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INDUSTRIAL PROJECT: PLANNING & DESIGN

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INDUSTRIAL PROJECT: PLANNING & DESIGN

A project and thesis
By

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In partial fulfillment of the requirement for the Degree of Bachelor of Science (B.Sc.)

in Civil Engineering.

SEPTEMBER 2020

DECLARATION

We, Md. Daud Hossain Majumder, Md. Modassher Billah, Md Abdul Manna & Md. Saifur Rahman Shamim, the students of B.Sc. in Civil Engineering hereby solemnly declare that the works presented in this project and thesis have been carried out by us and have not previously submitted to any other university/organization for any academic qualification/certificate/diploma/degree, etc.

The work that I have presented does not breach any existing copyright and no portion of this report is copied from any work done earlier for a degree or otherwise.

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ACKNOWLEDGMENT

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First of all. We are like to our highest gratitude to the Almighty Allah for his kindness on us that make it possible to complete the study and preparation of this project and thesis.

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ABSTRACT

This study was out in the Department of Civil Engineering of Stamford University of Bangladesh will the objectives of Industrial Project: Planning & Design.

The objective of this report is to analyze the structural condition of the industrial building planning and design, the structure accordingly. The implementation aims to help the construction of structurally safe building more economically, identify structural challenges, selection of best possible pre-fabricated structural sections; preparing the bill of quantities to submit a competitive offer and preparing general assembly drawings based on which the buildings will be constructed after the client awarding the contract. In doing so, the report assesses the efficiency, effectiveness, value-added, impact, user satisfaction, coherence and sustainability of construction of their desired buildings.

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CHAPTER 1
INTRODUCTION

1.1 General

Industrial buildings, which are developed as a response to population growth, rapid urbanization, and economic cycles, are indispensable for metropolitan city development. This statement holds for today; however, the relationship between cost and benefit is more complex in today's global market place. The current trend for constructing buildings is to build higher and higher, and developers tend to compete with one another on heights. Tenants also appreciate landmark address and politicians are conscious of the symbolic role of the industrial building. Nonetheless, industrial buildings are more expensive to construct per square meter, they produce less usable space and operation costs are more expensive than conventional industrial buildings. Space efficiency, which is determined by the size of the floor slab, dimension of the structural elements and rationalized core goes along with financial benefit.

1.2 Background of study

The severity of ground shaking at a given location during an earthquake can be minor, moderate and strong. Relatively speaking, minor shaking occurs frequently; moderate shaking occasionally and strong shaking rarely. The engineers do not attempt to make earthquake; such buildings that will not get damaged even during the rare but strong earthquake; such building will not vibrate and also too expensive. Instead, the engineering intention is to make buildings heavy load resistant; such buildings resist the effects of ground shaking, although they may get damaged severely but would not collapse during the heavy loaded machinery working time. Thus, the safety of people and contents is assured in heavy load resistant industrials buildings, and thereby a disaster is avoided. This is a major objective of design code throughout the world.

The conventional approach to the heavy loaded resistant design of buildings depends upon providing the building with strength, stiffness and inelastic deformation capacity which are great enough to withstand a given level of machinery-generated force. This is generally accomplished through the selection of an appropriate structural configuration and the careful detailing of structural members, such as beams and columns.

Based on the above considerations, this study focuses on the planning & design of 10 storied factory building and 6 storied office administrative building structures having edge supported floor systems.

1.3 Objective of the study

- How to allocate different items of a factory & office administrative building.
- How to allocate Proper amenities & services such as passenger and cargo lift, car parking, loading-unloading zone, raw fabric store, finish good store, sample zone, cutting facility, sewing facility, finishing facilities, security facilities, etc. for factory & office administrative buildings.
- How to specify the minimum design forces including dead loads, live loads, wind and earthquake loads, miscellaneous loads and their various combination.

1.4 Organization of the Thesis Works

The thesis has been arranged in the following order also including references as well as appendices used for the study.

Chapter I ***:*** This includes the introduction, the objectives and the scope of the study.

Chapter II ***:*** Includes a literature review.

Chapter III ***:*** Includes the methodology of the study.

Chapter IV ***:*** Description o the structure.

Chapter V ***:*** Includes conclusions and recommendations for future study.

References

Appendices

1.5 Scopes/limitations of the study

- Edge supported floor system was considered.
- Plumbing, electrification, brickworks, etc. were not considered.
- Estimation & cost analysis was omitted.

CHAPTER 2

LITERATURE REVIEW

2.1 General

This chapter elaborated on a detailed literature review that was required for though understanding and proper conducting of analysis of the whole structure.

2.2 Edge Supported floor system

If the slab is supported by beams on all four sides as shown in figure 2.2 the loads are transferred to all four beams, assuming rebar in both directions.

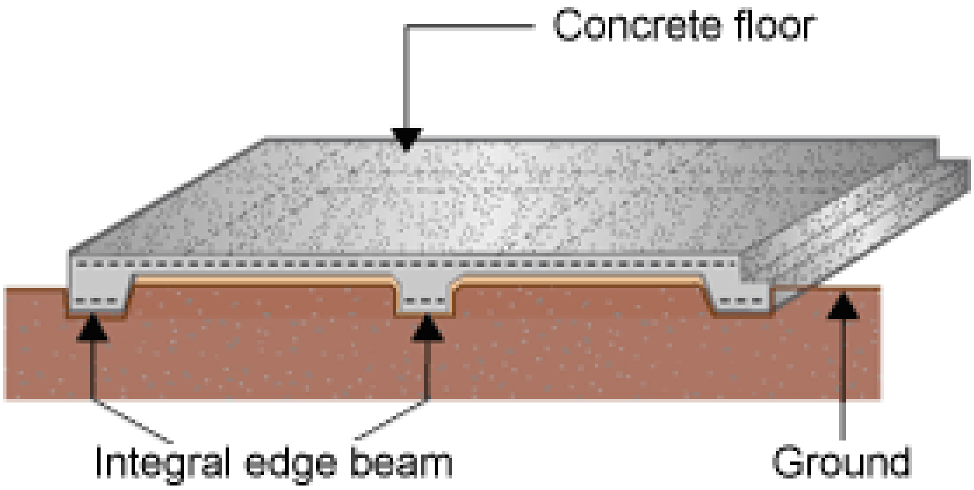


Figure 2.2: An edge supported floor system

The edge supported slab system has the following advantage disadvantage and applications:

- Advantages:

- Increase gravity and lateral load resistance
- Increase torsional resistance.
- Economical for longer spans and High loads

- Disadvantages:

- Presence of Beams may require greater story height
- Requires a regular column layout.

- Typical Applications:
- Economical for the more heavily loaded span from 25-35 feet
- Generally used for retail developments warehouse stores etc.

2.4 Lateral Loads

A. Wind Load

The actual intensity of wind pressure depends on several factors like the angle of incidence of the wind roughness of surrounding area effects of architecture features i.e. shape of the structure etc. And lateral resistance of the structure apart from these the maximum design wind load pressure depends on the duration and amplitude of the gusts and the probability of accordance of UN exceptional wind in the lifetime of building it is possible to take into account the above factors in determining the wind pressure.

The lateral load due to wind is the major factor that causes the design of high rise buildings to differ from those of low rise to medium-rise buildings for buildings up to about 10 storied end girl properties and the design is affected by the wind loads above this height however the increase in the size of the structural members and the possible rearrangement of the structure to account for wind load in cars AC cost premium that increases progressively with height Innovations in architectural treatment increases in the strangers as materials and advanced in the method of analysis tall building structures become more efficient and lighter and consequently more prone to deflect and even to sway under wind load.

Code provisions for wind load

The minimum design wind load on buildings and components is determined based on the velocity of the wind the shape and size of the building and the trend exposure condition of the site provides to the calculation of design wind loads for the primary framing system and the individual structural components of the buildings. Provisions are included for forces due to along wind response of regular-shaped buildings caused by the common wind storms including cyclones thunderstorm and nonwestern.

a. Basic wind speed

The basic wind speed for the design is taken from basic wind speed map on Bangladesh (BNBC, 1993) where it is in km/h for any location in Bangladesh having isobaths representing the fastest Mile wind speed at 10 meters above the ground with terrain exposure b for a 50 years recurrence interval the minimum value of basic wind speed set in the map in 130 km/h and the maximum is 260 km/h.

b. Exposure category.

Expose A: urban and sub-urban areas, industrial areas, wooded areas hilly or other Tremaine Covering at least 20% of the area with observations of 6 meters or more in height and extending from the site at least 500 meters or 10 times the height of the structure, whichever is greater.

Exposure B: Open terrain with scattered obstruction having heights generally less than 10 m extending 800 meters or more from the site in any full Quadrant. this category includes Airfields, open park-land, sparely built up out-skirts of towns, flat open country, and grassland

Exposure C: Flat and unobstructed open terrain, coastal areas and riverside facing large bodies of water, over 1.5 km or more in width. Exposure C extends inland from the shoreline 400m or 10 times the height of the structure, whichever greater. The basic wind speed for a selected location in Bangladesh are given BNBC in appendix C, table C,1

B. Earthquake Load

Selection of lateral force method

Seismic lateral forces on primary framing systems shall be determined by using either the equivalent static force method of the dynamic response method with the restriction given below.

The equivalent static force method:

1.All structures regular or irregular, in seismic zone 1 and structure importance

2. Regular structures less than 75 m in height with lateral force resistance provided by structural systems listed in BNBC except case 4 below.

3.Irregular structures not more than 20 m in height.

4. A tower-like building or structure having a flexible upper portion supported on a rigid lower portion where:

- Both portions of the structure considered separately can be classified as regular structures,
- The average story stiffness of the lower portion is at least ten times the average story stiffness of the upper portion.

Seismic dead load:

Seismic dead load w is the total dead load of a building for the structure including permanent partitions and applicable portions of Aadhar loads listed below:

1) in storage and warehouse occupancy a minimum of 25 percent off the floor live load shall be applicable.

2) where an allowance for partitions load is included in the floor design by following BNBC all such loads but not less than 6 Km/m2 shall be applicable.

3) The total weight of permanent equipment shall be included.

Determination of the Minimum Base Shear:

This is an approximate method that has e evolved because of the difficulties involved in carrying out realistic dynamic analysis. Code practice inevitably relies mainly on the simpler static force approach and incorporate varying degrees of refinement in an attempt to simulate the real behavior of The structures used gives a crude means of determining the total horizontal force (base share) V honor structure UBC 1994 uses equivalent horizontal static forces to design the building for maximum earthquake motion. Using Newton's second law of motion that total literal seismic force, also called the base share is determined by the relation.

$V = Ma = F_1 + F_2 + F_3 = m_1 a_1 + m_2 a_2 + m_3 a_3$

W= building weight.

V= total horizontal seismic force over the height of the building.

M= mass of the building.

g = acceleration due to gravity.

a = the maximum acceleration of the building.

M = mass of building

C = a/g

"C" is the seismic coefficient who is representing the ratio ab maximum earthquake acceleration. The acceleration due to gravity (Taranath, 1988) but in BNBC it refers as "z" an important feature of equivalent static load requirements in most code of practice

Till 2000 is the fact that the calculated seismic forces are considerably less than those which occur in the large earthquakes in the area concerned.

Code Provisions for Earthquake Load

The UBC states that the structure is designed for a minimum total lateral seismic load Which is assumed to act nonconcurrent in orthogonal directions parallel to the main axes of the structure, where V is the calculated from the formula, UBC (1994), BNBC (1993) as

V = ZICWIR

C = 1.25S/7

T = C

V = base shear

Z = seismic zone coefficient

I = structural importance coefficient

R = response modification coefficient for the structural system

C = Numerical coefficient

W = total dead load +25%live load in storage and warehouse occupancies

S = site coefficient for soil

T = fundamental period of vibration

Ct = 0.073 for reinforced concrete moment resisting frames and eccentrically braced steel frame and 0.049 for all other structural systems

HN = Height in meters above the base to level n.

The design base shear equation provides the level of the seismic design loading for a given structural system; assuming that the structure undergoes inelastic deformation during a major earthquake

Vertical Distribution of Lateral Forces

The total design base shear V is distributed over the height of the structure as described below:

Ft is the concentrated lateral force applied at the top of the structure,

Ft = 0.07TV<0.25 V for T < 0.7 sec

Limitation of Height Fundamental Period Time in Code Provisions for Earthquake Analysis.

The main restitution that has been imposed by different codes to the quasi-static method is structural hide in every code regular in irregular structures of certain height is analyzed by the quasi-static method.

The height restriction is given by different codes

UBC (1994) 73m (240'-0")

IS (1994) 90m (295'-0")

BSLJ (1987) 60m (197'-0")

Fundamental time period T, indifferent codes.

BNBC (1993),

T=Ct x hn ¾

hn = height in feet above the base two-level n

C1 = 0.083 for settle moment resisting frames.

C1 = 0.073 for reinforced concert moment-resisting frames and eccentrically braced frames.

C1 = 0.049 for all other buildings.

A Tectonic earthquake occurs deep into the Earth's interior the Centre of occurrence is called the focus point vertically above focus on earth’s surface is called epicenter it impossible to find out the depth focus by proportion to their distance. The Tectonic map of Bangladesh is furnished here. The flats shown are most crucial for the favor of an earthquake.

According to the depth of focus techno tech earthquake is classified as:

Shallow: The depth of focus is less than 60 km.

Intermediate: depth of focus between 60 to 70 km if.

Deep: depth of focus above 70 km i.e.

Earthquakes deeper than 700 i.e. are seldom field on the earth’s surface except by instruments.

Scale: The scale of earthquake intensities was conveniently classified into 12 categories, till 1935, when C.F Richer devised a scale indicating numerical J5; the magnitude of the intensity of the earthquake, 10 is the higher on this scale. The greater the number more is the damaging power.

The intensity of the earthquake in terms of Richer scale is expressed as:

1) Instrumental: detected by a seismograph, magnitude 1-3;

2) Feeble: noticed only by sensitive people

3) Slight: is like a vibration of passing lorry, felt on upper floors, magnitude'. 3.5 to 4.2

4) Moderate: felt while walking, magnitude 4.3

5) Rather strong: most sleeper awakened, magnitude 5.5-6;

6) Strong: trees sway suspended objected swing, falling loose, objects, magnitude 4.9-5.4;

7) very strong: walls crake, plaster fall, magnitude 5.5-6;

8) Destructive: chimneys fall; building damage magnitude 6.8;

9) Ruinous: house collapse, ground cracks, pipes break open, magnitude 6.9;

10) Disastrous: ground crack badly budges. Destroyed, rail lines bent, magnitude 7-7.3;

11) Very disastrous: few buildings remain standing; bridges destroyed, great landslide and floor, magnitude 7.4-8.7

12) Catastrophic: Destruction. Objects are thrown into the air, the ground rises and falls in waves, magnitude 8.2 and above.

Seismic Factors: The committee has suggested three zones namely zone I, being most active, zone II, zone III, being the minimum possible intensity of the earthquake.

To design building basic, seismic coefficient denoted by Z may be taken as follows:

Zone I,	Z = 0.08 g
Zone II,	Z = 0.05 g
Zone III,	Z = 0.04 g

Seismic probable Magnitude

Zone	(Richer scale)
I	7.0
II	6.5-7.0
III	6.0-6.5

CHAPTER 3
METHODOLOGY OF THE STUDY

3.1 General

This chapter gives the outlines of the procedures that were followed to complete this study. Finally, the deflection of the whole structure is provided to obtain the effects of lateral loadings on the structure.

3.2 Study procedures

Step-I: Selection and planning of the structure

10 storied factory and 6 storied office & administrative buildings having Edge Supported floor system structure had been selected. Two models of this were done with the help of ETABS. The description of the whole structure is provided in *Chapter 4*.

Step-II: Selection of material properties & loadings

As per discussions made in *Chapter 2* and based on design code/specifications of *ACI/BNBC*, material properties (compressive strength of concrete, the yield stress of steel, unit weight of concrete, soil, brick, etc.) and loadings (standard live load, floor finish, etc.) were selected. Wind and earthquake loads were also considered.

Step-III: Analysis of the structure

Two models of the structure were separately analyzed considering i) gravity + wind + earthquake loadings by using ETABS. *Chapters 5* provides detailed analysis and findings of the study.

Step-IV: Conclusions & Recommendations

Based on the study, a few concluding remarks were drawn. To carry out further study on this topic, recommendations were proposed in *Chapter 6*.

3.3 Design data and specifications considered in this study

The whole study was carried out based on a few considerations and specifications which are summarized in Table 3.1.

Table 3.1: Summary of the design considerations and specification of study

Items	Description
Design code	• American Concrete Institute (*ACI*) Building design code, 2014. • Bangladesh National Building Code (*BNBC*), 1993.
Loadings	• Floor plus ceiling finish = 25 *psf*. • Live load = 100 *psf* for all floors. • Earthquake and wind load are considered.
Building components	• Column type = Tied • Thickness of partition walls = 5 *inch & 10 inch*
Material properties	• Yield strength of reinforcing bars, f_y = 60,000 *psi*. • Concrete compressive strength, f'_c = 3,000 *psi*. • Normal density concrete having w_c = 150 *pcf*.

3.4 Load Calculation

3.4.1 Dead load calculation for all structures

Self-weight of slab = $\frac{7}{12} \times 150 = 87.5$ *psf*

Floor finish = 25 *psf*

Total partition wall load = 60 *psf*

3.4.2 Live load calculation for all structures

Live load for Residential space = 100 *psf*

Live load for Stair = 150 *psf*

3.4.3 Seismic load calculation

Height of building

- Office building &
  Administrative building = 71.5 *ft* = 21.80 *m*
- Industrial building = 113.5 *ft* = 34.60 *m*

Seismic zone Coefficient (Dhaka zone) = 0.15

Intermediate moment resisting frame, *R* = 8

Importance Coefficient for residential building, *I* = 1.0

Period of Vibration, T $= C_t \times h^{\frac{3}{4}}_n = 0.073 \times 45.72^{\frac{3}{4}}$

$= 1.28$

Seismic Coefficient, C $= \frac{1.25 \times S}{T^{2/3}} = \frac{1.25 \times 1.50}{1.28^{2/3}} = 1.590$

Story range = Base to Roof.

3.4.4 Wind load calculation

a) Office & Administrative Building:

Length of building, L = 120 *ft*

Width of building, B = 100 *ft*

Height of building, H = 71.5 *ft*

Wind pressure in Dhaka city, V_b = 210 *Km/h*

Importance coefficient for building, C_i = 1

b) Industrial Building:

Length of building, L = 350 *ft*

Width of building, B = 125 *ft*

Height of building, H = 113.5 *ft*

Wind pressure in Dhaka city, V_b = 210 *Km/h*

Importance coefficient for building, C_i = 1

CHAPTER 4

DESCRIPTION OF THE WHOLE STRUCTURE

4.1 Introduction

This chapter provides a detailed description of the whole structure. The main features can be summarized as below:

a) Office & Administrative Building:

Height of building : 71.5 *ft*

Length of building : 120 *ft*

Width of building : 100 *ft*

Total floors : 6nos (covered with floor tiles).

Types of floors : 1. Ground Floor

2. Typical Floor

b) Industrial Building:

Height of building : 113.5 *ft*

Length of building : 350125 *ft*

Width of building : 125 *ft*

Total floors : 10nos (covered with floor tiles).

Types of floors : 1. Ground Floor

2. Typical Floor

4.2 Description of the floors

a) Office & Administrative Building:

Ground Floor:

- 1' -6" high from road level.
- The floor height 10'-6".
- The total floor area is 12000*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 2 stairs & 2-passenger lift.
- Provision for the medical center, reception & waiting zone car parking.

The plan view of a typical floor with all facilities is shown in Figure 4.1.

1st Floor:

- 12' -0" high from road level.
- Floor height 10'-6".
- The total floor area is 12000*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 2 stairs & 2-passenger lift.
- Provision for table tennis & pool.

The plan view of a typical floor with all facilities is shown in Figure 4.2

2nd Floor:

- 22' -6" high from road level.
- Floor height 10'-6".
- The total floor area is 12000*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 2 stairs & 2-passenger lift.
- Provision for V.I.P conference room, V.I.P training room, Merchandiser & QUC room, Inspection room & Meeting room.

The plan view of a typical floor with all facilities is shown in Figure 4.3.

3rd Floor:

- 33' -0" high from road level.
- Floor height 10'-6".
- The total floor area is 12000*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 2 stairs & 2-passenger lift.
- Provision for officer’s room Cutting & Swing section.

The plan view of a typical floor with all facilities is shown in Figure 4.4.

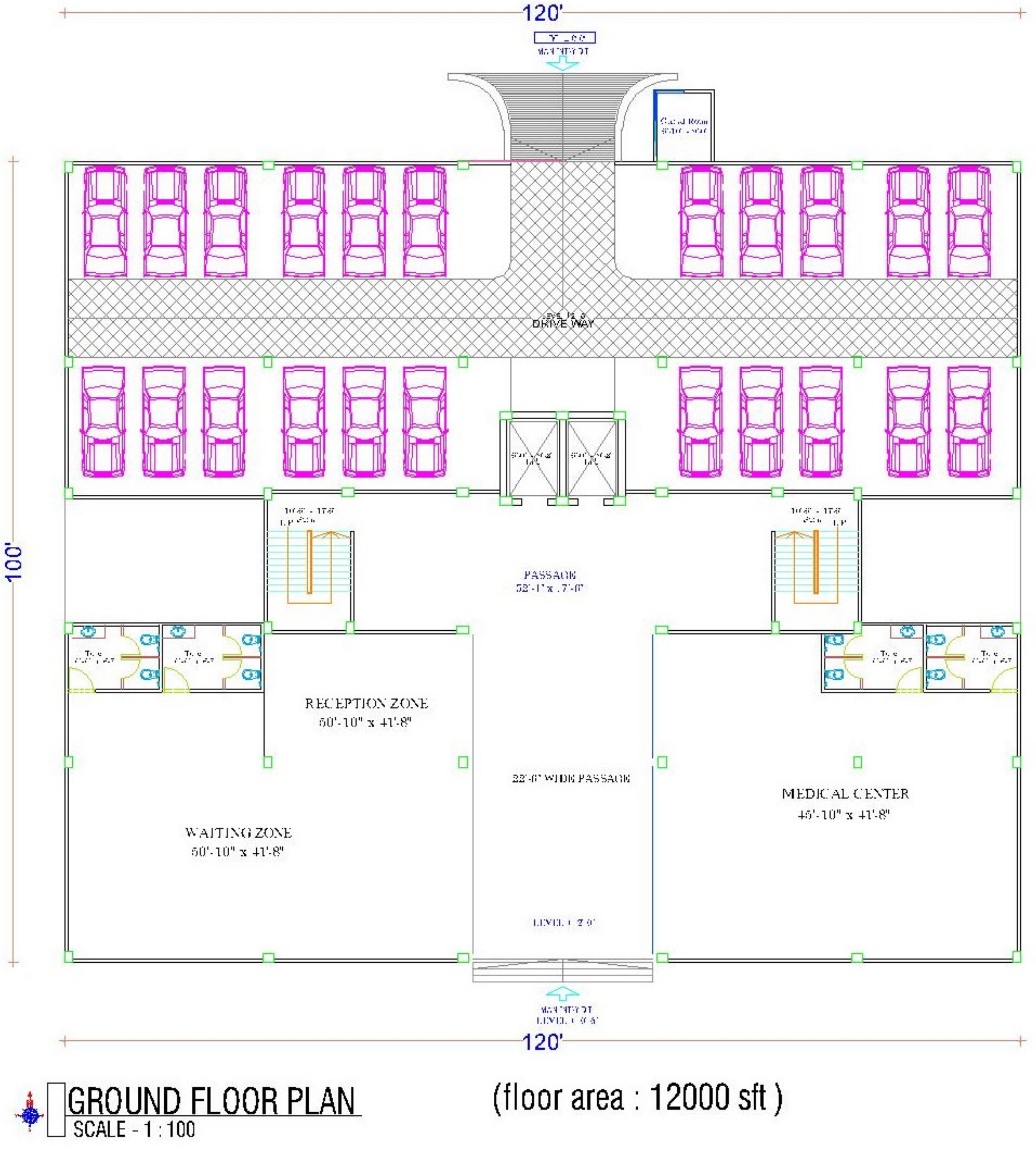


Figure 4.1: A typical plan view of the Ground Floor

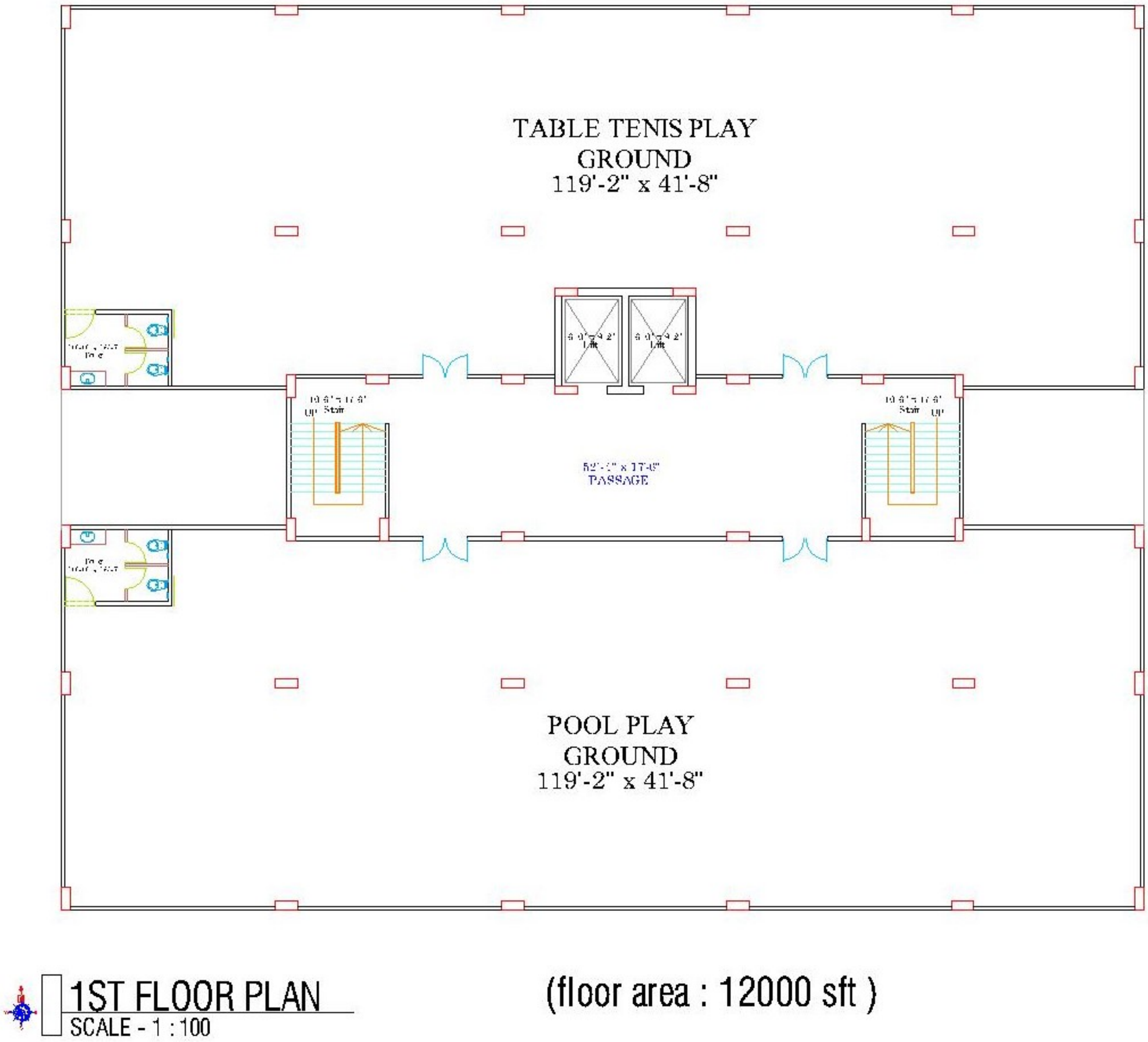


Figure 4.2: A typical plan view of 1st Floor

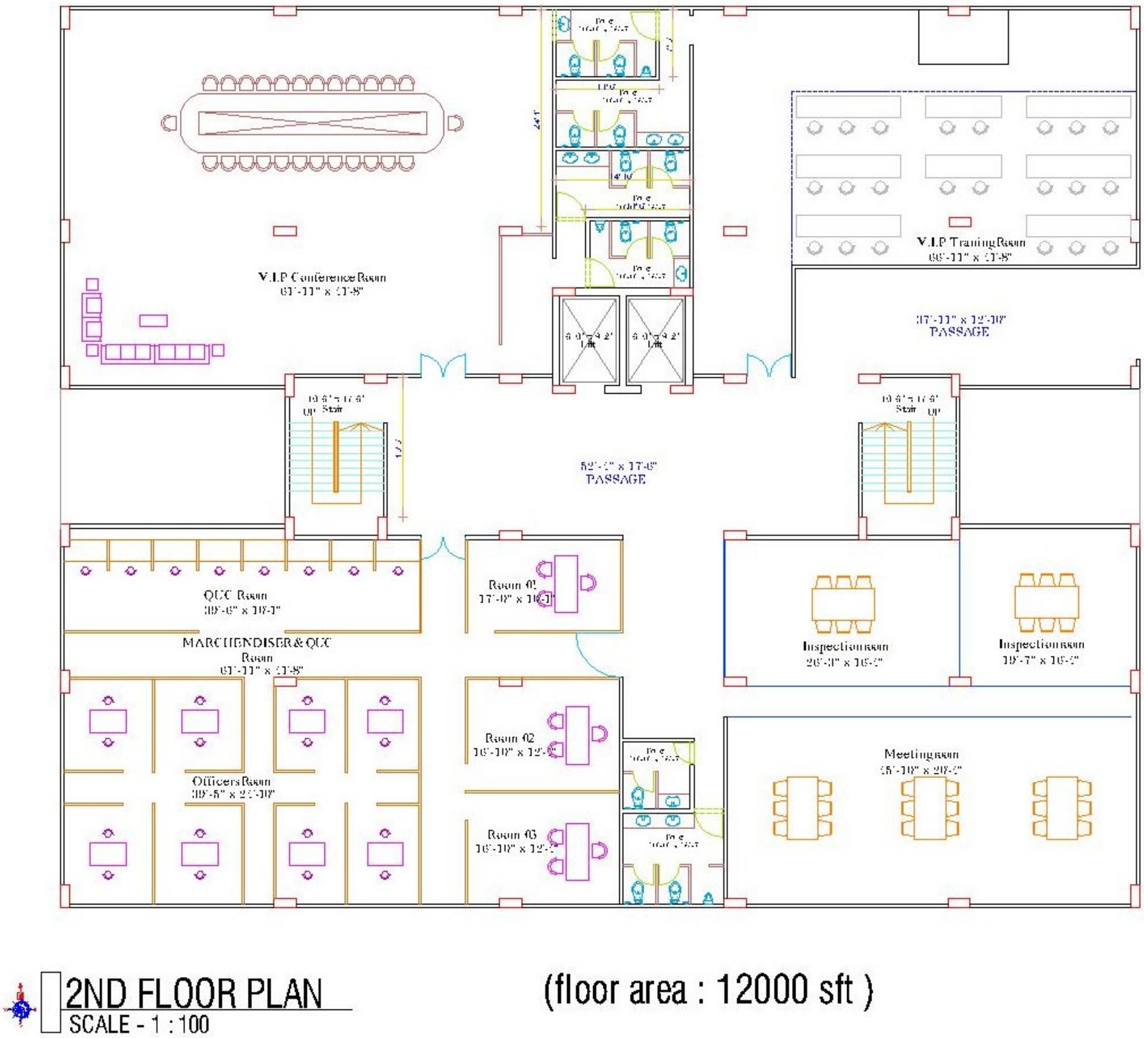


Figure 4.3: A typical plan view of 2nd Floor

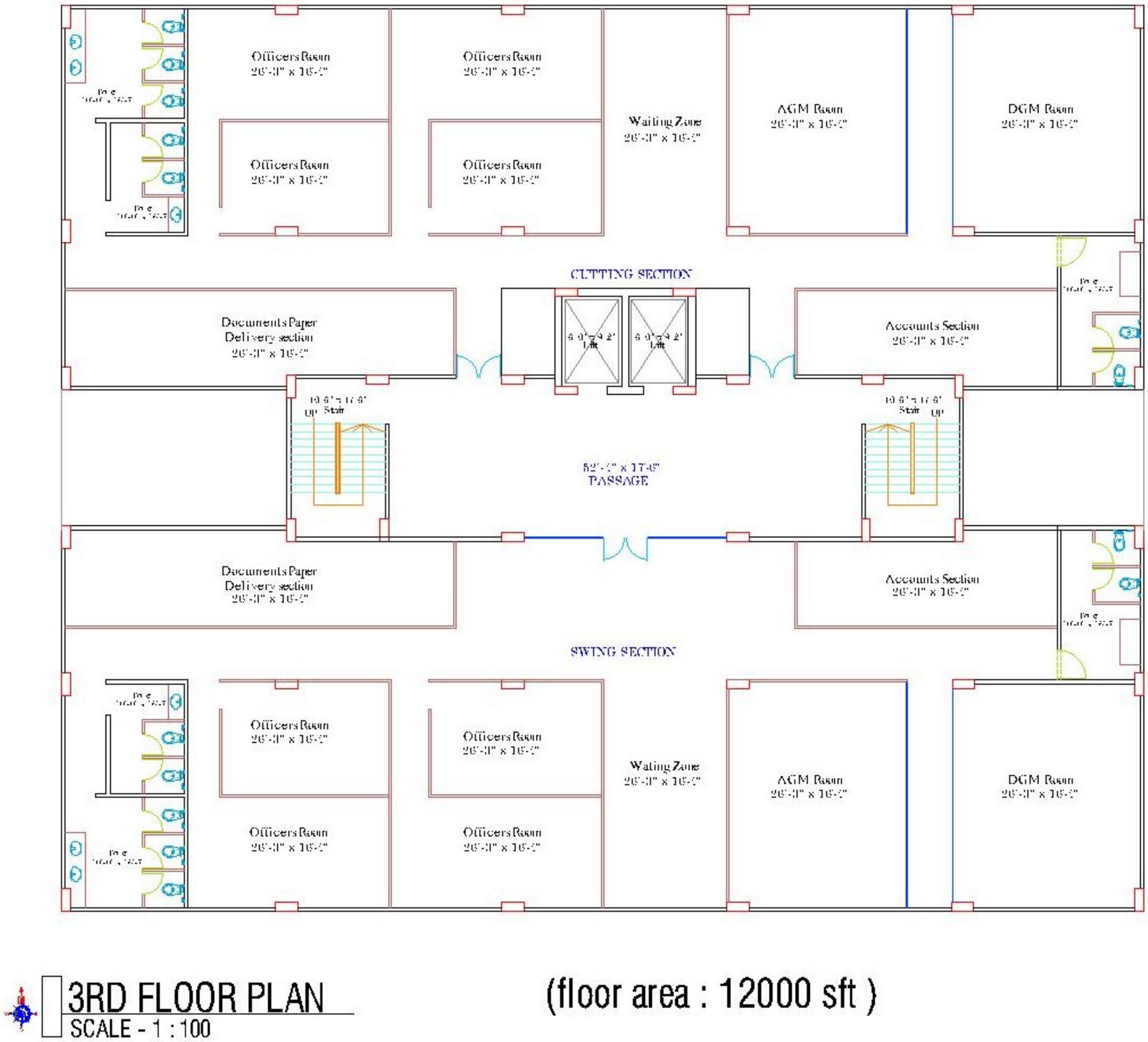


Figure 4.4: A typical plan view of 3rd Floor

<u>4th Floor:</u>

- 43' -6" high from road level.
- Floor height 10'-6".
- The total floor area is 12000*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 2 stairs & 2-passenger lift.
- Provision for Chairman room, GM room, lab, IT section, account section & officer's room finishing section.

The plan view of a typical floor with all facilities is shown in Figure 4.5.

<u>5th Floor:</u>

- 54' -0" high from road level.
- Floor height 10'-6".
- The total floor area is 12000*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 2 stairs & 2-passenger lift.
- Provision for V.I.P hall room, prayer room, V.I.P refreshment zone, dining room.

The plan view of a typical floor with all facilities is shown in Figure 4.6.

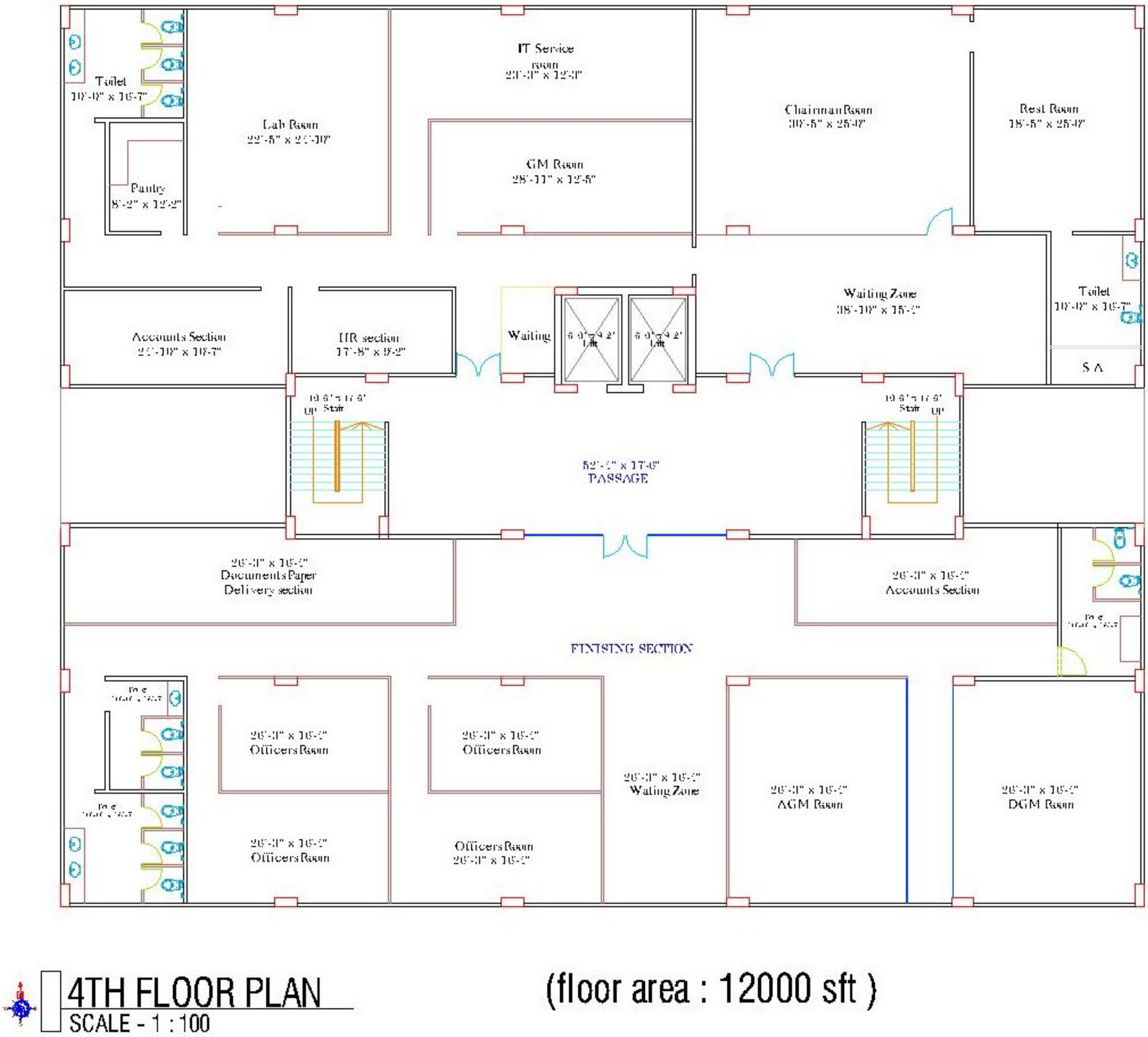


Figure 4.5: A typical plan view of 4th Floor

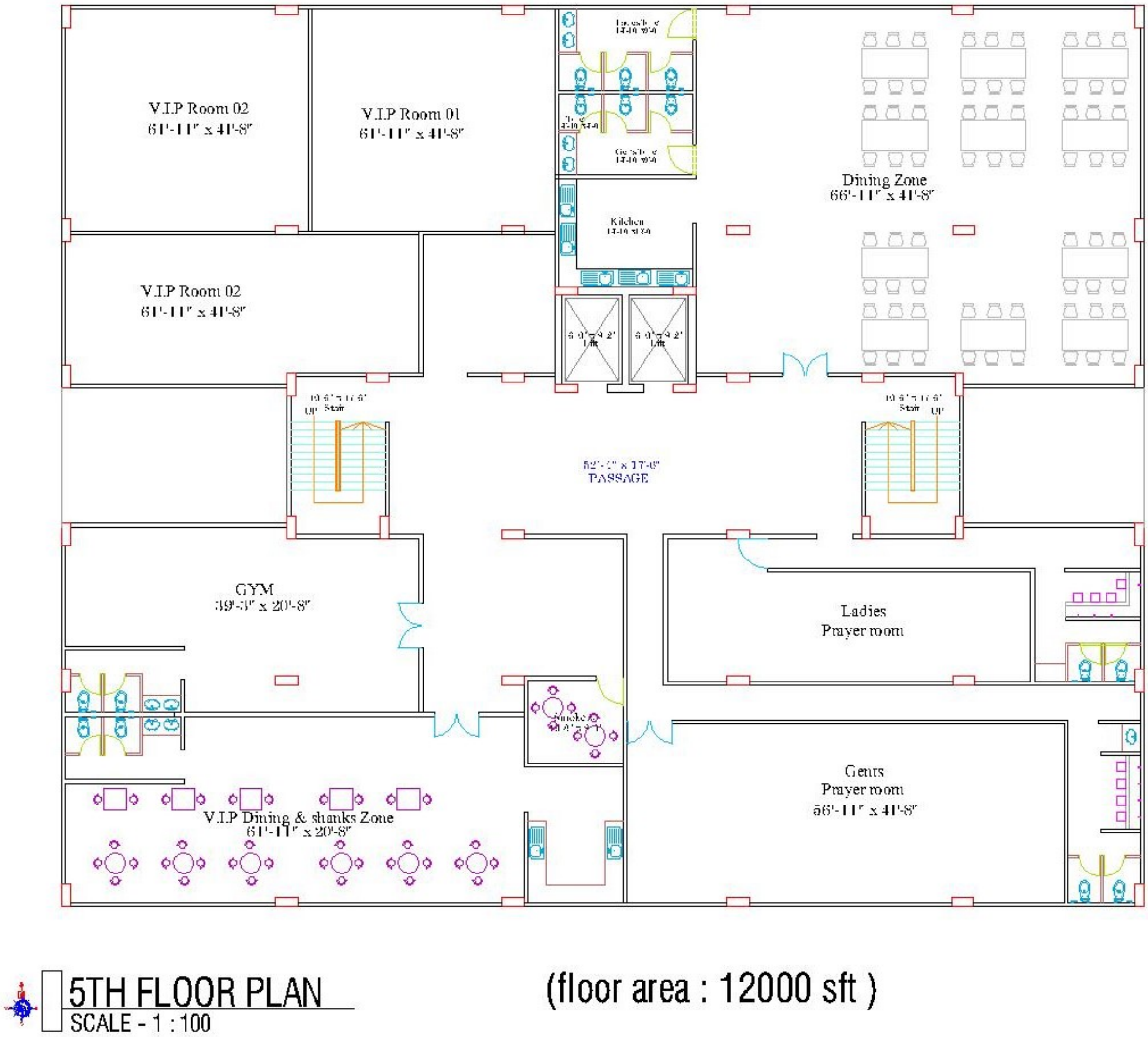


Figure 4.6: A typical plan view of 5th Floor

b) Industrial Building:

Ground Floor:

- 1' -6" high from road level.
- Floor height 10'-6".
- The total floor area is 43750*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 3 stairs, 4-passenger lift & 2 cargo lift.
- Provision for loading and unloading & raw fabrics store.

The plan view of a typical floor with all facilities is shown in Figure 4.7.

1st Floor:

- 12' -0" high from road level.
- Floor height 10'-6".
- The total floor area is 43750*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 3 stairs, 4-passenger lift & 2 cargo lift.
- Provision for finish goods store, sample zone, accessories store, office, maintenance, inspection & account section.

The plan view of a typical floor with all facilities is shown in Figure 4.8.

2nd Floor and 3rd Floor:

- 22' -6" & 33' -0" high from road level.
- Floor height 10'-6".
- The total floor area is 43750*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 3 stairs, 4-passenger lift & 2 cargo lift.
- Provision for finishing facilities.

The plan view of a typical floor with all facilities is shown in Figure 4.9.

Figure 4.7: A typical plan view of the Ground Floor

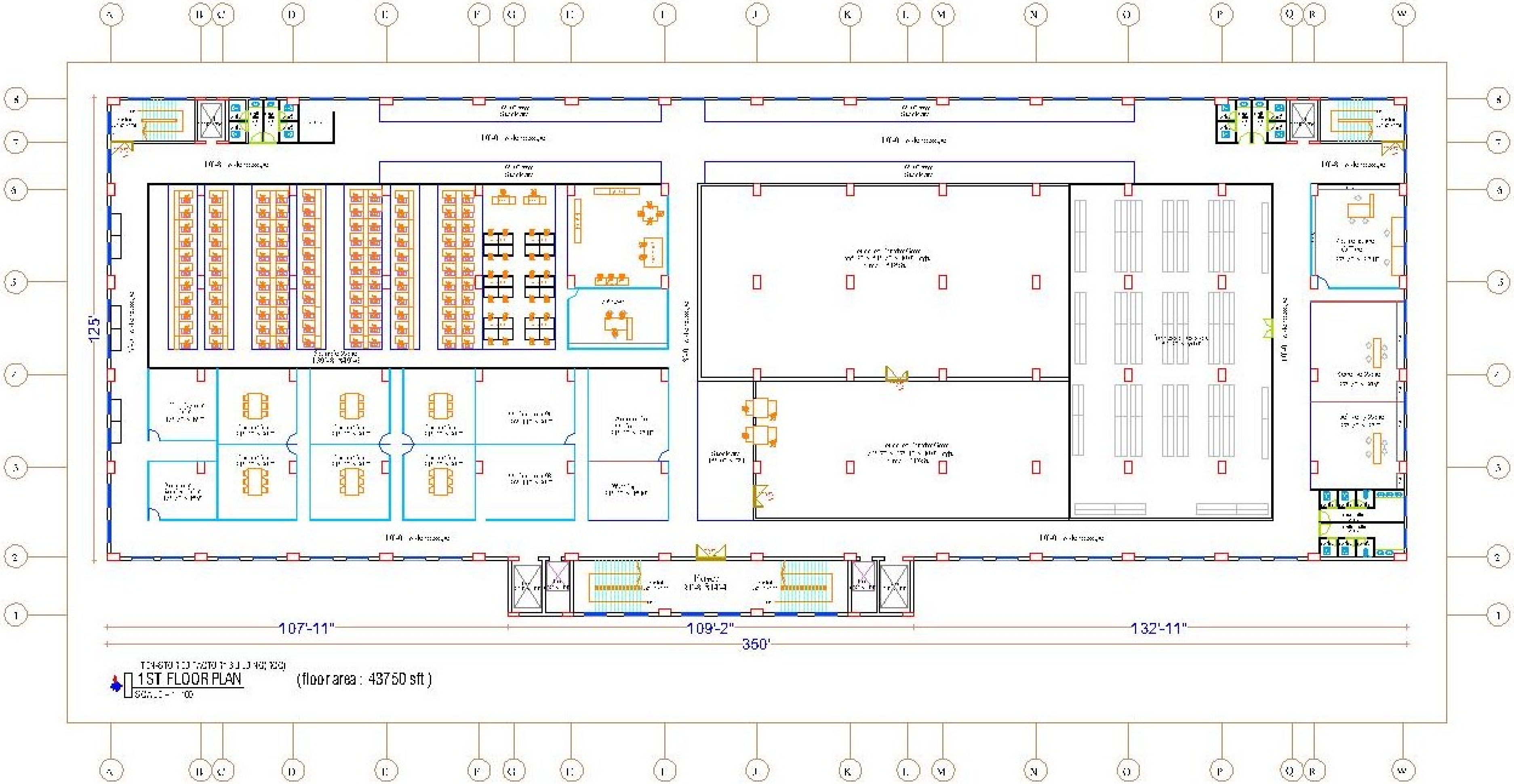


Figure 4.8: A typical plan view of 1st Floor

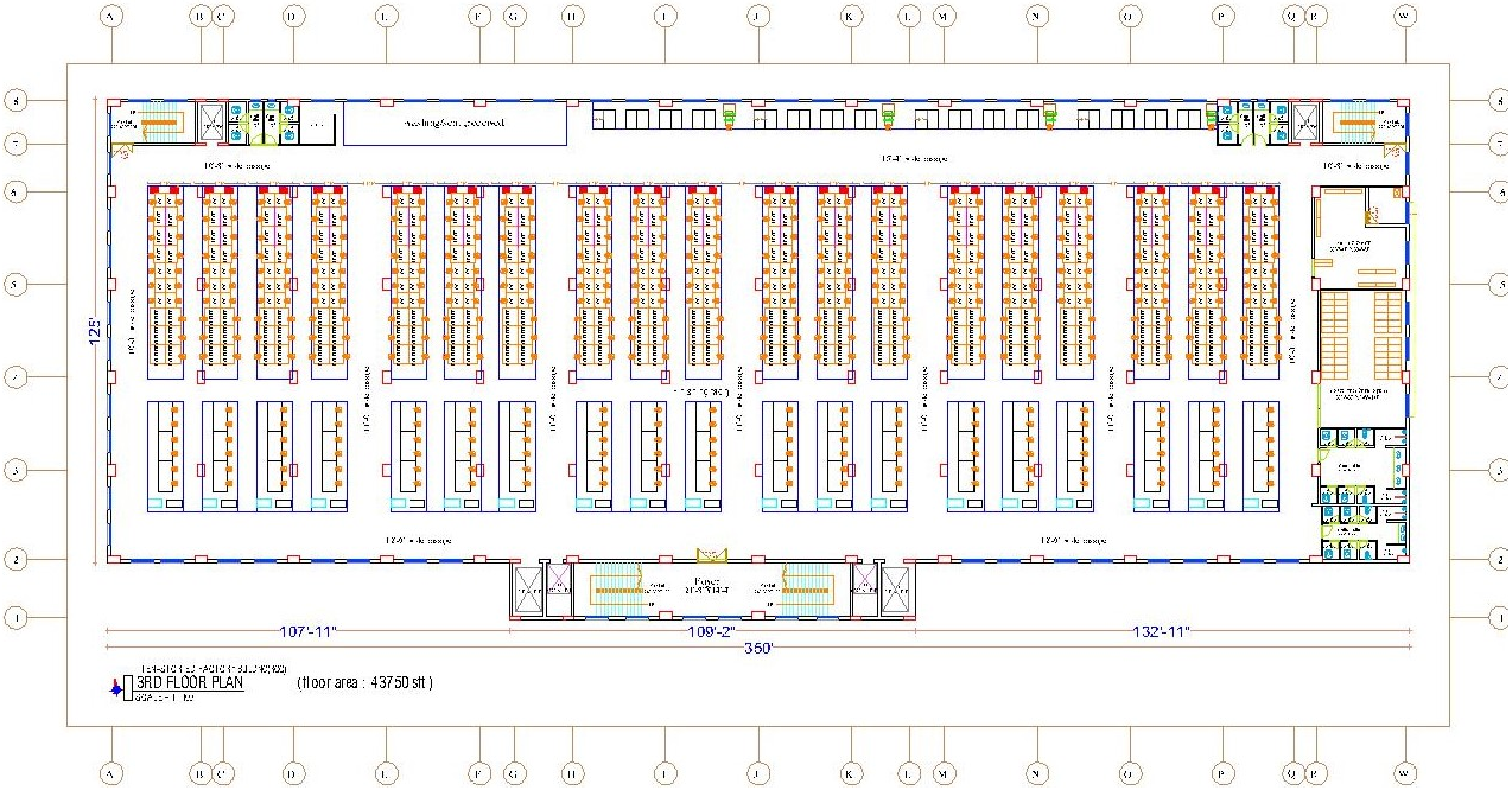


Figure 4.9: A typical plan view of 2nd & 3rd Floor

<u>4th Floor and 5th Floor:</u>

- 43' -6" & 54' -0" high from road level.
- Floor height 10'-6".
- The total floor area is 43750*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 3 stairs, 4-passenger lift & 2 cargo lift.
- Provision for swing facilities.

The plan view of a typical floor with all facilities is shown in Figure 4.10.

6th Floor and 8th Floor:

- 64' -6" to 85' -6" high from road level.
- Floor height 10'-6".
- The total floor area is 43750*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 3 stairs, 4-passenger lift & 2 cargo lift.
- Provision for cutting facilities.

The plan view of a typical floor with all facilities is shown in Figure 4.11.

9th Floor:

- 96' -0" high from road level.
- Floor height 10'-6".
- The total floor area is 43750*sft*.
- Entry & Exit by stairs.
- Connected with other floors by 3 stairs, 4-passenger lift & 2 cargo lift.
- Provision for dining space, training zone & prayer space.

The plan view of a typical floor with all facilities is shown in Figure 4.12.

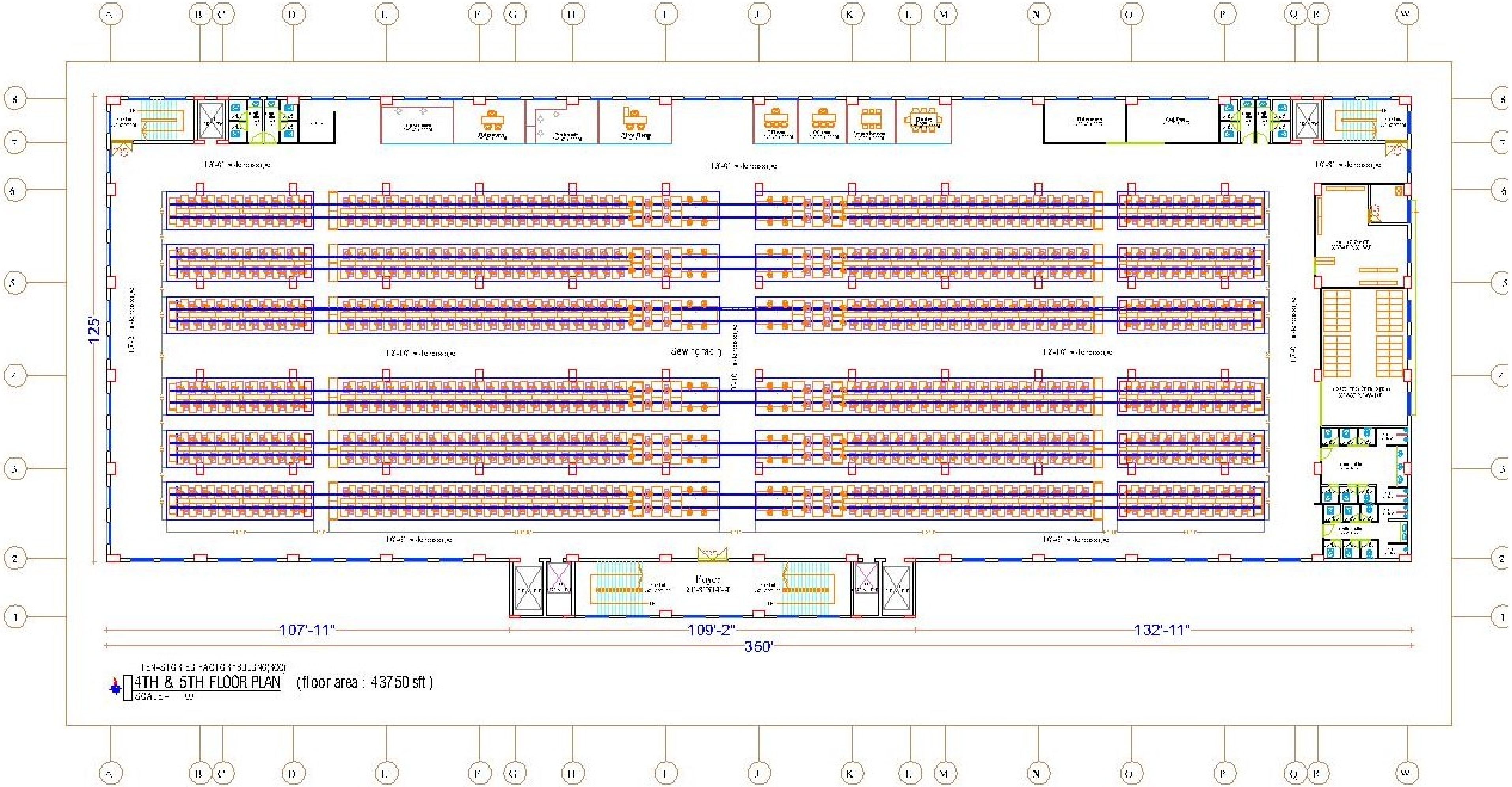


Figure 4.10: A typical plan view of 4th & 5th Floor

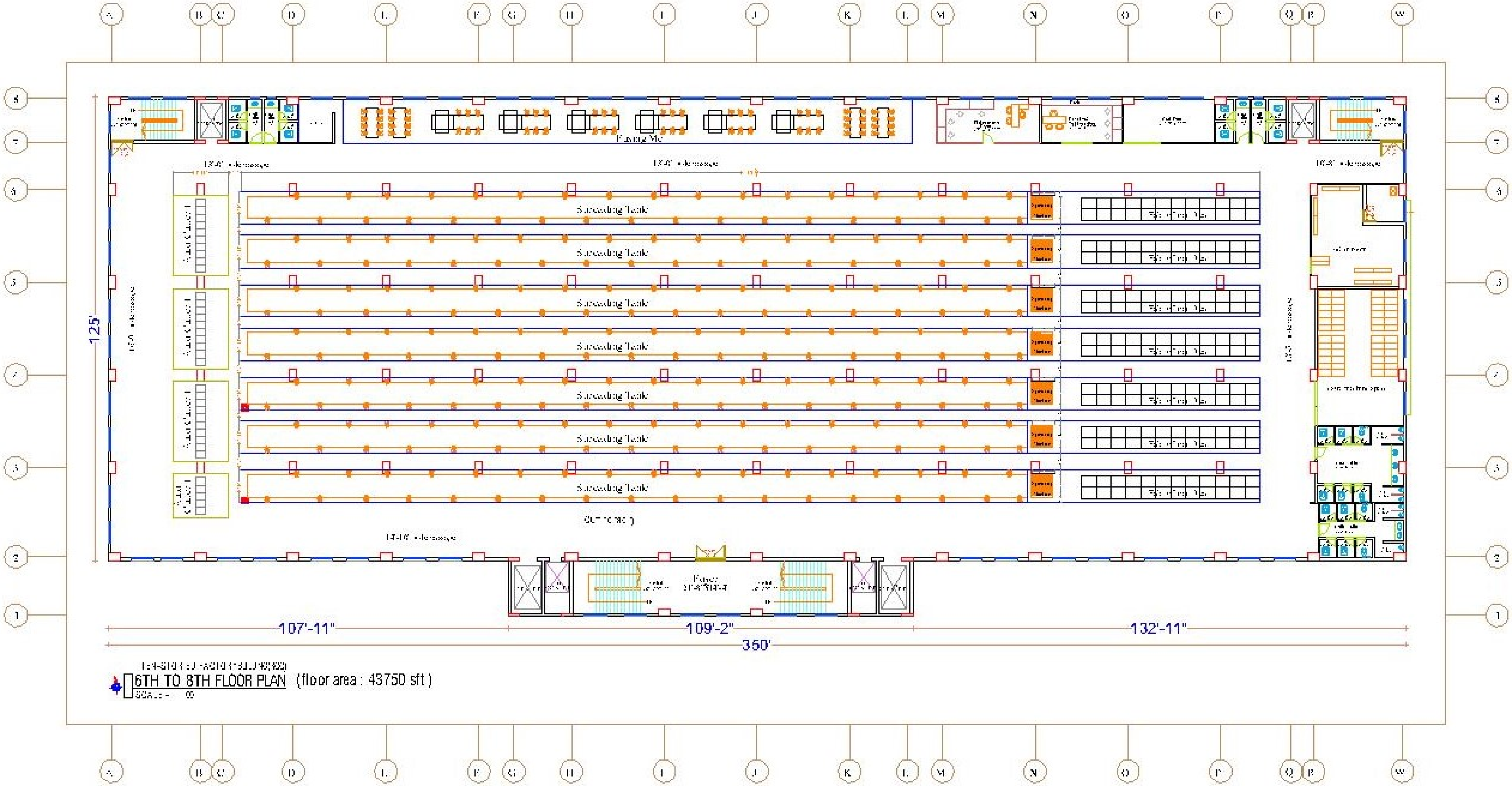


Figure 4.11: A typical plan view of 6th floor to 8th Floor

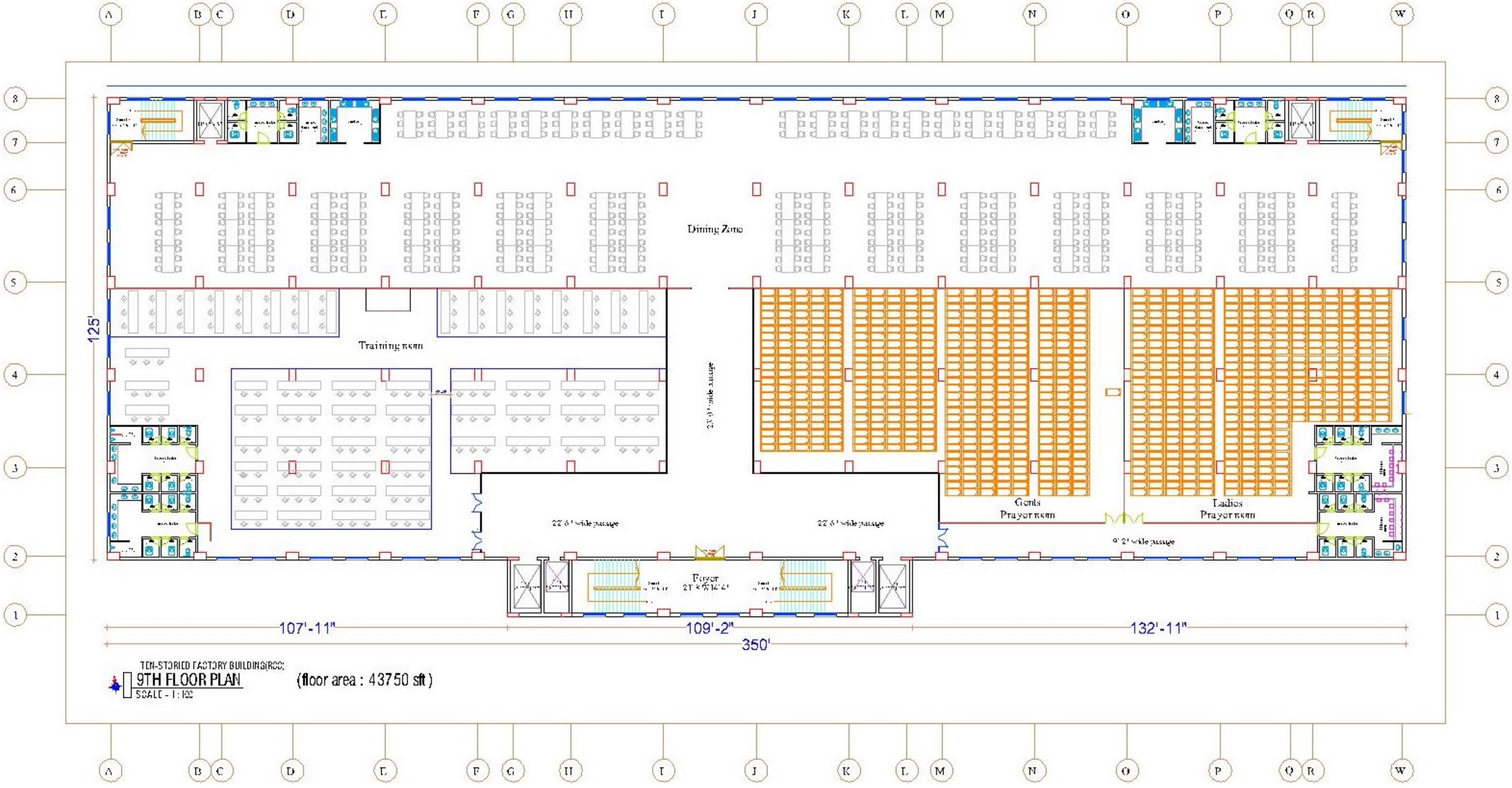


Figure 4.12: A typical plan view of 9th Floor

CHAPTER 5
DESIGN OF SLAB, BEAM & COLUMN

5.1 Introduction

This chapter provides detailing of the factory as BNBC by analyzing the effects of the lateral loads on 10 storied & 6 storied the structure having an edge supported floor system. The structure is divided into several grids in ETABS plan: 1-7 in horizontal grids and A-K in vertical grids

Figueres 5.1 a shown the 3-D ETABS model view of the Rigid Frame Structure. Also Figures 5.2 presents plan views at Ground floor plan which give clear pictures on the presence of columns, floor beams that consider in the study.

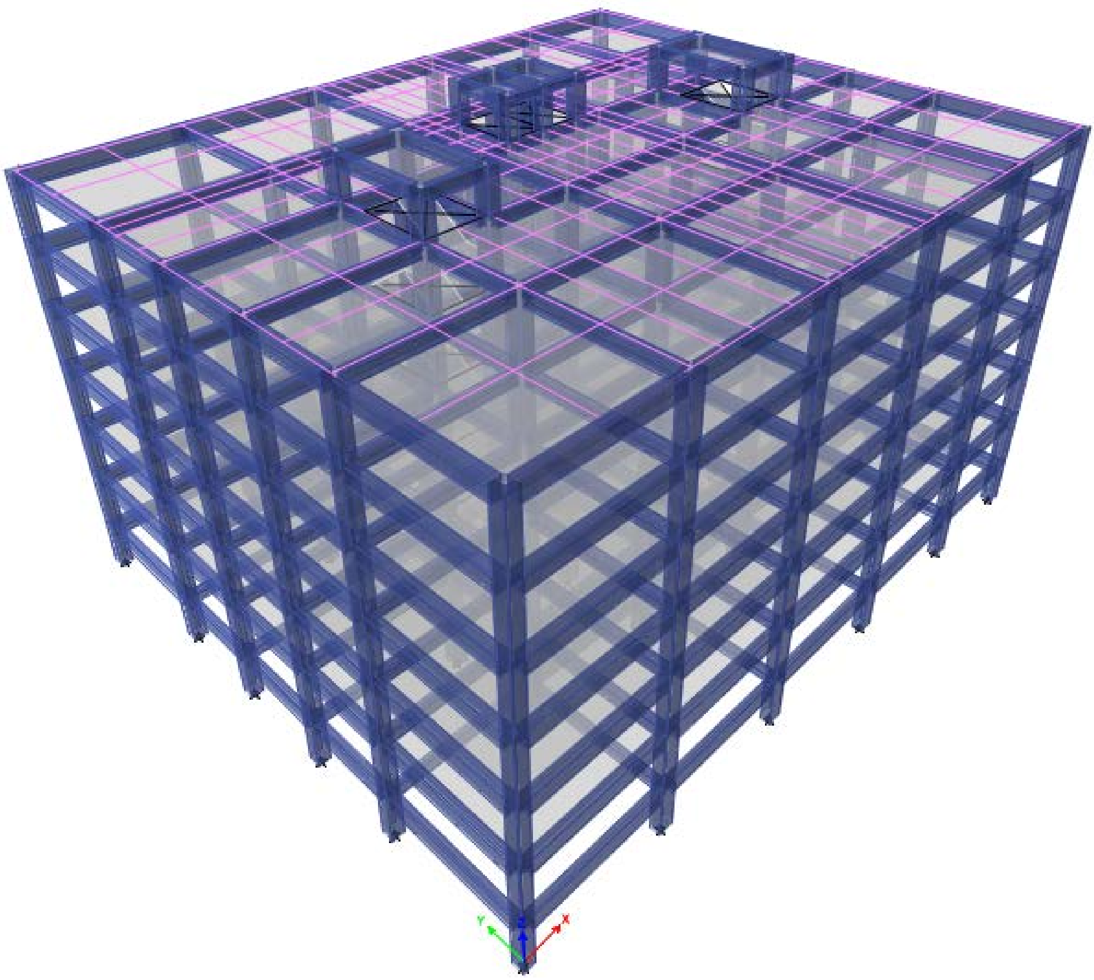


Figure 5.1: 3-D Model View of the Office

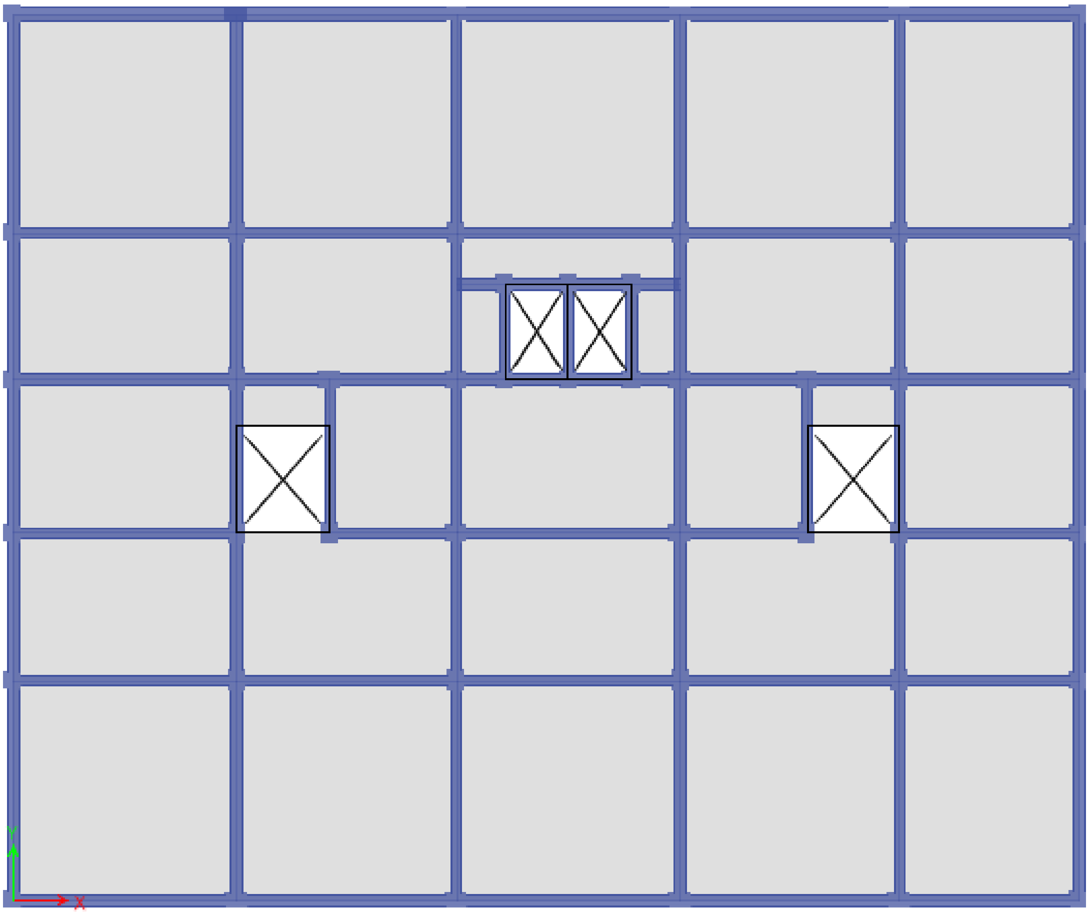


Figure 5.2: Ground Floor plan

5.2 Slab design (Ground Floor to 5th Floor)

A slab is designed by ETABS software. There are several panels in each other. These panels based on different dimensions and continuity conditions. Panel plan is given in Figure 5.3. Also, reinforcement details of the roof floor which are detailed by ETABS 16.2.1 are given Figures 5.4 – 5.5 respectively.

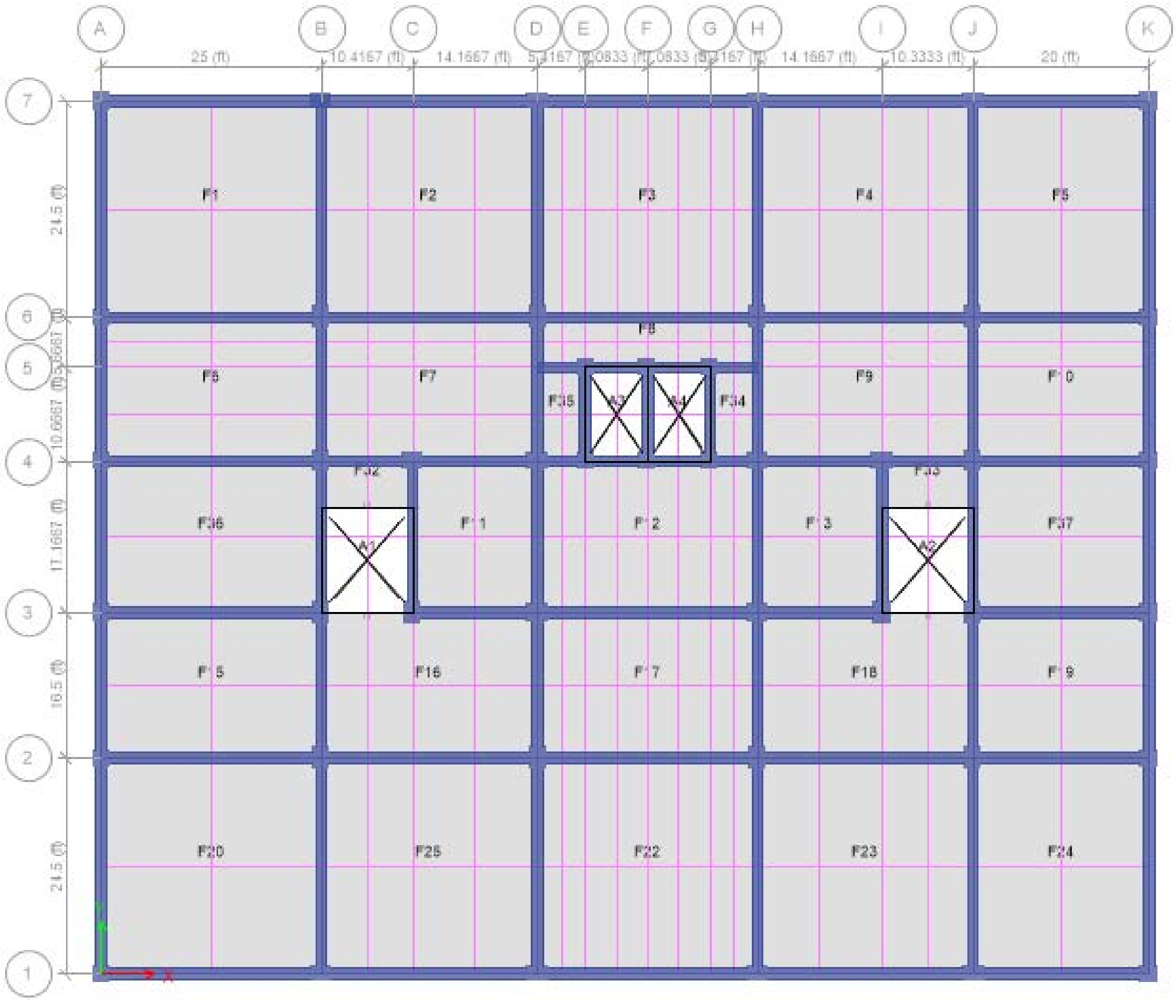


Figure 5.3: Slab framing plan (roof)

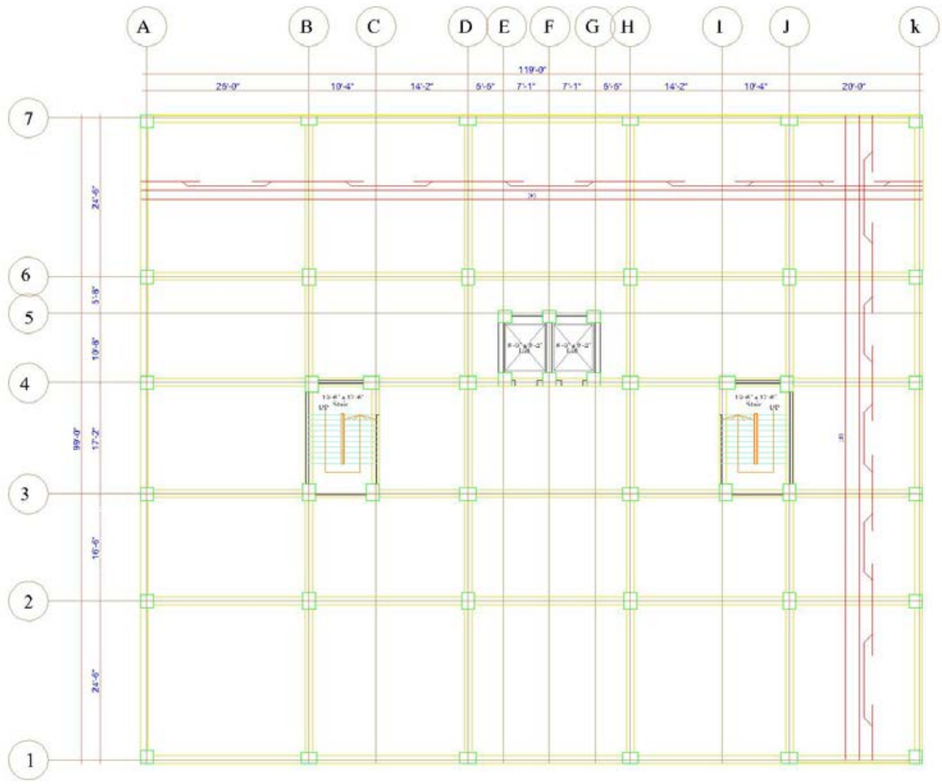


Figure 5.4: Arrangements of the bar (roof)

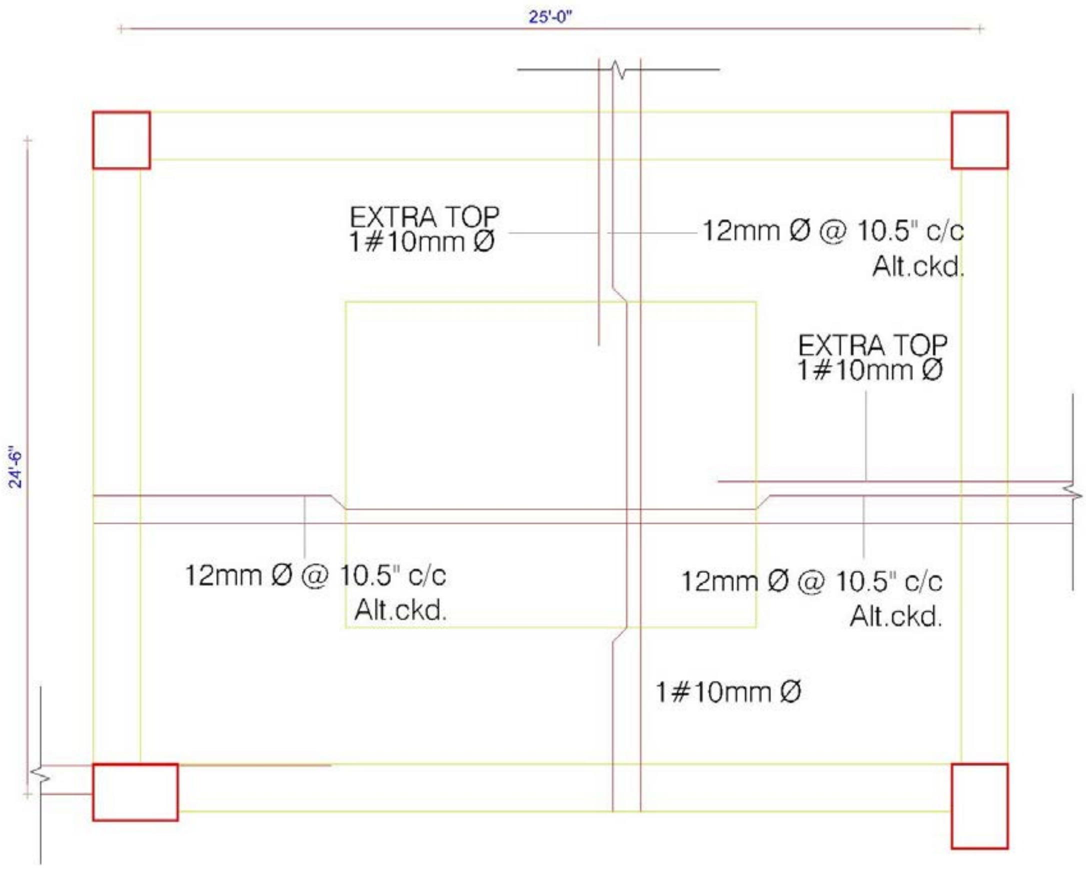


Figure 5.5: Arrangements of bars of F1 panel (roof)

5.3 Design of Beam (Ground Floor to 5th Floor)

This gives details of floor beam design for lateral loadings as per code. There are several floor beams in this picture. All beam analyzed and designed by ETABS software. Beam framing plans of ground and roof floors are shown in Figures 5.6 and 5.7 respectively. For space limitations, the design of B 14X20 and GB 14X24 are presented here.

Details of sections, moment, shear, torsion as well as reinforcement of GB 14X20 of the ground floor and B 14X24 of the roof are given in tables 5.1 – 5.2 respectively. Details of the reinforcement arrangement of B11 of the ground floor and B11 of the roof are given in figures 5.8 - 5.9 respectively.

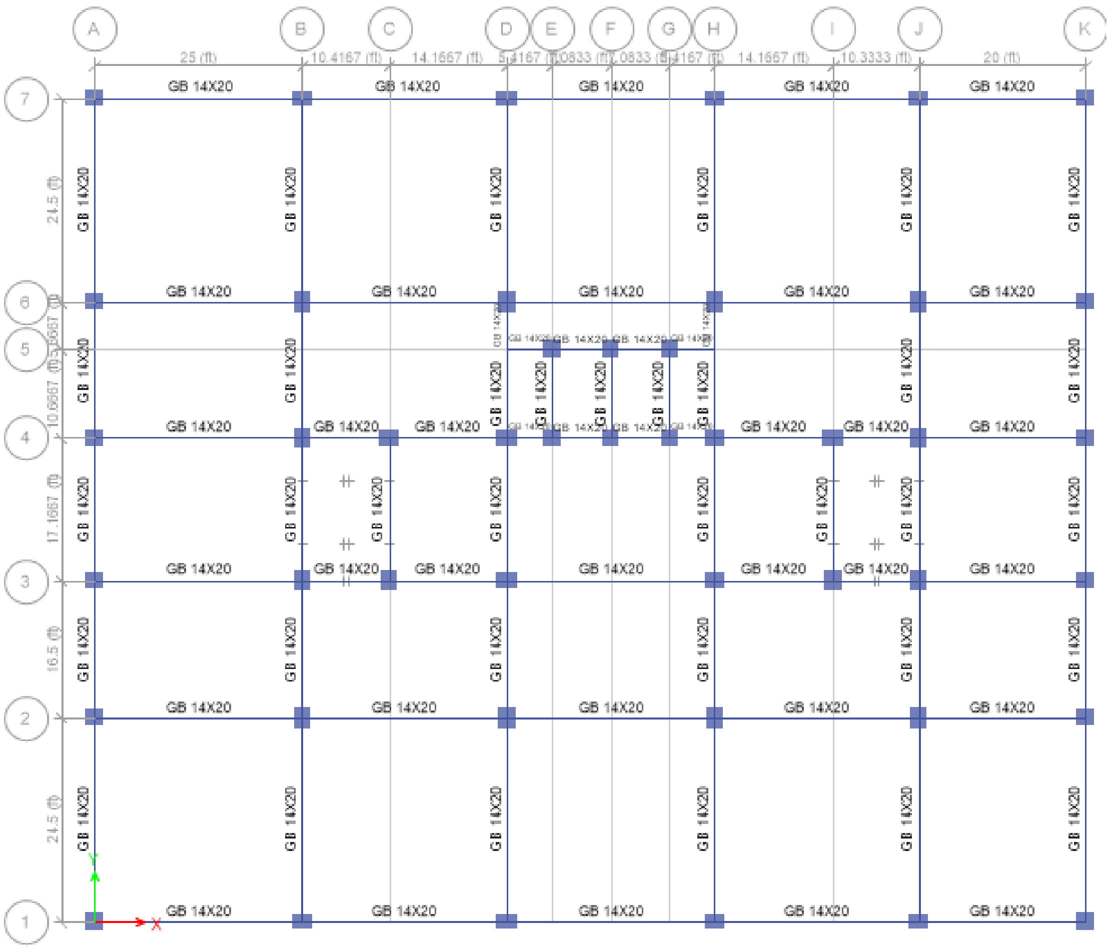


Figure 5.6: Beam framing plan (ground floor)

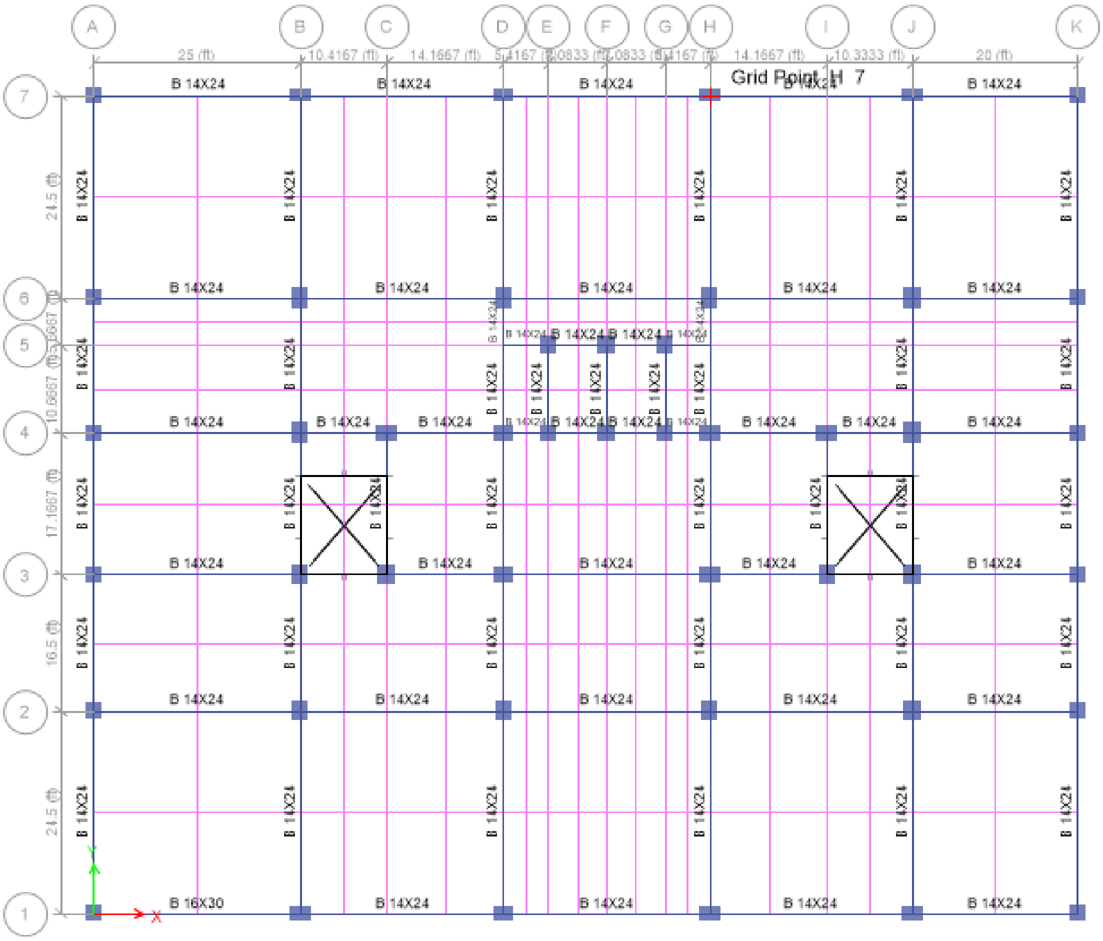


Figure 5.7: Beam framing plan (roof)

1. Dimension of the beam:

Assume the Size of Beam = b × h = 14″×20″

2. Longitudinal reinforcement of beam:

Moment, Shear and Steel area of the Beam and details from ETABS analysis are given below by table.

Table 5.1a: Details of Longitudinal Reinforcement (Bottom)

Beam Portion		**End - I**	**Middle**	**End - J**
Bottom	Moment, Mu	+73.53 K-ft	+197.65 K-ft	+71.67 K-ft
	As (Required)	1.00 in^2	2.24 in^2	1.00 in^2
		Use 2ϕ22mm Bar +1ϕ16mm Bar	Use 2ϕ22mm Bar +1ϕ16mm Bar	Use 2ϕ22mm Bar +1ϕ16mm Bar
	As (Provided)	2.37 in^2	2.37in^2	2.37 in^2

Table 5.1b: Details of Longitudinal Reinforcement (Top)

Beam Portion		**End - I**	**Middle**	**End - J**
Top	Moment, Mu	- 193.73 K-ft	- 43.52 K-ft	-217.59 K-ft
	As (Required)	2.19 in^2	0.61 in^2	2.49 in^2
		Use 4 ϕ22mm Bar	Use 2 ϕ22mm Bar	Use 4 ϕ22mm Bar
	As (Provided)	2.40 in^2	1.64 in^2	2.40 in^2

3. Transverse/Shear Reinforcement:

There will be 3 types of reinforcement for share.

Table 5.2: Details of Shear Reinforcement

Beam Portion	**End - I**	**Middle**	**End - J**
Shear Force, V_u	55.73 Kip	3.33 Kip	53.81 Kip
Shear Steel	0.38in^2/ft	0.19 in^2/ft	0.36in^2/ft

(a) Seismic Stirrup:

Use ϕ16mm as Seismic Stirrups.

- **Spacing**

S_{max} = $\frac{d}{4}$ = $\frac{17.5}{4}$ = 4.375 ≡ 4.0" c/c

S_{max} = 8×minimum dia. of main bar = 8×1= 8″ c/c

S_{max} = 24×hoops bar dia. =24 x $\frac{5}{8}$ = 15″ c/c

∴ S_{max} = 4.0" c/c is selected.

A closed hoop with seismic hook will be provided. The first one is placed 2" from each face of column. The others are placed @ 4.0" c/c within *2h* = 2 x 20= 40″ from both faces of column.

Here, $\frac{A_v}{s} = 0.38$

A_v= 0.38 × 4.0

=1.52 in²

For 2-leg 16mm stirrup and A_v= 0.31 × 2 =0.62 in²

So, A_v =1.52-0.62= 0.90 in²

3ϕ16mm cross ties are required.

(b) Splicing Stirrups:

- **Splice Length**

For top bars, class A and for bottom bars, class B lap splices will be provided.

Total splice length for top bars = l_d

Total splice length for bottom bars = 1.3 l_d

Here,

- For ϕ22mm bar

$l_d \geq \frac{0.04 \times A_b \times fy}{\sqrt{f'_c}} = \frac{0.04 \times 0.60 \times 60000}{\sqrt{3000}}$ = 26.29″

$l_d \geq 0.0003\, d_b\, f_y$ = 0.0003 × .60 × 60000 = 10.8″

l_d = 12″

Selected, l_d = 26.29″

Splice length for ϕ22mm (bottom) bars = 1.3 ×26.29 ″ = 34.18″

Splice length for ϕ22 (top) bars = 26.5″

❖ **Splice Location**

(i) Lap splices of the bars should not be placed within the beam-column joint and within a distance *2h* = 2×20= 40" from both support faces.

(ii) Lap splices of bottom bars should be made immediately beyond the *2h* distance.

(iii) Lap splices of top bars should be placed at or near mid span.

❖ **Spacing**

Total splice length of top & bottom bars should be confined by close hoops with seismic hooks.

Spacing, *S* = *d/4* = 17.5/ 4 = 4.375"≡ 4.0" c/c

or S = 4.0" c/c

So, provide ϕ16mm closed hoops with seismic hook @ 4.0" c/c along the total splice length.

(c) <u>Regular Stirrups:</u>

Except confinement zone & lap splices length for top & bottom bars, the regular stirrup ϕ16mm will be provided spacing @ $\frac{d}{2} = \frac{17.5}{2} = 8.75'' \equiv 8.5''$ c/c

Selected spacing 8.5″ c/c for regular stirrup.

Other beams of the frame were designed as per similar procedure Details of reinforcement arrangement is shown in Figure 8.8.

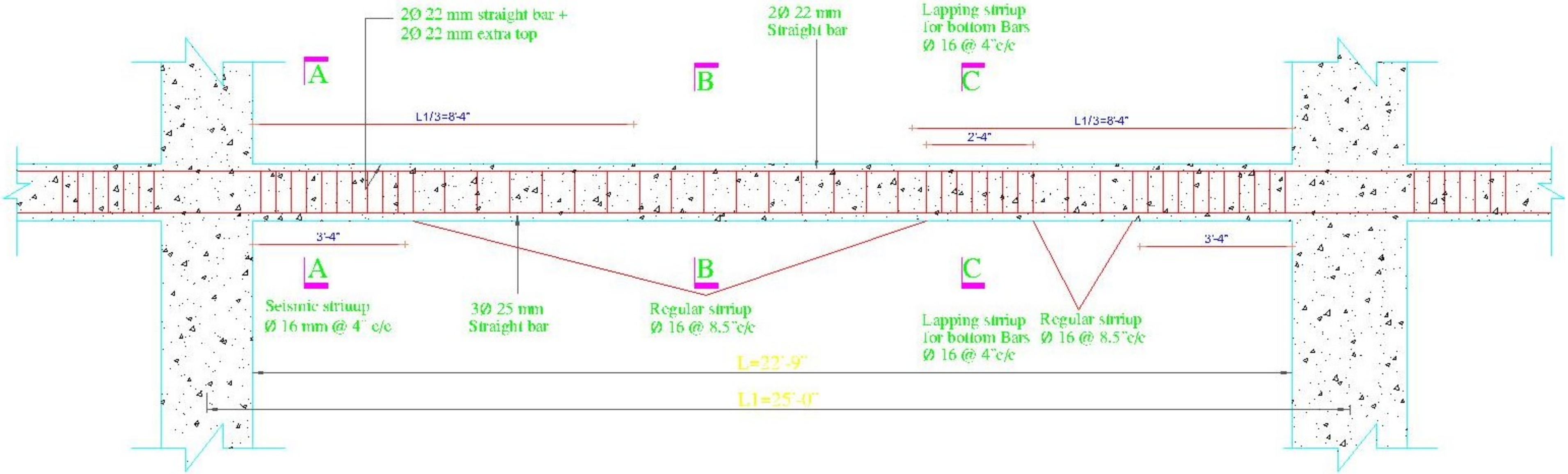


Figure 5.8a: Longitudinal Section Reinforcement of Beam

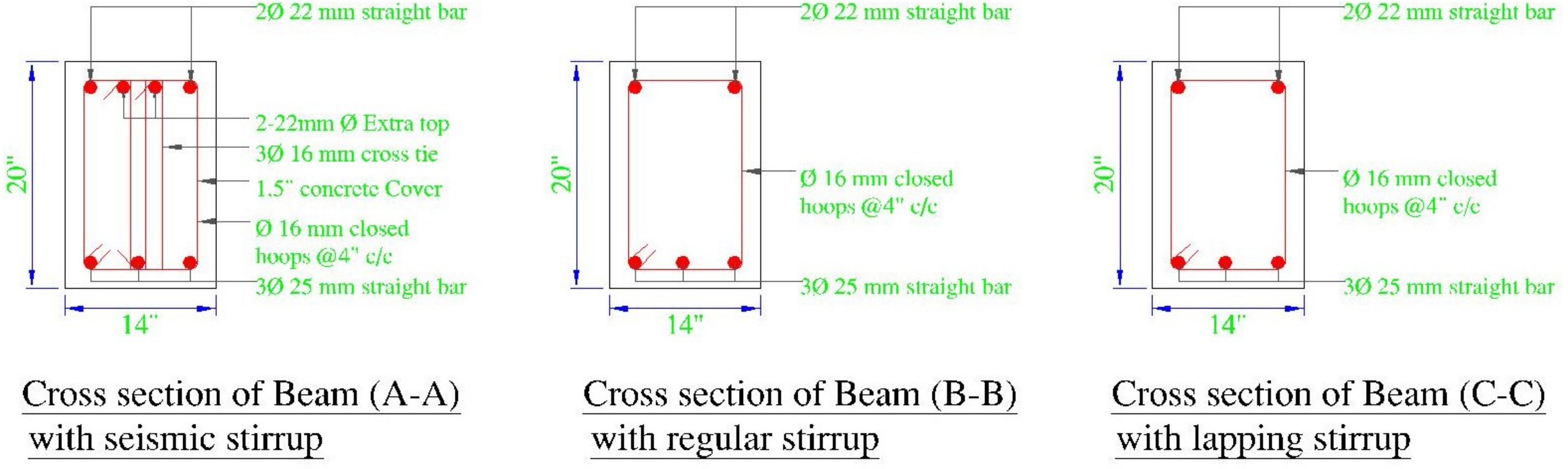
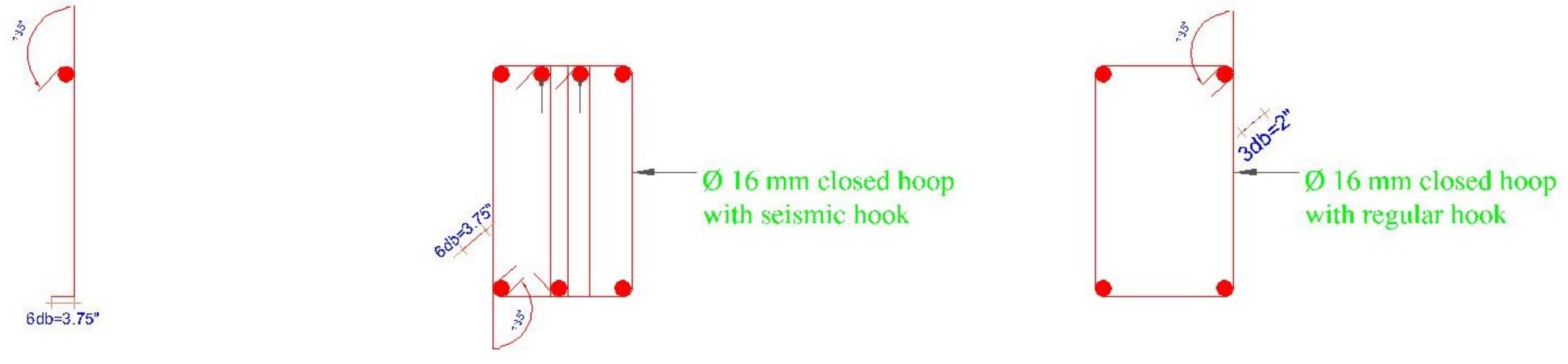


Figure 5.8b: Cross Section at three locations of Beam



Detail of hook for Ø 16 mm Longitudinal Bar

Detail of Ø 16 mm closed hoop with seismic hook and cross ties

Detail of Ø 16 mm closed hoop with regular hook

Figure 5.8c: Details of closed Hoop and Cross Stirrup of Beam

5.4 Design of Column (Ground Floor to 5th Floor)

This gives details of floor Column design for lateral loadings as per code. There are several floor beams in this picture. All beam analyzed and designed by ETABS software. Column framing plans of ground and roof floors are shown in Figures 5.10.a and 5.10.b respectively. For space limitations, the design of C1, C13, and C21 are presented here.

Details of sections, moment, shear, torsion as well as reinforcement of C1, C13, and C21 are given in tables 5.3 – 5.5 respectively. Details of the reinforcement arrangement of C1, C13 and C21 are given in figures 5.10 - 5.13 respectively.

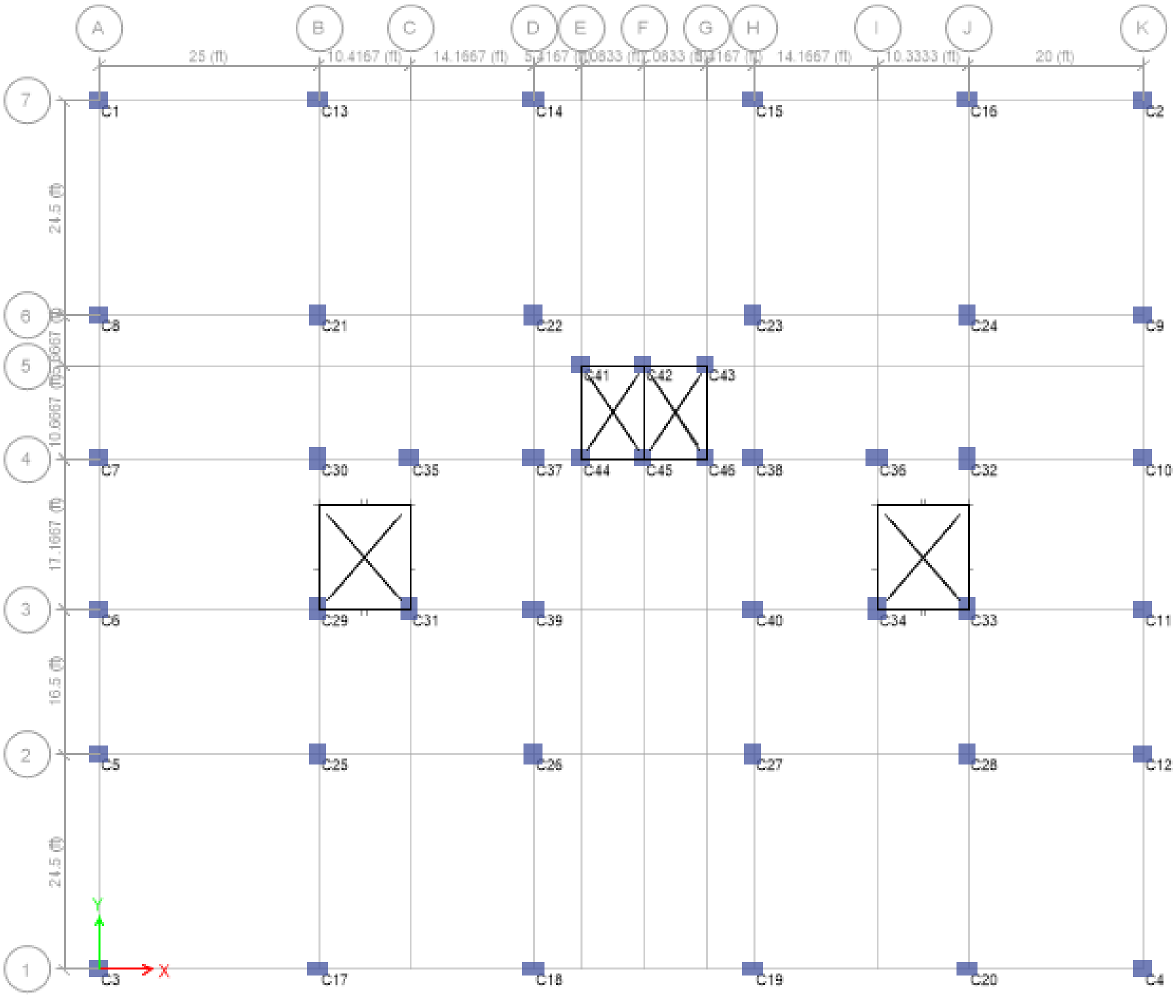


Figure 5.9.A: Concrete column layout

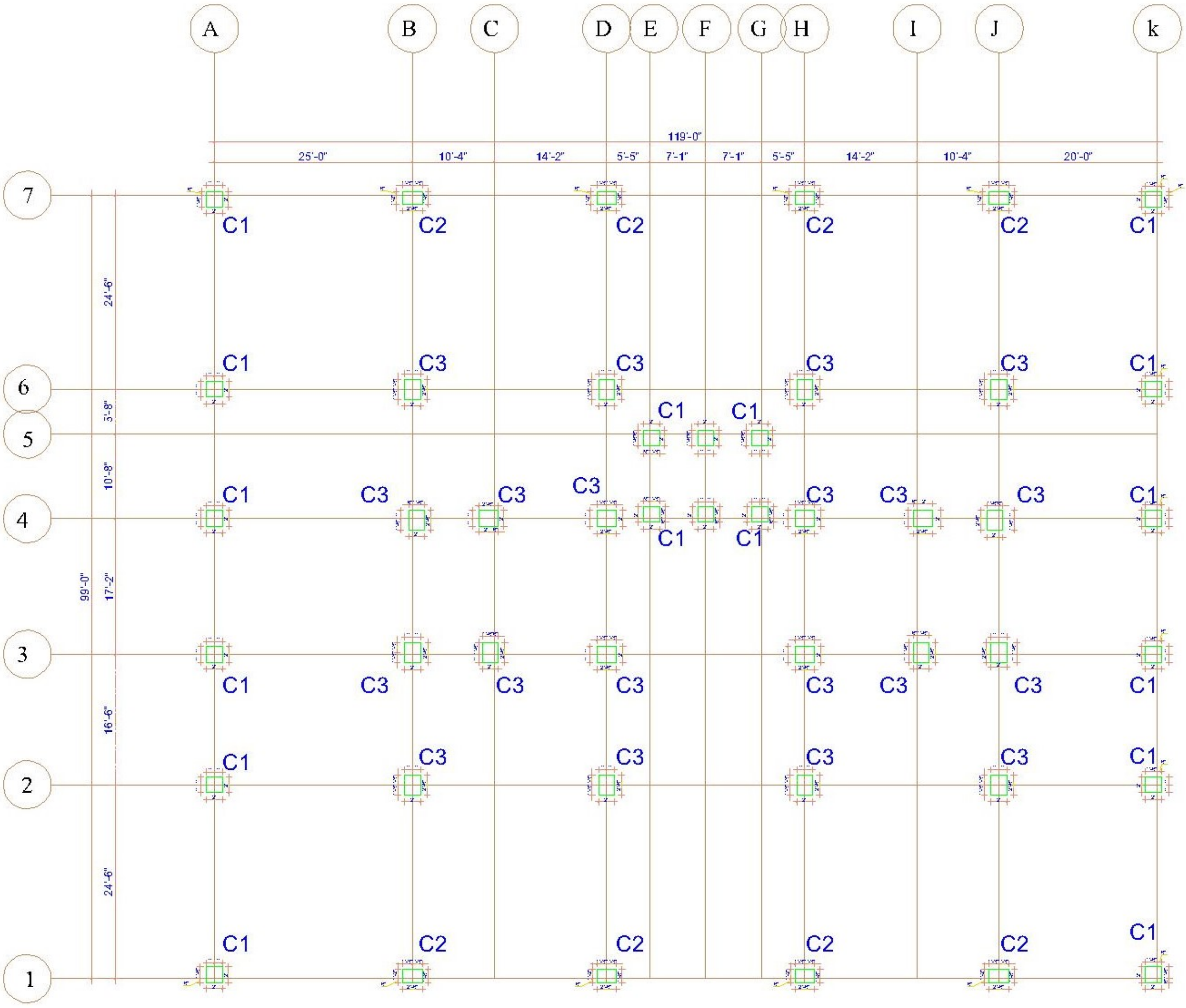


Figure 5.9.B: Concrete column layout

1. Dimension of the column:

Assume the Size of column = b × h = 24″×24″

2. Longitudinal reinforcement of column:

Moment, Shear, Axial Force and Steel area of the Column and details from ETABS, given below by Table 4.3.

Table 5.3: Longitudinal reinforcement of column

	Top	**Bottom**
Axial Force, Pu	33.988 Kip	203.401 Kip
Moment, Mu	- 44.285 k-ft	+ 148.554 k-ft
As (Required)	5.76 in^2	5.76 in^2
Required Bar	Use 14ϕ32mmBar	Use 14ϕ32mmBar
As (Provided)	6.38 in^2	6.38 in^2

3. Transverse reinforcement of column:

There are three types of ties.

Table 5.4: Transverse/Shear Reinforcement of Column

	Top	**Bottom**
Shear Force, Vu	16.83 Kip	16.83 Kip
Av/s **(Required)**	0.0 in^2	0.0 in^2

(a) Seismic Tie

Here no use closed hoops with seismic hook.

❖ **Confinement length for transverse steel:**

First condition-

l_o = Depth of Column = 24"

Second Condition-

$$l_o = \frac{Clear\ \ span\ \ of\ \ column}{6} = \frac{10-\frac{24}{12}}{6} \times 12 = 16.0"$$

Third Condition

l_o= 18"

Provided confinement length from both center of joints, l_o= 24" = 2.0'

Total confinement length =2 l_o = 2 x 2.0' = 4'

(b) Splice Tie

It is a closed hoop with seismic hook provided for splicing length of longitudinal bars. Generally lapping of bars is done at or near mid height of column.

❖ **Splicing length-**

Splicing length must be 1.3 times of development length for ϕ32mm main bar which is calculated as below.

First condition

$l_d = 0.04 \times A_b \times \frac{fy}{\sqrt{f'c}} = 0.04 \times 0.79 \times \frac{60000}{\sqrt{3000}} = 34.62''$

Second condition-

$l_d = 0.0004 \times d_b \times f_y = 0.0004 \times 0.79 \times 60000 = 30.48''$

Third condition-

l_d = minimum 12"

From above condition, selected l_d = 34.62"

Provided splicing length = 1.30 × 34.62" = 45.00"= 3.9′.

❖ **Spacing of tie**

According to the ACI code the whole splicing zone should be confined by closed hoops with seismic hooks having the spacing calculated as below:

First condition-

$S_{max} = \frac{d}{4} = \frac{24-1.5-0.5-\frac{1}{2}}{4} = 5.375'' \cong 5.0''$

Second condition-

S_{max} = minimum 4 " c / c

So, use ϕ12mm splicing ties @ 4 " c / c

(c) Regular Tie

Provided closed hoops having the spacing calculated as below:

First condition-

$S_{max} = 6d_b = 6 \times 0.79 = 4.74 \equiv 4.5''$ c/c

Second condition-

$S_{max} = 6''_{c/c}$

So, use ϕ12mm regular ties @ 4.5 " c / c

Other columns of the frame were designed as per similar procedure. The reinforcement detail of the column is shown in the Figures 5.10.

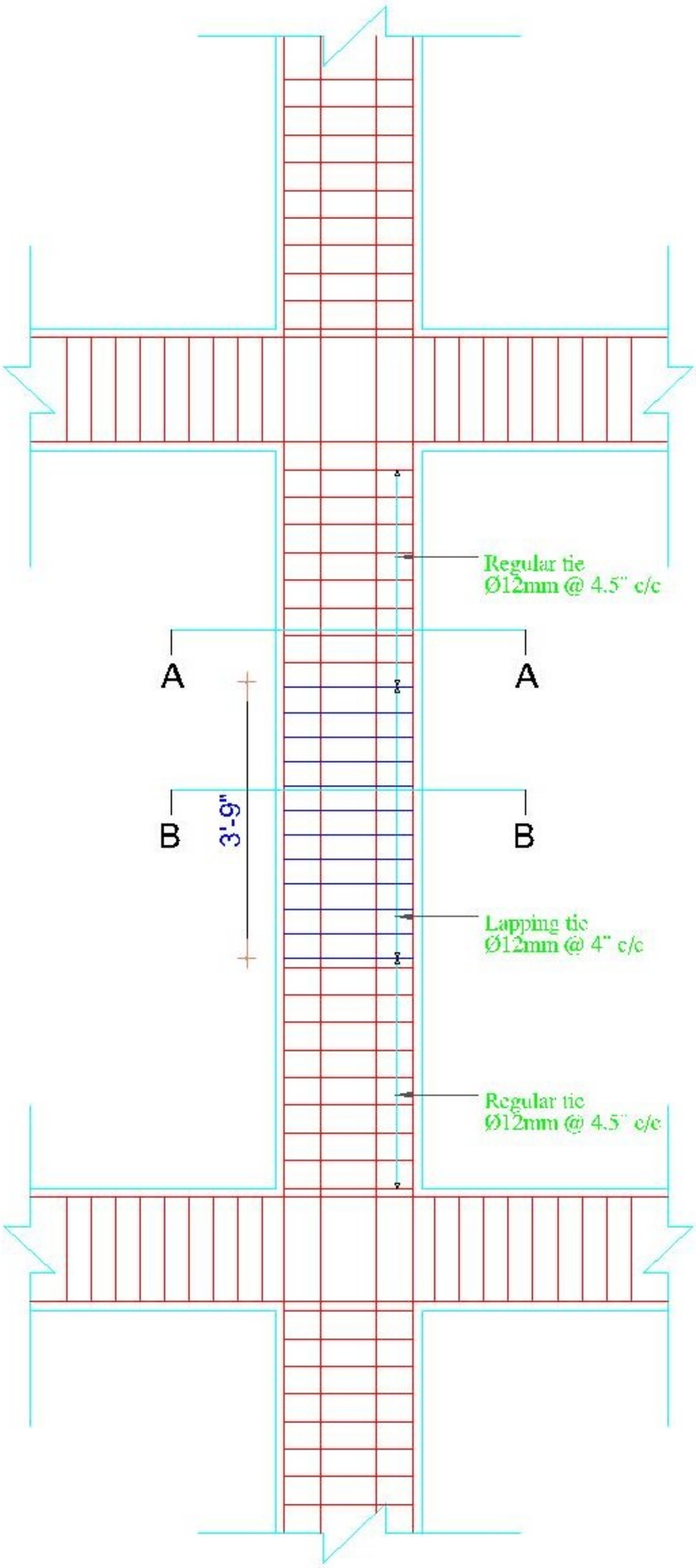


Figure 5.10a: Details of Longitudinal section of Column

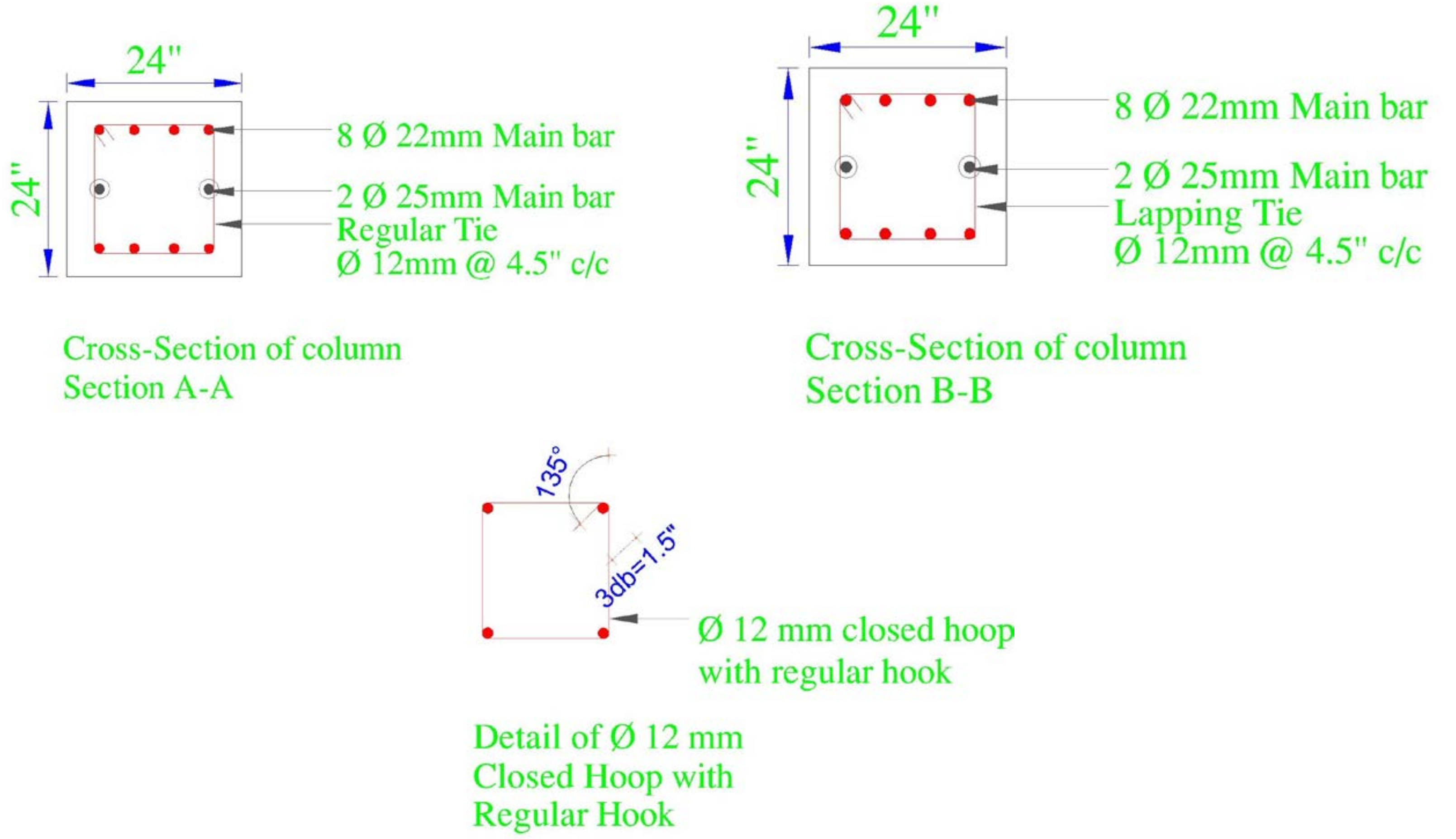


Figure 5.10b: Details of Cross Sections, Ties and Closed Hoop

CHAPTER 6

DESIGN OF STAIR

6.1 Detailing of Stair

There is one stair starts from ground floor and ends at roof top. The plan view of the Stair is shown in Figure 6.1.

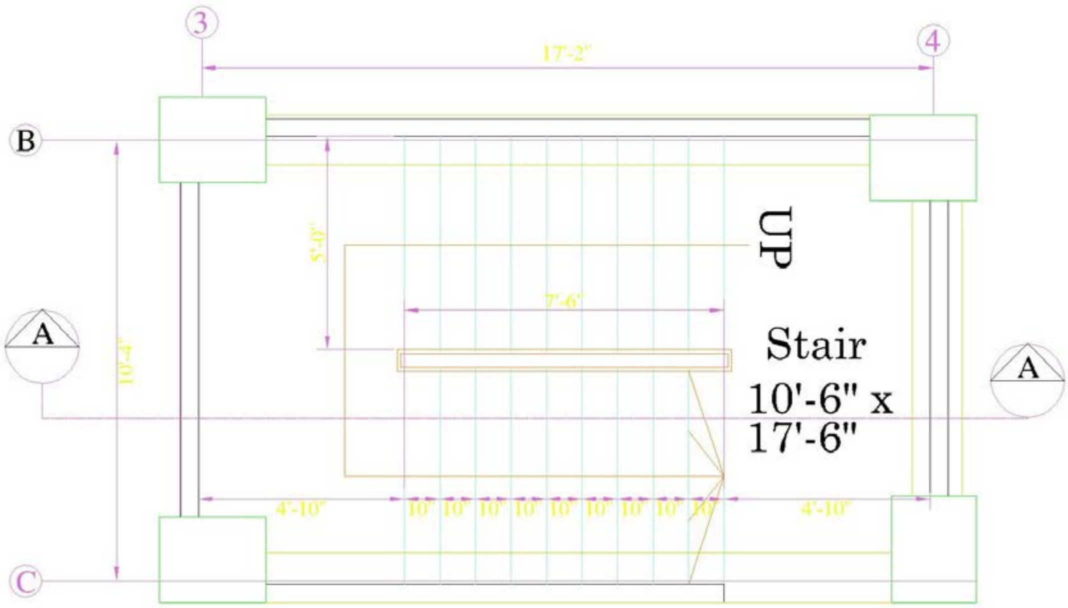


Figure 6.1a: Stair Plan

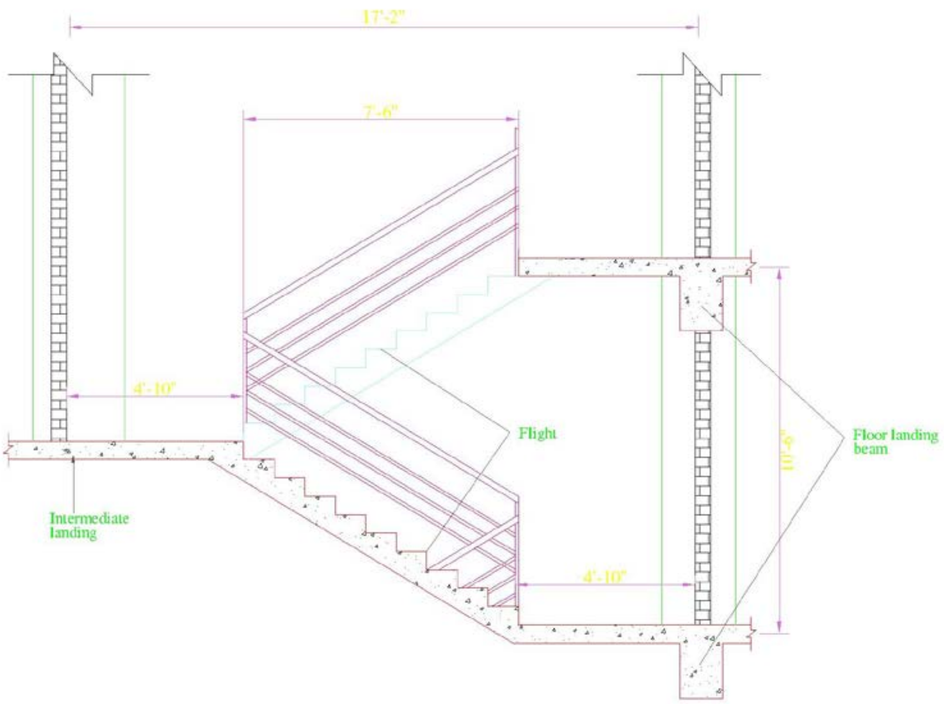
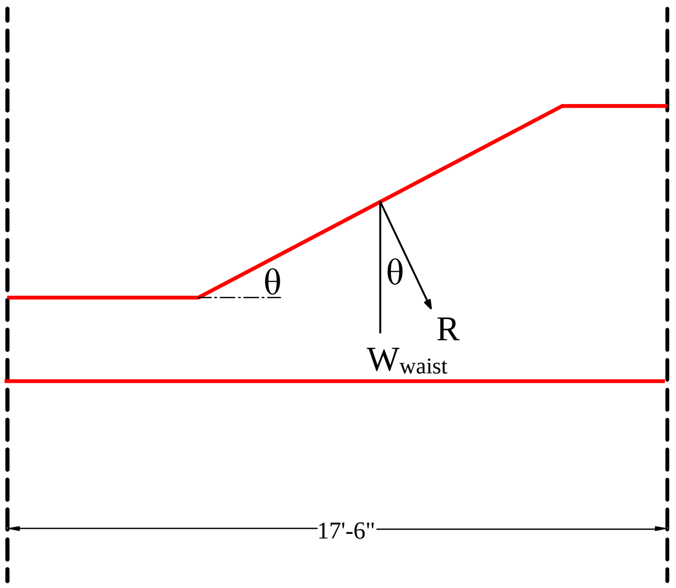


Figure 6.1b: Stair Elevation

Design Data:

Stair size	= 17'-6" x 10'-6"
Assume all beam width	= 16.0"
Story height	= 10'-6"
Height of each flight	= 5'-3"
Width of each flight	= 5'-0"
Tread (T)	=10"
Rise (R)	= 6"
No. of Rise	$= \frac{Ht.\ of\ flight}{ht.\ of\ rise} = \frac{5.25*12}{6} = 10.5 nos. \cong$ 11 nos.
No. of tread	= No. of Rise -1 =11-1 = 10 nos.



Analysis and Design of Stairs presented below:

Effective span length = 17'-6''

Assume, waist slab thickness = 6"

Effective depth, *d* = 6-1 =5"

Here,

W_{waist} = R*Cos θ

$$\mathbf{R} = \frac{W_{waist}}{\cos\theta} = \frac{\frac{Thickness\ of\ waist\ slab}{12} * unit\ wt.\ of\ conc.}{\cos\theta}$$

I) Load calculation:

- Self-weight of waist slab $= \frac{6}{12} * 150 * \frac{1}{\cos\theta}$ $\quad [\cos\theta = \frac{10}{\sqrt{5^2+10^2}} = 0.894$

  $= \frac{6}{12} \times 150 \times \frac{1}{0.894}$ = 83.89 *lb/ft*

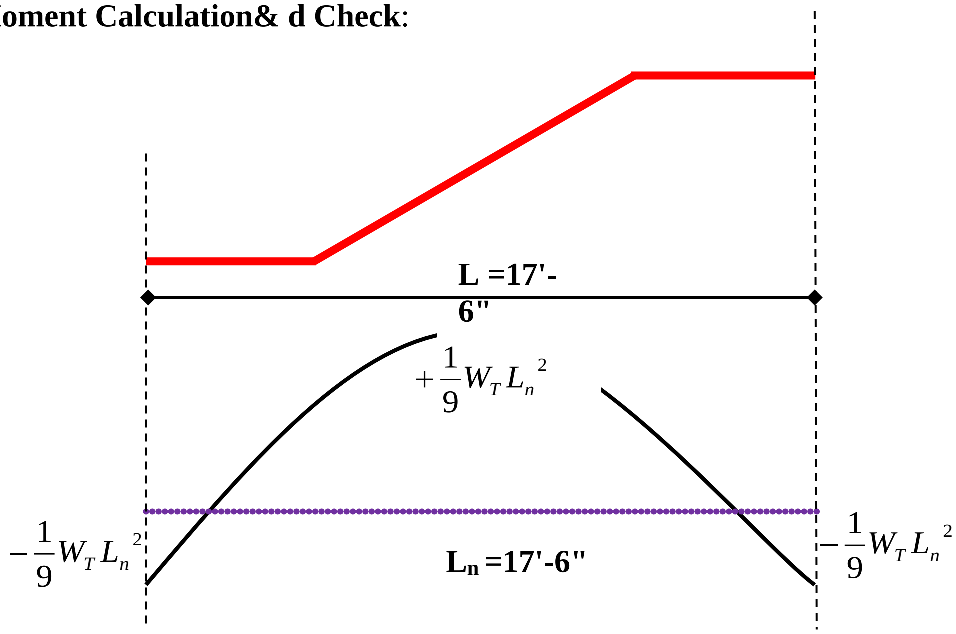
- Self-weight of steps $= \frac{\frac{1}{2} \times \frac{12}{12} \times \frac{6}{12} \times 10.50 \times 150}{10}$ = 39.375 *lb/ft*
- Floor finish = 30.0 *lb/ft*
- Live load = 150 *lb/ft*

Total unfactored dead load = 83.89+39.75+30 = 153.64 *lb/ft*

Total unfactored live load = 150 *lb/ft*

Total factored load, W_t = 1.2×153.64 +1.6×150 = 424.368 *lb/ft*

II) Moment Calculation& d Check:



-ve moment at both supports $= \frac{1}{9} W_T l_n^2 = \frac{424.368 \times 17.5^2}{9}$ = 14440.30 *lb-ft* = 14.44 *k-ft*

+ve moment at mid span $= \frac{1}{9} W_T l_n^2 = \frac{424.368 \times 17.5^2}{9}$ = 14440.30 *lb-ft* = 14.44 *k-ft*

Effective depth will be checked for maximum moment which is 14.44 *k-ft*.

$\rho_b = 0.85\beta_1 \frac{f'_c}{f_y} \times \frac{87000}{87000+f_y} = 0.85 \times 0.85 \times \frac{3000}{60000} \times \frac{87000}{87000+60000}$ = 0.02138

$\rho_{\max} = 0.75\rho_b = 0.75 \times 0.02138 = 0.016035$

$$M_u = \phi\rho bd^2 f_y\left(1 - 0.59\rho\frac{f_y}{f'_c}\right)$$

$$=> 14.44 * 12 = 0.90 \times 0.016035 \times 12 \times d^2 \times 60 \times \left(1 - 0.59 \times 0.016035 \times \frac{60}{3}\right)$$

So, $d = 4.53" < 5"\ ok.$

III) A_s calculation:

Main Steel:

+Ve steel for mid span

M_u= 14.44 *k-ft*   b = 1*ft* strip of waist slab =12"

Selected bar is *Ø12mm* and A_b = 0.20 in^2

- Minimum steel requirement for 60-grade bar, $A_{st} = 0.0018bh$ = 0.0018x12x6 = 0.13 in^2/ft

  Total required reinforcement, $As_{req} = \rho bd$

$$= \frac{0.85x3}{60}\left[1 - \sqrt{1 - \frac{2 \times 14.44 \times 12}{0.85 \times 0.90 \times 3 \times 12 \times 5^2}}\right] \times 12 \times 5 = 0.753in^2/ft$$

$$> 0.13in^2/ft$$

$$= \frac{0.85f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2M_U}{0.85\varphi f'_c bd^2}}\right] bd$$

So provided steel area, $A_s = 0.753\ in^2/ft.$

- Maximum spacing, S_{max} = Smaller of 3h or 18″= Smaller of 3x6″ or 18″= 18

- Spacing $= \frac{A_b \times 12}{A_s} = \frac{0.20 \times 12}{0.753}$= 3.19" c/c = 3.0" c/c

Use *Ø12mm* bar @ 3.0" c/c,

<u>-Ve steel for both supports</u>

Moment for mid span and both supports are same as ***14.44 k-ft***. Reinforcement requirement and spacing of the main bars at both supports will also be same as mid span which is *Ø12mm* bar @ 3.0" c/c.

So, use 1*Ø12mm* extra top in between two bars in order to maintain the spacing 3.0" at those locations.

Temperature & Shrinkage Steel:

For 60-grade bar, $A_{st} = 0.0018bh$ = 0.0018x12x6 = 0.13 in^2/ft

Use *Ø10mm* bar, area is 0.11 in^2

Spacing $= \frac{0.11*12}{0.13}$ =10.15"=10" c/c

Use *Ø10mm* bar @10" c/c above the main bars at mid span and below at both supports in opposite direction of the main bars.

Reinforcement details of stair is given Figure 6.2

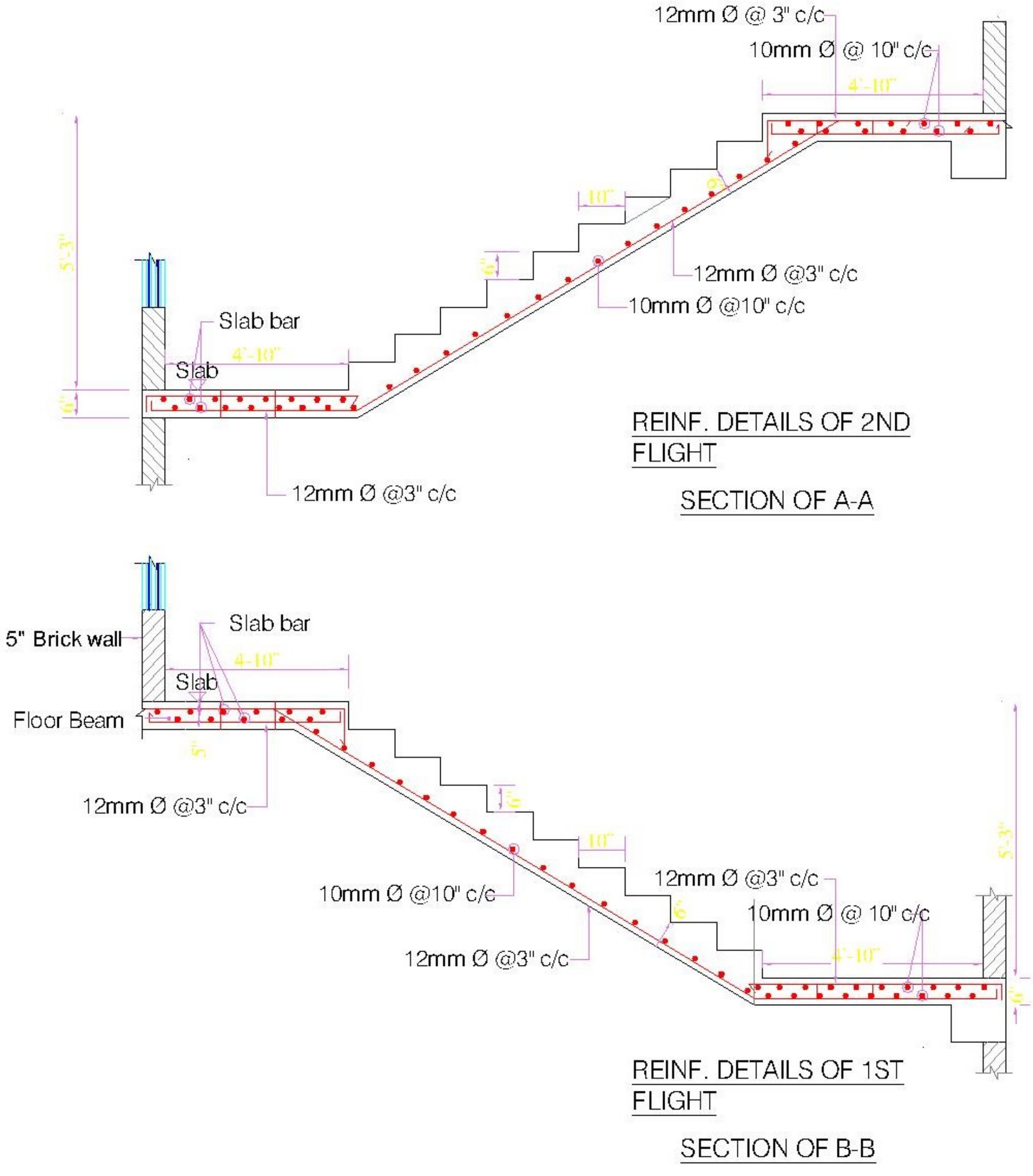


Figure 6.2: Reinforcement Details of Stair

CHAPTER 7

DESIGN OF OVER HEAD WATER TANK

7.1 Detailing of Overhead Water Tank

There is one overhead tank constructed. For space limitations, design of OHWT is presented here.

Length	= 14’-2”
Width	= 10’-8”
Tank height	= 5’-0”
Free board	= 0’-6”
Total tank height	= 5’-0” + 0’-6” = 5’-6”
Select tank size	14’-2” X 10’-8” X 5’-6”

Overhead water tank is analyzed, designed and detailed by ETABS and SAP software. Details of reinforcement arrangement of OHWT are shown in Figure 7.1 – 7.3

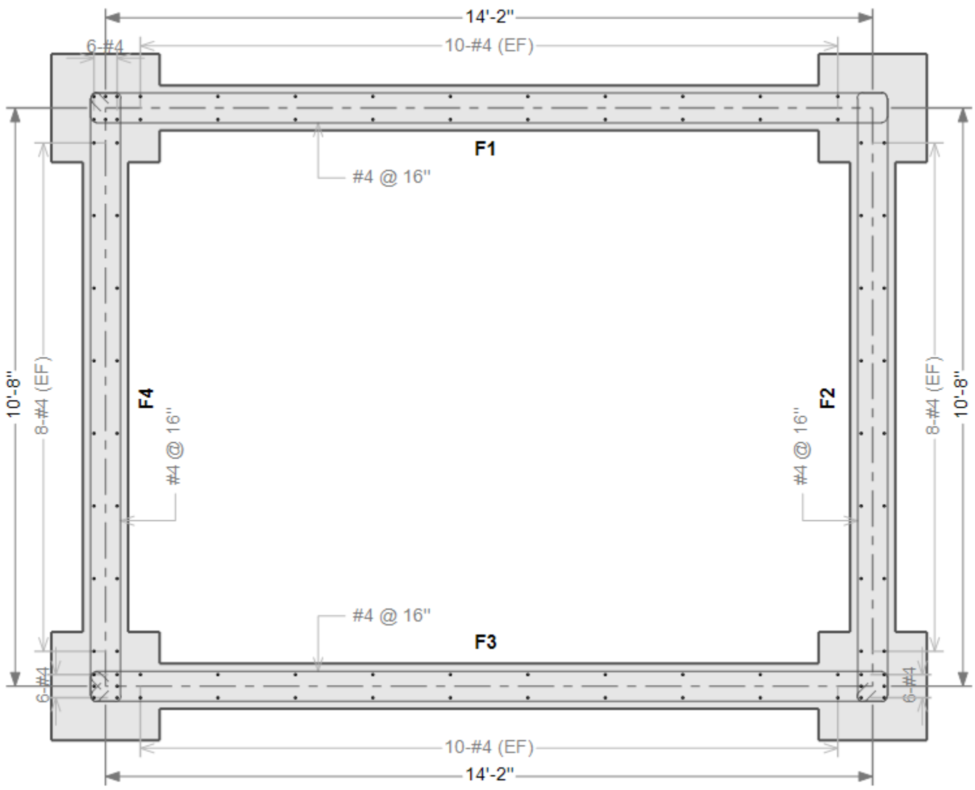


Figure 7.1: Reinforcement details of OHWT wall cross section

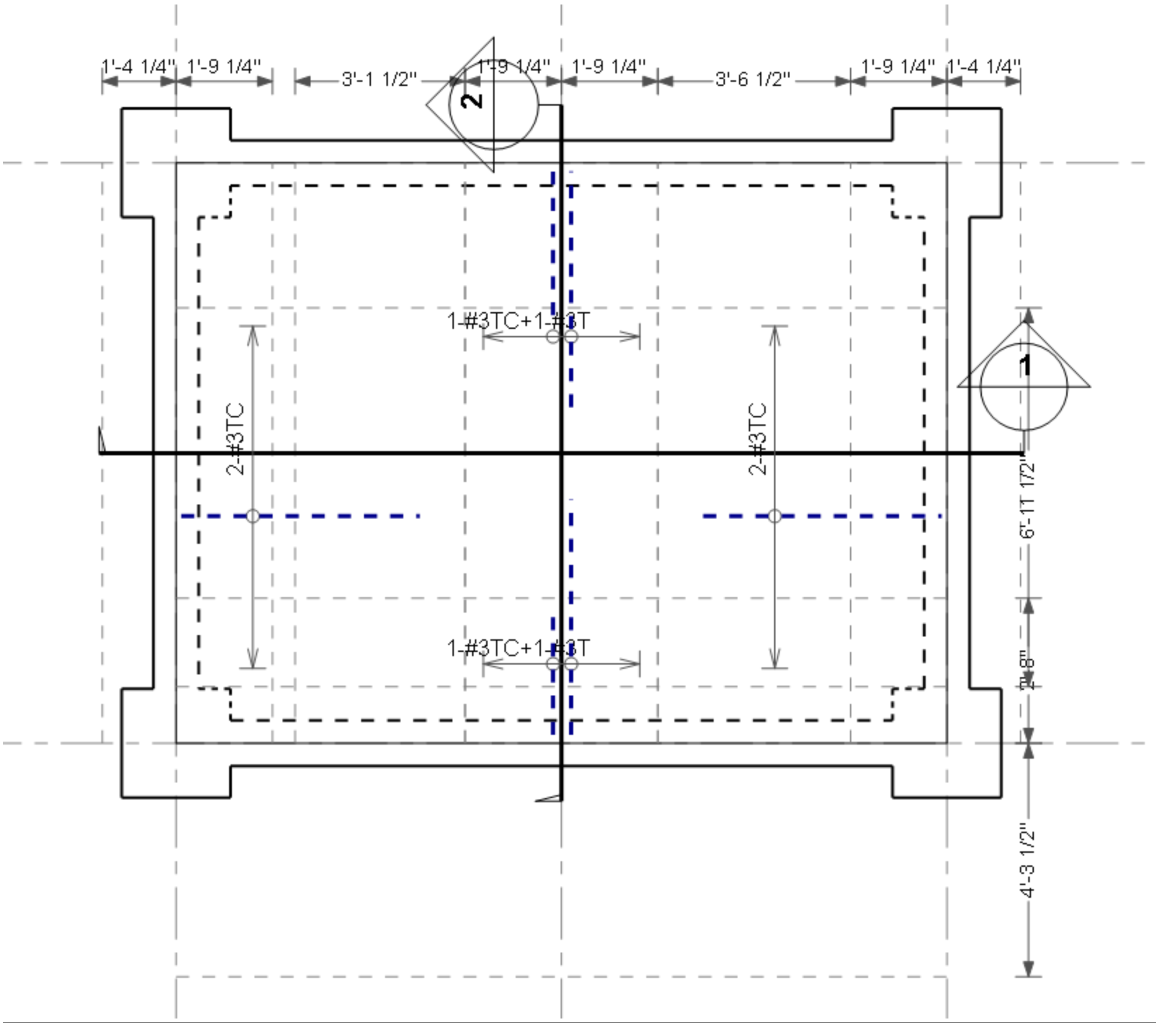


Figure 7.2: Slab detailing of OHWT TOP

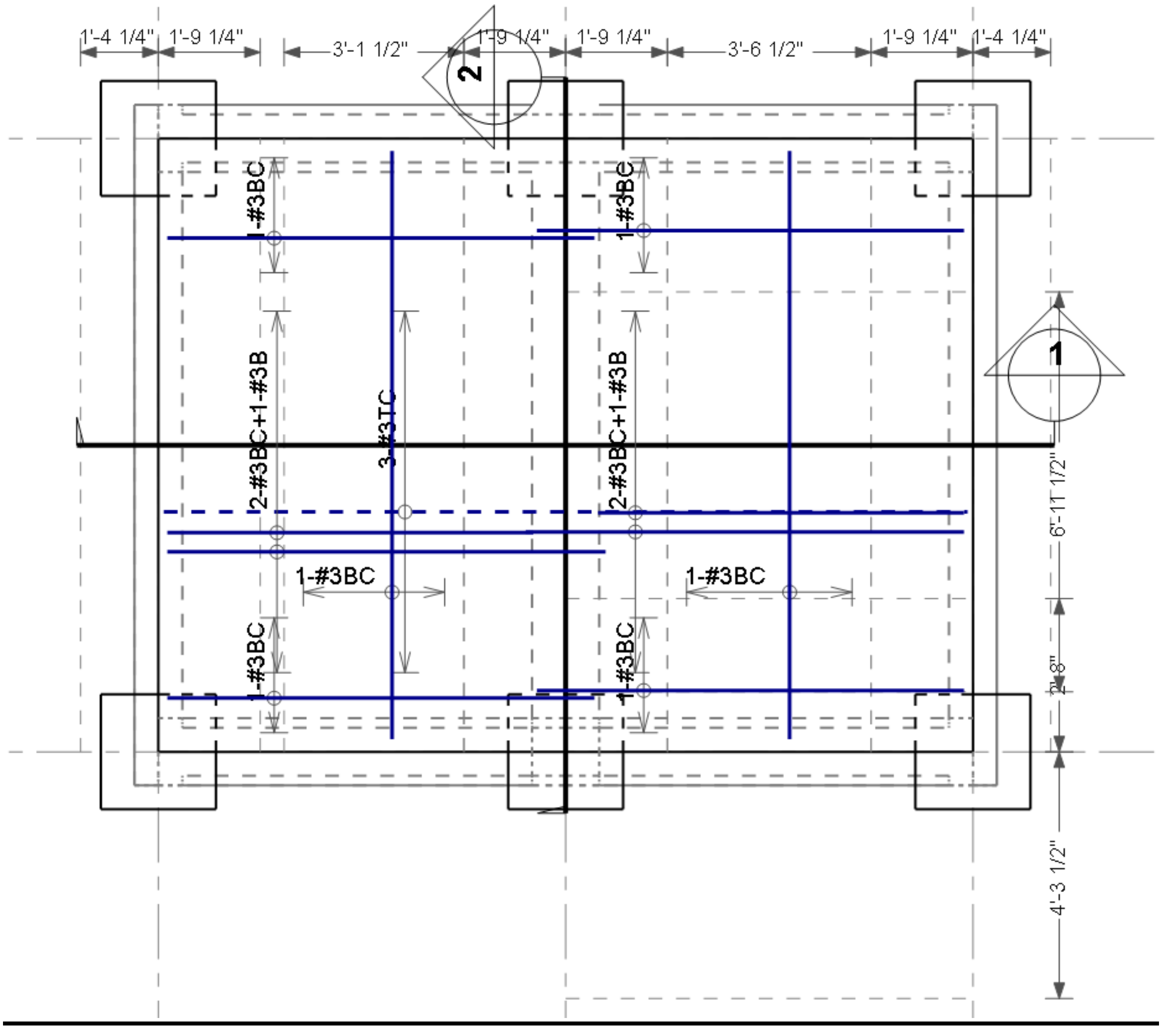


Figure 7.3: Slab detailing of OHWT BOTTOM

CHAPTER 8

DESIGN OF UNDER GROUND WATER RESERVOIR

8.1 Detailing of Underground Water Reservoir

There is one underground water reservoir (15'-0" x 22'-6") on soil for fulfilling the water demand of the Factory & Gurage. Water storage is made also considering one-hour Fire Fighting works. The design of this water tank will be done as per WSD.

Determination of water requirement

*** Factory purposes**

Water requirement = 40 *L/P/D*

No. of Persons = 250 (Assume)

Total required volume of water = 40×250 = 10000 *L/P/D*

*** Garage purposes**

Water requirement = 70 *L/P/D*

No. of Persons = 20 (Assume)

Total required volume of water = 70×20 = 1400 *L/P/D*

* **Fire safety purposes**

Consider, water storage for 1 hr. fighting,

Water requirement for 1 floor = 265 *gal/min*

= 265 × 60 *gal/hr* = 15900 *gal/hr*

∴ Water requirement for two floor = 2 × 15900 *gal*

= 31800 *gal*

= 120376 *L*

Total water requirement for whole structures = (120376 +10000+1400)

= 131776 *L*

∴ Water requirement for tank = 131776 L = 131.77 m^3

= 131.77× 3.28^3 ft^3

= 4649.84 ft^3

= 4649.8 ft^3

Tank dimension

Inside width dimension, B = 15 *ft*

Height = 6.5 *ft*

Free board = 0.5 *ft*

Final height = 6.5+0.5 = 7 *ft*

So inside length dimension, L = 4649.84/ (15×7) = 44.28*ft* = 45*ft*

Hence the dimension of the tank 2 compartment will be 15 ft wide and 22.5 ft long.

A. Design of long walls

Both long walls will be designed considering empty condition.

Pressure exerted by dry soil $= wh\dfrac{1-\sin\phi}{1+\sin\phi}$

$$=15 \times 2.13 \times \frac{1-\sin 30}{1+\sin 30} = 11.99 kN/m^2$$

$$\therefore p = 11.99 kN/m^2$$

Thickness of the wall:

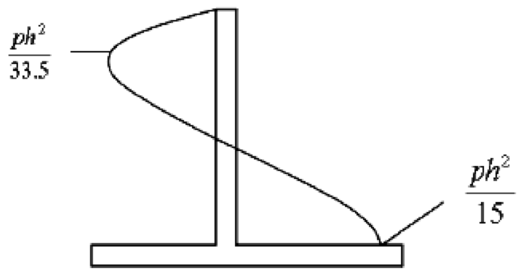
Moment at outer face of long wall

$$=\frac{ph^2}{33.5} = \frac{11.99 \times 2.13^2}{33.5} = 1.624 \ kN\text{-}m/m$$

$$= \frac{1.624 \times 1000}{4.448 \times 0.3048} \times \frac{12}{1000}$$

$$= 14.37 k - in$$

(Per meter run)



So, tension per feet run= 14.37 ×0.3048 = 4.38 *k-in/ft*

Moment at inner face of long wall,

$$M_{\max} = \frac{ph^2}{15} = \frac{11.99x\,2.13^2}{15} = 3.63 \ kN - m/m$$

= 32.13 *k-in* (per meter run) = 9.80 *k-in* (per ft run)

From cracking consideration, the thickness of long wall will be determined.

D = total thickness of tank wall,

$$M = \frac{F_{ct} b D^2}{6}$$

$$\therefore \ D^2 = \frac{6x9.80}{0.410x12}$$

$$D = 3.46" \equiv 10.0" \ (preferable \text{ minimum } thickness)$$

[Here $f_{ct} = (6 \rightarrow 8)\sqrt{f_c'}$ and let, $f_{ct} = 7.5\sqrt{f_c'} = 7.5 \times \sqrt{3000} = 410.79 psi$]

∴ Effective depth = 10-1.5= 8.50 *inch*

Vertical reinforcement:

$f'_c = 3000\ psi$

$f_c = 0.45\ f'_c = 0.45 \times 3000 = 1350\ psi$

$f_y = 60{,}000\ psi$.

$f_s = 0.50\ f_y = 0.50 \times 60000 = 30000\ psi$

$E_s = 29 \times 10^6\ psi$

$E_C = 57{,}000\sqrt{3000} = 3.1 \times 10^6\ psi$

$n = \frac{E_S}{E_C} = \frac{29 \times 10^6}{3.1 \times 10^6} = 9$

$r = \frac{f_S}{f_C} = \frac{30000}{1350} = 22.22$

$k = \frac{n}{n+r} = \frac{9}{9 + 22.22} = 0.288$

$J = 1 - \frac{K}{3} = 1 - \frac{0.288}{3} = 0.904$

Vertical Reinforcement for inner face of wall

M = 11.57 *k- in* (per ft run)

Steel requirement, $A_S = \frac{M}{f_s jd} = \frac{9.80*1000}{30000*0.904*8.5} = 0.043\ in^2/ft$

$Minimum\ A_s = 0.003bh = 0.003x12x10 = 0.36\,in^2\ /\ ft$

Selected $A_s = 0.36\ in^2/ft$.

Use ϕ12mm, spacing = $\frac{0.20x12}{0.36} = 6.67'' \equiv 6.5''\,c/c$

$S_{max} = 3h = 3 \times 10'' = 30''$

$S_{max} = 18''$

So, use ϕ12mm @ 6.5"c/c.

Vertical Reinforcement for outer face of wall

M = 5.19 *kip-in* (per ft run).

$A_S = \frac{M}{f_s jd} = \frac{4.38*1000}{30000*0.904*8.5} = 0.019\ in^2/ft$

$Minimum\ A_s = 0.003bh = 0.003x12x10 = 0.36\,in^2\ /\ ft$

Selected $A_s = 0.36\ in^2/ft$.

Use ϕ12mm, spacing = $\frac{0.20x12}{0.36} = 6.67'' \equiv 6.5'' c/c$

So, use ϕ12mm @ 6.5"c/c.

- **Horizontal reinforcement:**

Minimum steel will be placed as binder.

$Minimum\ A_s = 0.003bh = 0.003x12x10 = 0.36 in^2 / ft$

Use ϕ12mm, spacing = $\frac{0.20x12}{0.36} = 6.67'' \equiv 6.5'' c/c$

So, use ϕ12mm @ 6.5"c/c at both faces.

B. Design of short wall

Earth pressure at the bottom, P =11.99 kN/m^2

Max moment at the center, M = $\frac{PL^2}{12}$

$$L = 15 + \frac{10}{12} = 15.83\,ft = 4.82\,m$$

$$M = \frac{11.99x6.35^2}{12} = 23.21\ k - in/meter = 7.08\,k - in/ft$$

Now check ‘*d*’,

$$M_{\max} = \frac{f_c}{2} jkbd^2$$

$$\therefore d = \sqrt{\frac{2x7.08}{1.35x0.288x0.904x12}} = 1.83''\ < provided\, d = 8.5''\ ok.$$

- **Vertical reinforcement:**

M = 7.08 *kip-in* (per ft run).

$$A_S = \frac{M}{f_s jd} = \frac{7.08*1000}{30000*0.904*8.5} = 0.031\ in^2/ft$$

$Minimum\ A_s = 0.003bh = 0.003x12x10 = 0.36 in^2 / ft$

Selected A_s = 0.36 in^2/ft.

Use ϕ12mm, spacing = $\frac{0.20x12}{0.36} = 6.67'' \equiv 6.5'' c/c$

So, use ϕ12mm @ 6.5"c/c at both faces.

- **Horizontal reinforcement:**

Minimum steel will be placed as binder.

$Minimum\ A_s = 0.003bh = 0.003x12x10 = 0.36\, in^2\ /\ ft$

Use ϕ12mm, spacing = $\frac{0.20x12}{0.36} = 6.67'' \equiv 6.5''c/c$

So, use ϕ12mm @ 6.5"c/c at both faces.

C. Design of top slab

$\frac{L}{B} = \frac{45}{15} = 3 > 2$

So, it is a one- way slab.

Minimum thickness of the slab,

$h = \frac{15}{28}x12 = 6.43''$ And take h = 10″

Load calculation:

Live load	=	10 *psf*
Self-weight of the slab	=	$\frac{10}{12}x150$ = 124.5 *psf*
Floor finish (assume)	=	10 *psf*
Vehicular load	=	50 *psf*

Total load, W = 194.5 *psf* = 0.195 *ksf*

d check:

$\frac{a}{b} = 15/45 = 0.33$

Case 1

$M_{a,neg} = 0$

$M_{b,neg} = 0$

For positive moment, $C_a dl = 0.0954$

$C_b dl = 0.0061$

$M_{a.positive}$ = 0.0954× 0.195 × 15^2 = 50.23 *k-in*

$M_{b.positive}$ = 0.0061 × 0.195 × 45^2 = 28.90 *k-in*

$$M_{max} = \frac{f_c}{2} jkbd^2$$

$$\therefore d = \sqrt{\frac{2x50.23}{1.35x0.288x0.904x12}} = 4.88" < provided\, d = 10 - 1 = 9"\ ok.$$

Reinforcement calculation:

- **Main steel (short direction)**

M = 50.23 *kip-in* (per ft run).

$$A_S = \frac{M}{f_s jd} = \frac{50.23*1000}{30000*0.904*9} = 0.21\ in^2/ft$$

$$Minimum\ A_s = 0.003bh = 0.003x12x10 = 0.36 in^2 / ft$$

Selected A_s = 0.36 in^2/ft.

Use ϕ12mm, spacing = $\frac{0.20x12}{0.36} = 6.67" \equiv 6.5" c/c$

So, use ϕ12mm @ 6.5"c/c.

- **Main steel (long direction)**

M = 28.90 *kip-in* (per ft run).

$$A_S = \frac{M}{f_s jd} = \frac{28.90*1000}{30000*0.904*9} = 0.11\ in^2/ft$$

$$Minimum\ A_s = 0.003bh = 0.003x12x10 = 0.36 in^2 / ft$$

Selected A_s = 0.36 in^2/ft.

Use ϕ12mm, spacing = $\frac{0.20x12}{0.36} = 6.67" \equiv 6.5" c/c$

So, use ϕ12mm @ 6.5"c/c.

D. Design of bottom slab:

$\frac{L}{B} = \frac{22.5}{15} = 1.5 \leq 2$

So, it is a Two- way slab.

Let the thickness of the slab is 20″.

Load calculation:

Water pressure	= 62.5×6.5	= 406.25 *psf*
Self-weight of the slab	$= \frac{20}{12} \times \frac{4}{12} \times 150$	= 250 *psf*
Floor finish & LL (assume)		= 20.0 *psf*

Total load, W = 676.25 *psf* = 0.676 *ksf*

Check for depth *d*:

$\frac{a}{b}$ = 15/22.5= 0.67

Case 1

$M_{a,neg} = 0$

$M_{b,neg} = 0$

For positive moment, C_a dl = 0.0716

C_b dl = 0.0118

$M_{a,positive} = 0.0716 \times 0.676 \times 15^2 = 130.68$ *k-in*

$M_{b,positive} = 0.0118 \times 0.676 \times 22.5^2 = 48.87$ *k-in*

$$M_{max} = \frac{f_c}{2} jkbd^2$$

$$\therefore d = \sqrt{\frac{2x130.68}{1.35x0.288x0.904x12}} = 7.87'' < provided\, d = 20 - 1 = 19''\ ok.$$

Check against floatation:

The whole tank must be checked against floatation when it is empty.

Because of saturated subsoil, there will be uplift pressure on the bottom slab.

- Total up- ward flottation force, $P_u = \gamma h \times B \times L$

  = (62.5×7) × (15+1.67) × (22.5+1.67) = 176.27 *k*

- Weight of 10″ thick long and short walls

  = 0.83 (2×16.67+2×24.17) × 7 ×150 = 71.18 *k*

  Weight of 10″ top slab = 0.83×16.67×24.17 ×150 = 50.16 *k*

  Weight of 20″ base slab = 1.67 ×16.67×24.17×150 = 100.93 *k*

  Total downward weight = 222.27 *k*

This is greater than floatation force 176.27 *k*, so reservoir is safe in design.

Reinforcement calculation:

- **Main steel (short direction)**

  M = 130.68 *kip-in* (per ft run).

  $A_S = \frac{M}{f_s jd} = \frac{130.68*1000}{30000*0.904*19} = 0.25\, in^2/ft$

  $Minimum\ A_s = 0.003bh = 0.003x12x20 = 0.72\, in^2/ft$

  Selected A_s = 0.72 in^2/ft.

  Use ϕ12mm, spacing = $\frac{0.20x12}{0.72} = 3.33" \equiv 3"\, c/c$

  So, use ϕ12mm @ 3"c/c.

- **Main steel (long direction)**

  M = 48.87 *kip-in* (per ft run).

  $A_S = \frac{M}{f_s jd} = \frac{48.87*1000}{30000*0.904*19} = 0.10\, in^2/ft$

  $Minimum\ A_s = 0.003bh = 0.003x12x20 = 0.72\, in^2/ft$

  Selected A_s = 0.72 in^2/ft.

  Use ϕ12mm, spacing = $\frac{0.20x12}{0.72} = 3.33" \equiv 3"\, c/c$

  So, use ϕ12mm @ 3"c/c

Reinforcement details of underground water reservoir are shown in Figures 8.1~8.3

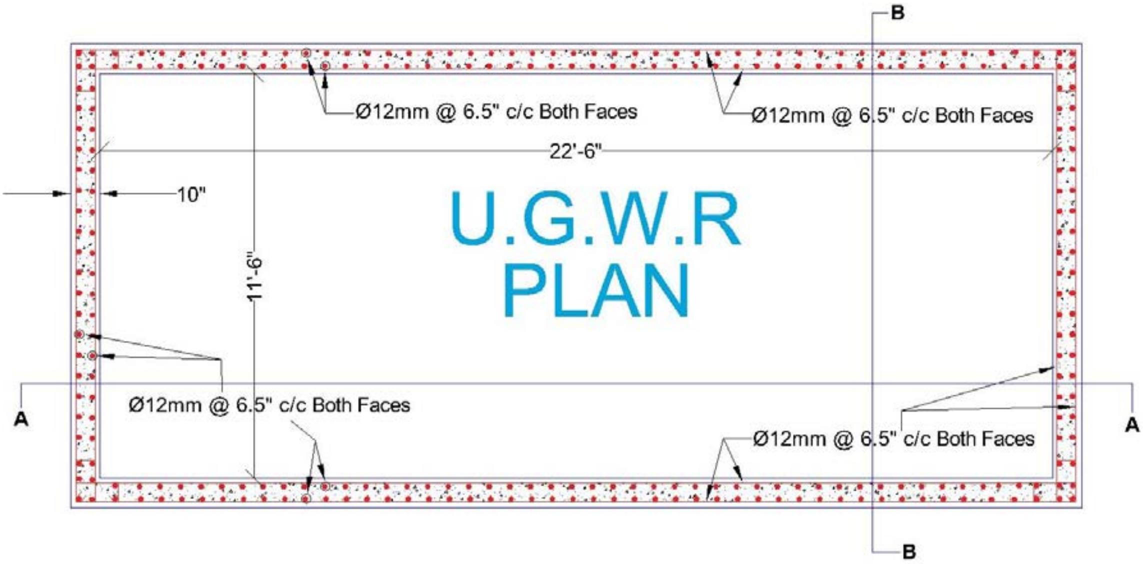


Figure 8.1: Details of reinforcement arrangement of Wall of the underground water reservoir.

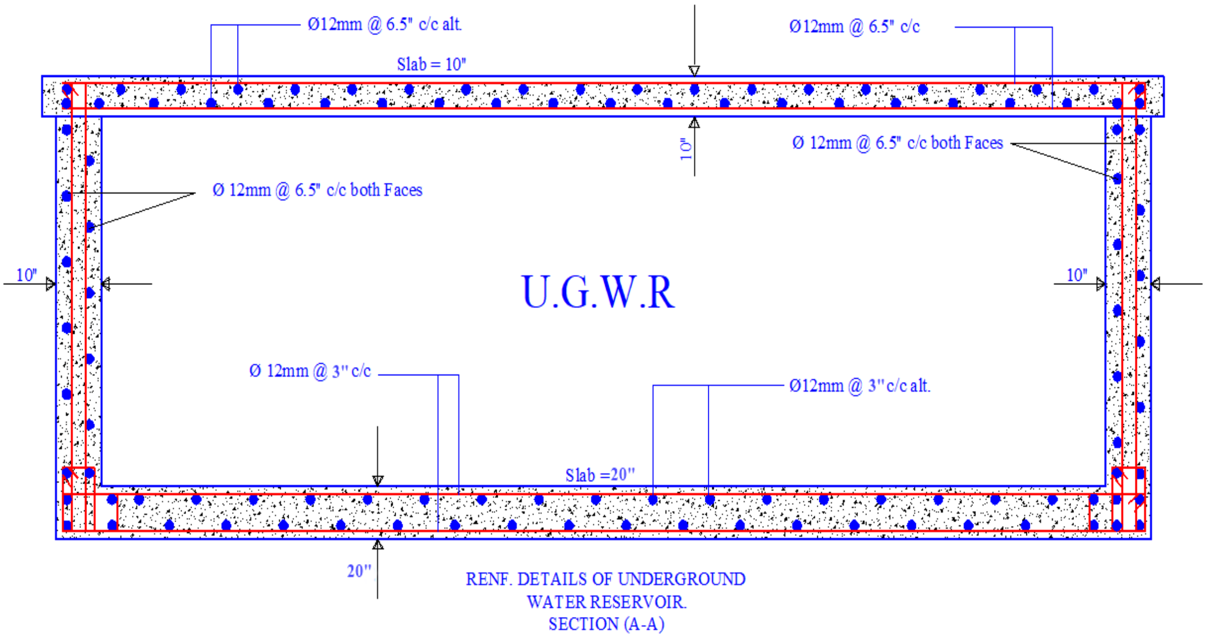


Figure 8.2a: Details of section A-A of the underground water reservoir.

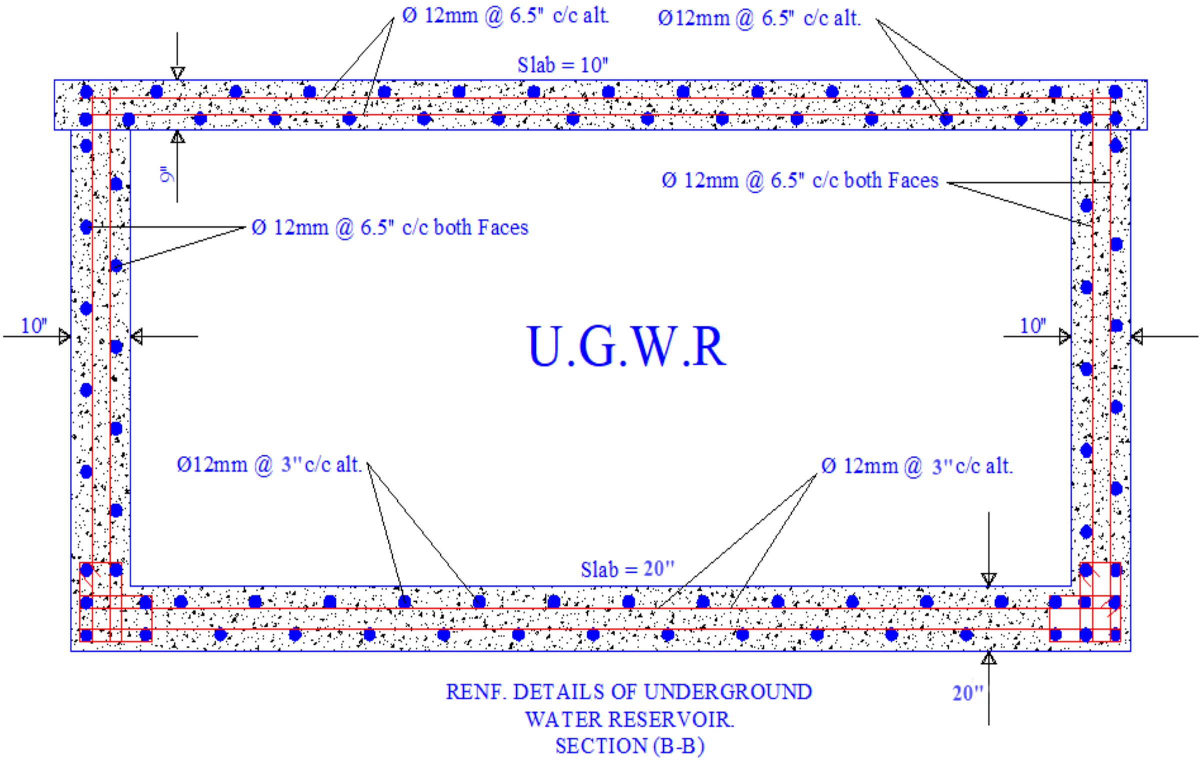


Figure 8.2b: Details of section B-B of the underground water reservoir.

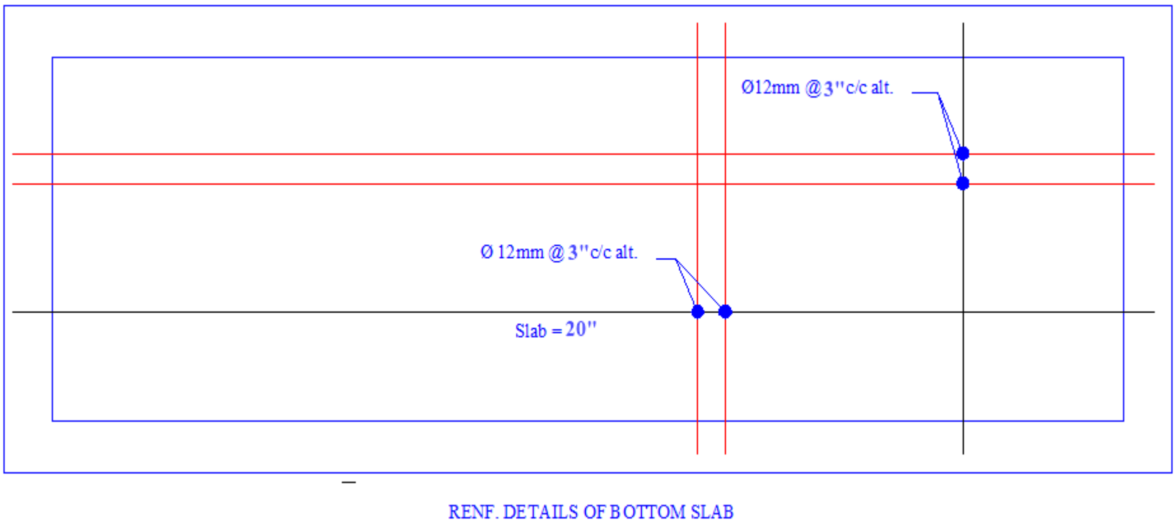


Figure 8.3a: Reinforcement details of bottom slab of the underground water reservoir.

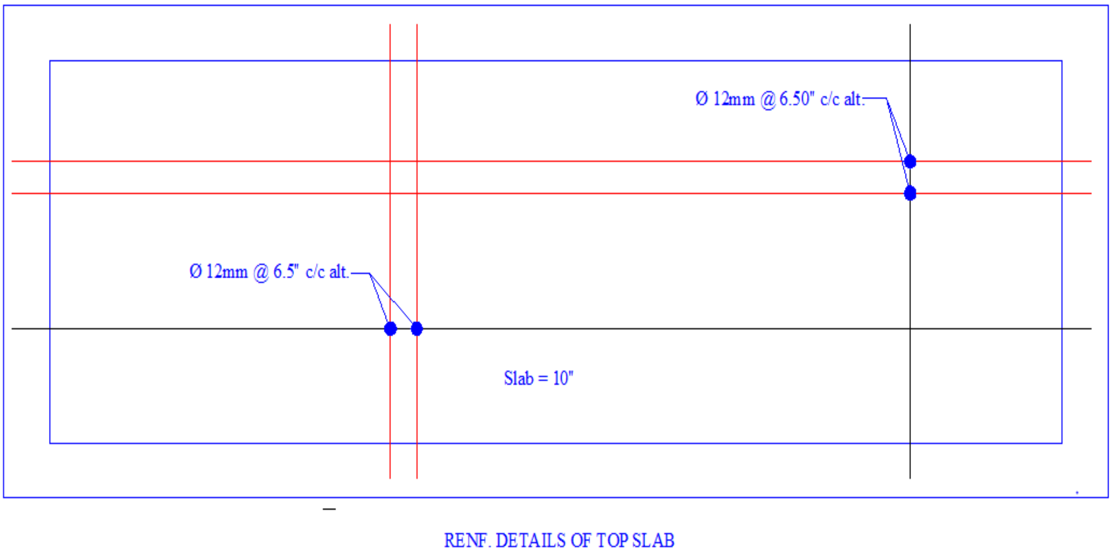


Figure 8.3b: Reinforcement details of top slab of the underground water reservoir.

CHAPTER 9
CONCLUSIONS & RECOMMENDATIONS

9.1Conclusions

From the study, it is observed that

1. Garments and factory base, we can ensure the perfect planning & design.
2. All kinds of practical needs, like ensuring water supplement, fire and other rescue supplement, garbage or wastage maintaining system.
3. Knowing the building codes and design as per those codes is one of the important considerations' factors in industrial building design.
4. The selection of materials and their properties should meet the requirement of building codes properly otherwise the integrity of the structure will fail at risk.
5. Ensuring a better position for transportation, far from house hold, environmentally friendly.

9.2 Recommendations

Based on the objectives, scopes, and limitations of the study (stated in *chapter I)*, few recommendations can be proposed for further studies:

- Another project such as: Spinning mills, Knitting Factory and Dyeing factory can be plan and design.
- Design of the Factory building can be done.
- Sub station room, generator room, canteen and guard room added here for getting better opportunities. Landscape will be including for beautification outer portion of the factory.
- Some of those important sections are as like as: officers' quarter, staff quarter and medical center etc. can be added in site plan.
- Cost benefit ratio is more important factor for manufacturing a garments industry. In future we can find out the cost benefit ratio.

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