Dia= 8 $\quad A_v$ = 0.000101

s = 0.06 m

Max S = 0.200 m Use **0.065**

Support Reinforcement Design for main girder beam (critical axis)

support	Supp.mom	R_n	ρ_{min}	ρ_{cal}	A_{scal}	# of bars	# of bars	Axis
1	99.95	1586.5	0.002	0.0066	1380	4.39	5Φ20	Axis C and Story-3
2	139.90	2220.6	0.002	0.0096	2010	6.40	7Φ20	
3	75.50	1198.4	0.002	0.0049	1020	5.08	6 Φ16	
4	69.40	1101.6	0.002	0.0045	930	6.05	7 Φ14	
1	106.00	1682.5	0.002	0.0070	1470	4.68	5 Φ20	Axis C and Story-7
2	137.80	2187.3	0.002	0.0094	1970	6.27	7 Φ20	
3	74.50	1182.54	0.002	0.0048	1010	5.03	6 Φ16	
4	67.7.	1074.6	0.002	0.0043	910	5.91	6 Φ14	

Span Reinforcement Design for main girder beam (critical axis)

support	Span.mom	R_n	ρ_{min}	ρ_{cal}	A_{scal}	# of bars	# of bars	Axis
1&2	108.20	1717.5	0.002	0.0072	1510	4.81	5Φ20	Axis C and Story-3
2&3	37.70	598.40	0.002	0.0024	500	3.25	4 Φ14	
3&4	74.00	1174.60	0.002	0.0048	1000	4.98	5Φ 16	
-	-	-	-	-	-	-	-	
1&2	111.00	1761.90	0.002	0.0074	1550	4.94	5Φ20	Axis C and Story-7
2&3	35.50	563.50	0.002	0.0022	470	3.05	4Φ14	
3&4	76.60	1215.80	0.002	0.0049	1040	5.18	6Φ16	
-	-	-	-	-	-	-	-	

Support Reinforcement Design for girder beam parallel to Ribs (critical axis)

support	Supp.mom	R_n	ρ_{min}	ρ_{cal}	A_{scal}	# of bars	# of bars	Axis
A	16.60	337.70	0.002	0.0013	530	3.45	4Φ14	Axis C and Story-3
B	22.50	457.70	0.002	0.0018	530	3.45	4Φ14	
C	31.50	640.80	0.002	0.0025	470	3.05	4Φ14	
D	19.00	386.50	0.002	0.0015	530	3.45	4Φ14	

Span Reinforcement Design for girder beam parallel to Ribs (critical axis)

support	Span.mom	R_n	ρ_{min}	ρ_{cal}	A_{scal}	# of bars	# of bars	Axis
A&B	21.30	433.30	0.002	0.0017	530	3.45	4Φ14	Axis C and Story-3
B&C	7.90	160.70	0.002	0.0006	530	3.45	4Φ14	
C&D	30.40	617.42	0.002	0.0024	450	2.9	3Φ14	
-	-	-	-	-	-	-	-	

6.2 COLUMN DESIGN

Columns are vertical members which support both the axial loads and the moment from beams and slabs to the foundation. Columns are primary compression members and also they have to resist bending forces due to some eccentricity.

For the purpose of design calculations, structural members may be classified as Sway or Non-sway depending on their sensitivity to second order effect due to lateral displacements.

- ✓ Our code EBCS 2, 1995 suggests that a frame may be classified as non sway for a given load case if the critical load ratio $\frac{N_{sd}}{N_{cr}}$ for that load case satisfied the criteria:

$$\frac{N_{sd}}{N_{cr}} \leq 0.1 \quad \text{(EBCS-2, 1995, Art 4.4.4.2 (3))},$$

Where Nsd - design value of total vertical load

N_{cr} - critical value (critical buckling load) for the failure in sway mode

- ✓ The buckling load of a story may be assumed to be equal to that of substitute beam column frame defined in the figure4.6 EBCS 2, 1995 and may be determined from the equation

$$N_{cr} = \frac{\Pi^2 EI_e}{L_e^{\ 2}} \quad \text{EBCS-2, 1995, Art 4.4.12 (1)}$$

Where EI_e – is the effective stiffness of the substitute column

L_e – is the effective length

- ✓ The effective stiffness EI_e may be taken as $EI_e = 0.2E_cI_c + E_sI_s$ EBCS-2, 1995, Art 4.4.12(2)

Where Ec = 1100fcd

Es = modulus of elasticity of steel

Ic, Is = Moment of inertial of concrete and reinforcement sections respectively of the substitute column with respect to the centroid of the concrete section.

- ✓ The effective length of a column (Le) may be defend as that height which corresponds to the height of a pin ended column which can carry the same axial load, or alternatively the effective length or height may be considered as the height between pointes of contra flexural of the buckling column
- ✓ According to EBCS-2, 1995 art 4.4.7.1 the effective buckling length of a compression member in a given plane may be obtained from the following

For sway mode the slenderness ratio of the column (Le/L)

$$\frac{L_e}{L} = \sqrt{\frac{7.5 + 4(a_1 + a_2) + 1.6a_1a_2}{7.5 + a_1 + a_2}} \geq 1.15$$ EBCS-2, 1995, Art 4.4.7

Or conservatively $\frac{L_e}{L} = \sqrt{1 + 0.8a_m} \geq 1.15$

For non sway mode

$$\frac{le}{l} = \frac{\alpha m + 0.4}{\alpha m + 0.8} \geq 0.7$$

Where α_1 and α_2 stiffness coefficients and are obtained form

$$a_2 = \frac{k_1 + k_c}{k_{11} + k_{12}} \qquad a_2 = \frac{k_2 + k_c}{k_{21} + k_{22}} \qquad a_m = \frac{a_1 + a_2}{2}$$

Where:- K_1 and K_2 are column stiffness coefficients (EI/L)

K_c is the stiffness coefficient (EI/L) of the column being designed

Kij is the effective beam stiffness coefficient (EI/L)

β = 1.0 opposite end elastically or rigidly restrained

= 0.5 opposite end free to rotate

= 0 for a cantilever beam

a= 1.0 if a base is designed to resist the column moment

6.2.1 Design Steps of column

1. Find design moment and design load (from ETABs)
2. Calculate the moment of inertia (take column inertia as reference value)
3. Calculate stiffness of the materials(EI/L) such as beam and column
4. Sway or non-sway analysis,

If $\frac{N_{sd}}{N_{cr}} \leq 0.1$ (EBCS-2, 1995) => non sway frame (i.e ignore the pΔ(second order effect)

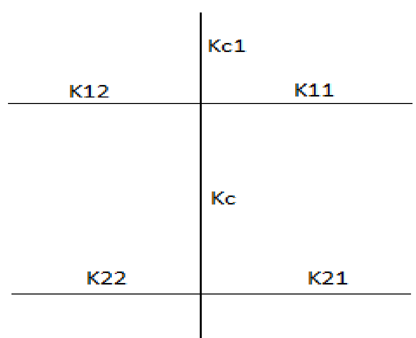
$$Ncr = \frac{\Pi^2 EIe}{Le^2}$$

5. Slenderness analysis,

   a) Calculate Limiting slenderness ratio $\lambda \max = 50 - 25\frac{M_1}{M_2}$ for non-sway frames and greater of $\lambda \max \leq 25$ & $\lambda \max = \frac{15}{\sqrt{vd}}$ for sway frames

   b) Calculate Slenderness ratio of isolated column due to cross section λ=Le/i , effective buckling length (Le) calculated as $\frac{L_e}{L} = \frac{\alpha_m + 0.4}{\alpha_m + 0.8} \geq 0.70$ for non-sway mode and $\frac{L_e}{L} = \sqrt{1 + 0.8\alpha_m} \geq 1.15$ sway mode, where

$$\alpha m = \frac{\alpha 1 + \alpha 2}{2}, \alpha_1 = \frac{K_1 + Kc}{K_{11} + K_{12}}, \alpha 2 = \frac{K_2 + K_c}{K_{21} + K_{22}}$$



If λmax<< λ it is long column (slender column) and failed by buckling. In this case we have to consider buckling effect (i.e second order effect.) and if λmax>> λ it is short column (non slender column) and it failed by crushing.

6. Find total eccentricity and design load calculation

   Additional eccentricity $e_a = \frac{Le}{300} \geq 20mm$

7. Reinforcement calculation

We have designed the column based of EBCS2, 1995. For this case we will present the sample column design for column 11 (at axis (D, 4)

19 columns to be designed and obtained from (ETABS-2015) under COMB-7

NO	STORY	JOI.LABEL	UN.NAME	CASE	F_z
1	BASE	61	252	COMB7 Max	1279.178
2	BASE	63	81	COMB7 Max	1566.418
3	BASE	65	79	COMB7 Max	1594.724
4	BASE	66	77	COMB7 Max	1930.632
5	BASE	67	75	COMB7 Max	1200.896
6	BASE	76	84	COMB7 Max	1320.257
7	BASE	77	86	COMB7 Max	1702.995
8	BASE	78	88	COMB7 Max	1781.058
9	BASE	79	73	COMB7 Max	1448.708
10	BASE	88	144	COMB7 Max	1266.985
11	BASE	89	176	COMB7 Max	1656.618
12	BASE	90	152	COMB7 Max	1741.958
13	BASE	91	160	COMB7 Max	1958.579
14	BASE	92	168	COMB7 Max	2283.133
15	BASE	93	184	COMB7 Max	1089.361
16	BASE	94	192	COMB7 Max	2514.677
17	BASE	95	200	COMB7 Max	2456.078
18	BASE	96	208	COMB7 Max	2155.015

19	BASE	97	216	COMB7 Max	1405.496

Grouping of columns in four categories depending on their magnitude of axial force

NO	STORY	JOI.LABEL	UN.NAME	CASE	Fz	COL.ID
1	BASE	61	252	COMB7 Max	2514.677	C1
2	BASE	63	81	COMB7 Max	2456.078	C1
3	BASE	65	79	COMB7 Max	2283.133	C1
4	BASE	66	77	COMB7 Max	2155.015	C1
5	BASE	67	75	COMB7 Max	1958.579	C2
6	BASE	76	84	COMB7 Max	1930.632	C2
7	BASE	77	86	COMB7 Max	1781.058	C2
8	BASE	78	88	COMB7 Max	1741.958	C2
9	BASE	79	73	COMB7 Max	1702.995	C2
10	BASE	88	144	COMB7 Max	1656.618	C3
11	BASE	89	176	COMB7 Max	1594.724	C3
12	BASE	90	152	COMB7 Max	1566.418	C3
13	BASE	91	160	COMB7 Max	1448.708	C3
14	BASE	92	168	COMB7 Max	1405.496	C3
15	BASE	93	184	COMB7 Max	1320.257	C3
16	BASE	94	192	COMB7 Max	1279.178	C4
17	BASE	95	200	COMB7 Max	1266.985	C4

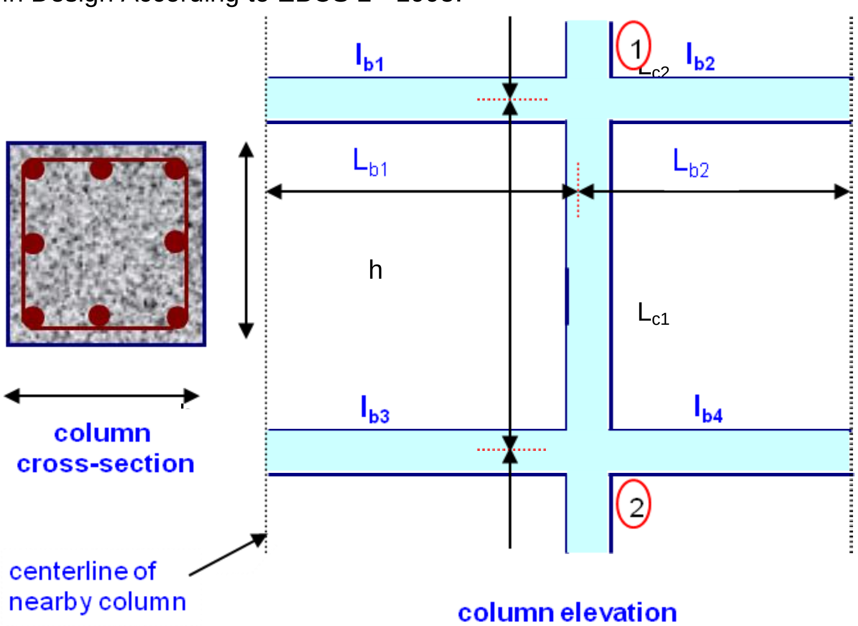
18	BASE	96	208	COMB7 Max	1200.896	C4
19	BASE	97	216	COMB7 Max	1089.361	C4

6.2.2 Sample critical Isolated Column(C-27) or C-8

Story	Column	Unique Name	Load Case/Combo	P kN	V2 kN	V3 kN	M2 kN-m	M3 kN-m
STORY1	C27	161	COMB1	-2514.68	-4.9792	9.3912	6.4869	-3.5318
STORY2	C27	167	COMB1	-2379.16	1.5721	11.6444	16.1273	-1.7219
STORY3	C27	166	COMB1	-1987.94	7.4241	13.366	20.773	11.1113
STORY4	C27	165	COMB1	-1600.04	7.5669	12.1822	18.2285	11.0451
STORY5	C27	164	COMB1	-1213.32	8.2132	12.3193	18.5385	12.1489
STORY6	C27	163	COMB1	-827.671	8.6247	11.6284	17.7933	12.8515
STORY7	C27	162	COMB1	-443.184	9.2063	13.4751	18.9938	13.6045
STORY7-1	C27	210	COMB1	-57.0716	4.3407	4.1201	10.1997	8.5686

FOUNDATION COLUMN C-8

Column Design According to EBCS 2 - 1995.



Material Properties

Concrete - Grade C-25					Steel - Grade S300		
F_{ck}	20,000	kpa	[EBCS 2 Table 2.3]		F_{yk}	300,000	kpa
F_{ctk}	1,500	kpa	[EBCS 2 Table 2.4]		PSF	1.15	
PSF	1.5		[EBCS 2 Table 3.1]		F_{yd}	260,870	kpa
F_{cd}	11333.33	kpa			E_s	200,000,000	kpa
F_{ctd}	1,000	kpa			$A_{s\ (min)}$	0.00128	
E_{cm}	29,000,000	kpa	[EBCS 2 Table 2.5]		$A_{s\ (max)}$	0.0128	

Analysis Result:

Axial	Mom. x-x	Mom y-y
2514.7	6.5	3.53
2514.7	3.53	6.5

Structure Classification:

Second-order global elastic analysis was done. Therefore both sway and non-sway are taken care of.

Dimensions in x-x direction:

	Depth	Width
B_1 =	0.30	0.70
B_2 =	0.30	0.70
B_3 =	0.00	0.00
B_4 =	0.00	0.00
C_1 =	0.40	0.40
C_2 =	0.40	0.40
C_3 =	0.00	0.00
L_{b1} =	5.00	
L_{b2} =	6.00	
L_{c1} =	2.00	
L_{c2} =	3.00	
L_{c3} =	0.00	

Dimensions in y-y direction:

	Depth	Width
B_1 =	0.30	0.70
B_2 =	0.30	0.70
B_3 =	0.00	0.00
B_4 =	0.00	0.00
C_1 =	0.40	0.40
C_2 =	0.40	0.40
C_3 =	0.00	0.00
L_{b1} =	6.00	
L_{b2} =	4.00	
L_{c1} =	2.00	
L_{c2} =	3.00	
L_{c3} =	0.00	

Limits of Slenderness:

$$\lambda \leq 50-25\left(\frac{M_1}{M_2}\right)$$ EBCS - 2, 4.4.6

$\frac{M_1}{M_2}$ = 0.543 $\frac{M_1}{M_2}$ = 0.543

$\lambda \leq$ 36.423 $\lambda \leq$ 36.423

But, the slenderness ratio is:

$\lambda = \frac{L_e}{i}$ i = Radius of gyration = $\sqrt{\frac{I_g}{A_g}}$ = 0.115 A_g = 0.160 I_g = 0.002

- -

L_e =Effective buckling length

Effective Buckling Length:

NON-SWAY $$L_e = \frac{\alpha_m + 0.4}{\alpha_m + 0.8} L \geq 0.7L$$

$$\alpha_1 = \frac{I_{c1}/L_{c1} + I_{c2}/L_{c2}}{I_{b1}/L_{b1} + I_{b2}/L_{b2}}$$

SWAY Le = $\sqrt{1+0.8\alpha_m}$ L ≥1.15 L

$$\alpha_m = \frac{\alpha_1 + \alpha_2}{2}$$

$$\alpha_2 = \frac{I_{c1}/L_{c1} + I_{c3}/L_{c3}}{I_{b3}/L_{b3} + I_{b4}/L_{b4}}$$

About x-x direction.				**About y-y direction.**			
I_{c1} =	0.00213	I_{c1} =	0.00213	I_{c1} =	0.00213	I_{c1} =	0.0021
I_{c2} =	0.00213	I_{c3} =	0.00213	I_{c2} =	0.00213	I_{c3} =	0.0000
I_{b1} =	0.00158	I_{b3} =	0.00000	I_{b1} =	0.00158	I_{b3} =	0.0000
I_{b2} =	0.00158	I_{b4} =	0.00000	I_{b2} =	0.00158	I_{b4} =	0.0000
α_1 =	**3.078**	**α_2 =**	**1.000**	**α_1 =**	**2.709**	**α_2 =**	**1.000**
α_m =	2.039			**α_m =**	1.854		

- the effective buckling length:

	x-x	y-y
SWAY	$L_e =$ 3.224	$L_e =$ 3.113
NON-SWAY	$L_e =$ 1.718	$L_e =$ 1.699

- Slenderness ratio:

x-x	y-y
$\lambda =$ 27.917	$\lambda =$ 26.957
Not Slender	Not Slender
Ignore Secondary Effects	Ignore Secondary Effect

Design Actions:

Calculate Eccentricities in the x-x Direction

Total = $e_{tot} = e_e + e_a + e_2$

$e_e =$ *max of* $\begin{cases} 0.6e_{02} + 0.4e_{01} \\ 0.4e_{02} \end{cases}$

$e_{01} =$ 0.0014

$e_{02} =$ 0.0026

$e_e =$ 0.0021

$e_a =$ $\frac{L_e}{300} \geq 200mm$

= 0.02

$e_2 =$ 0.02

$e_{tot} =$ 0.0377

Calculate Eccentricities in the y-y Direction

Total = $e_{tot} = e_e + e_a + e_2$

$e_e =$ *max of* $\begin{cases} 0.6e_{02} + 0.4e_{01} \\ 0.4e_{02} \end{cases}$

$e_{01} =$ 0.0014

$e_{02} =$ 0.0026

$e_e =$ 0.0021

$e_a =$ $\frac{L_e}{300} \geq 200mm$

= 0.02

$e_2 =$ 0.01

$e_{tot} =$ 0.0366

$N_{sd} =$ 2514.700KN

$M_{sd\ x\text{-}x} =$ 94.802KN-m

$M_{sd\ y\text{-}y} =$ 92.153KN-m

Reinforcement Calculation:

Cover ratio = 0.1

$$\upsilon_{sd} = \frac{N_{sd}}{f_{cd}A_c} \qquad \mu_{sd,x-x} = \frac{M_{sd,x-x}}{f_{cd}A_c h} \qquad \mu_{sd,y-y} = \frac{M_{sd,y-y}}{f_{cd}A_c b}$$

ν_{sd} = 1.39 $\mu_{sd,x-x}$ = 0.131 $\mu_{sd,y-y}$ = 0.127

INTRPOLATION

ν	ω
0.6	0.4
1.39	1.187
0.7	0.5

ω = 1.187 EBCS 2 - 1995: Part 2 Biaxial Chart No.9, 10

$$A_{s,tot} = \frac{\omega A_c f_{cd}}{f_{yd}}$$

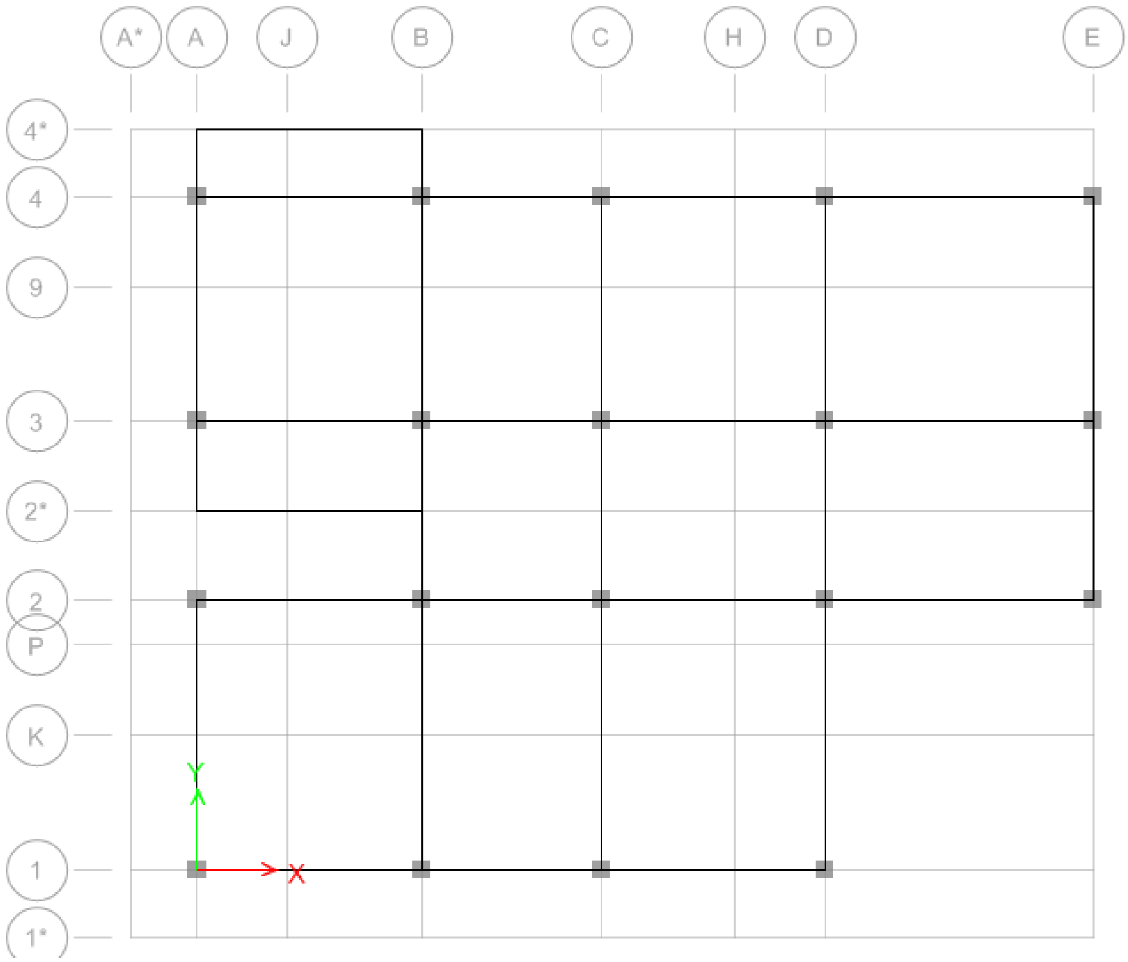
$A_{s,tot}$ = 0.008249 m^2

Using design templates and according to EBCS-2,1995 we designed critical column C_{27}

Story	Column	P	M_2	M_3	ω	As	#of bars	#of bars
1	**C_{27}**	2514.70	6.50	-3.53	1.187	8249	17.2	18Φ24
2	**C_{27}**	2379.20	16.12	-1.72	1.112	7730	15.09	16 Φ24
3	**C_{27}**	1987.94	20.80	11.11	0.896	6230	13.8	14 Φ24
4	**C_{27}**	1600.04	18.23	11.05	0.682	4743	9.5	10 Φ24
5	**C_{27}**	1213.32	18.54	12.15	0.469	3261	7.21	8Φ 24
6	**C_{27}**	827.70	17.80	12.90	0.256	1783	5.68	6Φ20
7	**C_{27}**	443.20	19.99	13.60	0.044	1280	5.03	6 Φ18

CHAPTER SEVEN

8. FOUNDATION DESIGN



7.1 Design of Isolated Footing

An isolated footing is a footing that carries a single column. Its function is to spread the column load laterally to the soil so that the stress intensity is reduced to a value that the soil can safely carry.

The approximate contact pressure under a given symmetrical foundation can be obtained from the flexural formula, provided that the considered load lies within the kern of the footing

$$\sigma_{\substack{max \\ min}} = \frac{P}{BL}\left(1 \pm \frac{6e_x}{L} \pm \frac{6e_y}{L}\right) \quad Where:\ e_x = \frac{M_y}{P};\ \ e_y = \frac{M_x}{P}$$

The thickness of a given footing that determined by checking the thickness needed for punching shear criteria and wide beam shear criteria. The greater of the two governs the depth of the footing.

Design axial loads and bending moments are obtained from ETABs analysis and the combinations are selected depending on large reinforcement requirements. The max load and moment of the footing is as following. And then, we grouped the load in to 4 groups. so we have 4 isolated footing .

19 columns to be designed and obtained from (ETABS-2015) under COMB-7

NO	STORY	JOI.LABEL	UN.NAME	CASE	F_z	M_x	M_y
1	BASE	61	252	COMB7 Max	1279.178	-2.6645	0.8503
2	BASE	63	81	COMB7 Max	1566.418	-6.8079	2.6456
3	BASE	65	79	COMB7 Max	1594.724	1.847	1.2103
4	BASE	66	77	COMB7 Max	1930.632	-2.31	2.3412
5	BASE	67	75	COMB7 Max	1200.896	3.2306	-3.7843
6	BASE	76	84	COMB7 Max	1320.257	-2.0244	3.4846
7	BASE	77	86	COMB7 Max	1702.995	6.5192	-0.7048
8	BASE	78	88	COMB7 Max	1781.058	6.3209	0.5869
9	BASE	79	73	COMB7 Max	1448.708	-3.3578	-1.8571
10	BASE	88	144	COMB7 Max	1266.985	5.1983	0.0269
11	BASE	89	176	COMB7 Max	1656.618	-0.3632	3.5941
12	BASE	90	152	COMB7 Max	1741.958	4.3707	-0.2131
13	BASE	91	160	COMB7 Max	1958.579	-1.3584	0.2247
14	BASE	92	168	COMB7 Max	2283.133	1.7046	-0.5627

15	BASE	93	184	COMB7 Max	1089.361	-0.8904	-4.3881
16	BASE	94	192	COMB7 Max	2514.677	6.4869	3.5318
17	BASE	95	200	COMB7 Max	2456.078	-1.5197	0.8982
18	BASE	96	208	COMB7 Max	2155.015	-3.5989	-1.4896
19	BASE	97	216	COMB7 Max	1405.496	2.839	4.2094

Grouping of columns in four categories

NO	STORY	JOI.LABEL	UN.NAME	CASE	Fz	M_x	M_y	COL.ID
1	BASE	61	252	COMB7 Max	2514.677	6.4869	3.5318	C1
2	BASE	63	81	COMB7 Max	2456.078	-1.5197	0.8982	C1
3	BASE	65	79	COMB7 Max	2283.133	1.7046	-0.5627	C1
4	BASE	66	77	COMB7 Max	2155.015	-3.5989	-1.4896	C1
5	BASE	67	75	COMB7 Max	1958.579	-1.3584	0.2247	C2
6	BASE	76	84	COMB7 Max	1930.632	-2.31	2.3412	C2
7	BASE	77	86	COMB7 Max	1781.058	6.3209	0.5869	C2
8	BASE	78	88	COMB7 Max	1741.958	4.3707	-0.2131	C2
9	BASE	79	73	COMB7 Max	1702.995	6.5192	-0.7048	C2
10	BASE	88	144	COMB7 Max	1656.618	-0.3632	3.5941	C3
11	BASE	89	176	COMB7 Max	1594.724	1.847	1.2103	C3
12	BASE	90	152	COMB7 Max	1566.418	-6.8079	2.6456	C3
13	BASE	91	160	COMB7 Max	1448.708	-3.3578	-1.8571	C3

14	BASE	92	168	COMB7 Max	1405.496	2.839	4.2094	C3
15	BASE	93	184	COMB7 Max	1320.257	-2.0244	3.4846	C3
16	BASE	94	192	COMB7 Max	1279.178	-2.6645	0.8503	C4
17	BASE	95	200	COMB7 Max	1266.985	5.1983	0.0269	C4
18	BASE	96	208	COMB7 Max	1200.896	3.2306	-3.7843	C4
19	BASE	97	216	COMB7 Max	1089.361	-0.8904	-4.3881	C4

7.2 Steps in footing Design

1. Proportioning of footing presumptive allowable soil pressure.

   q_{all}=P/A Where: P= un factored super structure load
2. Thickness determination
3. Check for punching and wide beam shear

   Punching shear resistance $V_{RD}=0.25f_{ctd}K_1K_2U_d$

   Where: $K_1= (1+50\rho) \leq 2$ $\rho=0.5/f_{yk}$

   $K_2=1.6-d$ d= effective depth

   Wide beam shear resistance, $V_{Rd}= 0.25f_{ctd}K_1K_2bd$

$$fctd = \frac{0.21 f_{ck}^{2/3}}{1.5}$$

4. Reinforcement Calculation

Un factored maximum axial loads and moment transferred from the supper structure for each categories of footing are summarized as follow

FOOTING ID	COLUMN ID	PIECES	F_z	M_x	M_y
F1	C1	4	2514.70	6.5	3.53
F2	C2	5	1958.6	-1.36	0.23
F3	C3	6	1656.62	-0.3632	3.59
F4	C4	4	1279.20	-2.67	0.85

7.3 Typical isolated footing design for footing type F1

Materials and design assumptions

- C-25
- S-300
- Depth of footing = 2m

7.3.1 Soil Type

Since there is no any given profile of the soil, we assumed the ground soil type as medium dense sand and gravel. Hence from EBCS-7, Table 6.3 we found the presumed design bearing resistance to be 420KPa. And the soil pressure distribution was assumed to be planar

Soil γ =18KN/m^3

a= 2.8m, b=2.8m, D= 0.75

7.4 Determination of Footing Dimensions

7.4.1Maximum Loading

P = 2514.70KN

M_X =6.50KNm

M_Y =3.53 KN

Column dimension=40x40cm

Cover =50mm

Weight of footing = 0.75*2.5*2.5*25 = 117.20 KN

Weight of soil = ((2.5*2.5 – (0.4*0.4)) 2 * 18= 219.20KN

Total design load = 2514.7 +117.2 +219.20

= **2850.64 KN**

7.4.2 Calculation of Eccentricity

$$e_x = \frac{My}{P} \Rightarrow e_x = \frac{-3.530}{2850.64} = 0.0012$$

$$e_y = \frac{Mx}{P} \Rightarrow e_y = \frac{6.50}{2850.64} = 0.0022m$$

7.4.3 Area proportioning

Assuming the footing to be square of side ‘a’ and using the general formula for rigid footing stress distribution due to vertical loading on soil;

$\delta all = \frac{p}{A}\left(1 \pm \frac{6e_x}{a} \pm \frac{6e_y}{b}\right)$ Where σ_{all} is the allowable soil bearing capacity which is taken to be equal to 400Mpa and size of a = b, area of footing A = ab = b^2, substituting relevant values:

$$400 = \frac{2938.18}{b^2}\left(1 \pm \frac{6*0.0012}{a} \pm \frac{6*0.0022}{b}\right)$$

By trial and error: a = 2.80m, b = 2.80m

$$\delta max = \frac{3001.14}{2.8^2} * \left[1 + \frac{6*0.0012}{2.8} + \frac{6*0.0022}{2.8}\right] = 386.57KN/m^2$$

$\Rightarrow 386.57KN/m^2 < 420KN/m^2$………… OK!

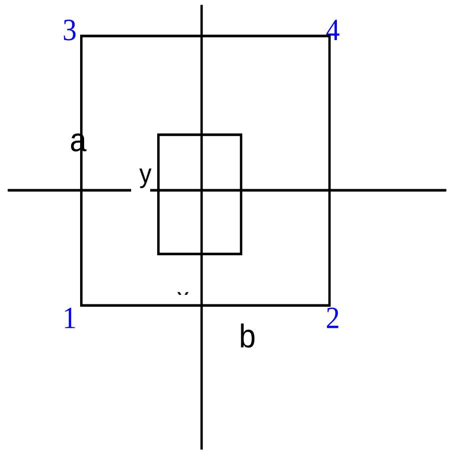
$\delta min = 379.03KN/m^2 > 0\ KN/m^2$………… OK!

There is no tensile stress in the stress distribution under the pad.

ISOLATED FOOTING DESIGN-F1

Calculation of Stress at all Corners

Max. Design Load	Pmax=	2851.10 KN
	Mx=	6.50 KNm
	My=	3.53 KNm
Column dimensions	c=	0.40 m
	d=	0.40 m
Depth of footing column =		2 m
Footing dimensions	a=	2.8 m
	b=	2.8 m
Allowable soil pressu	s_{all}=	420 KN/m^2



$$\delta all = \frac{p}{A}\left(1 \pm \frac{6e_x}{a} \pm \frac{6e_y}{b}\right)$$

Where s= the allowable soil pressure

A= axb

P= max. load sustained by the footing

e_x = 0.0012

e_y = 0.00220

Therefore the stress at all corner will be

s_1= 364.47

s_2= 366.40

s_3= 360.92

s_4= 362.85

savg= 364.47

Structural design

use concrete strength 25 Mpa

Fcd= 11.333 Mpa

Fctd= 1.167 Mpa

ρ_{min}= 0.00200

Steel S-300

Vup= 641.67 KN/m^2

Vud= 385.00 KN/m^2

A) **Punching Sheer**

d= 0.836

taking concrete cover to be =			50	mm			
D=	885.849	mm					
~D=	886.000	mm					
hence d=	856.000	mm					

B) <u>**Wide beam shear**</u>

da=	626.08696
db=	626.08696
d=	**856.000**

C) <u>**Bending moment**</u>

Ma-a=	262.42
Mb-b=	262.42

D) <u>**Reinforcement**</u>

Asmin=	**1712**	mm^2					
Kma-a=	18.93		Ksa-a=	3.9	Asa-a=	1712	,**Φ14c/c140**mm mm^2
Kmb-b=	18.93		Ksb-b=	3.9	Asb-b=	1712	,**Φ14c/c140**mm

Table of Contents

Contents **Pages**