STAMFORD UNIVERSITY BANGLADESH
DEPARTMENT OF CIVIL ENGINEERING

Planning & Design of a Residential cum Commercial Building

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Planning & Design of a Residential cum Commercial Building

A Project and Thesis

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

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The project and thesis titled "**Planning & Design of a Residential cum Commercial Building" s**ubmitted by **Md. Mehedi Hasan,** ID # CEN 059-09112; **Md. Imran Hossain,** ID # CEN 059-09115 and **Md. Muniruzzaman,** ID # CEN 059-09184, students of the Department of Civil Engineering has been examined thoroughly and accepted as partial fulfillment of the requirements for the degree of Bachelor of Science (B. Sc.) in Civil Engineering on 12th September, 2020.

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DECLARATION

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ABSTRACT

This study was carried out in the Department of Civil Engineering of Stamford University Bangladesh with the objectives of obtaining knowledge on design of Edge Supported Wall-Frame Residential.

Major cities, Specially Dhaka have been witnessing a renaissance in urban development. One of the current development patterns is a steady demand for urban projects due to continuous city migration by empty nesters and young professionals. Concerns over sprawl, traffic congestion and higher energy price will accentuate the desirability of urban living environments. This reflects a global trend taking place in major international cities due to demographic changes and rediscovery of center city living creating a need for more urban residential housing.

But what is driving this boom in high rise living? Major demographic shifts, life-style, health choices, environmental awareness, sustainability, investments in real estate and smart growth principles are contributing factors to the rise in a demand for more high-rise residential living. These issues are not only changing the way we live, but they are impacting and reinventing how we build high rise residential buildings.

High-rise residential buildings became possible with the invention of the elevator (lift) and cheaper, more abundant building materials. The materials used for the structural system of high-rise buildings are reinforced concrete and steel. High-rise structures pose particular design challenges for structural and geotechnical engineers, particularly if situated in a seismically active region or if the underlying soils have geotechnical risk factors. They also pose serious challenges to firefighters during emergencies in high-rise structures. Wind load impacts are to be carefully considered during design.

Despite the drive to meet a growing demand for housing by building faster and cheaper, a movement has begun to “raise the bar” on the quality of high-rise residential architecture from both performance and design aesthetic viewpoints. This study focuses all issues while planning and design the proposed model of the residential building.

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CHAPTER 1

INTRODUCTION

1.1 General

High-rise buildings, which are developed as a response to population growth, rapid urbanization and economic cycles, are indispensable for a metropolitan city development. This statement holds true for today; however, the relationship between cost and benefit is more complex in today's global marketplace. The political ideology of the city plays an important role in the globalization process (Newman and Tornely, 2005; Abu-Ghazalah, 2007). 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 high-rise buildings. The international and high technology styles have accompanied nearly all new tall buildings and became landmark of our cities (McNeill and Tewdwr-Jones, 2003). Nonetheless high-rise buildings are more expensive to construct per square meter, they produce less usable space and their operation costs are more expensive than conventional office buildings. The space efficiency, as well as the shape and geometry of the high-rise building need to satisfy the value and cost of the development equation. Space efficiency, which is determined by the size of the floor slab, dimension of the structural elements and rationalized core goes along with the financial benefit.

1.2 Background of study

Constructing a commercial cum residential building requires careful thought and planning. This design can be challenging to retail developers because of space constraints and the tendency of shoppers to prefer horizontal, rather than vertical. However, the massive land values of dense urban centers and high levels of traffic sometimes make vertical designs. A commercial building consumes more energy than any residential one. Designers now build systems to integrate artificial and natural light, air-conditioning units, acoustics and natural resources to make the commercial building more efficient.

The technical classification of a commercial building for zoning purposes is that it has more than half of its floor space used for commercial activities. Such buildings are owned by various individuals and group entities who construct them or build them for profit.

Developers build commercial cum residential properties with the intent to resell for profit or to lease for income. Other investors enter after construction for similar investment purposes.

Over the history of building structures, the changes in technology have been tremendous. Part of this comes from the daily strategies of human living. In recent years, the trend is the construction of mixed-uses structures as limited natural resources, the expenses, time and stresses of commuting draw people back into the city center. As a result, urban centers include now mostly structures with a storefront next to the street, offices in the stories immediately above, and, finally in the upper levels, apartments for city dwellers. But these types of buildings are difficult to arrange to take total advantage of structural and mechanical systems. The living quarters, with their more intimate spaces, need closer column spacing, and have fewer vents and wires required meeting needs of comfort. Shallow floor-to-floor heights in the apartment areas are possible since they can be accommodated by a flat plate/flat slab design. Offices need grid or pan systems covered by drop ceilings to allow HVAC and electrical systems to be delivered to desired locations within each square.

Hence, according to the need of rentable spaces, owner desires, aesthetics, cost, safety and comfort, architects and engineers are now facing the challenges of structural design to accommodate people's total daily life in one single structure. As outcomes, multiplan and multifunctional structures are now being constructed with different types of concrete floor systems considering lateral loads impacts which is a major concern to the designers and this study reflects this scenario.

1.3 Objectives and the study

- How to allocate Multiplan Multistoried Building.
- How to allocate modern amenities & services such as Escalator, Passenger Elevator, Observation Lift, Car Parking, Security Facilities etc. for a commercial & residential high rise structure.
- How to prepare floor plans for different purposes such as commercial, residential etc. in one structure.
- How to design different structural elements such as beams, columns, shear wall, ramp etc. as per lateral loads requirements of BNBC & ACI Codes.

1.4 Scopes/limitations of the study

1. This study had been made based on High rise structural design concept. Following parameters were not considered in the design of the structure:
   ► Deflections and sway effects
   ► Design of beam-column joints

2. Edge supported floor system was considered.
3. Etabs-2016 & Safe-2014 was used for analysis, design & detailing.
4. Stair, Underground water reservoir, Linear ramp, Septic tank etc. were designed manually.
5. Architectural Plan was done according to BNBC code.
6. Plumbing, electrification, brick works etc. were not considered.
7. Estimation & Cost analysis of the structure were not done.

CHAPTER 2

LITERETURE REVIEW

2.1 Edge Supported Floor Systems

If the slab is supported by beams on all four sides (as shown in Figure 2.1), the loads are transferred to all four beams, assuming rebar in both directions.

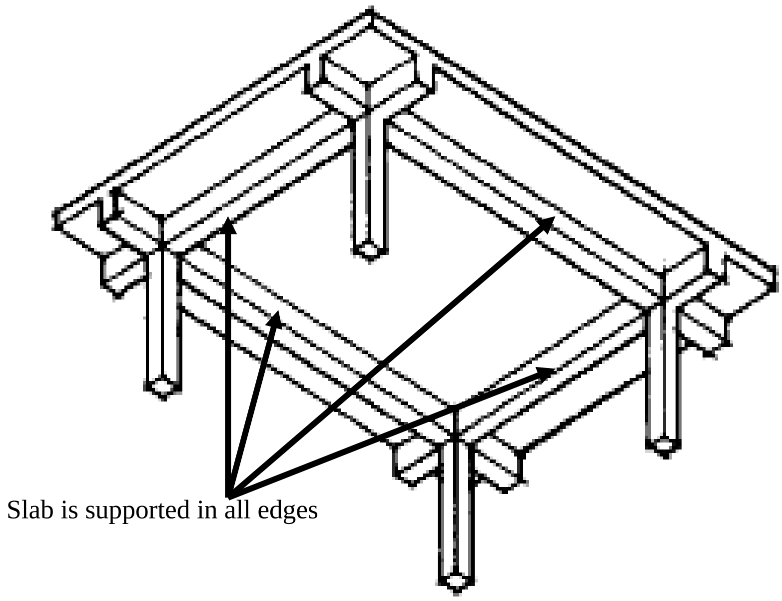


Figure 2.1a: Two way edge supported slab

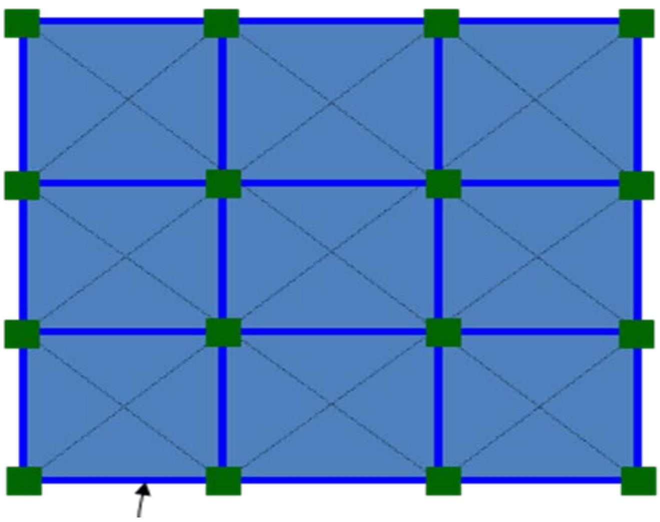


Figure 2.1b: Plan of an edge supported slab system

The edge supported slab system has the following advantages, disadvantages and applications:

- Advantages:
- Increased gravity and lateral load resistance
- Increased torsional resistance
- Decreased slab edge displacements
- Economical for longer spans and high loads

- Disadvantages:
- Presence of beams may require greater storey height
- Requires a regular column layout
- Grid of downstand beams deters fast formwork recycling.
- Flexibility of partition location and horizontal service distribution may be compromised.

- Typical Applications:
- Economical for more heavily loaded spans from 25 to 35 ft.
- Generally used for retail developments, warehouses, stores, etc.

2.2 Wall-frame structure

It is a combination from shear walls and rigid frames. In this combinations the walls tend to deflect in a flexural configuration and the frames tend to deflect in shear mode are constrained to adopt a common deflected by a horizontally rigidity of the girders and the slabs. As a consequence, the walls and the frames interact horizontally, especially at the top, to produce stiffer and stronger structure. The interacting wall –frame combination is appropriate for buildings in the 40 to 60 storey range, well beyond that of rigid frames or shear walls alone.

2.3 Seismic design requirements of RCC Beams

 # Material Strength

- Minimum specified compressive strength of all types of concrete, f_c' = 3,000 *psi*
- Maximum specified compressive strength of light-weight concrete, f_c'= 5,000 *psi*
- Maximum specified yield strength of reinforcement, f_y = 60,000 *psi*

 # Clear span of the beam

- Clear span > four times the effective depth i.e. $l_n > 4d$

 # Sectional dimensions of the beam

- Width-to-depth ratio ≥ 0.3 i.e. $b/h \geq 0.3$
- Minimum width ≥ 10 *inch*
- Minimum width of the beam ≤ [width of the supporting column + *1.5h*]

 # Main reinforcement

- $\rho_{min} \geq 200/f_y$
- $\rho_{max} \leq 0.0250$
- Two continuous bars should be at both top and bottom of the member.
- At any section, the top or bottom steel should not be less than ¼ of the steel for the maximum –ve moment at the supports.
- At each support, minimum bottom +ve steel must be equal to ½ of the –ve moment steel.

 # Splicing of the Main reinforcement

- Splice shall not be used (i) within joints (ii) within *2h* from the column face.
- Splices are to be confined by hoops or spiral reinforcement with maximum spacing or pitch of *d/4* or *4 inch* whichever is smaller.

 # Transverse reinforcement details

Such reinforcement (details in Figure 2.2a) is provided in the form of a closed hoop with cross tie(s) and must satisfy the following requirements:

- Total required steel area $A_v = \dfrac{\dfrac{V_u}{\phi}}{f_y \times d} \times s$
- Confinement reinforcement is provided in the form of hoops, as shown in figure 2.2b.
- Hoops are required over a distance 2*h* (*h* = depth of beam) from faces of both supports.
- First hoop will be placed at 2 *inch* from face of support.
- Maximum hoop spacing is the smaller of the followings:

  i) *d/4*

  ii) 8 x diameter of smallest longitudinal bar

  iii) 24 x diameter of the hoop bar

  iv) 12 *inch*
- Where hoops are not required (beyond confinement zone and splicing), stirrups with seismic hooks at both ends (detail A) shall be spaced not more than *d/2* throughout the length of the member.

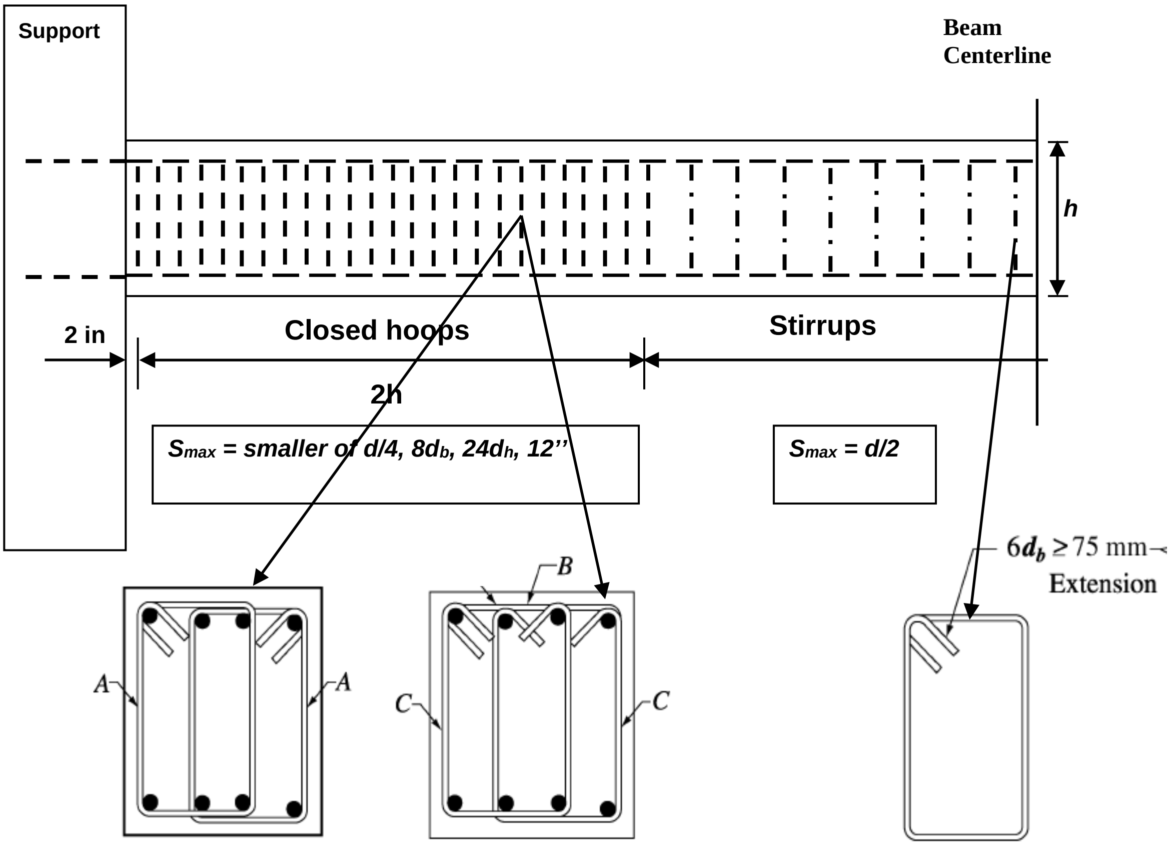


Figure 2.2a: Arrangement of Transverse Reinforcement in RCC Beam

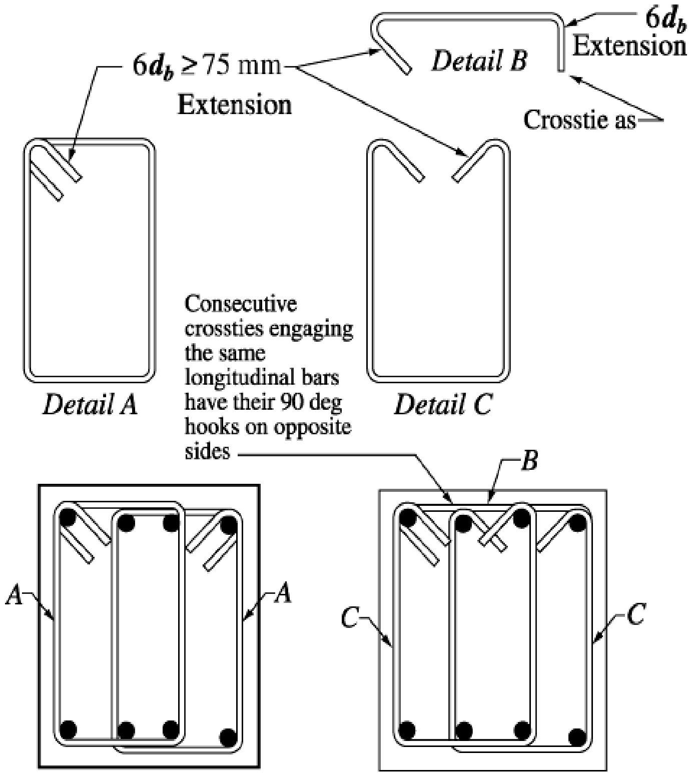


Figure 2.2b: Details of Transverse Reinforcement

2.4 Seismic Considerations for Column Design

Material Strength

- Minimum compressive strength of all types of concrete, f_c'= 3,000 *psi*
- Maximum compressive strength of light-weight concrete, f_c'= 5,000 *psi*
- Maximum yield strength of reinforcement, f_y = 60,000 *psi*
- Normal density concrete is preferable, w_c = 140~150 *pcf*

Sectional dimensions of the column

- Width-to-depth ratio ≥ 0.4 i.e. $b/h \geq 0.4$
- Least dimension ≥ 12 *inch*

Main reinforcement ratio

- $\rho_{min} = 0.01$
- $\rho_{max} = 0.06$
- Preferable $\rho = 0.04$

Splicing of the Main reinforcement

- Lap splice shall be used only within the center of the column.
- Welded splices may be used at any section of column, provided that:

  a) They are used only alternate longitudinal bars at a section

  b) The distance between splices along the longitudinal axis of reinforcement ≥ 24"
- Splices are to be confined by hoops or spiral reinforcement with maximum spacing or pitch of *d/4* or *4 inch* whichever is smaller.
- Splice length:

  Splice length = 1.3 l_d (class B splice)

Where, l_d = development length of the bars

$$l_d \geq \begin{Bmatrix} \dfrac{0.04A_b f_y}{\sqrt{f_c'}},\ A_b = \text{Bar area} \\ 0.0004 d_b f_y \\ 12'' \end{Bmatrix}$$

Transverse reinforcement

Such reinforcement is provided as closed hoops for tied column or circular hoops for spiral column.

a) Circular hoops: steel ratio of circular/spiral hoops,

$$\rho_s \geq \begin{cases} 0.12\dfrac{f_c'}{f_{yh}} \\ 0.45\left(\dfrac{A_g}{A_{ch}} - 1\right)\dfrac{f_c'}{f_{yh}} \end{cases}$$

Where,

f_{yh} = yield strength of hoop reinforcement.

A_{ch} = core area of column section measured to the outside of hoop reinforcement.

b) Closed hoops: Total cross-sectional area of closed hoops,

$$A_{sh} \geq \begin{cases} 0.09\, s_o h_c \dfrac{f_c'}{f_{yh}} \\ 0.3 s_o h_c\left(\dfrac{A_g}{A_{ch}} - 1\right)\dfrac{f_c'}{f_{yh}} \end{cases}$$

Where,

h_c = cross-sectional dimension of column core measured center-to-center of hoop reinforcement.

s_o = vertical spacing of hoop reinforcement.

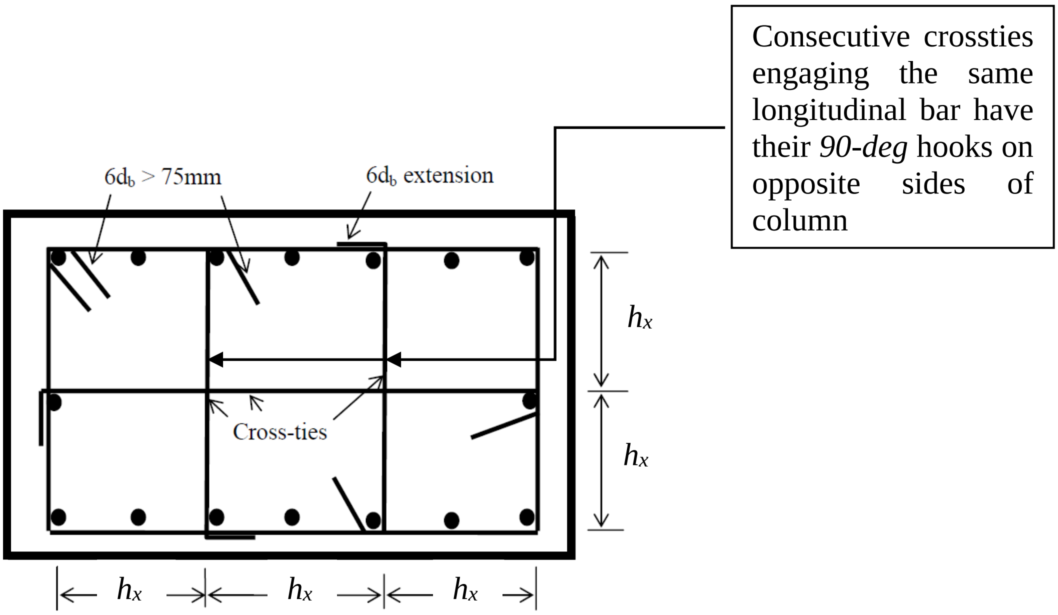
- ***Confinement length***: confinement reinforcement is to be provided over a length l_o from each joint face.

$$l_o \geq \begin{cases} \dfrac{1}{6}(clearspan\ of\ the\ column) \\ depth\ of\ member \\ 18'' \end{cases}$$

First hoop will be placed at *2 inch* from the joint.

- ***Spacing of the hoops***:

$$s_o \leq \begin{Bmatrix} \frac{1}{4}(\,least\,member\,dimension\,) \\ 6d_b \\ s_x = 4 + \frac{14 - h_x}{3}\,; 4'' \leq s_x \leq 6'' \end{Bmatrix}$$



Where,

h_x = maximum horizontal spacing of hoop or crosstie legs on all faces of the column $\leq$ 14".

- Special Notes:

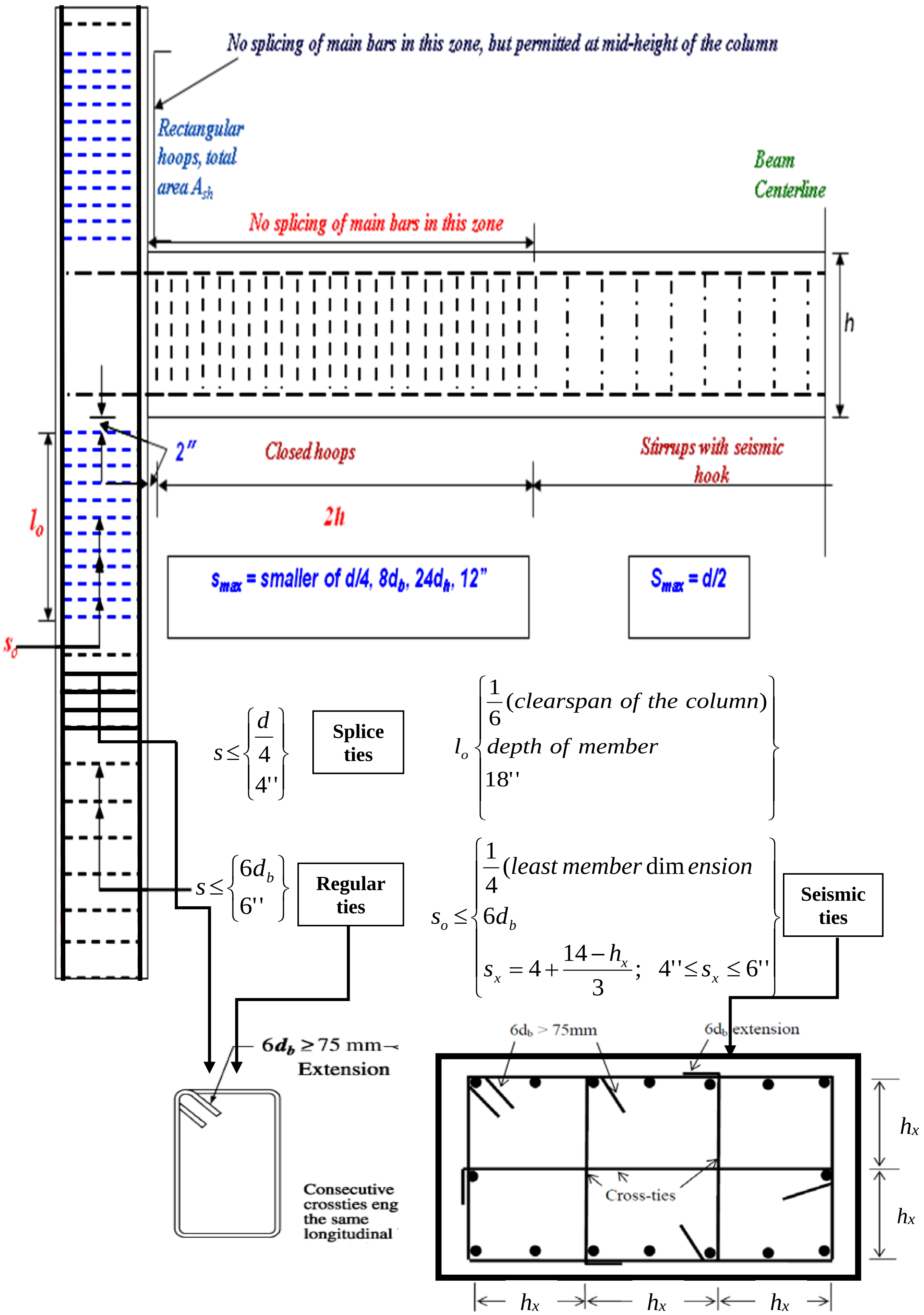
a) If column support shear wall, confinement reinforcement is to be provided over the full height of the column.

b) If column terminates on a footing, confinement reinforcement shall extend at least 12" into the footing.

c) Beyond the length l_o and splice length, the maximum spacing of tie or pitch of spiral will be,

$$s \leq \begin{Bmatrix} 6d_b \\ 6'' \end{Bmatrix}$$

A detail of seismic requirements for column is shown in Figure 2.3.

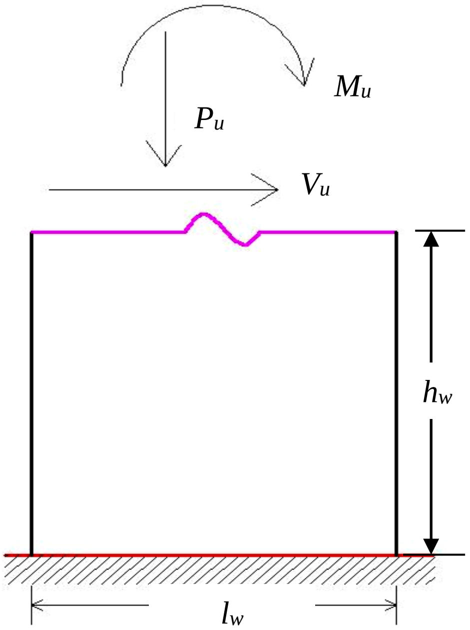


- $h_x \le 14''$

Figure 2.3: Seismic Requirements of Column

2.5 Shear wall design considerations

A shear wall may be subjected to the following forces. The typical design procedure of a shear wall is presented here as per ACI code.



Step-1: Calculate External Load

M_u
V_u » These can be obtained by software analysis.
P_u

Step-2: Boundary element check

$$I_g = \frac{bh^3}{12} = \frac{b_w l_w{}^3}{12}$$

$$f_c = \frac{P}{A} \pm \frac{MC}{I_g}$$

If $f_c < 0.2 f_c'$, then boundary element will not be required.

If $f_c \geq 0.2 f_c'$, then boundary element will be required.

Step-3: Obtain the dimension of the shear wall

Length of the shear wall = l_w

Width of the shear wall = b_w

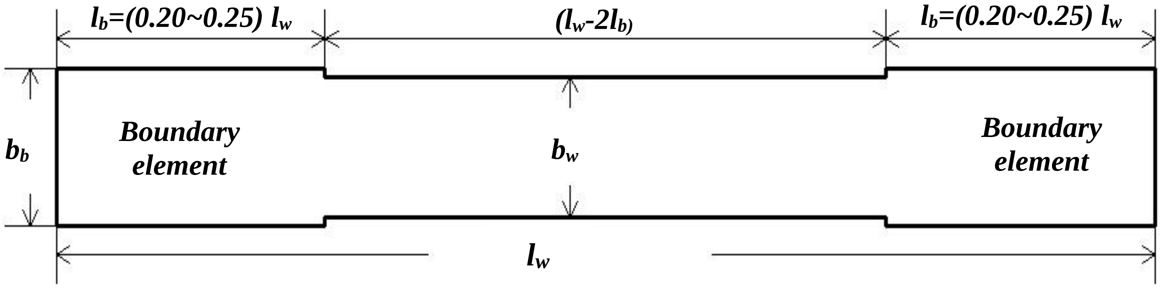
Height of the shear wall = h_w

Step-4: Determine the dimension of the boundary element

As per following figure, calculate:

Thickness of the boundary element= b_b

Length of the boundary element = l_b



Also remember the following conditions:

- The minimum section dimension of the boundary zone shall be $l_w/16$ i.e. $b_b \geq l_w/16$ and $l_b \geq l_w/16$.
- Boundary zones shall have a minimum length of 18 *inches* (measured along the length) at each end of the wall i.e. $l_b \geq 18"$.

Step-5: Check requirement of longitudinal & transverse reinforcements

According to the ACI code, two sets of reinforcement curtains, each having bars running in the longitudinal and transverse directions, will be required

- If $\frac{A_{cv} \times \sqrt{f'_c}}{6} < V_u$, where $A_{cv} = l_w b_w$
- If thickness of the wall > 10".

Step-6: Calculate the longitudinal & transverse reinforcements

Steel ratios:

$$* if \;\; V_u > \frac{A_{cv}\sqrt{f_c^{'}}}{6},$$

$$\rho_v = 0.0025 \;\;\&\;\; \rho_h = 0.0025$$

$$* if \;\; V_u \leq \frac{A_{cv}\sqrt{f_c^{'}}}{6},$$

for bar $\leq \phi 16$:

$$\rho_v = 0.0012 \;\;\&\;\; \rho_h = 0.0020$$

for bar $> \phi 16$:

$$\rho_v = 0.0015 \;\;\&\;\; \rho_h = 0.0025$$

Total steel areas:

Total longitudinal reinforcement per feet of wall, $A_{sv} = \rho_v \times 12 \times b_w$

Total transverse reinforcement per feet of wall, $A_{sh} = \rho_h \times 12 \times b_w$

Spacing:

Required spacing of bars having areas A_b per feet of wall [in two curtains, $A_v = 2A_b$]:

$$S = \frac{A_v x 12}{A_s}$$

Maximum spacing:

$S_{\max}$ will be smaller of the followings:

$S_{\max} = 3h = 3b_w$

$$S_{\max} = \frac{l_w}{5}$$

$S_{\max} = 18''$

Step-7: Check shear strength of concrete of wall to prevent V_u

- For walls with a height-to-width ratio $h_w/l_w \geq 2.0$, the shear strength of concrete is to be determined using the expression:

$$\phi V_n = \phi A_{cv}\left(2\sqrt{f_c^{'}} + \rho_n f_y\right)$$

Where,

φ = 0.60, unless the nominal shear strength provided exceeds the shear corresponding to development of nominal flexural capacity of the wall.

A_{cv}= net area = $l_w b_w$

h_w = height of entire wall or of segment of wall considered

l_w= width of wall (or segment of wall) in direction of shear force

ρ_n= reinforcement ratio in per foot of wall corresponding to plane perpendicular to plane of A_{cv}

$$= \frac{2x\, area\, of\, the\, selected\, horizontal\, bar}{b_w x 12}$$

- For walls with $h_w/l_w < 2.0$, the shear strength of concrete may be determined from

$$\phi V_n = \phi A_{cv}\left(\alpha_c \sqrt{f_c^{'}} + \rho_n f_y\right)$$

Where the coefficient α_c varies linearly from a value of 3.0 for h_w/l_w = 1.5 to 2.0 for h_w/l_w = 2.0.

Where the ratio h_w/l_w <2.0, ρ_v cannot be less than ρ_h.

Step-8: Reinforcement for boundary elements

Determine $\dfrac{M_u}{A_g \times l_w}$ and $\dfrac{P_u}{A_g}$

From interaction diagrams, corresponding $\dfrac{P_u}{A_g}$ and $\dfrac{M_u}{A_g \times l_w}$ value, reinforcement ratio ρ can be obtained.

Total reinforcement required for the shear wall, $A_s = \rho A_g$

Therefore, reinforcement required for boundary element

= Total steel requirement – vertical reinforcement required for non-boundary elements

$A_{sb} = A_s - A_{sv}$

For each boundary element, use $A_v = \dfrac{A_{sb}}{2}$

Minimum A_v should be larger of the followings:

$$A_v > \begin{cases} 0.005\, x\ area\ of\ the\ boundary\ zone \\ 2\#5\ bars\ at\ each\ edge\ of\ the\ boundary\ zone \end{cases}$$

Step-9: Design of Transverse reinforcement for boundary elements

Spacing, S_o:

Transverse reinforcement spacing will be the smaller of the followings:

First condition

$$S_o = \frac{\text{minimum dimension of wall}}{4}$$

Second condition

$$S_o = 6d_b$$

Third condition

$$S_x = \frac{14 - h_x}{3} + 4; 4'' \leq S_x \leq 6''.$$

Maximum spacing will be smaller of the followings:

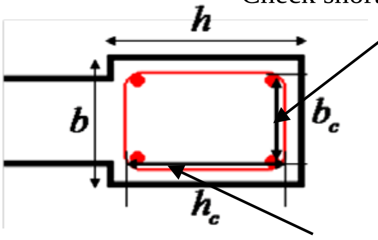
$$S_{\max} = \begin{cases} 6'' \\ 6x\,dia\,of\,largest\,vertical\,bar \end{cases}$$

Total steel area A_{sh}:

Total transverse reinforcement in long or short direction will be larger of the followings:

$$A_{sh} > 0.09 \times S_o \times h_c \times \frac{f_c'}{f_{yh}}$$

$$A_{sh} > 0.30 \times S_o \times h_c \times \left(\frac{A_g}{A_{ch}} - 1 \right) \times \frac{f_c'}{f_{yh}}$$



Where,

h_c = cross-sectional dimension of boundary element core measured center-to-center of hoop reinforcement.

S_o= vertical spacing of hoop reinforcement.

f_{yh} = yield strength of hoop reinforcement.

A_{ch} = core area of boundary element section measured to the outside of hoop reinforcement.

A_g = gross area of boundary element section.

Typical reinforcement arrangement in a shear wall is shown in Figure 2.4

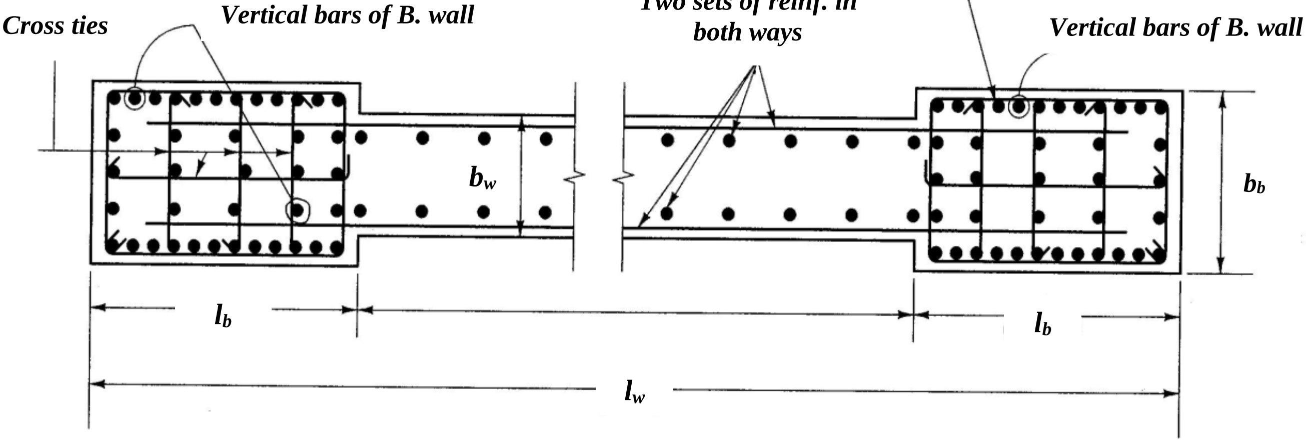


Figure 2.4: Shear Wall with Typical Reinforcement Arrangement

2.6 Water Requirements as per BNBC

Water Requirement for Domestic Use

Water requirements for daily domestic use of a building shall be assessed on the basis of the one or a combination of the following two methods:

a) Number of occupants according to their occupancy classification and their water requirements as specified in Table 2.1.

b) Peak demand or maximum probable flow.

Table 2.1: Guideline for Water Requirements for Various Occupancies and Facility- Groups in Litres Per Capita Per Day (LPCD)

Class of Occupancy	Occupancy Groups	For Full[a] Facilities (LPCD)	For Restricted Facilities (LPCD)
Occupancy A: Residential	A1: Single Family Dwelling	400	135
	A2: Flats or Apartments	225	135
	A3: Mess, Hostels, or Boarding House	135	70
	A4: Minimum Standard Housing	-	70
	A5: Hotels or Lodging House (Per bed)	300	135
Occupancy B: Educational	B1: Educational Facilities	70	45
	B2: Preschool Facilities	50	35
Occupancy C: Institutional	C1: Institution for Children's Care	180	100
	C2: Custodian Institution for Capable	180	100
	C3: Custodian Institution for Incapable	120	70
	C4: Penal and Mental Institution	120	70
Occupancy D: Health Care	D1: Normal Medical Facilities	450	225
	D2: Emergency Medical Facilities	300	135
Occupancy E: Assembly	E1: Large Assembly with Fixed Seats (per seat)	90	45
	E2: Small Assembly with Fixed Seats (per seat)	90	45
	E3: Large Assembly without Fixed Seats [b]	8	5
	E4: Small Assembly without Fixed Seats	8	5
	E5: Sports Facilities	8	5
Occupancy F: Business and Mercantile	F1: Offices	45	30
	F2: Small Shops and Markets	45	30
	F3: Large Shops and Markets	45	30
	F4: Garage and Petrol Stations	70	45
	F5: Essential Services	70	45
Occupancy G: Industrial	G1: Low Hazard Industries	40	25
	G2: Moderate Hazards Industries	40	25
Occupancy H: Storage	H1: Low Fire Risk Storage	10	6
	H2: Moderate Fire Risk Storage	10	6
Occupancy J: Hazardous	J1: Explosive Hazard Building	8	5
	J2: Chemical Hazard Building	8	5
Occupancy K[c] Miscellaneous	K1: Private Garage & Special Structure	8	5
	K2: Fences, Tanks and Towers	-	3

a For full facility in occupancy classifications A, B, C and D, the water requirement value includes 25% hot water.

b In the case of mosques, the water requirements given above shall be adequate for ablution and other uses of one devotee per prayer. The appropriate LPCD value may be calculated on this basis.

c Water requirement for occupancy K is shown as a provision for unknown visitors only.

Water Requirement for Fire Fighting

The water requirement for firefighting shall be in accordance with Table 2.2.

Table 2.2: Fire Protection Flow Requirements

Building Type	Sprinkler System (l/min.)*	Standpipe and hose System (l/min.)*	Duration** (minute, min.)
Light hazard- I	1000	1000	30
Light hazard- II	1900	1900	50
Ordinary hazard- I	2650	1900	75
Ordinary hazard - II	3200	1900	75
Ordinary hazard - III	4800	1900	75

Notes:

* Values will be for one riser serving floor area of 1000 m^2.

* * These durations shall be for a building up to the height of 51m. For greater height of 51-102m and above 102m, the duration will be 1.25 times and 1.5 times of the specified values respectively.

Light hazard - I : Occupancy groups, A1, A2, A4
Light hazard - II : Occupancy groups, A3, A6, A7, A8, B, C, D, E2, E4, E7, F1 & F2
Ordinary hazard - I : Occupancy groups, E1, E3, E5, F3, F4, F5, F6, F7, G1 & G4
Ordinary hazard- II : Occupancy groups, G2 & H1
Ordinary hazard- III : Occupancy groups, G3 & H2

2.7 Wind Loads

a. Basic Wind Speed

The basic wind speed for the design is taken from basic wind speed map of 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 the basic wind speed set in the map is 130 km / h and maximum is 260 km/h. The basic wind speed for selected locations in Bangladesh are given in Table 2.3

Table 2.3: Basic Wind Speeds for Selected Locations in Bangladesh

Location	Basic Wind Speed (km/h)	Location	Basic Wind Speed (km/h)
Angarpota	150	Lalmonirhat	204
Bagerhat	252	Madaripur	220
Bandarban	200	Magura	208
Barguna	260	Manikganj	185
Barisal	256	Meherpur	185
Bhola	225	Maheshkhali	260
Bogra	198	Moulvibazar	168
Brahmanbaria	180	Munshiganj	184
Chandpur	160	Mymensingh	217
Chapai		Naogaon	175
Nawabganj	130	Narail	222
Chittagong	260	Narayanganj	195
Chuadanga	198	Narsinghdi	190
Comilla	196	Natore	198
Cox’s Bazar	260	Netrokona	210
Dahagram	150	Nilphamari	140
Dhaka	210	Noakhali	184
Dinajpur	130	Pabna	202
Faridpur	202	Panchagarh	130
Feni	205	Patuakhali	260
Gaibandha	210	Pirojpur	260
Gazipur	215	Rajbari	188
Gopalganj	242	Rajshahi	155
Habiganj	172	Rangamati	180
Hatiya	260	Rangpur	209
Ishurdi	225	Satkhira	183
Joypurhat	180	Shariatpur	198
Jamalpur	180	Sherpur	200
Jessore	205	Sirajganj	160
Jhalakati	260	Srimangal	160
Jhenaidah	208	St. Martin’s Island	260
Khagrachhari	180	Sunamganj	195
Khulna	238	Sylhet	195
Kutubdia	260	Sandwip	260
Kishoreganj	207	Tangail	160
Kurigram	210	Teknaf	260
Kushtia	215	Thakurgaon	130
Lakshmipur	162		

b. Exposure Category

Exposure A: Urban and sub-urban areas, industrial areas, wooded areas, hilly or other terrain covering at least 20 percent of the area with obstructions 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 10m extending 800m 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 grass land.

Exposure C: Flat and unobstructed open terrain, coastal areas and riversides 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 structure, whichever greater.

2.8 Earthquake Load

Structural System for EQ

a) **Bearing Wall System**: A structural system having bearing walls or bracing systems without a complete vertical load carrying frame to support gravity loads. Resistance to lateral loads is provided by shear walls or braced frames.

b) **Building Frame System**: A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral loads is provided by shear walls or braced frames separately.

c) **Moment Resisting Frame System**: A structural system with an essentially complete space frame providing support for gravity loads. Moment resisting frames also provide resistance to lateral load primarily by flexural action of members, and may be classified as one of the following types:

i) Special Moment Resisting Frames (SMRF)
ii) Intermediate Moment Resisting Frames (IMRF)
iii) Ordinary Moment Resisting Frames (OMRF).

The framing system, IMRF and SMRF shall have special detailing to provide ductile behavior for concrete and steel structures respectively. OMRF need not conform to the ductility requirements.

d) **Dual System**: A structural system having a combination of the following framing systems :

i) Moment resisting frames (SMRF, IMRF or steel OMRF), and ii) Shear walls.

The moment resisting frames shall be capable of resisting at least 25% of the applicable total seismic lateral force, even when wind or any other lateral force governs the design.

Selection of Lateral Force Method

Seismic lateral forces on primary framing systems shall be determined by using either the Equivalent Static Force Method or the Dynamic Response Method complying with the restrictions given below :

a) The Equivalent Static Force Method may be used for the following structures:

i) All structures, regular or irregular, in Seismic Zone 1 and in Structure Importance Category IV in Seismic Zone 2, except case b(iv) below.

ii) Regular structures under 75 metres in height with lateral force resistance provided by structural systems except case b(iv) below.

iii) Irregular structures not more than 20 metres in height.

iv) A tower like building or structure having a flexible upper portion supported on a rigid lower portion where:

1) both portions of the structure considered separately can be classified as regular structures,

2) the average storey stiffness of the lower portion is at least ten times the average storey stiffness of the upper portion, and

3) the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

b) The Dynamic Response Method may be used for all classes of structure, but shall be used for structures of the following types:

i) Structures 75 metres or more in height except as permitted by case a(i) above.

ii) Structures having a stiffness, weight or geometric vertical irregularity of Type I, II, or III or structures having irregular features.

iii) Structures over 20 metres in height in Seismic Zone 3 not having the same structural system throughout their height.

iv) Structures, regular or irregular, located on Soil Profile Type S_4, which have a period greater than 0.7 second.

Equivalent Static Force Method

This method may be used for calculation of seismic lateral forces for all structures.

Design Base Shear: The total design base shear in a given direction shall be determined from the following relation :

$$V = \frac{ZIC}{R} W$$

where, Z = Seismic zone coefficient

I = Structure importance coefficient.

R = Response modification coefficient for structural systems.

W = The total seismic dead load

C = Numerical coefficient given by the relation:

$$= \frac{1.25S}{T^{2/3}}$$

S = Site coefficient for soil characteristics.

T = Fundamental period of vibration in seconds, of the structure for the direction under consideration.

Table 2.4:

Seismic Zone Coefficients, *Z*

Seismic Zone	Zone Coefficient
1	0.075
2	0.150
3	0.250

Table 2.5:

Structure Importance Coefficients *I, I′*

Structure Importance Category	Structure Importance Coefficient	
	I	*I′*
I Essential facilities	1.25	1.50
II Hazardous facilities	1.25	1.50
III Special occupancy structures	1.00	1.00
IV Standard occupancy structures	1.00	1.00
V Low-risk Structures	1.00	1.00

The value of C need not exceed 2.75 and this value may be used for any structure without regard to soil type or structure period. Except for those requirements where Code prescribed forces are scaled up by 0.375R, the minimum value of the ratio C/R shall be 0.075.

Definition of different facilities as mentioned in Table 2.5 is summarized in Table 2.6.

Structure Period: For all buildings the value of T may be approximated by the following formula:

$$T = C_t\,(h_n)^{3/4}$$

where, C_t = 0.083 for steel moment resisting frames

= 0.073 for reinforced concrete moment resisting frames, and eccentric braced steel frames

= 0.049 for all other structural systems

h_n = Height in metres above the base to level *n*.

Table 2.6: Structure Importance Categories

Structure Importance Category	Occupancy Type or Functions of Structure	
	General	Particular
I	Essential Facilities	1. Hospital and other medical facilities having surgery and emergency treatment area. 2. Fire and police stations. 3. Tanks or other structures containing, housing or supporting water or other fire-suppression materials or equipment required for the protection of essential or hazardous facilities, or special occupancy structures. 4. Emergency vehicle shelters and garages. 5. Structures and equipment in emergency-preparedness centres, including cyclone and flood shelters. 6. Standby power-generating equipment for essential facilities. 7. Structures and equipment in government communication centres and other facilities required for emergency response.
II	Hazardous Facilities	Structures housing, supporting or containing sufficient quanti-ties of toxic or explosive substances to be dangerous to the safety of the general public if released.
III	Special Occupancy Structures	1. Covered structures whose primary occupancy is public assembly with capacity > 300 persons. 2. Buildings for schools through secondary or day-care centre with capacity > 250 students. 3. Buildings for colleges or adult education schools with capacity > 500 students. 4. Medical facilities with 50 or more resident incapacitated patients not included above. 5. Jails and detention facilities. 6. All structures with occupancy > 5,000 persons. 7. Structures and equipment in power-generating stations and other public utility facilities not included above, and required for continued operation.
IV	Standard Occupancy Structures	All structures having occupancies or functions not listed above.

V	Low Risk Structures	Buildings and Structures that exhibit a low risk to human life and property in the event of failure, such as agricultural buildings, minor storage facilities, temporary facilities, construction facilities, and boundary walls.

Table 2.7: Response Modification Coefficient for Structural Systems, *R*

Basic Structural System(1)	Description of Lateral Force Resisting System	***R***
a. Bearing Wall System	1. Light framed walls with shear panels	
	i) Plywood walls for structures, 3 storeys or less	8
	ii) All other light framed walls	6
	2. Shear walls	
	i) Concrete	6
	ii) Masonry	6
	3. Light steel framed bearing walls with tension only bracing	4
	4. Braced frames where bracing carries gravity loads	
	i) Steel	6
	ii) Concrete (2)	4
	iii) Heavy timber	4
b. Building Frame System	1. Steel eccentric braced frame (EBF)	10
	2. Light framed walls with shear panels	
	i) Plywood walls for structures 3-storeys or less	9
	ii) All other light framed walls	7
	3. Shear walls	
	i) Concrete	8
	ii) Masonry	8
	4. Concentric braced frames (CBF)	
	i) Steel	8
	ii) Concrete (2)	8
	iii) Heavy timber	8
c. Moment Resisting Frame System	1. Special moment resisting frames (SMRF)	
	i) Steel	12
	ii) Concrete	12
	2. Intermediate moment resisting frames (IMRF), concrete (3)	8
	3. Ordinary moment resisting frames (OMRF)	
	i) Steel	6
	ii) Concrete (4)	5
d. Dual System	1. Shear walls	
	i) Concrete with steel or concrete SMRF	12
	ii) Concrete with steel OMRF	6
	iii) Concrete with concrete IMRF (3)	9
	iv) Masonry with steel or concrete SMRF	8
	v) Masonry with steel OMRF	6
	vi) Masonry with concrete IMRF (2)	7
	2. Steel EBF	
	i) With steel SMRF	12
	ii) With steel OMRF	6
	3. Concentric braced frame (CBF)	
	i) Steel with steel SMRF	10
	ii) Steel with steel OMRF	6
	iii) Concrete with concrete SMRF (2)	9
	iv) Concrete with concrete IMRF (2)	6

Notes : (1)	Basic Structural Systems.
(2)	Prohibited in Seismic Zone 3.
(3)	Prohibited in Seismic Zone 3
(4)	Prohibited in Seismic Zones 2 and 3.

Table 2.8: Site Coefficient, *S* for Seismic Lateral Forces

Site Soil Characteristics		Coefficient, *S*
Type	**Description**	
S_1	A soil profile with either : a) A rock-like material characterized by a shear-wave velocity greater than 762 m/s or by other suitable means of classification, or b) Stiff or dense soil condition where the soil depth is less than 61 metres	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 61 metres	1.2
S_3	A soil profile 21 metres or more in depth and containing more than 6 metres of soft to medium stiff clay but not more than 12 metres of soft clay	1.5
S_4	A soil profile containing more than 12 metres of soft clay characterized by a shear wave velocity less than 152 m/s	2.0
Note : (1)	The site coefficient shall be established from properly substantiated geotechnical data. In locations where the soil properties are not known in sufficient detail to determine the soil profile type, soil profile S_3 shall be used. Soil profile S_4 need not be assumed unless the building official determines that soil profile S_4 may be present at the site, or in the event that soil profile S_4 is established by geotechnical data.	

Vertical Distribution of Lateral Forces: In the absence of a more rigorous procedure, the total lateral force, which is the base shear *V*, shall be distributed along the height of the structure:

$$V = F_t + \sum_{i=1}^{n} F_i$$

where, F_i = Lateral force applied at storey level -*i* and

F_t = Concentrated lateral force considered at the top of the building in addition to the force F_n.

The concentrated force, F_t acting at the top of the building shall be determined as follows:

F_t = 0.07 $TV \leq 0.25\ V$ when $T > 0.7$ second

F_t = 0.0 when $T \leq$ 0.7 second

The remaining portion of the base shear (V-F_t), shall be distributed over the height of the building, including level-n, according to the relation:

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

At each storey level-x, the force F_x shall be applied over the area of the building in proportion to the mass distribution at that level.

Any combination of Building Frame Systems, Dual Systems, or Moment Resisting Frame Systems may be used to resist design seismic forces in structures less than 50 m in height. Only combinations of Dual Systems and Special Moment Resisting Frames (SMRF) can be used to resist the design seismic forces in structures exceeding 50 m in height in Seismic Zone 3.

Drift of the Storey

Storey drift is the displacement of one level relative to the level above or below due to the design lateral forces. Calculated storey drift shall include both translational and torsional deflections and conform to the following requirements:

a) Storey drift, Δ, shall be limited as follows:

i) $\Delta \leq 0.04h/R \leq 0.005h$ for $T < 0.7$ second.

ii) $\Delta \leq 0.03h/R \leq 0.004h$ for $T \geq 0.7$ second.

iii) $\Delta \leq 0.0025h$ for unreinforced masonry structures.

where, h = height of the building or structure.

The period T used in this calculation shall be the same as that used for determining the base shear. The limits involving R in (i) and (ii) above shall be applicable only when earthquake forces are present.

b) The drift limits set out in (a) above may be exceeded where it can be demonstrated that greater drift can be tolerated by both structural and nonstructural elements without affecting life safety.

EQ Magnitude

According to the depth of focus, tectonic earthquake is classified as:

Shallow: depth of focus is "less than 60 km.
Intermediate: depth of focus between 60 to 70' km.
Deep: depth of focus above 70 km.

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

There are three zones namely zone 1, being most active, zone II, less active and zone III, being the minimum possible intensity of earthquake. Seismic probable magnitudes are:

Zone	(Richter scale)
I	>7.0
II	6.5-7.0
III	6.0-6.5

The design of buildings against earthquake should obviously be done in Zone I. However, for low height buildings additional provision of *33%* Reinforcement may be provided. For high rise buildings proper technical Design should be done by qualified Civil Engineers.

CHAPTER 3

DETAILS OF THE PROPOSED RESIDENTIAL CUM COMMERCIAL BUILDING

3.1 Introduction

This chapter gives completed the planning, modelling, analysis, design and detailing of the structural parts of Residential cum Commercial Building.

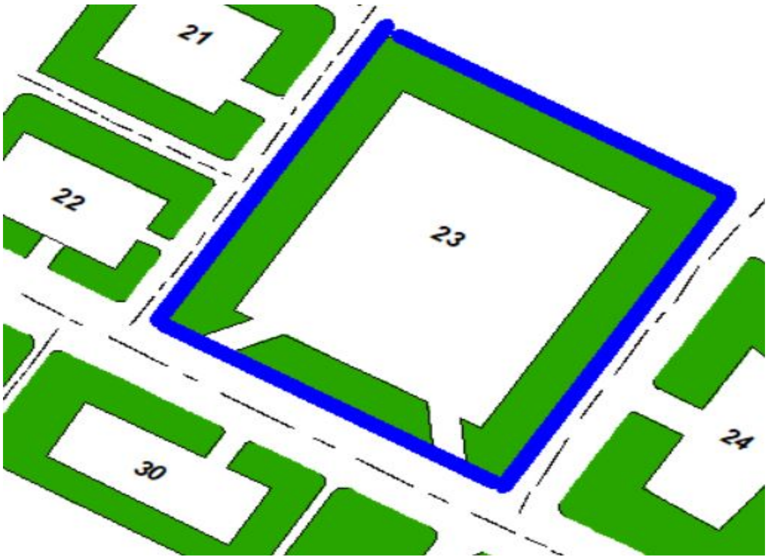


Figure 3.1: Site plan of the Proposed Building

This Chapter has been systemically arranged in the following manner:

- Article 3.2 presents details of Loads and Material Properties of proposed building.
  - 3.2.1 Load Calculation for Official & Commercial Building
  - 3.2.2 Load Calculation for Residential Building

- Article 3.3 presents Floor Plan details of proposed building.
  - 3.3.1 Basement floor
  - 3.3.2 Ground floor
  - 3.3.3 1st to 6th floors
  - 3.3.4 7th floor
  - 3.3.5 8th floor
  - 3.3.6 9th floor

- 3.3.7 10th floor
- 3.3.8 11th floor
- 3.3.9 12th floor
- 3.3.10 13th floor
- 3.3.11 14th floor
- 3.3.12 15th floor
- 3.3.13 16th floor
- 3.3.14 17th floor
- Roof top

- Article 3.4 present Structural details of proposed Building.
  - 3.4.1 Detailing of Slab.
  - 3.4.2 Detailing of Beam.
  - 3.4.3 Detailing of Column
  - 3.4.4 Detailing of Stair.
  - 3.4.5 Detailing of Shear Wall.
  - 3.4.6 Detailing of Overhead Water Tank.
  - 3.4.7 Detailing of Underground Water Reservoir.

3.2 Details of Loads and Material Properties of Proposed Building

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

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

Items	Description
Design code	• American Concrete Institute (*ACI*) Building design code, 2014. • Bangladesh National Building Code (*BNBC*), 1993.
Building components	• Column type = Tied • Footing type = Piling • Thickness of all partition walls = 5 *inch.* • Thickness of Slab (Two way) = 6.0 *inch.* • Thickness of Slab (One way) = 9.0 *inch.* • Thickness of Slab (Ramp) = 19 *inch.*

Material properties	• Yield strength of reinforcing bars, f_y= 60,000 *psi*. • Concrete compressive strength, f_c' = 4,000 *psi* • Normal density concrete, unit weight = 150 *pcf*. • Unit weight of brick = 120 *pcf*. • Unit weight of water = 62.5 *pcf*.

3.2.1 Load Calculation for Official & Commercial Building:

(a) Dead loads:

Self-weight of two way slab = (6/12) X 150 = 75 *psf*.

Self-weight of one way slab = (9/12) X 150 = 112.5 *psf*.

Self-weight of Ramp slab = (19/12) X 150 = 237.5 *psf*.

Floor finish = 30 *psf*.

5″ Partition wall Load calculation (commercial) = 21 *psf*.

(b) Other dead loads:

Floor finish for (1st – 11th) = 30 *psf*.

Floor finish for parking floor space = 15 *psf*.

. Floor finish for water tank = 10 *psf*.

Floor finish for stair = 25 *psf*.

(c) Live loads:

Live load for stair (commercial) = 150 *psf*.

Live load for water tank slab =10 *psf*.

Water pressure for water tank = 437.5 *psf*.

Live load for floor (commercial) =150 psf.

3.2.2 Load Calculation for Residential Building:

(a) Dead loads:

Self-weight of two way slab = (6/12) X 150 = 75 *psf*.

Self-weight of one way slab = (9/12) X 150 = 112.5 *psf*.

Self-weight of Ramp slab = (14/12) X 150 = 175 *psf*.

Floor finish = 30 *psf*.

5″ Partition wall Load calculation (residential) = 49 *psf*.

(b) Other dead loads:

Floor finish for (1st – roof top)	= 30 *psf.*
Floor finish for parking floor space	= 15 *psf.*
Floor finish for water tank	= 10 *psf.*
Floor finish for stair	= 25 *psf.*

(c) Live loads:

Live load for stair (residential)	= 100 *psf.*
Live load for water tank slab	=10 *psf.*
Water pressure for water tank	= 437.5 *psf.*
Live load for floor (residential)	= 40 *psf.*

- **Seismic load:**

Height of building	= 205 *ft* = 62.5 *m*
Seismic zone Coefficient (Dhaka zone)	= 0.15
Resapons modification coefficient, R [Dual System, Shear wall (IMRF)]	= 9.00
Importance Coefficient for residential building (standard occupancy), *I*	= 1.00
Story range	= Base to Parapet.

- **Wind load:**

Length of building	=158 *ft.* 1 *inch.*
Width of building	= 130 *ft. 5 inch*
Exposure Condition	= B
Wind Pressure in Dhaka city, V_b	= 131 *mph*
Importance Coefficient, I	= 1.00
Story range	= Ground Floor to Parapet.

Wind load and Earthquake load were auto-calculated by ETABS-2017 according to UBC-94 which is most acquainted with BNBC-93 code.

3.3 Floor Plan details of Commercial, Official & Residential part.

Building details

- Height of building = 205 *ft* = 62.5 m
- Length of building = 158 *ft.* 1 *inch.*
- Width of building = 130 *ft. 5 inch.*
- Total floors = 18 Nos.
- Types of floors = Basement as Car parking
  = GF to 6th floor as market (Commercial part)
  = 7th to 8th floor as restaurant & Cineplex (Commercial)
  = 9th to 11th floor as (Official part)
  =12th to 18th floor as (Residential part)

The 3D view from SKETCHUP and structural model view from ETABS of the whole structures are shown in Figures 3.2 and 3.3.

Figure 3.2a: 3D view of the proposed building

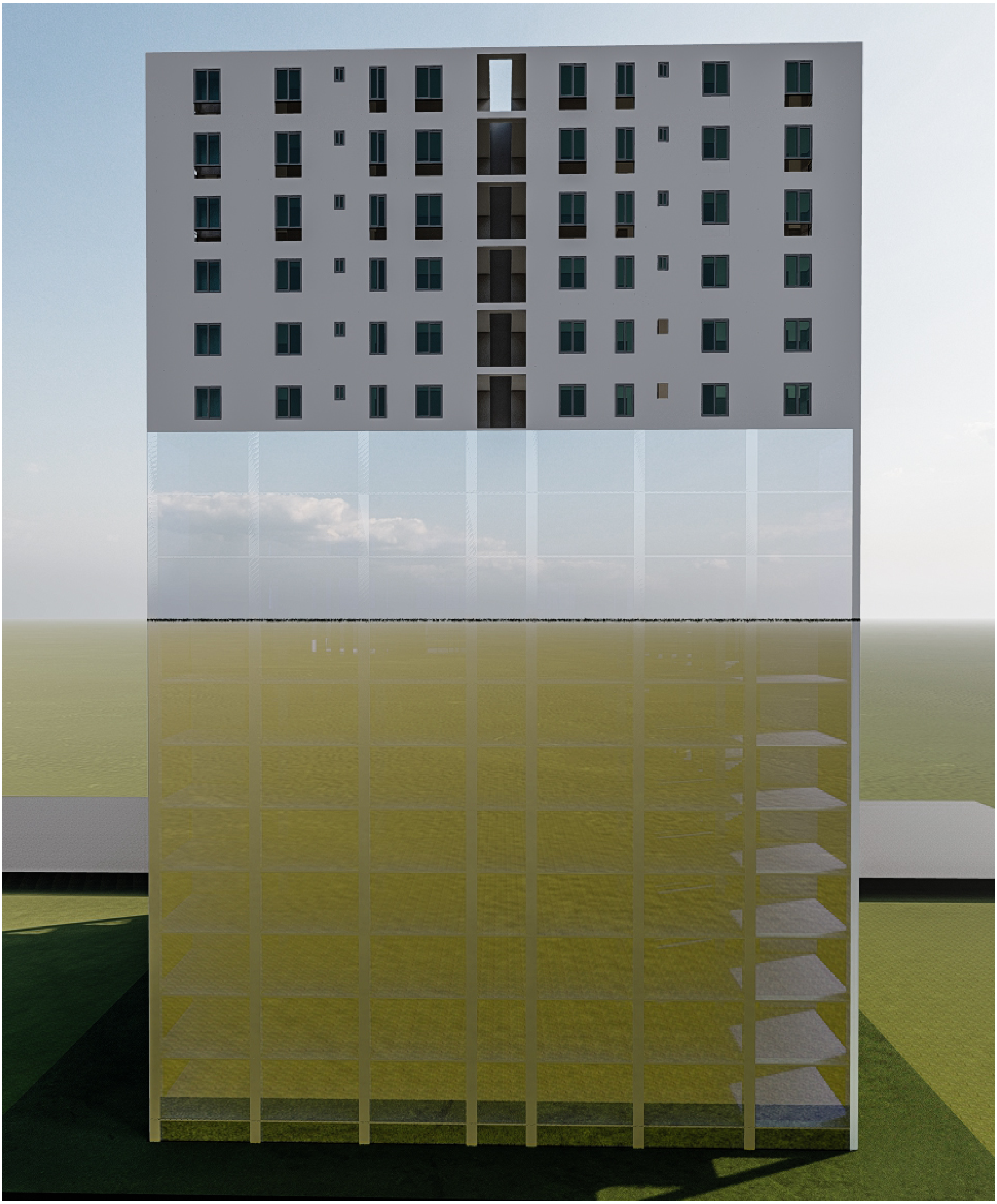


Figure 3.2b: Back side 3D View of the Proposed Building

Figure 3.2c: Right side 3D View of the Proposed Building

Figure 3.2d: Left side 3D View of the Proposed Building

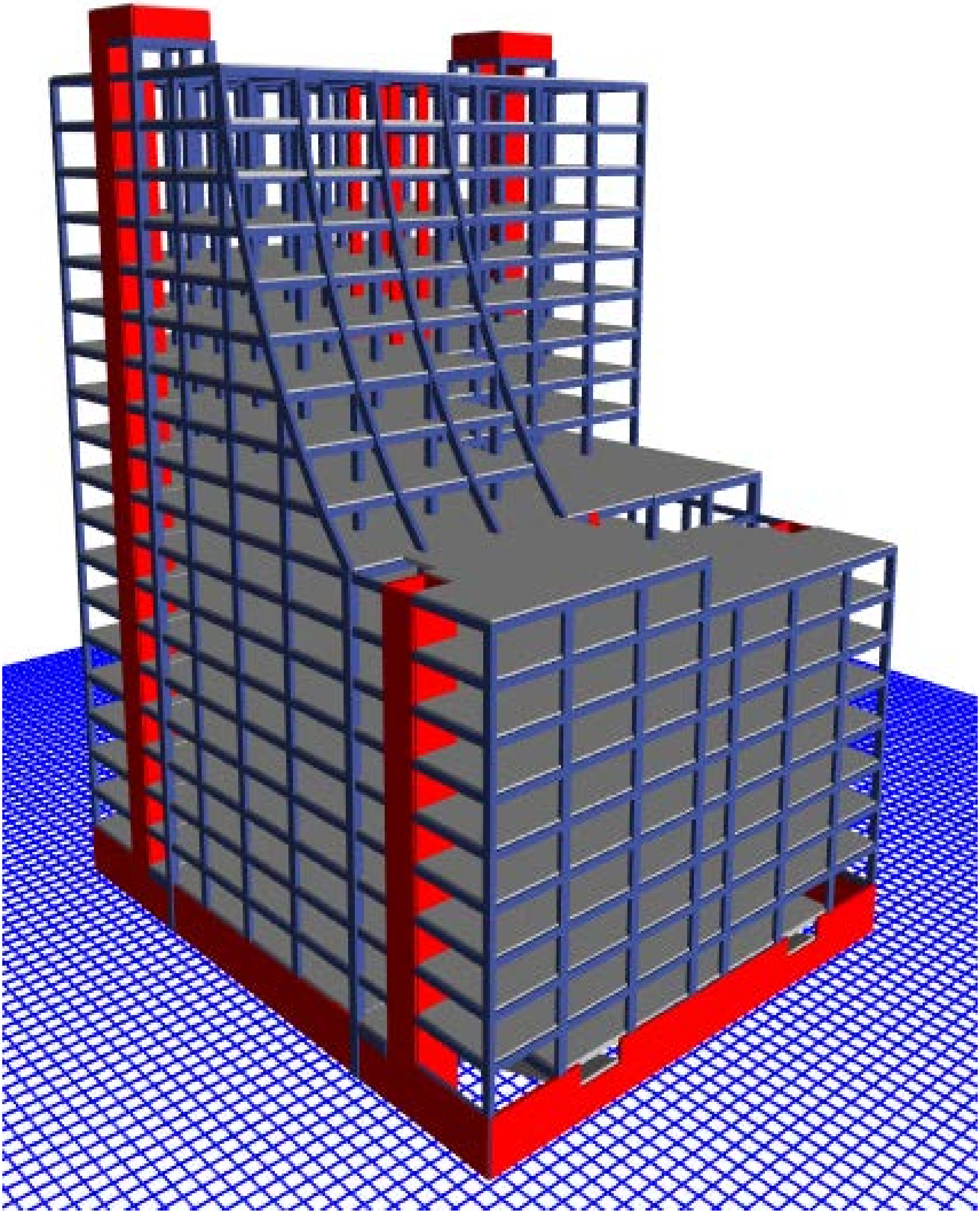


Figure 3.3: 3D Structural View (frame) as per ETABS

❖ 3.3.1 Basement

- 7’ bellow from road level and connected with other floors by four Stair & six passengers’ lift and also connected with road by two ramp.
- Floor area = 20628.14 ft^2.
- Floor height =12'-0".
- This floor has one generator room (15’-6”X6’-7”), one toilet (6’-7”X5’-0”), one driving waiting room (17’-10”X8’-0”), one guard room (7’-0”X8’-0”), one family flat for guard (380 ft^2), it also contain 48 car and 32 motor cycle.

❖ 3.3.2 Ground Floor (Commercial)

- 5’ above from road level and connected with 1^{st} to 7^{th} floors by four stair & six passenger lift.
- Floor area = 20628.14 ft^2.
- Floor height =10'-0".
- This floor has 51 shops, which 76sft has 2 shops, and the others shops are about 115sft, 125stt, 155sft, 165sft, 175sft, 185sft, 195sft and 300sft respectively 6 shops, 6 shops, 9 shops, 8 shops, 2 shops, 6 shops, 10 shops and 2 shops.

❖ 3.3.3 1^{st} to 6^{th} Floor (Commercial)

- Floor area = 20628.14 ft^2.
- Floor height =10'-0".
- This floor has 59 shops, which have 2 shops in 76sft, and the others shops are about 115sft, 125stt, 155sft, 165sft, 175sft, 185sft, 195sft, 205sft, 235sft and 300sft respectively 6 shops, 6 shops, 9 shops, 8 shops, 2 shops, 8 shops, 10 shops, 2 shops, 4 shops and 2 shops.

❖ 3.3.4 7th Floor (Commercial)

- Floor area = 20628.14 ft^2.
- Floor height =10'-0".
- This floor has 4 Restaurants, A Cineplex, A ticket counter, A Shop and a Washroom facility. Which have 4 Restaurants 2100sft, 1080sft, 916sft and 965sft respectively. Cineplex is 3024sft, tickets counter is 400sft, 280sft shop & 318sft spaces for Washroom facilities.

❖ 3.3.5 8th Floor (Commercial)

- Floor area = 20628.14 ft^2.
- Floor height =10'-0".
- A 2150sft Mosque, A shop of 360sft, a machine room of 924sft, 2nd floor of Cineplex is 2940sft, a shop of1320sft & 318sft spaces for Washroom facilities.

❖ 3.3.6 9th Floor (Commercial)

- Floor area = 12423 ft^2.
- Floor height =10'-0".
- There are 3 office spaces which are 2882sft, 2124sft & 4392sft.

❖ 3.3.7 10th Floor (Commercial)

- Floor area = 11890 ft^2.
- Floor height =10'-0".
- There are 3 office spaces which are 2882sft, 2124sft & 3460sft.

❖ 3.3.8 11th Floor (Commercial)

- Floor area = 11243 ft^2.
- Floor height =10'-0".
- There are 3 office spaces which are 2882sft, 2124sft & 2997sft.

- **3.3.9 12th Floor (Residential)**
  - Floor area = 11016.5 ft^2.
  - Floor height =10'-0"
  - There are 5 units, which are 2 units of 2071sft, 1 unit of 2091sft, 1 unit of 1640sft, and 1 unit of 1420sft.

- **3.3.10 13th Floor (Residential)**
  - Floor area = 10287.5 ft^2.
  - Floor height =10'-0"
  - There are 5 units, which are 2071sft, 2071sft, 2091sft, 1527sft and 1375sft respectively.

- **3.3.11 14th Floor (Residential)**
  - Floor area = 9801 ft^2.
  - Floor height =10'-0"
  - There are 5 units, which are 2071sft, 2071sft, 2091sft, 1460sft and 1297sft respectively.

- **3.3.12 15th Floor (Residential)**
  - Floor area = 9497 ft^2.
  - Floor height =10'-0"
  - There are 5 units, which are 2071sft, 2071sft, 2091sft, 1335sft and 1217sft respectively.

- **3.3.13 16th Floor (Residential)**
  - Floor area = 9376 ft^2.
  - Floor height =10'-0"
  - There are 4 units, which are 2071sft, 2071sft, 2091sft, 2156sft respectively.

- **3.3.14 17th Floor (Residential)**
  - Floor area = 9376 ft^2.
  - Floor height =10'-0"
  - There are 4 units, which are 2071sft, 2071sft, 2091sft, 2120sft respectively.

Details of Basement Floor, 1st to 17th Floors and Roof top are shown in Figures 3.4~3.17.

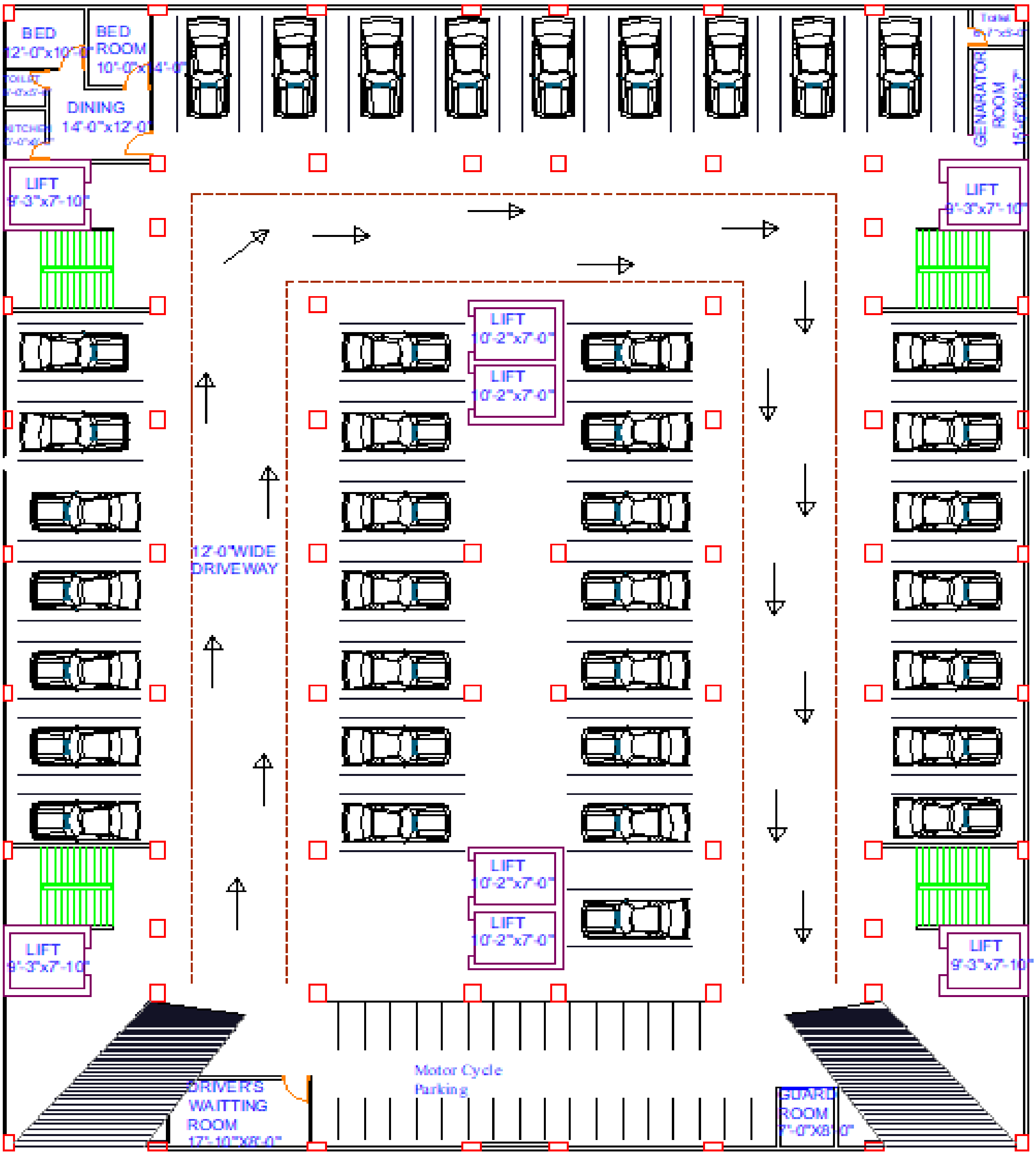


Figure 3.4: Plan view of Basement Floor.

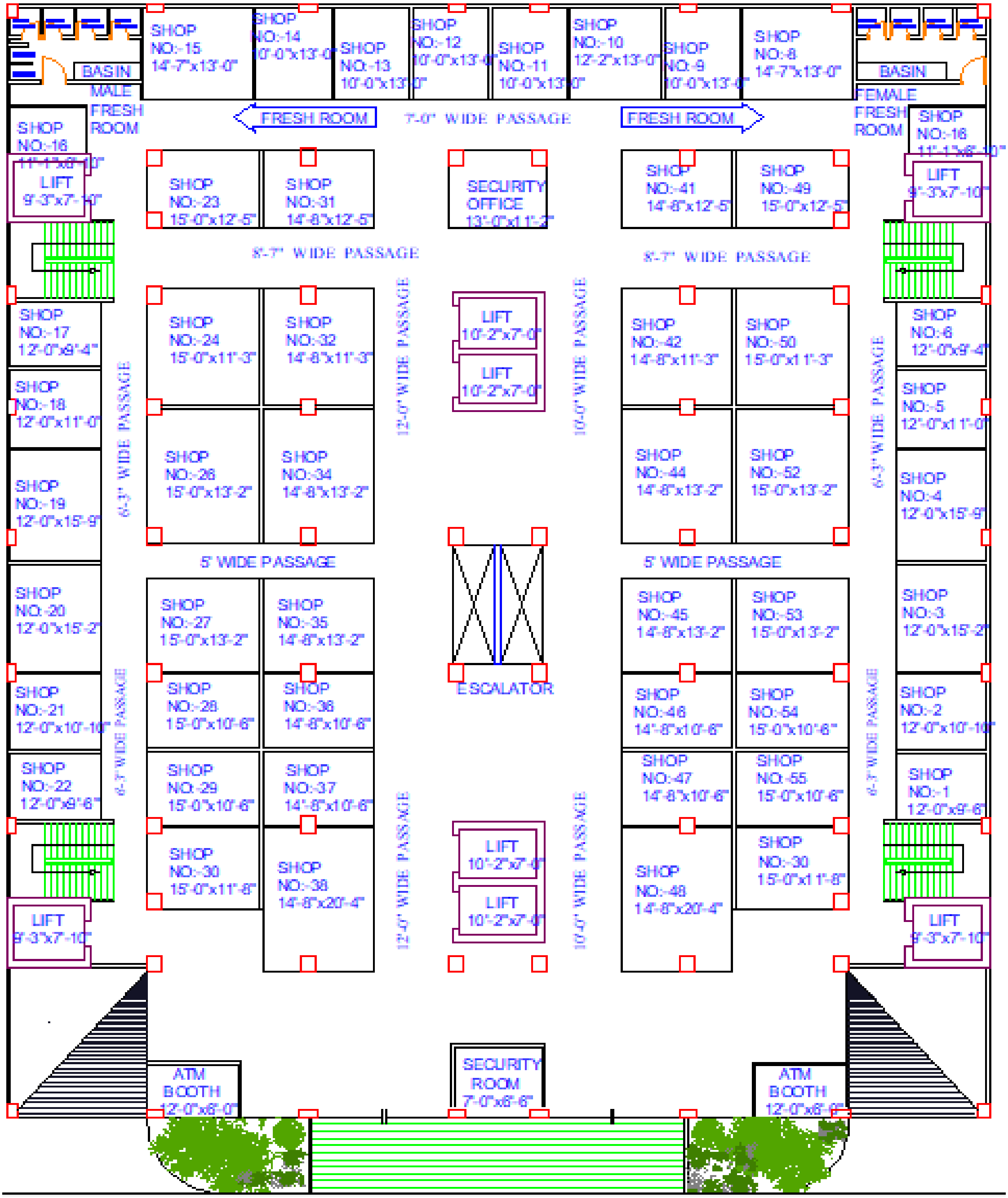


Figure 3.5: Plan view of Ground Floor.

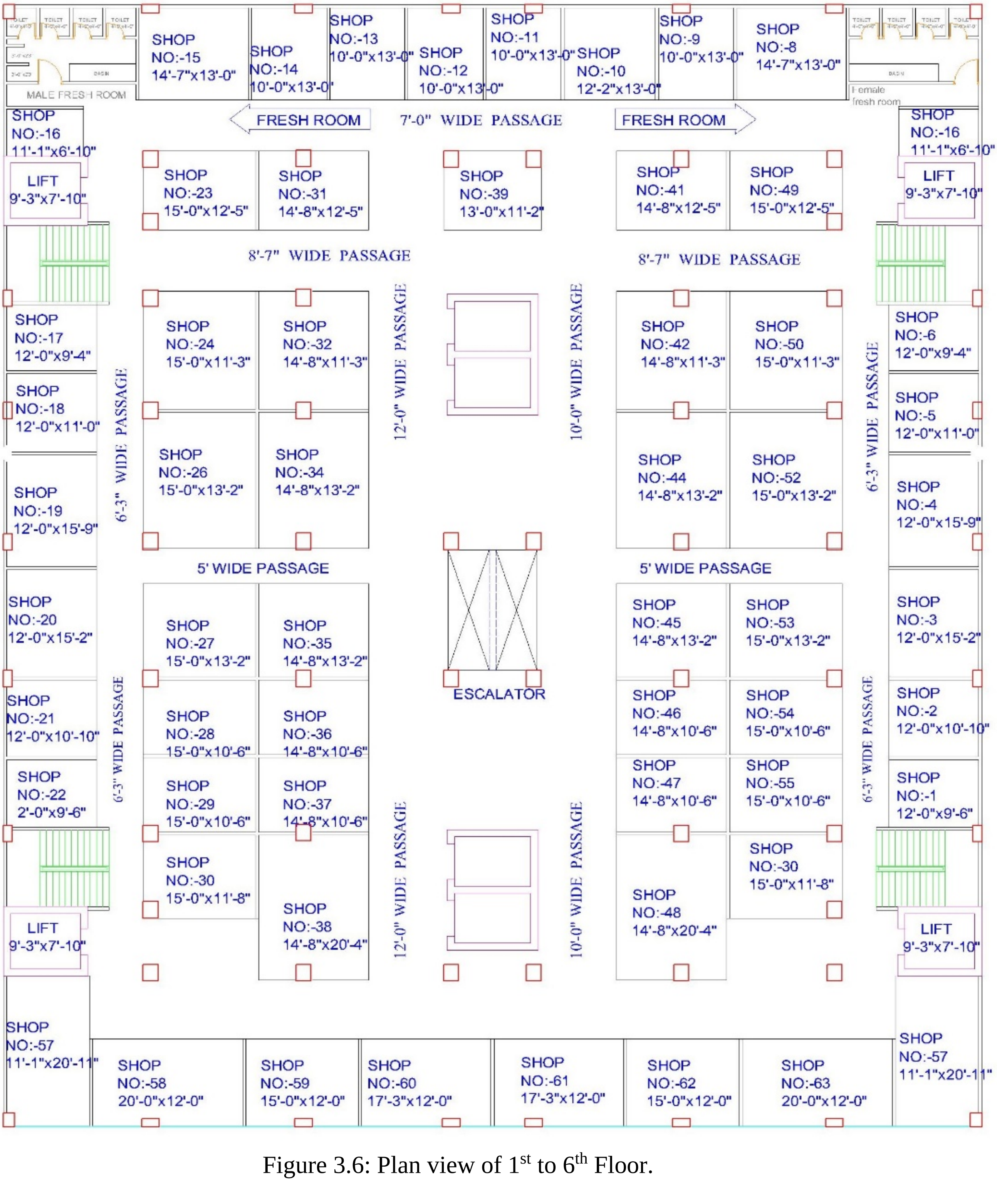


Figure 3.6: Plan view of 1st to 6th Floor.

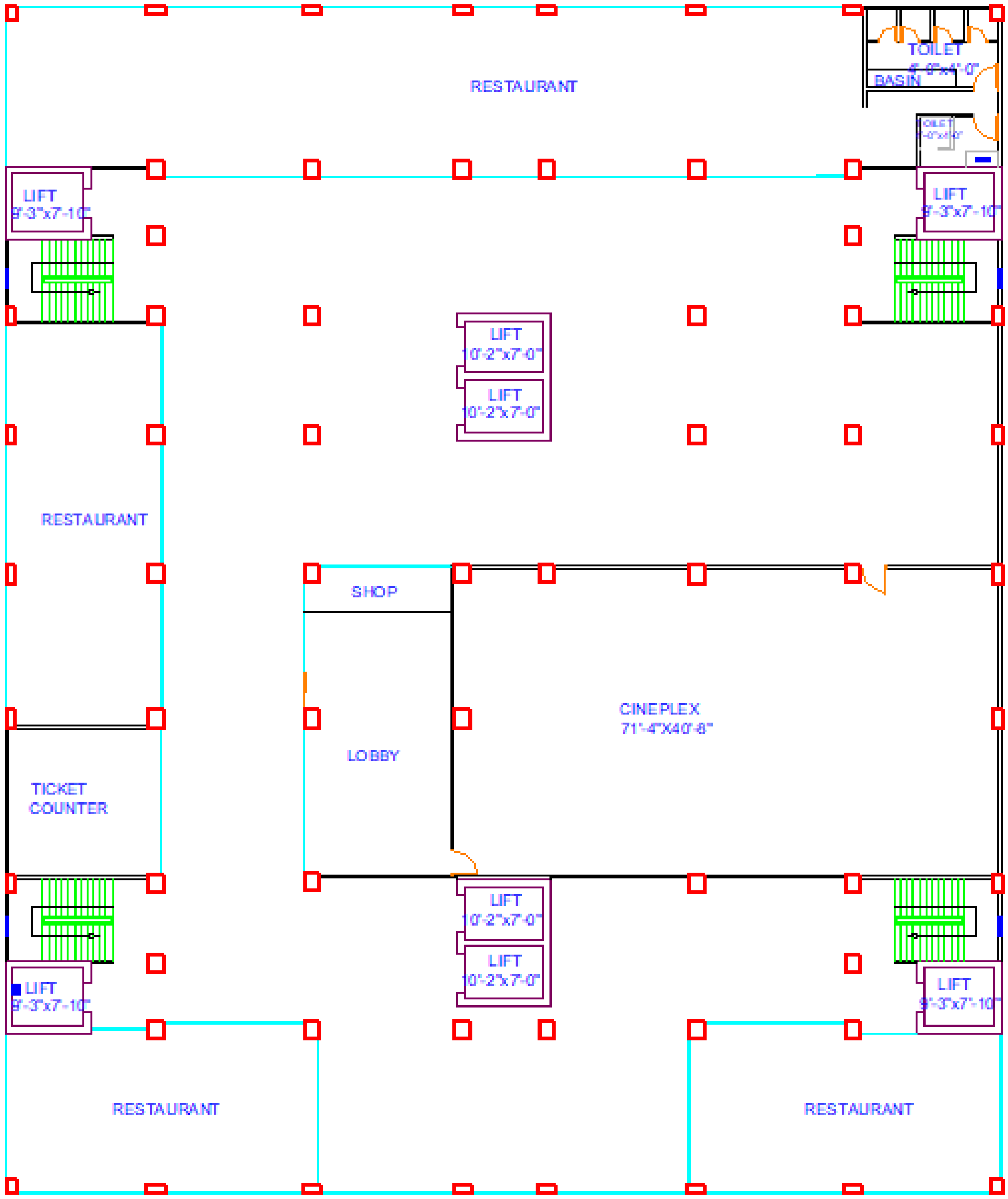


Figure 3.7: Plan view of 7th Floor.

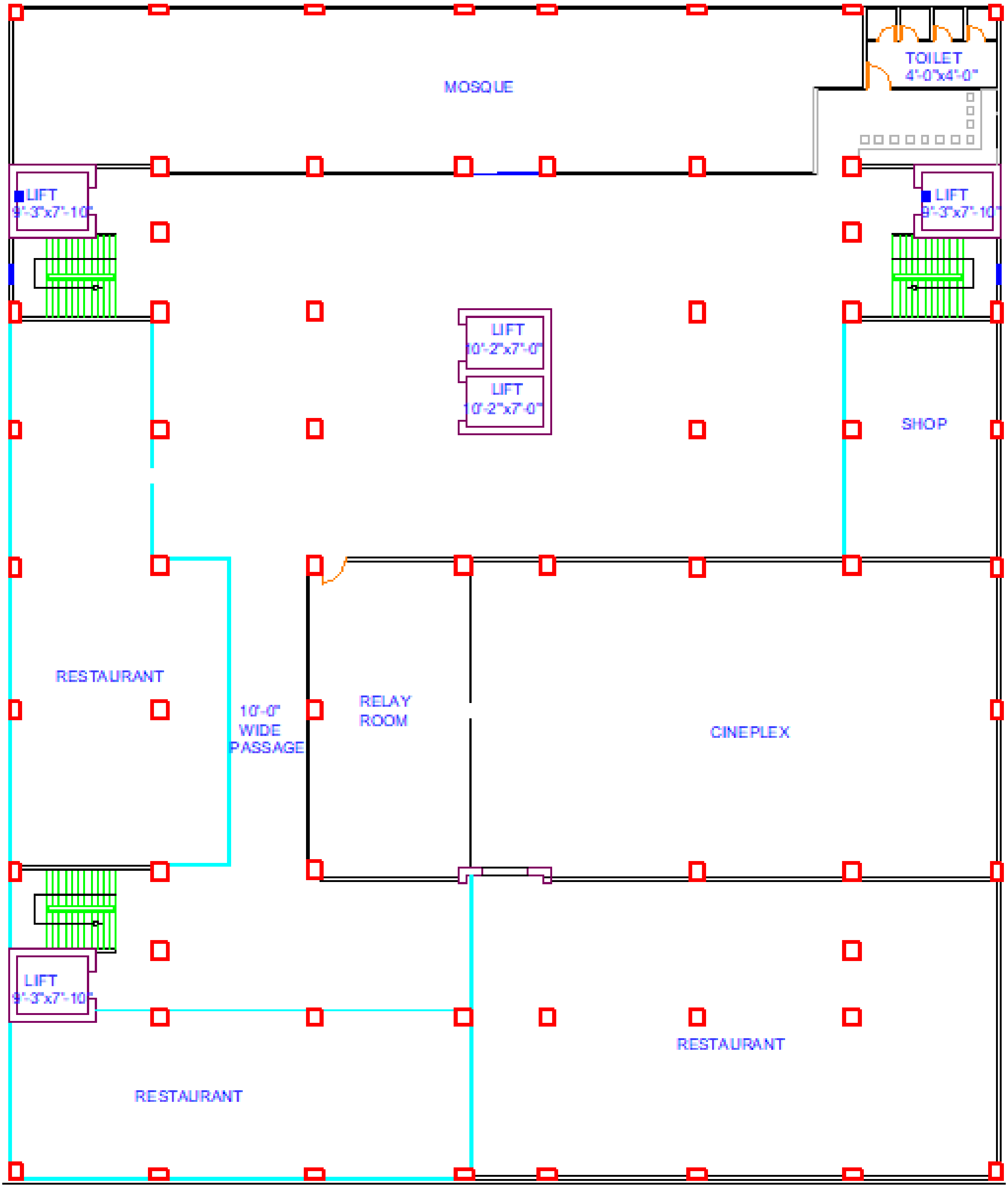


Figure 3.8: Plan view of 8th Floor.

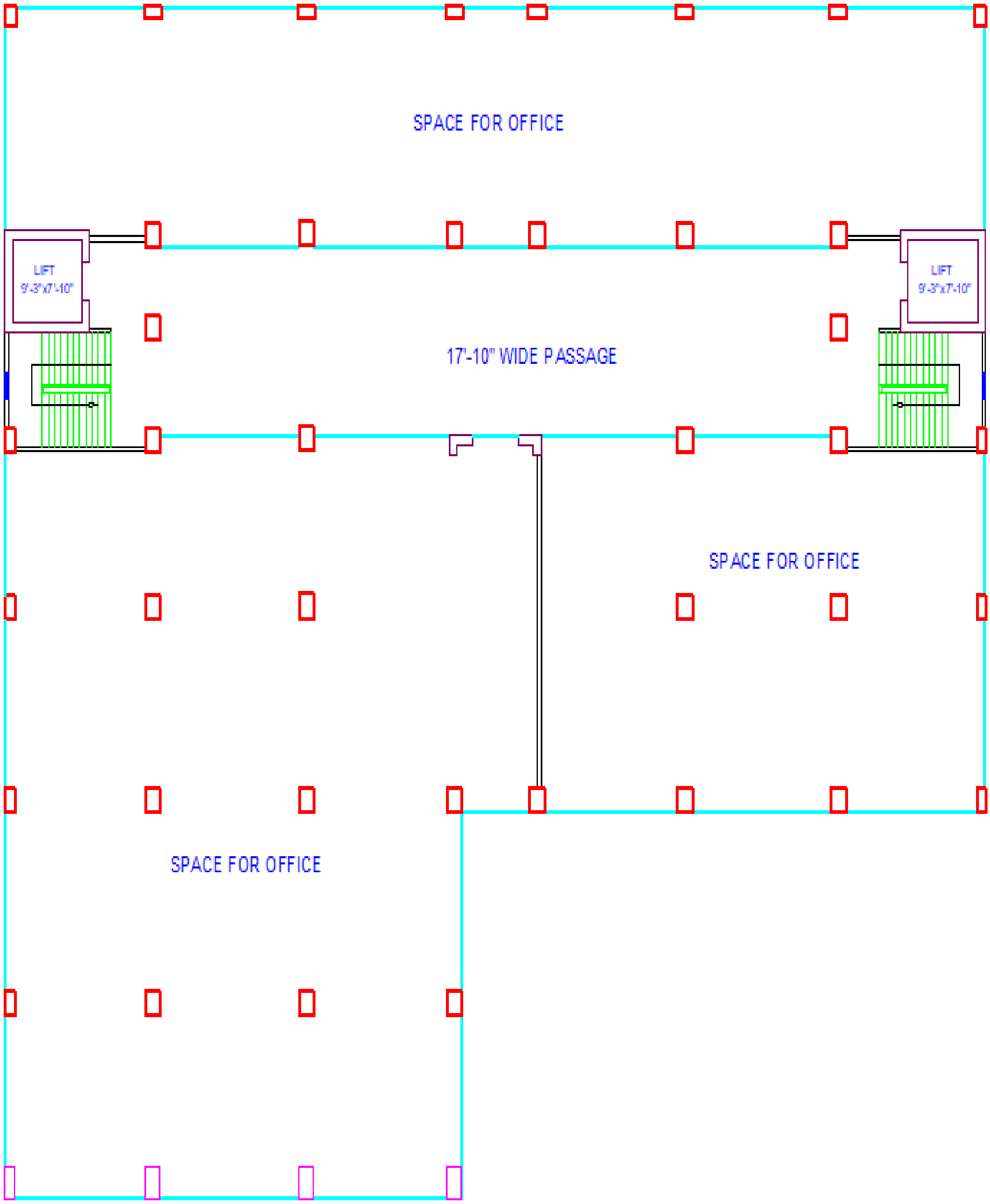


Figure 3.9: Plan view of 9th Floor (12423 ft^2).

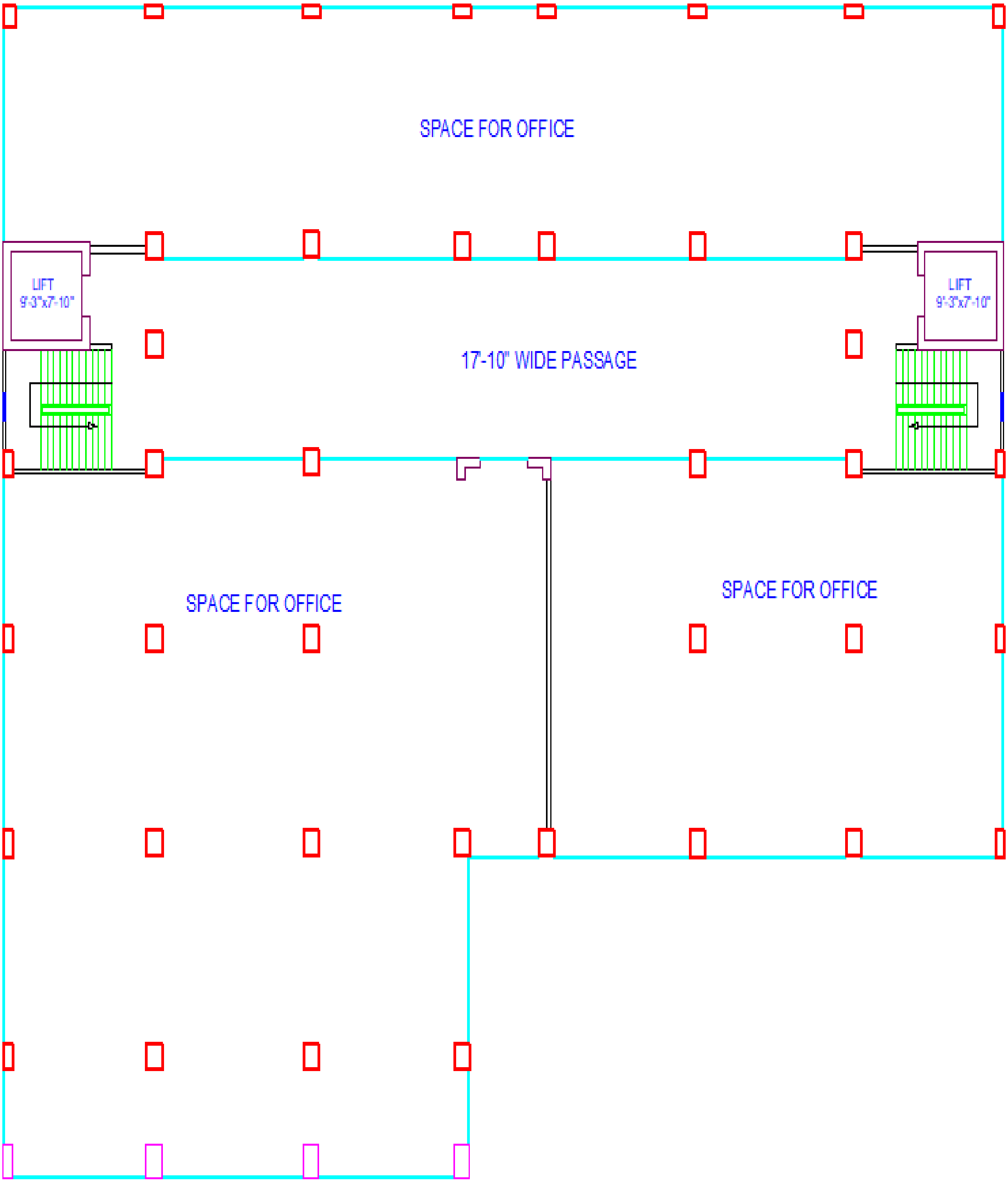


Figure 3.10: Plan view of 10th Floor (11890 ft^2).

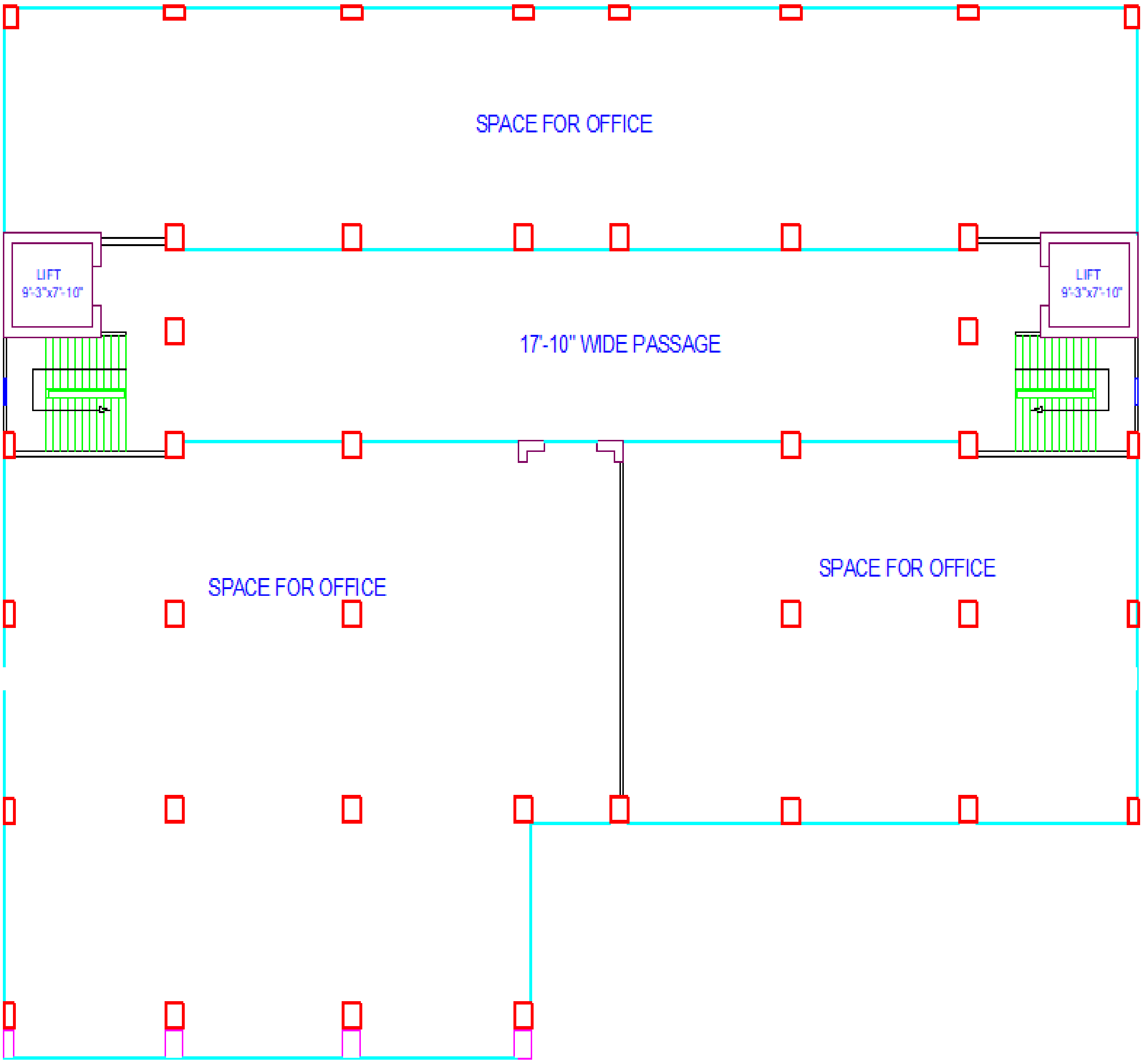


Figure 3.11: Plan view of 11th Floor (11243 ft^2).

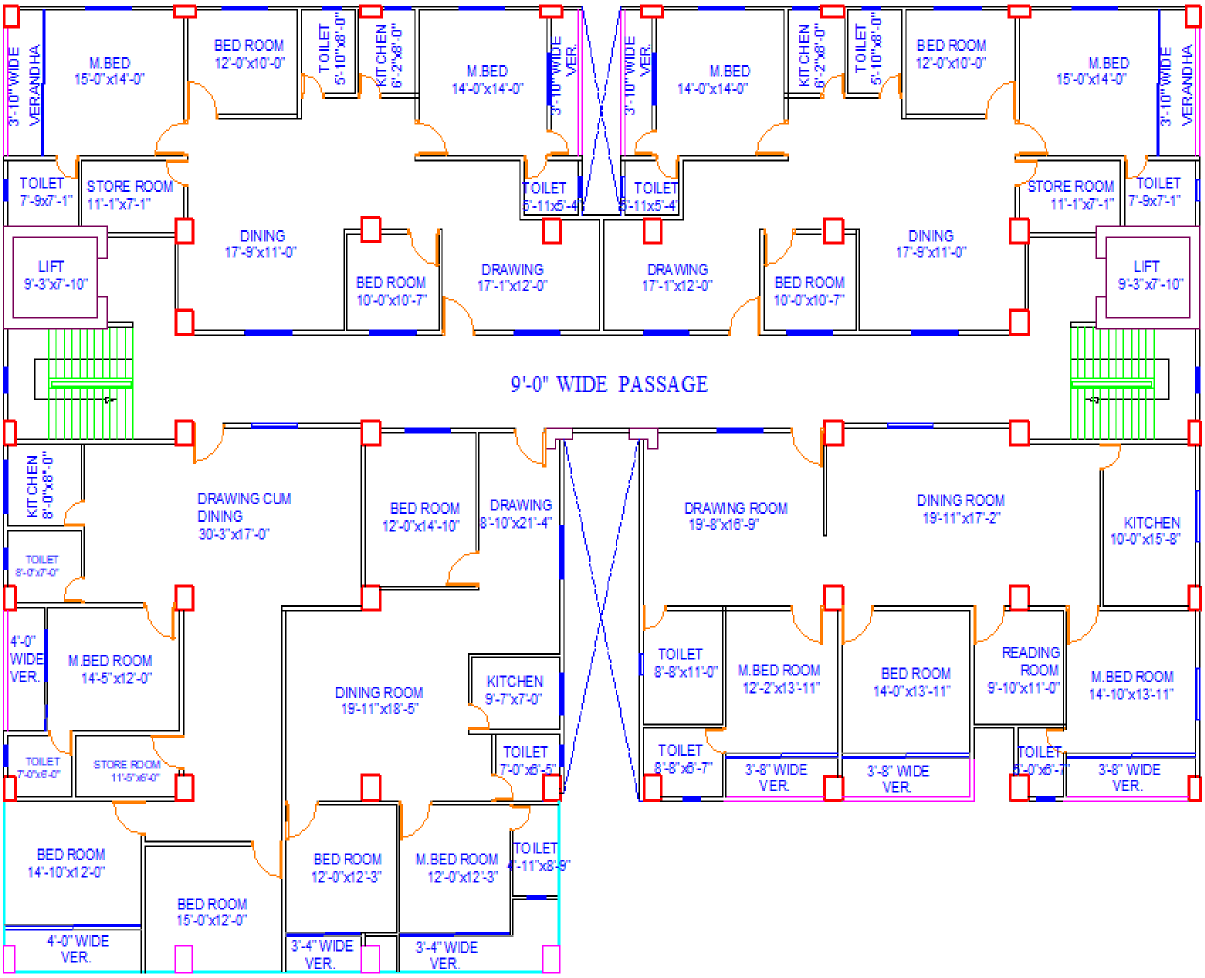


Figure 3.12: Plan view of 12th Floor (11016.5 ft^2).

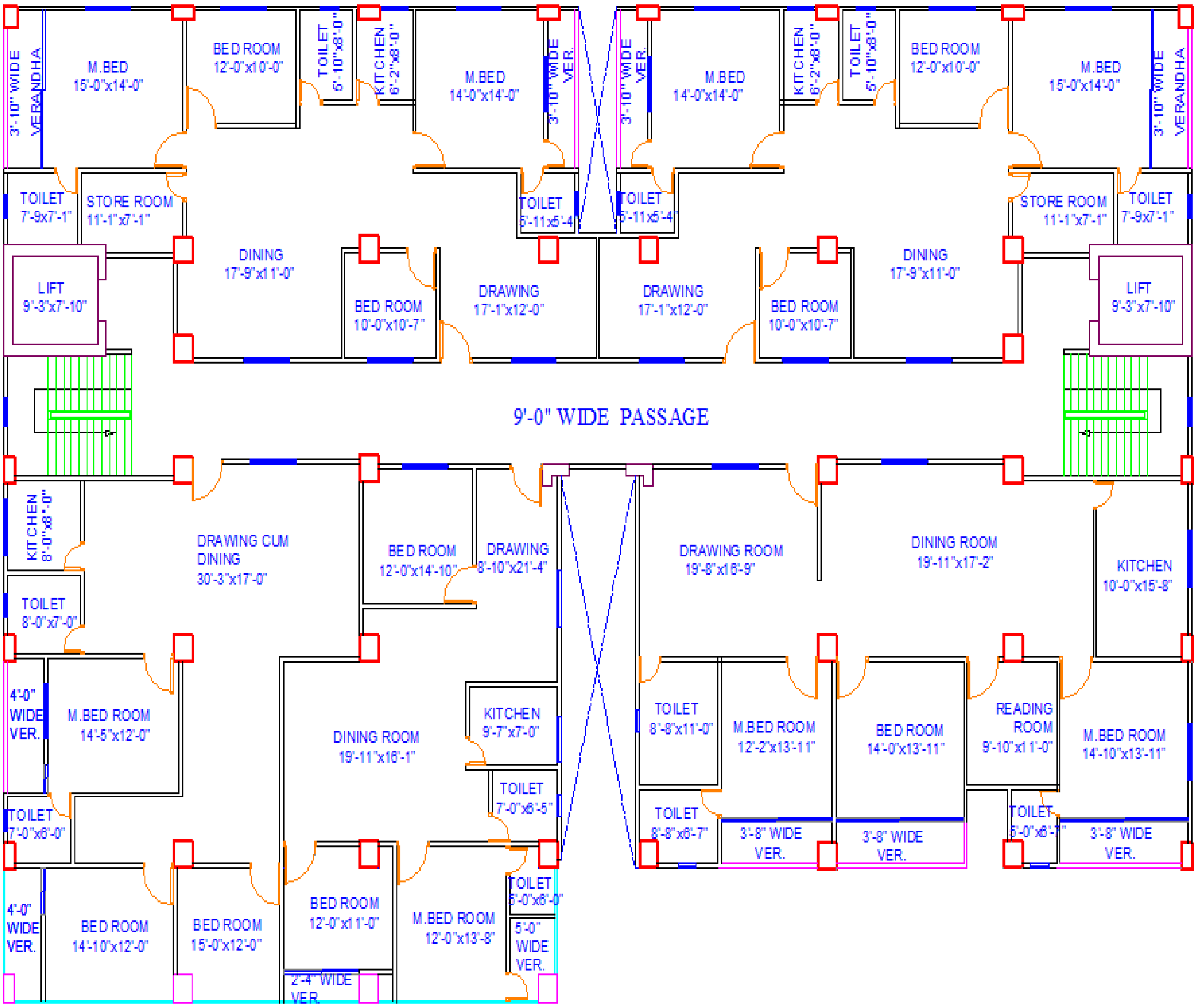


Figure 3.13: Plan view of 13th Floor (10287.5 ft^2).

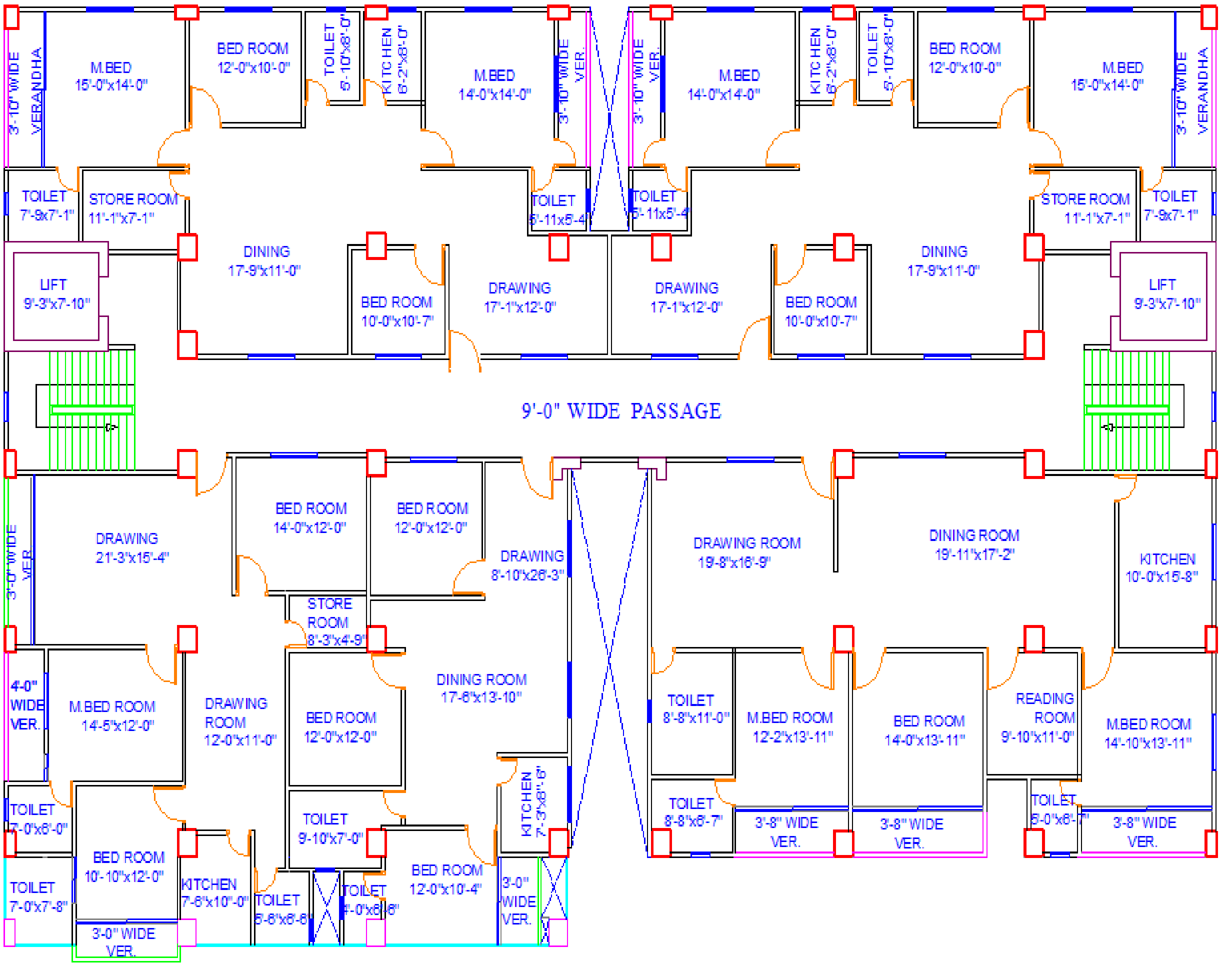


Figure 3.14: Plan view of 14th Floor (9801 ft^2).

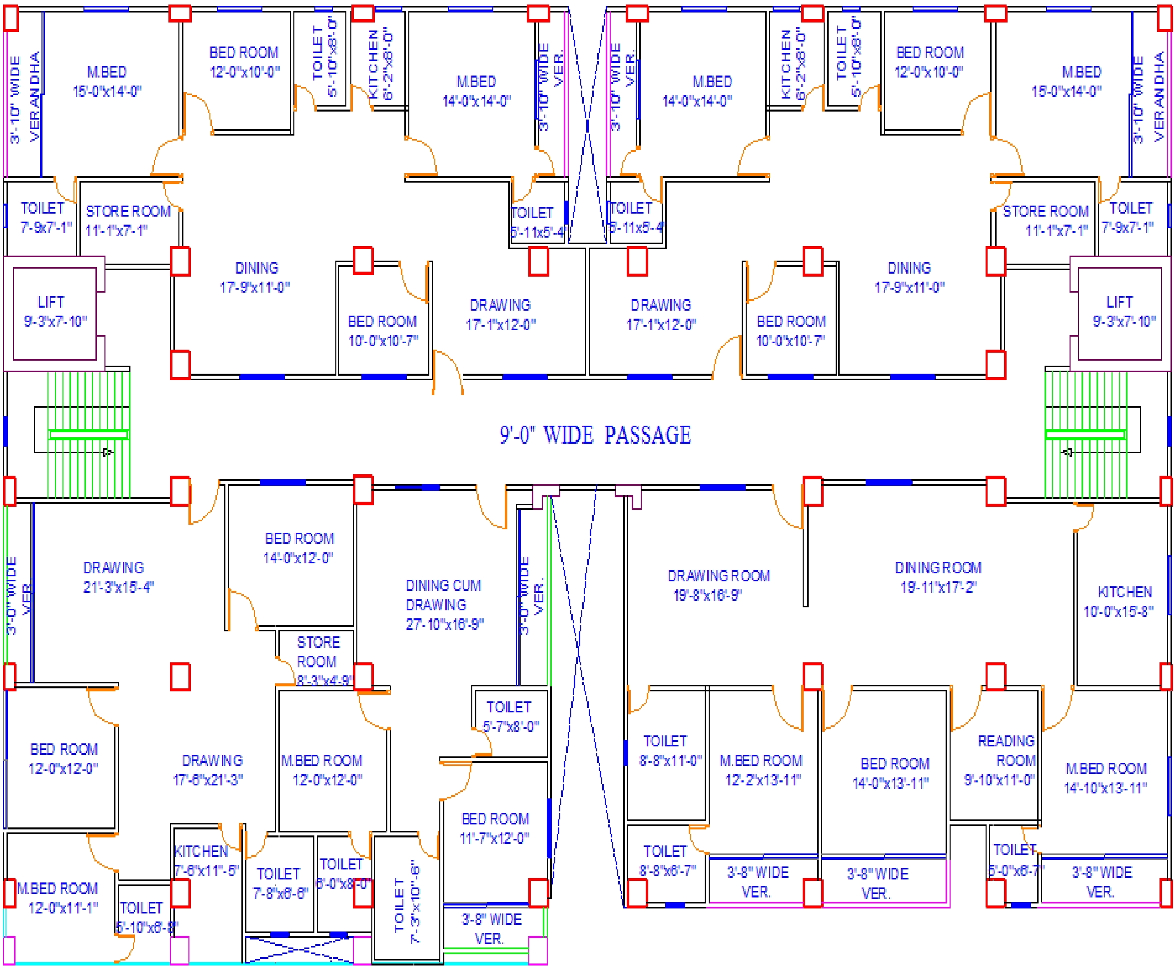


Figure 3.15: Plan view of 15th Floor (9497 ft^2)

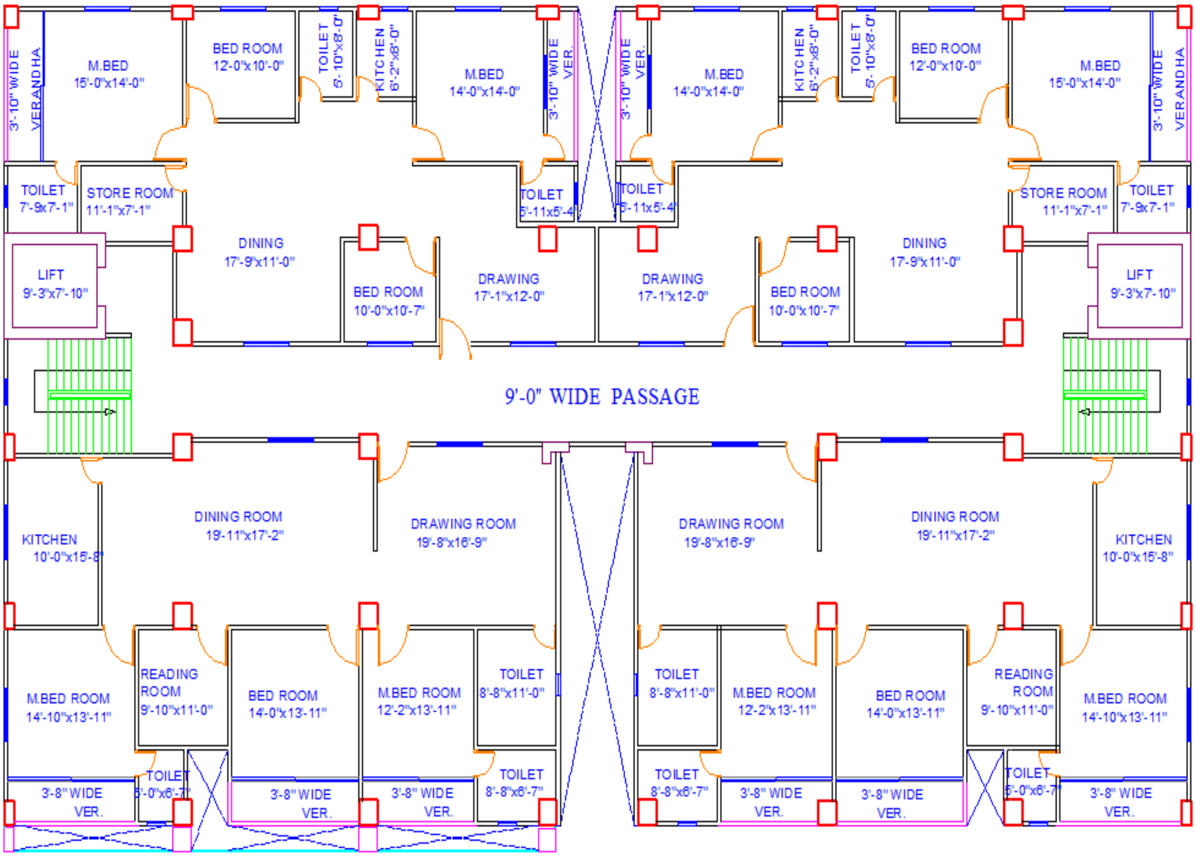


Figure 3.16: Plan view of 16th Floor (9376 ft^2).

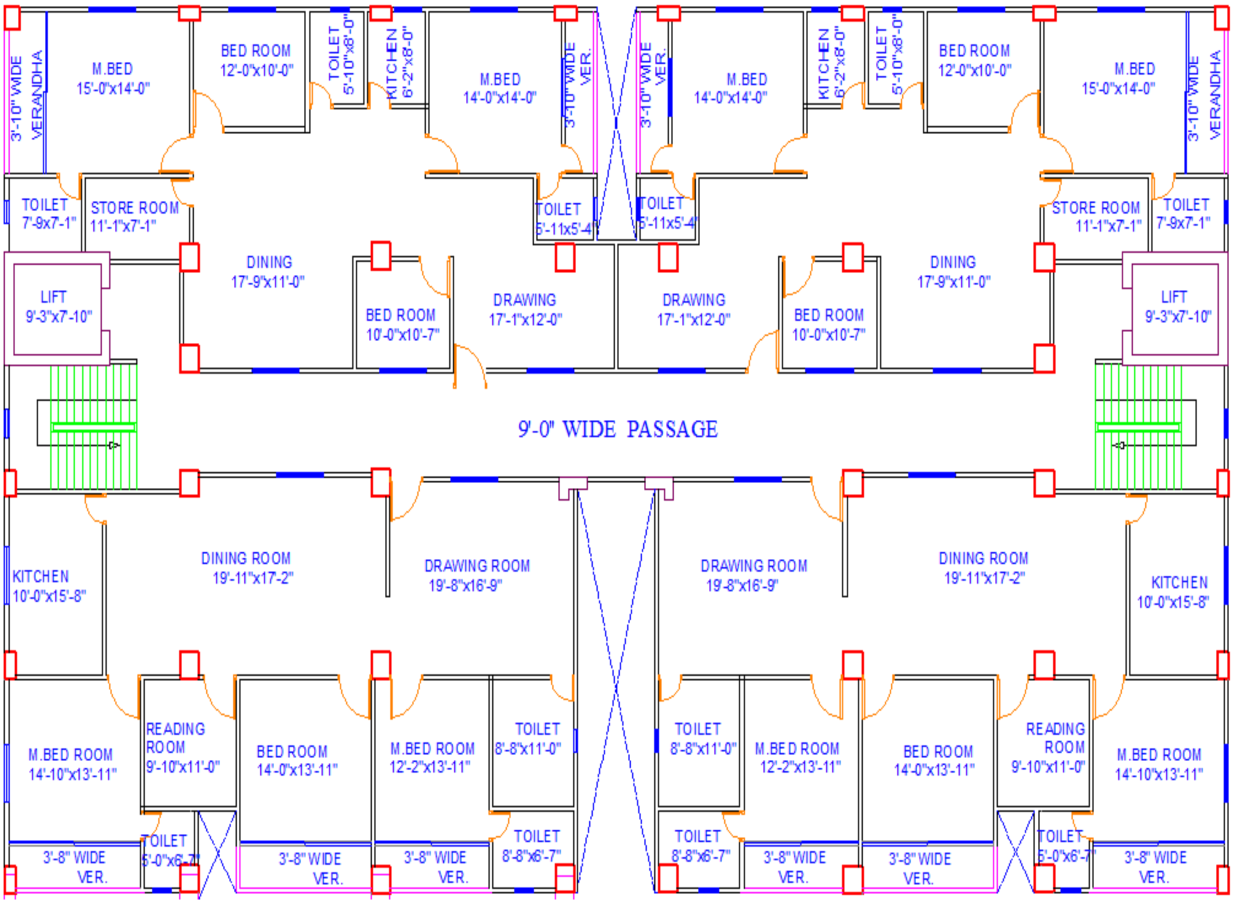


Figure 3.17: Plan view of 17th Floor (9376 ft^2).

3.4 Structural details of propose Building

This part includes the portions of results of analysis, design and detailing of Slab, Beam, Column, Stair, Lift core, Water tank etc.

3.4.1 <u>Detailing of Slab</u>

All slab panels are analyzed, designed and detailed by ETABS and SAP software. Detailing of different slab panels shown in Figure 3.18.

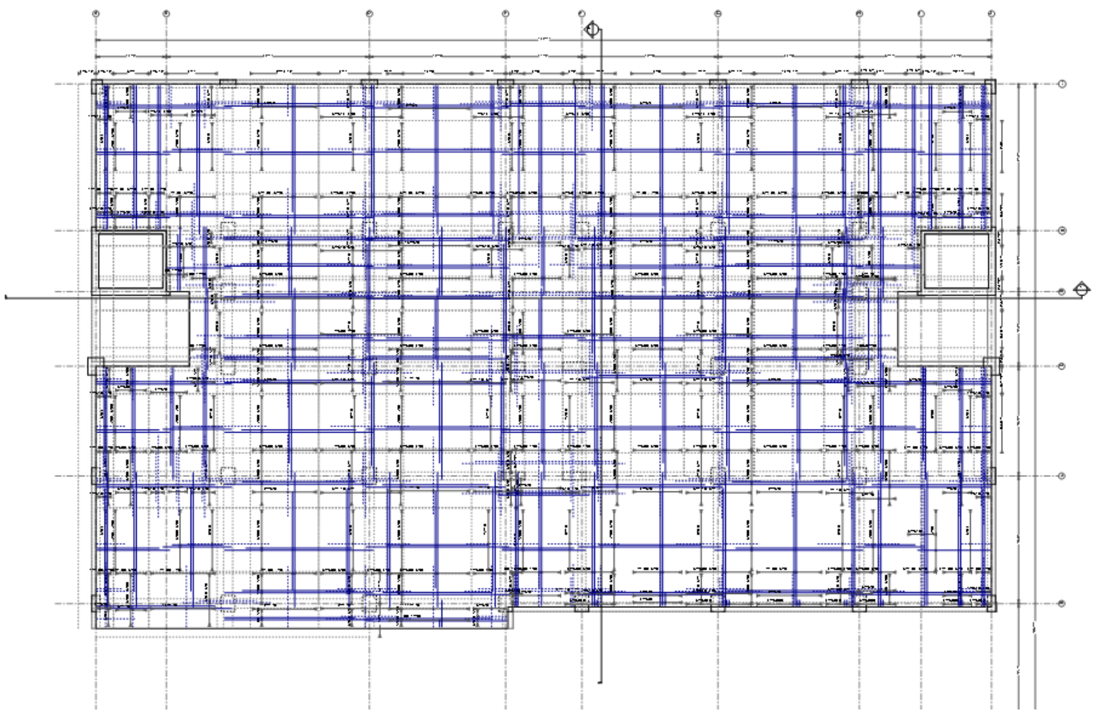


Figure 3.18a: Slab detailing of 16th Floor

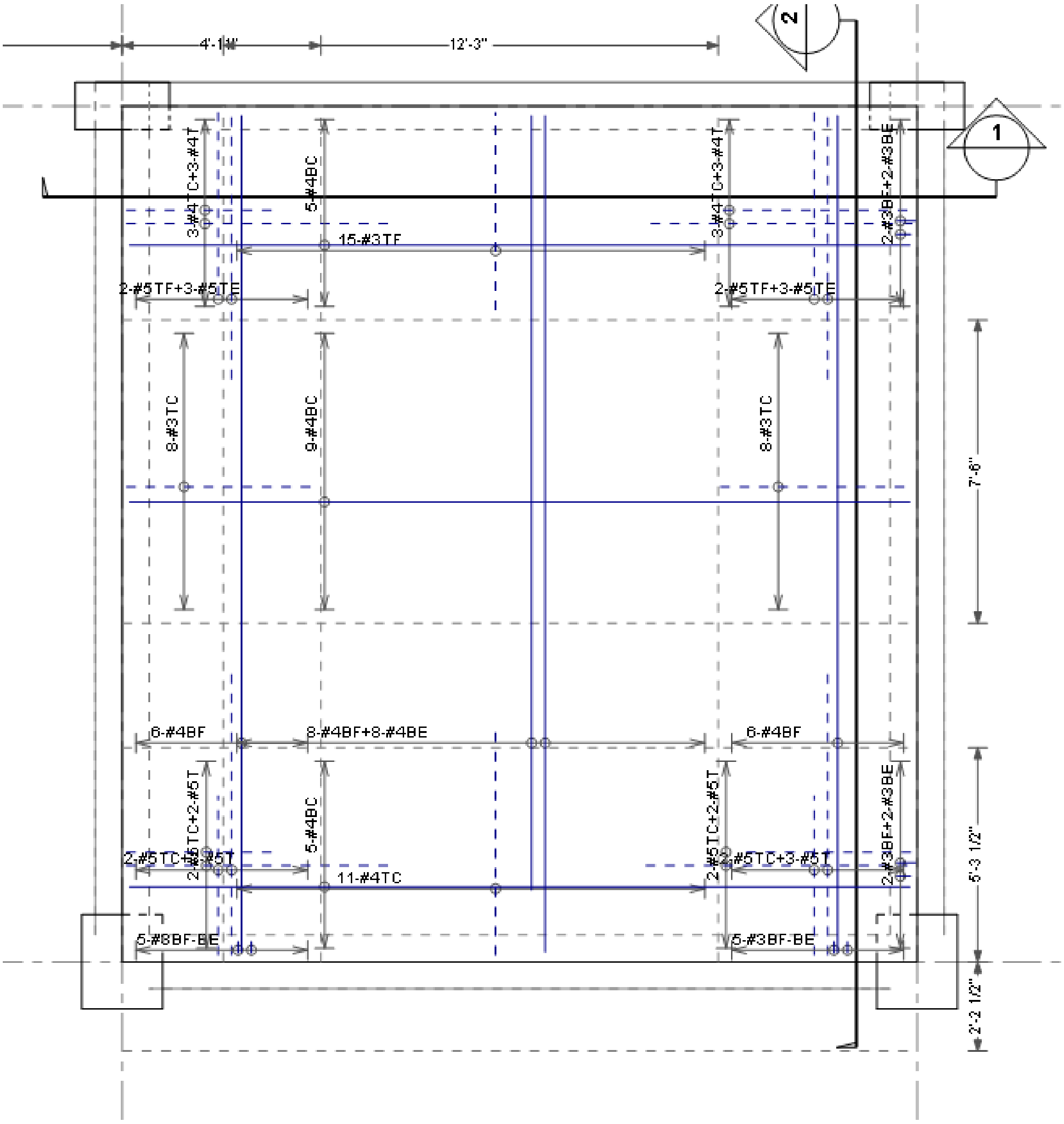


Figure 3.18b: Slab detailing of 16th Floor (Partial)

3.4.2 Detailing of Beam

This gives details of floor beam design for lateral loadings. There are several floor beams in this structure. All beams are analyzed by ETABS software. For space limitations, design of grid 8, frame D-E of 15th floor is presented here.

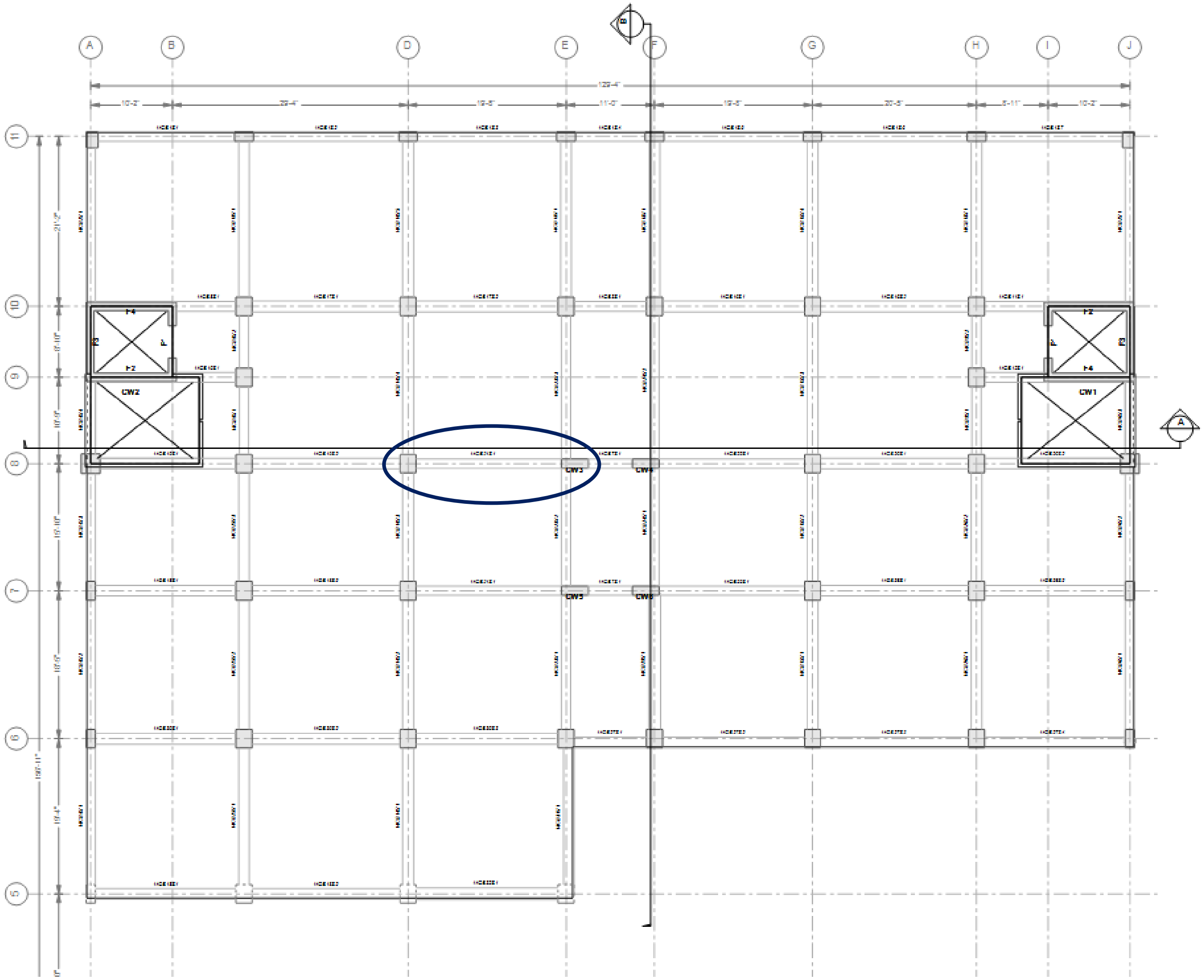


Figure 3.19: Floor beam layout of 15th floor

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 3.2a: Details of Longitudinal Reinforcement (Bottom)

Beam Portion		End - I	Middle	End – J
Bottom	Moment, Mu	+47.66 K-ft	+42.27 K-ft	+65.67 K-ft
	As (Required)	0.82 in^2	0.71 in^2	0.85 in^2
		Use 2ϕ20mm Bar	Use 2ϕ20mm Bar	Use 2ϕ20mm Bar
	As (Provided)	0.88 in^2	0.88 in^2	0.88 in^2

Table 3.2b: Details of Longitudinal Reinforcement (Top)

Beam Portion		End - I	Middle	End – J
Top	Moment, Mu	-106.21 K-ft	-28.38 K-ft	-114.97 K-ft
	As (Required)	1.41 in^2	0.47 in^2	1.50 in^2
		Use 2 ϕ20mm Bar +2ϕ16mm Bar	Use 2 ϕ20mm Bar	Use 2 ϕ20mm Bar +2ϕ16mm Bar
	As (Provided)	1.50 in^2	0.88 in^2	1.50 in^2

3. Transverse/Shear Reinforcement:

There will be 3 types of reinforcement for share.

Table 3.3: Details of Shear Reinforcement

Beam Portion	End - I	Middle	End – J
Shear Force, V_u	8.07 Kip	1.1 Kip	26.97 Kip
Shear Steel	0.14in^2/ft	0 in^2/ft	0.15in^2/ft

(a) Seismic Stirrup:

Use ϕ16mm as Seismic Stirrups.

- **Spacing**

S_{max} = $\frac{d}{4}$ = $\frac{17.5}{4}$ = 4.375 ≡ 4.25" c/c

S_{max} = 8×minimum dia. of main bar = 8×0.75= 6″c/c

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

$\therefore S_{max} = 4.25$" 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 @ 4.25"c/c within *2h* = 2 x 20= 40″ from both faces of column.

Here, $\frac{A_v}{s} = 0.15$

$A_v = 0.15 \times 4.25$

=0.64 in²

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

So cross tie is required.

So, A_v =0.64-0.62= 0.02 in² (negligible)

So, No cross tie 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 ϕ20mm bar

$$l_d \geq \frac{0.04 \times A_b \times fy}{\sqrt{f'_c}} = \frac{0.04 \times 0.44 \times 60000}{\sqrt{4000}} = 16.70''$$

$l_d \geq 0.0003\, d_b\, f_y$ = 0.0003 × 0.75 × 60000 = 13.5″

l_d = 12″

Selected, l_d = 16.70″

Splice length for ϕ20mm (bottom) bars = 1.3 × 16.70″ = 21.71″=22″

Splice length for ϕ20 (top) bars = 17″

- **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"≡ 4.25" 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 3.20.

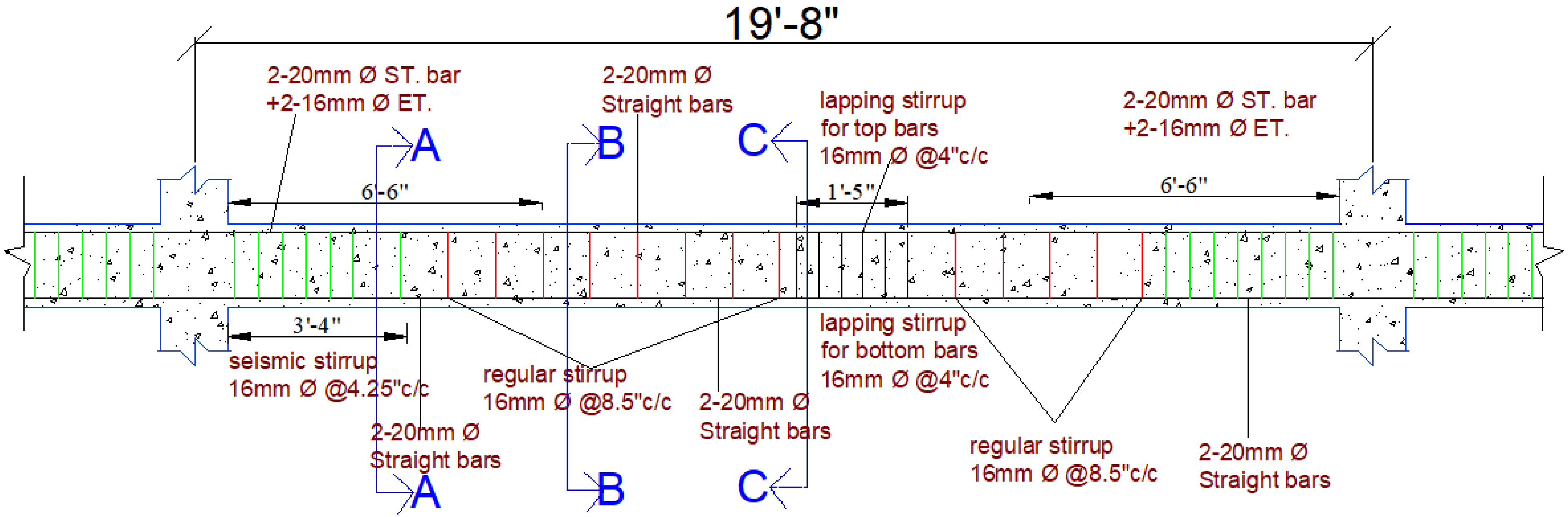
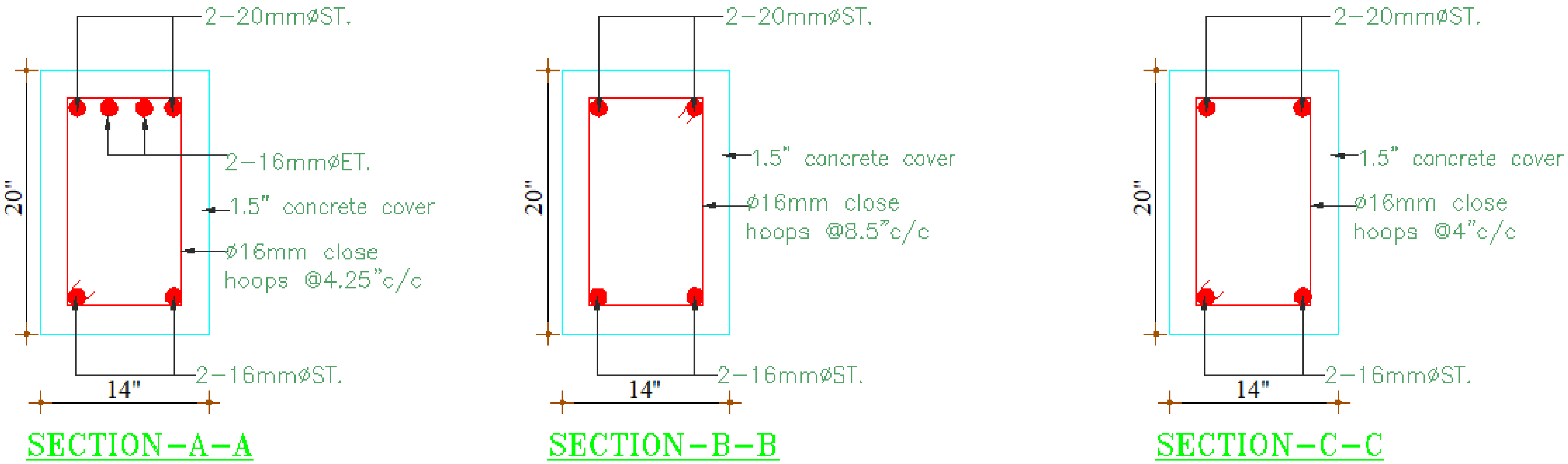


Figure 3.20a: Longitudinal Section Reinforcement of Beam



Cross section of Beam (A-A)

With Seismic stirrup.

Cross section of Beam (B-B)

With Regular stirrup.

Cross section of Beam (C-C)

With lapping stirrup.

Figure 3.20b: Cross Section at three locations of Beam

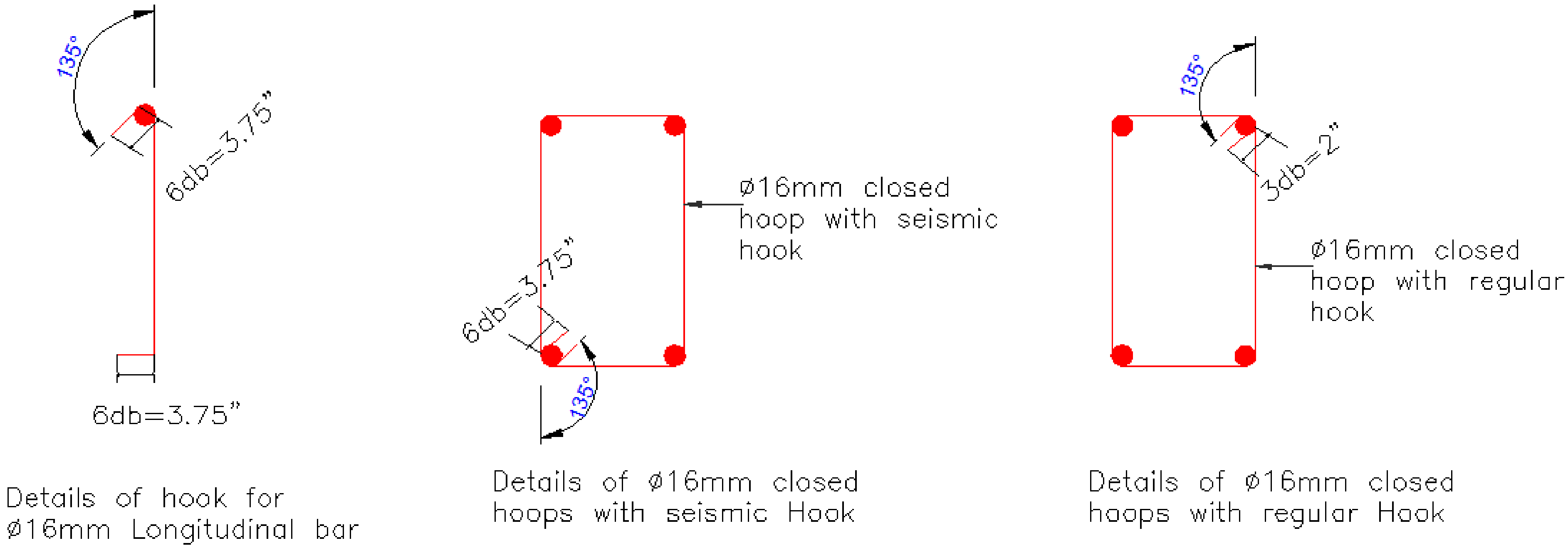


Figure 3.20c: Details of Closed Hoop Stirrup of Beam.

3.4.3 <u>Detailing of Column</u>

This presents the detailed design of reinforced columns there are several columns in this building. All columns are analyzed by ETABS software. For space limitations, design of grid J-4 (C-28x30) of GROUND floor is presented here.

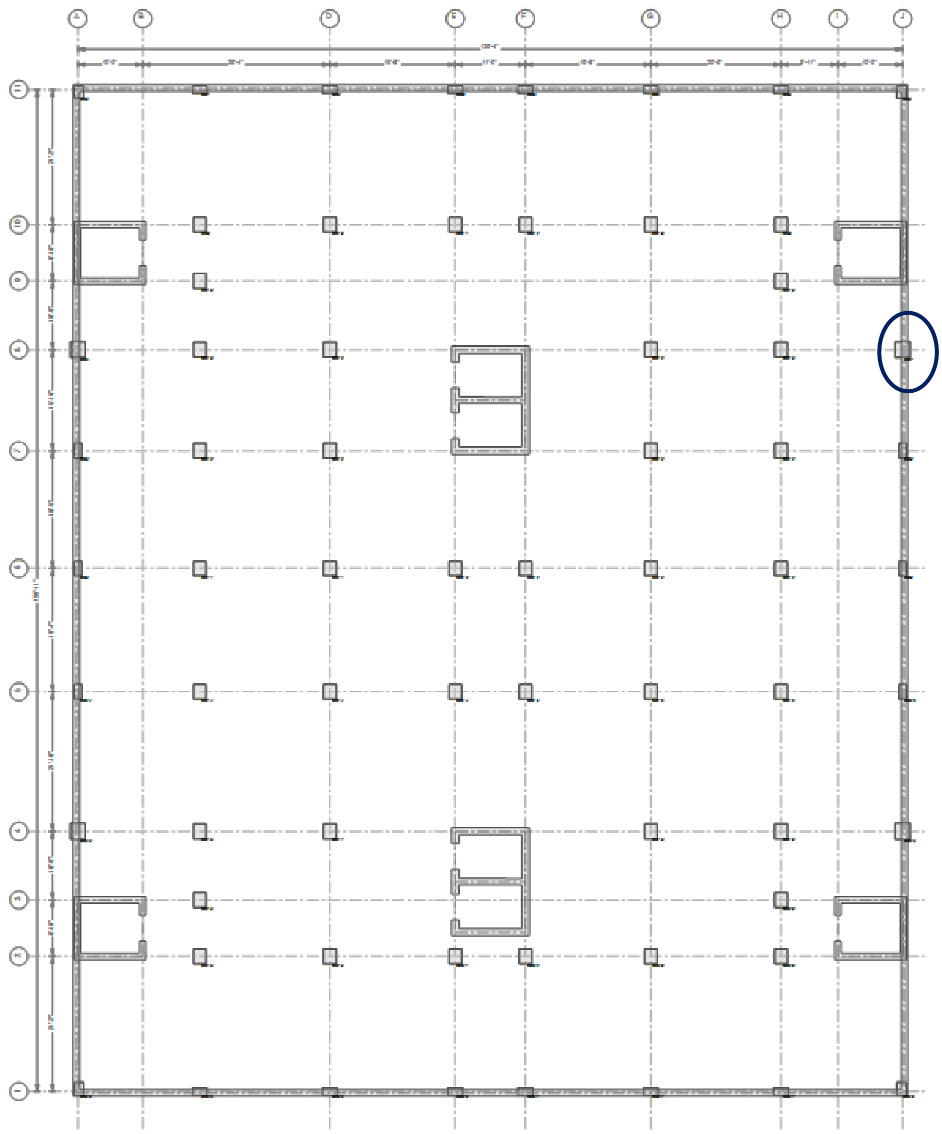


Figure 3.21: Ground Floor column layout

1. Dimension of the column:

Assume the Size of column = b × h = 28″×30″

2. Longitudinal reinforcement of column:

Moment, Shear, Axial Force and Steel area of the Column and details from ETABS, given below by Table 3.4.

Table 3.4: Longitudinal reinforcement of column (C-28x30) GRID J-4

	Top	**Bottom**
Axial Force, Pu	787.95 Kip	777.95 Kip
Moment, Mu	+999.94 k-ft	-676.52 k-ft
As (Required)	33.85 in^2	27.05 in^2
Required Bar	Use 22ϕ35mmBar	Use 22ϕ35mmBar
As (Provided)	34.32 in^2	34.32 in^2

3. Transverse reinforcement of column:

There are three types of ties.

Table 3.5: Transverse/Shear Reinforcement of Column

	Top	**Bottom**
Shear Force, Vu	8.35 Kip	8.35 Kip
$Av/_S$ **(Required)**	0.30 in^2	0.30 in^2

(a) Seismic Tie

We use closed hoops with seismic hook. It is provided at a specified distance near both joints. Useϕ 12mm tie.

- **Spacing of the seismic tie:**

First Condition-

$$S_o = \frac{minimum\ dimension\ of\ Column}{4} = \frac{28}{4} = 7''$$

Second Condition-

$$S_o = 6d_b = 6 \times \frac{11}{8} = 8.25''$$

Third Condition-

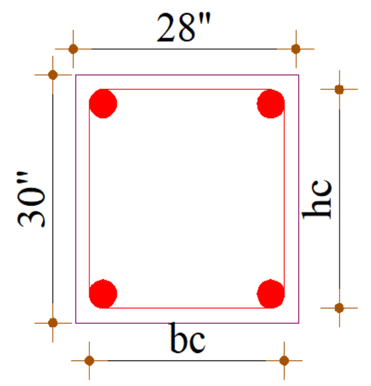
S_o = 4"

From above condition, the minimum spacing S_o= 4.0" c / c

❖ Transverse reinforcement:

Total transverse steel areas A_{sh}

Short direction



$h_c = 30" - 2 * [1.5 + \frac{4}{2x8}] = 26.50"$

A_{ch} = (30-2*1.5) x (28-2*1.5) = 675 in^2

Total Transverse reinforcement will be larger of the followings:

$A_{sh} > 0.09 \times S \times h_c \times \frac{f_c'}{f_{yh}} = 0.09 * 7 * 26.5 * \frac{4}{60} = 1.113\ in^2$ (governs)

Or $A_{sh} > 0.30 \times S \times h_c \times \left(\frac{A_g}{A_{ch}} - 1\right) \times \frac{f_c'}{f_{yh}} = 0.30 \times 7 \times 26.5 \times \left(\frac{30*28}{675} - 1\right) \times \frac{4}{60}$

$= 0.91\ in^2$

Use 2 ϕ12mm [1 outside closed hoop] & 4 ϕ12mm cross ties

Provided areas = 2x0.20+4x0.20 = 1.20 $in^2 > 1.113 in^2$.

Long direction

$h_c = 28" - 2 * [1.5 + \frac{4}{2x8}] = 24.50"$

A_{ch} = (30-2*1.5) x (28-2*1.5) = 675 in^2

Total Transverse reinforcement will be larger of the followings:

$A_{sh} > 0.09 \times S \times h_c \times \frac{f_c'}{f_{yh}} = 0.09 * 7 * 24.5 * \frac{4}{60} = 1.03\ in^2$ (governs)

Or $A_{sh} > 0.30 \times S \times h_c \times \left(\frac{A_g}{A_{ch}} - 1\right) \times \frac{f_c'}{f_{yh}} = 0.30 \times 7 \times 24.5 \times \left(\frac{30*28}{675} - 1\right) \times \frac{4}{60}$

$= 0.838\ in^2$

Use 2 ϕ12mm [1 outside closed hoop] & 4 ϕ12mm cross ties

Provided areas = 2x0.20+4x0.20 = 1.20 $in^2 > 1.03 in^2$.

❖ Confinement length for transverse steel:

First condition-

l_o = Depth of Column = 30"

Second Condition-

$$l_o = \frac{Clear \quad span \quad of \quad column}{6} = \frac{12 - \frac{30}{12}}{6} \times 12 = 19''$$

Third Condition

l_o= 18"

Provided confinement length from both center of joints, l_o= 30" = 2.5'

Total confinement length =2 l_o = 2 x 2.5' = 5'

(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 ϕ35mm main bar which is calculated as below.

First condition

l_d= 0.04$\times$ A_b $\times$ $\frac{fy}{\sqrt{f'c}}$ = 0.04$\times$ 1.56 $\times \frac{60000}{\sqrt{4000}}$ = 59.20"

Second condition-

l_d =0.0004 $\times$ d_b $\times$ f_y = 0.0004 $\times$ $\frac{11}{8}$ $\times$ 60000= 33"

Third condition-

l_d = minimum 12"

From above condition, selected l_d= 59.20"

Provided splicing length = 1.30 × 59.20" = 76.96"= 6.4′.

- **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-

$S_{max} = \frac{d}{4} = \frac{30 - 1.5 - 0.5 - \frac{1.375}{2}}{4}$ = 6.83 " $\cong$ 6.75"

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-

$S_{max} = 6d_b = 6 \times 1.56 = 9.36 \equiv 9.25''$ c/c

Second condition-

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

So, use ϕ12mm regular ties @ 6 " c / c

Other columns of the frame were designed as per similar procedure. The reinforcement detail of the column is shown in the Figures 3.22.

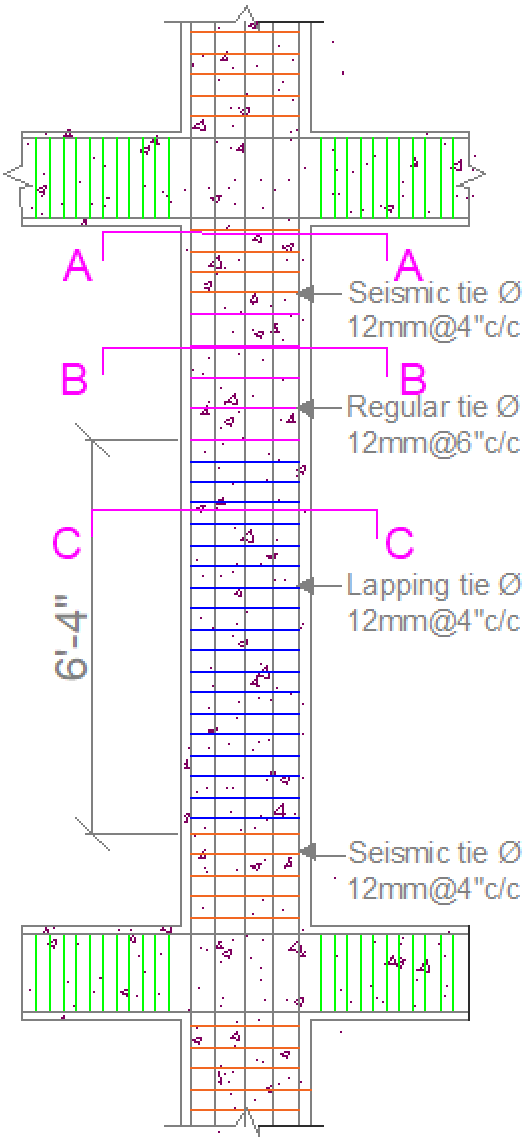


Figure 3.22a: Details of Longitudinal Section

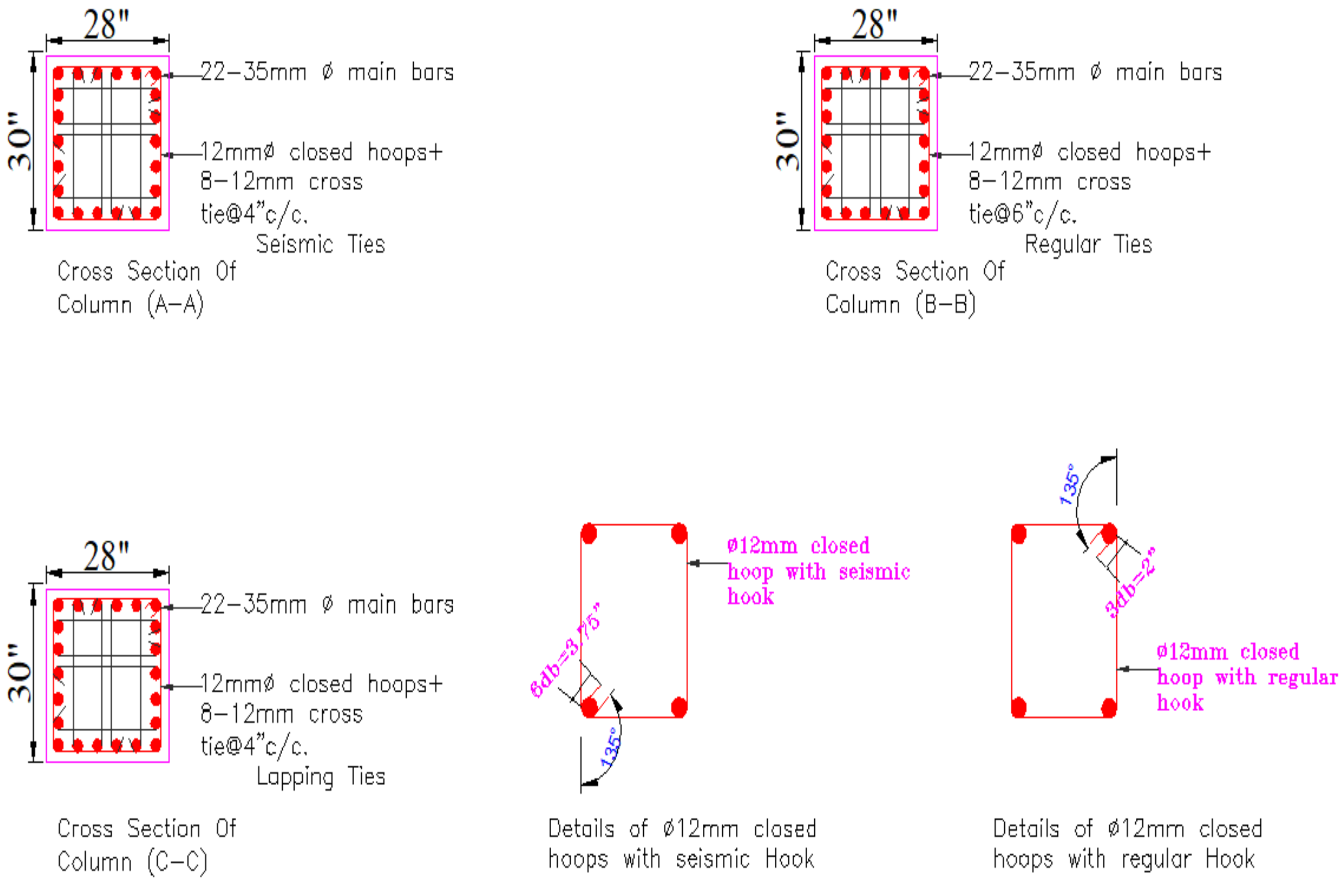


Figure 3.22b: Details of Cross Sections, Ties and Closed Hoops

3.4.4 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 3.23.

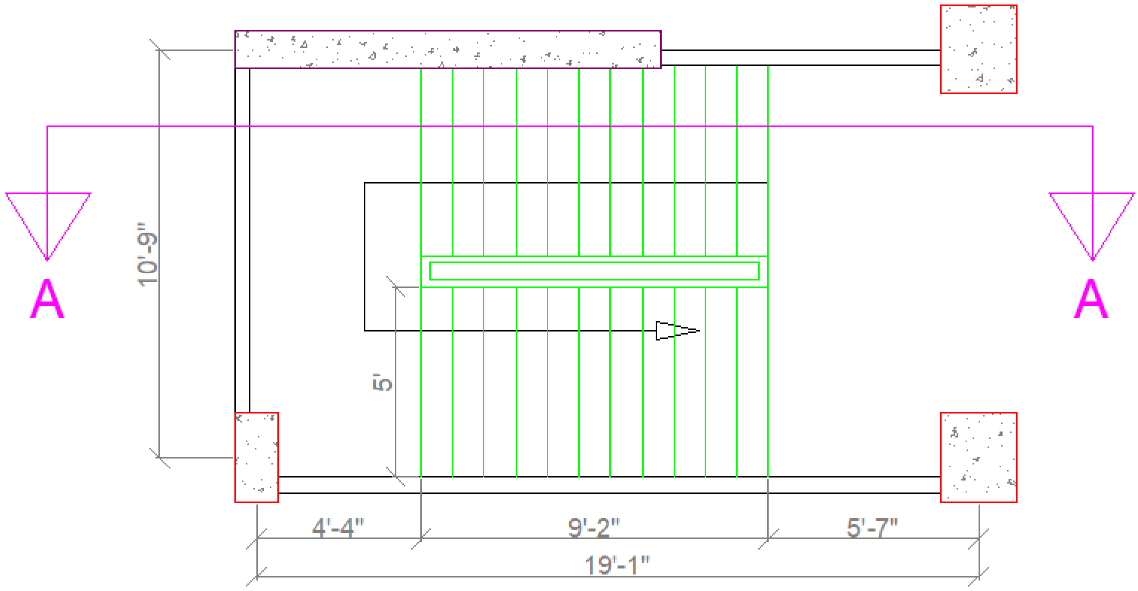


Figure 3.23a: Stair Plan

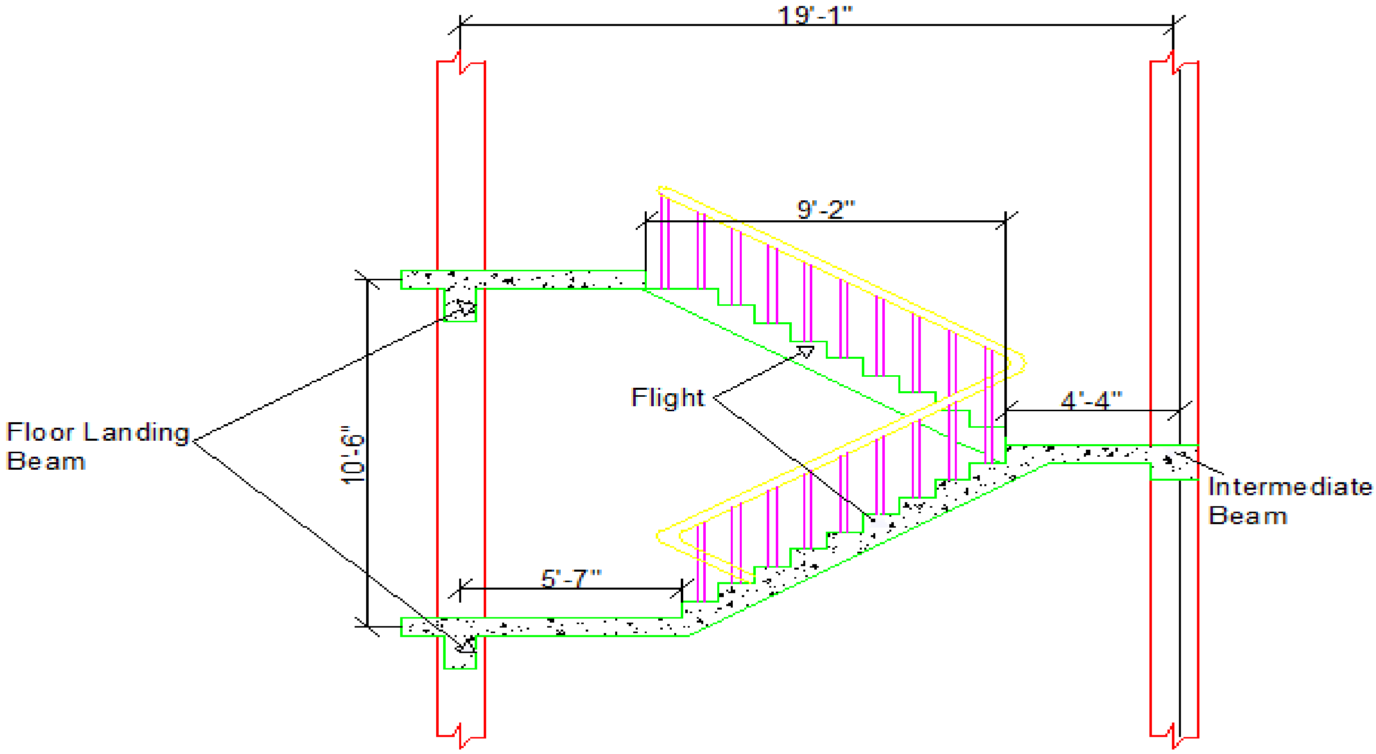
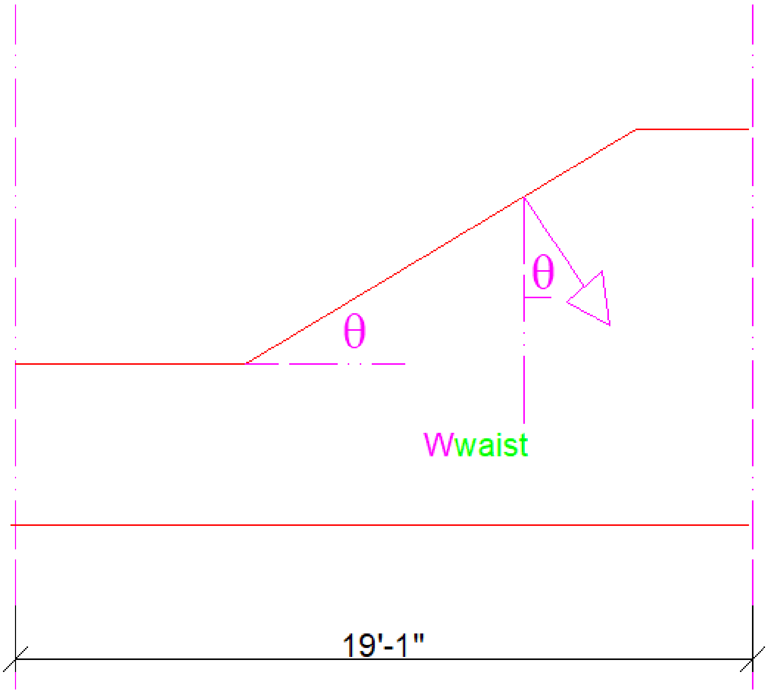


Figure 3.23b: Stair Elevation

Design Data:

Stair size	= 19'-1" x 10'-9"
Assume all beam width	= 16.0"
Story height	= 10'
Height of each flight	= 5'
Width of each flight	= 5'-0"
Tread (T)	=12"
Rise (R)	= 6"
No. of Rise	$= \frac{Ht.\ of\ \ flight}{ht.\ of\ \ rise} = \frac{5*12}{6} = 10\ nos.$
No. of tread	= No. of Rise -1 =10-1 = 9 nos.



Analysis and Design of Stairs presented below:

Effective span length = 19'-1''

Assume, waist slab thickness = 6"

Effective depth, d = 6-1 =5"

Here,

$W_{waist} = R*\cos\theta$

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

I) Load calculation:

- Self-weight of waist slab $= \frac{6}{12} * 150 * \frac{1}{\cos\theta}$ $\quad$ [$\cos\theta = \frac{\mathbf{9.167}}{\sqrt{\mathbf{5^2+9.167^2}}} = 0.878$

  $= \frac{6}{12} \times 150 \times \frac{1}{0.878}$ $\quad$ = 85.82 *lb/ft*

- Self-weight of steps $= \frac{\frac{1}{2}\times\frac{12}{12}\times\frac{6}{12}\times 10.44\times 150}{9}$ $\quad$ = 43.51 *lb/ft*

- Floor finish = 25.0 *lb/ft*

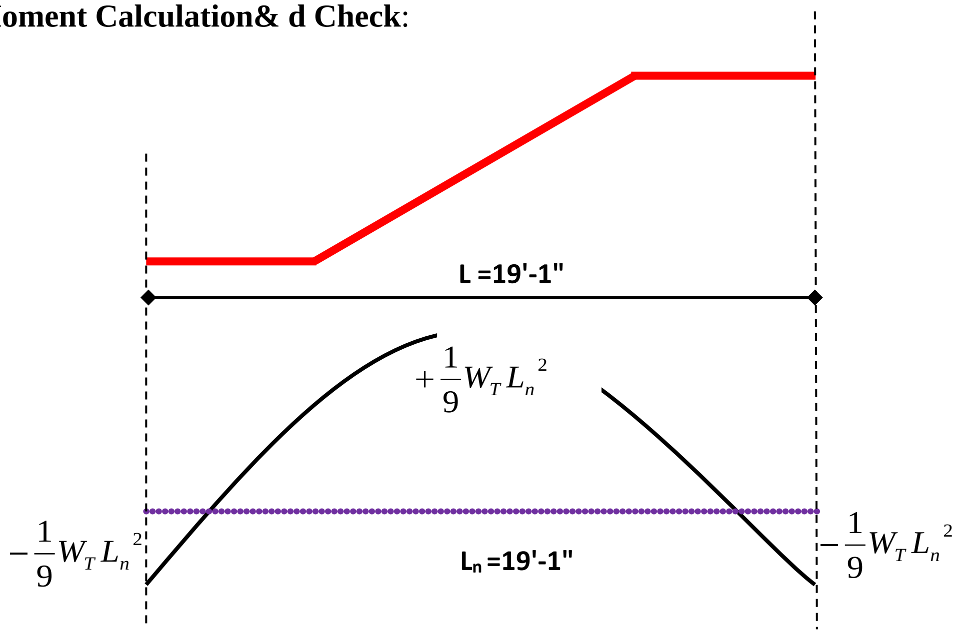
- Live load = 100 *lb/ft*

Total unfactored dead load = 85.82+43.51+25 = 154.33 *lb/ft*

Total unfactored live load = 100 *lb/ft*

Total factored load, W_t = 1.2×154.33 +1.6×100 = 345.196 *lb/ft*

II) Moment Calculation& d Check:



-ve moment at both supports $= \frac{1}{9}W_T l_n{}^2 = \frac{345.196 \times 19.167^2}{9}$ = 14090.67 *lb-ft* = 14.09 *k-ft*

+ve moment at mid span $= \frac{1}{9}W_T l_n{}^2 = \frac{345.196 \times 19.167^2}{9}$ = 14090.67 *lb-ft* = 14.09 *k-ft*

Effective depth will be checked for maximum moment which is 14.09 *k-ft*.

$\rho_b \ = 0.85\beta_1 \frac{f'_c}{f_y} \times \frac{87000}{87000+f_y} = 0.85 \times 0.85 \times \frac{4000}{60000} \times \frac{87000}{87000+60000}$ = 0.028506

$\rho_{\max} \ = \ 0.75\rho_b \ = \ 0.75 \times 0.028506 \ = \ 0.021380$

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

$$=> 14.09 * 12 = 0.90 \times 0.021380 \times 12 \times d^2 \times 60 \times \left(1 - 0.59 \times 0.021380 \times \frac{60}{4}\right)$$

So, $d = 3.88" < 5"\ ok$.

<u>III) A_s calculation:</u>

Main Steel:

<u>+Ve steel for mid span</u>

M_u= 14.09 *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,

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

$$= \frac{0.85 \times 4}{60}\left[1 - \sqrt{1 - \frac{2 \times 14.09}{0.85 \times 0.90 \times 4 \times 12 \times 5^2}}\right]12 \times 5 = 0.698 in^2/ft > 0.13 in^2/ft$$

So provided steel area, A_s = *0.698 in²/ft.*

- Maximum spacing, S_{max} = Smaller of 3h or 18″= Smaller of 3x6″ or 18″= 18
- Spacing $= \frac{Ab\ x12}{As} = \frac{0.20*12}{0.698}$ = 3.44" c/c = 3..25" c/c

Use *Ø12mm* bar @ 3.25" c/c,

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

Moment for mid span and both supports are same as ***14.09 k-ft***. Reinforcement requirement and spacing of the main bars at both supports will also be same as mid span which is *Ø12mm* bar @ 3.25" c/c.

So, use 1*Ø12mm* extra top in between two bars in order to maintain the spacing 3.25" 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 3.24.

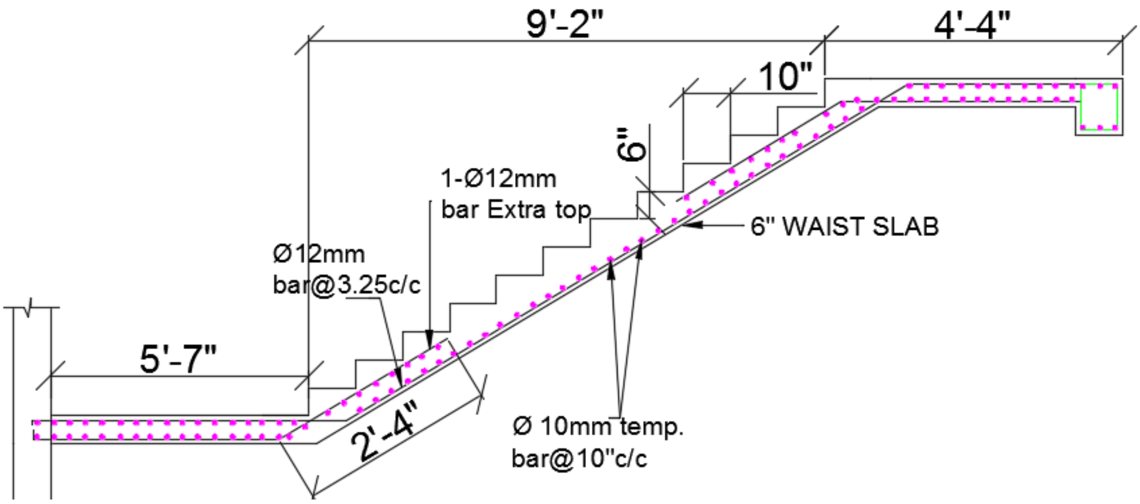


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Figure 3.24(a): Reinforcement Details of Stair (1st Flight)

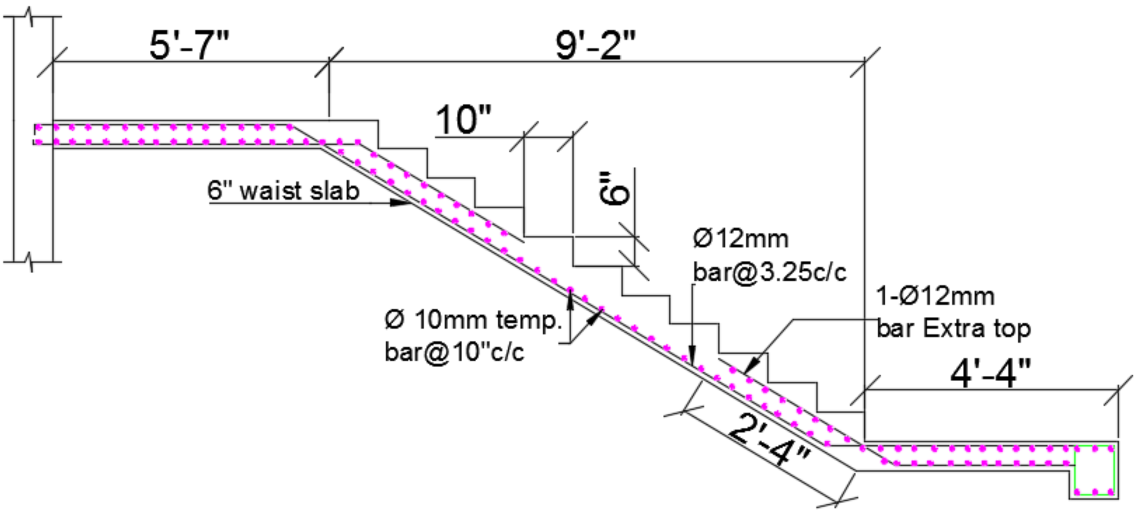


Figure 3.24(b): Reinforcement Details of Stair (2nd Flight)

Detailing of Lift Core (Shear Wall)

This gives details of lift core design for lateral loadings. There is one lift core in this building. Lift core was analyzed, designed and detailed by ETABS 2017 Software. Allocation of lift core, detailing of lift core of ground floor level and roof level is presented in Figures 3.25 ~ 3.27.

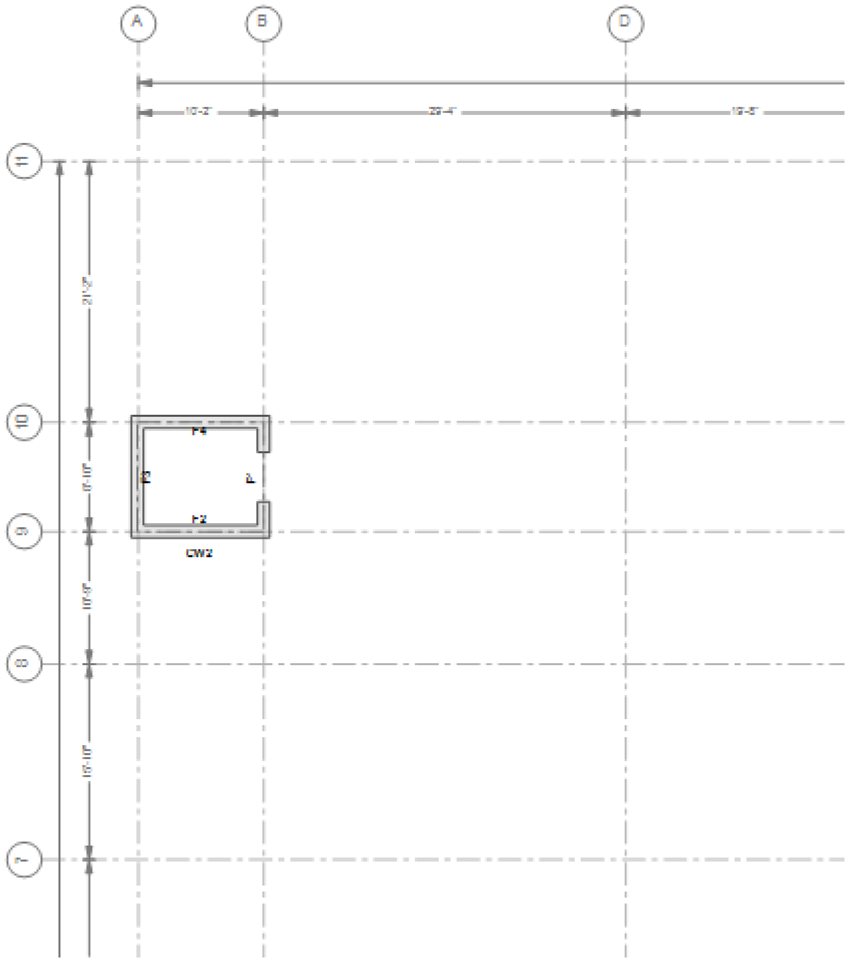


Figure 3.25: Wall Layout Plan

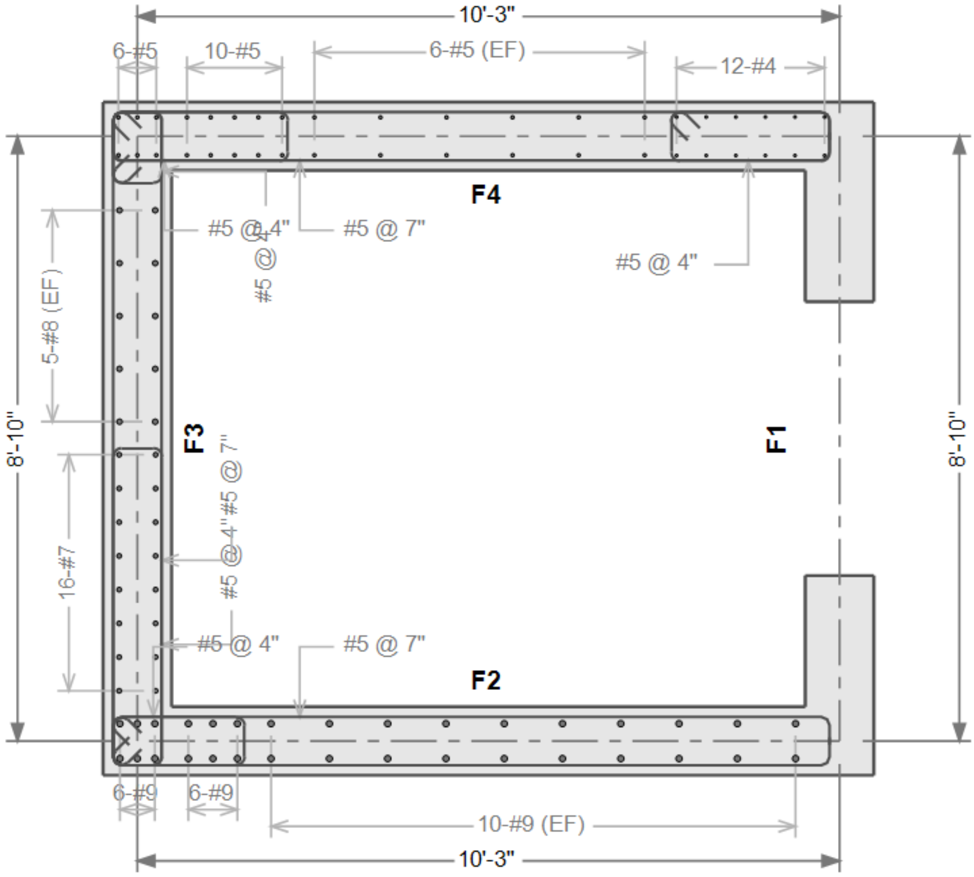


Figure 3.26: Lift Core Section (At GF Floor Level)

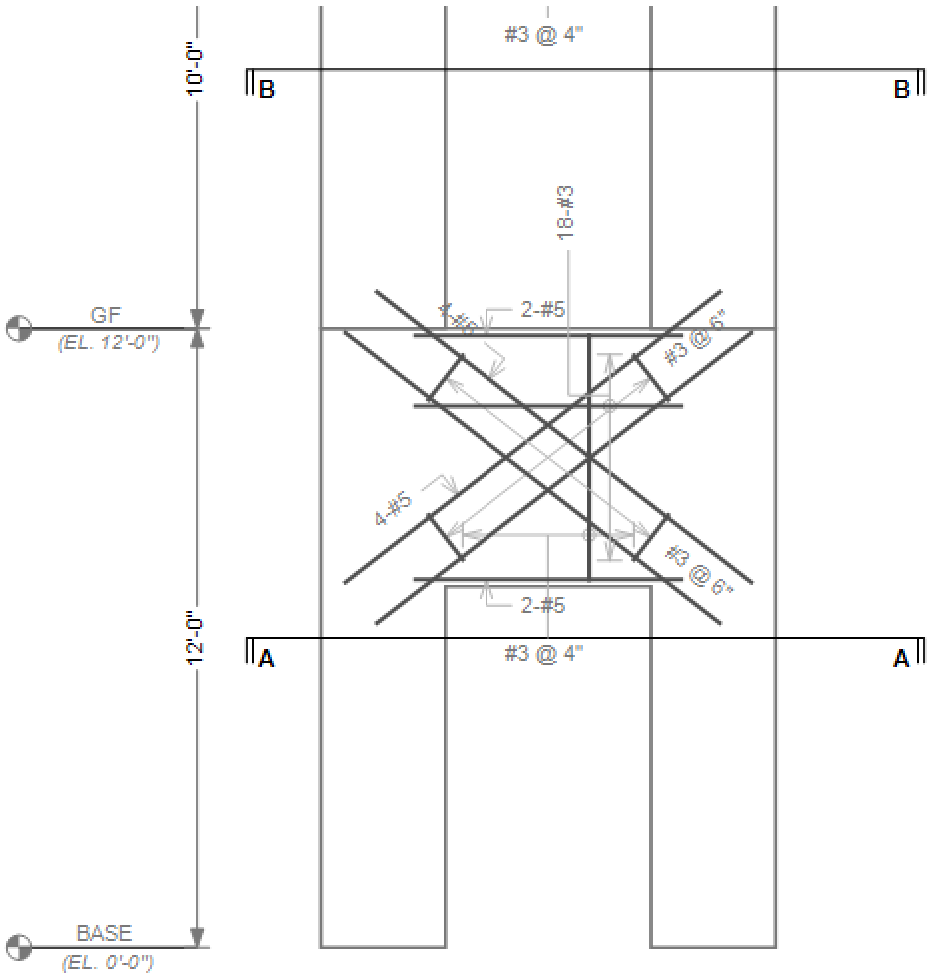


Figure 3.27: Reinforcement arrangement of Spandrel (At GF Floor Level)

<u>Detailing of Overhead Water Tank</u>

There are three overhead water tanks constructed. For space limitations, design of OWT, which located within grid 1-2, frame B-C is presented here.

No of OHWT	= 2
Length	= 19'-7″
Width	= 19'-1″
Tank height	= 5.5'
Free board	= 0.5'
Total tank height	= 5.5+0.5 = 6'
Select tank size	= 19'-7" × 19'-1" × 6'

Over Head Water Tank are analyzed, designed and detailed by ETABS and SAP software. The layout of OWT are shown in Figure 4.29. Details of reinforcement arrangement of OWT are shown in Figures 3.28-3.31.

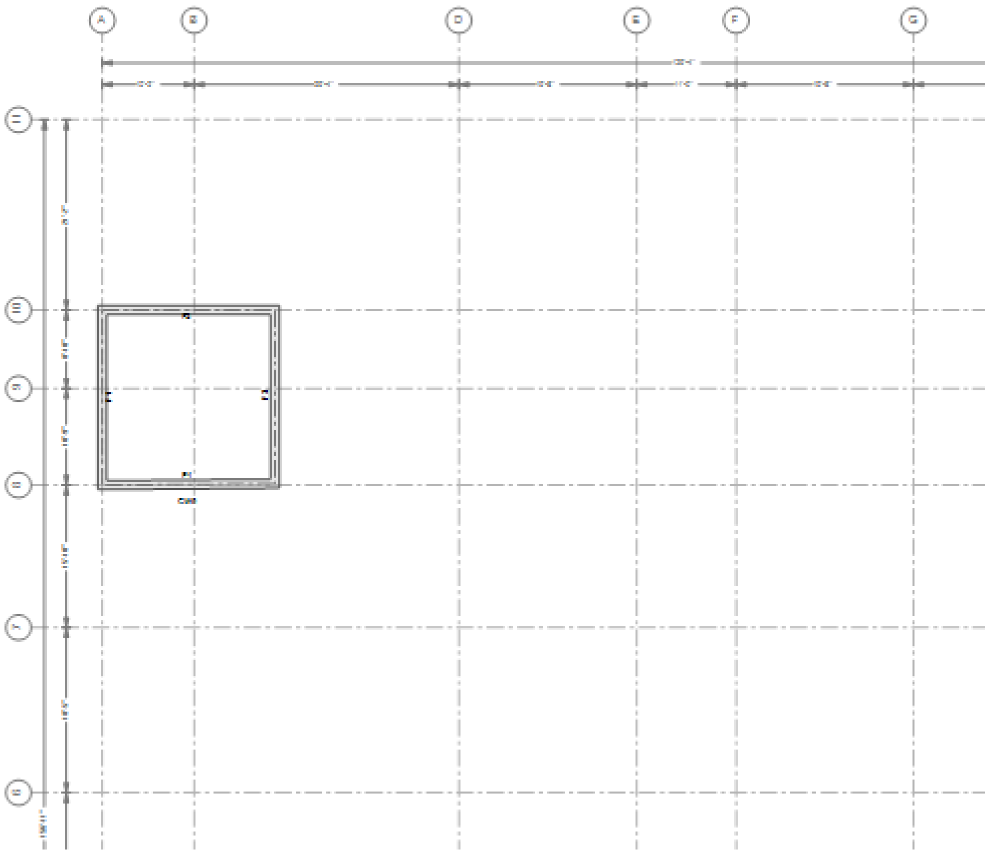


Figure 3.28: Overhead Water Tank Layout Plan

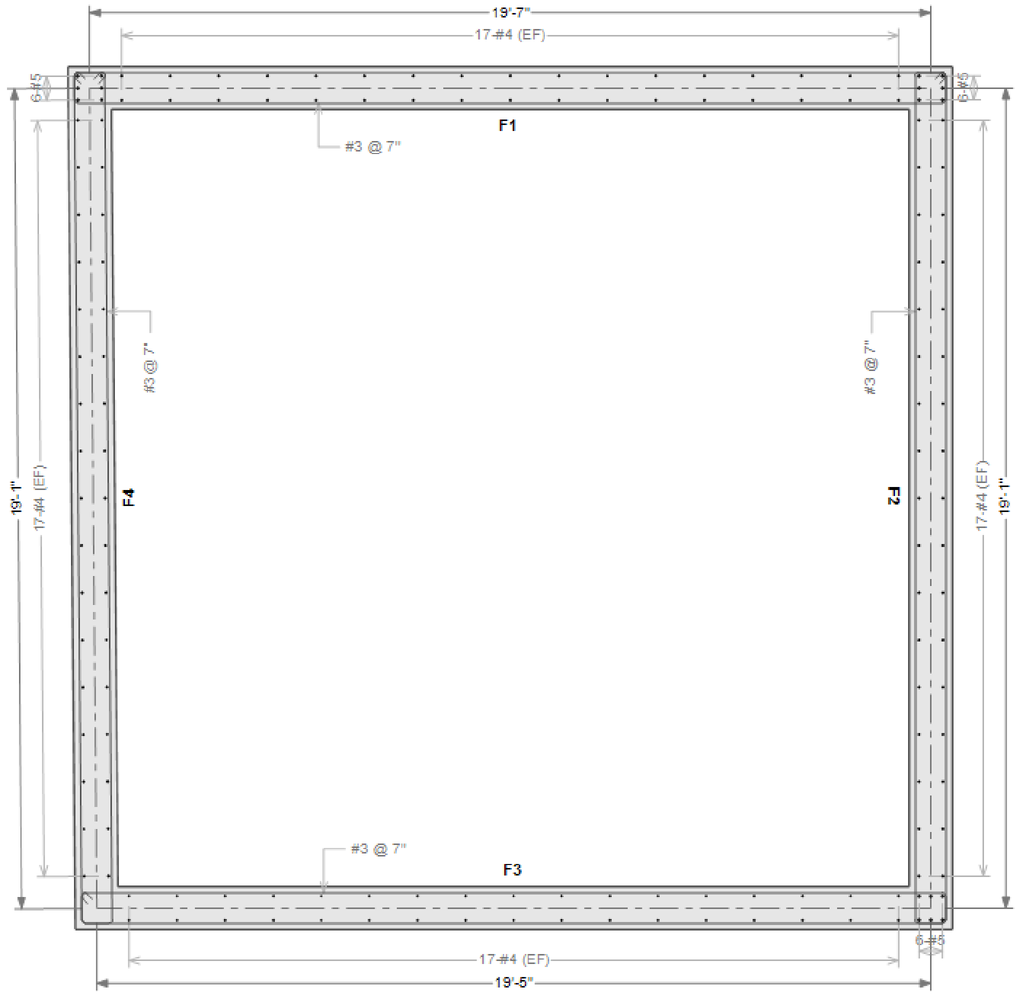


Figure 3.29: Reinforcement details of Over Head Water Tank wall cross section.

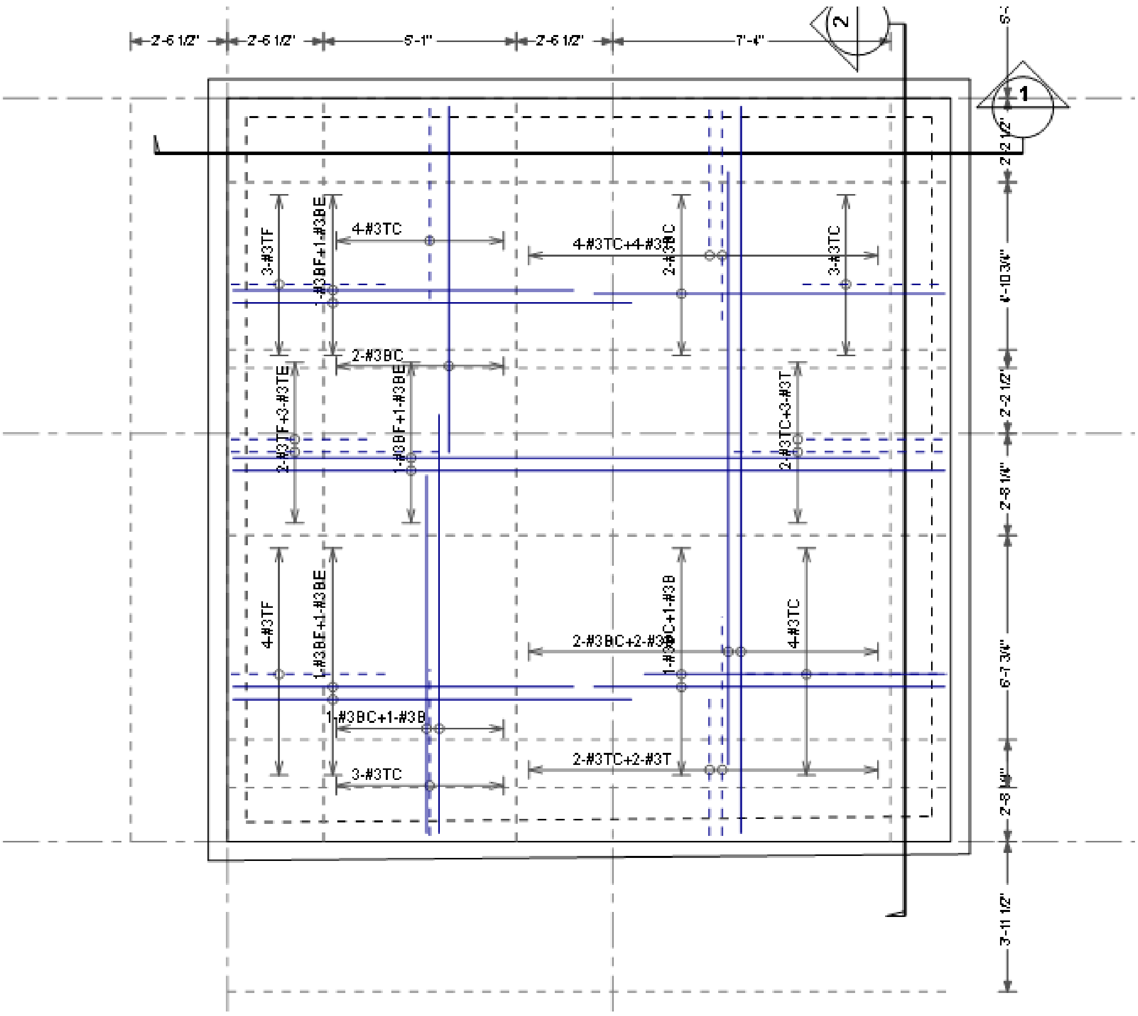


Figure 3.30: Slab detailing OHWT (Top)

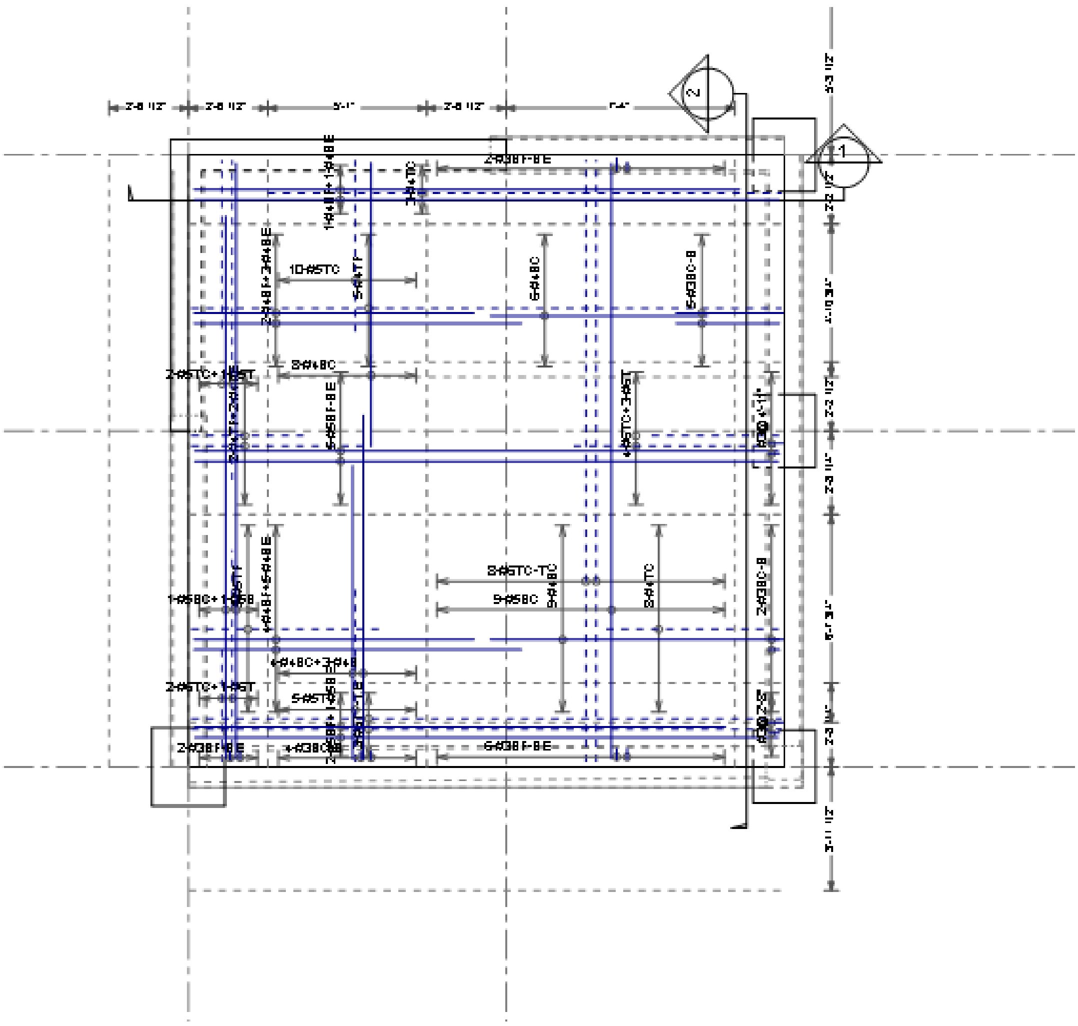


Figure 3.31: Slab detailing OWT (Bottom)

Detailing of Underground Water Reservoir

There are two underground water reservoirs (35' x 20'each) below basement floor surrounded by mat for fulfilling the water demand of the Building. 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

*** Residential Building purposes**

Water requirement	= 225 *L/P/D*
No. of floor	= 6
No. of Persons	= 250 (Assume)
Total req. vol. of water (Res.)	= 225×250= 56000 *L/P/D*

*** Commercial & Official Building purposes**

Water requirement = 45 *L/P/D*

No. of floor = 12

No. of Persons = 2600 (Assume)

Total req. vol. of water (Com) = 45×2600= 117000 *L/P/D*

So, Total required volume of water = 56000+117000 = 173000 *L/P/D*

* **Fire safety purposes**

Consider, water storage for 1 hr. fighting,

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

= 250 × 60 *gal/hr.* = 15000 *gal/hr.*

∴ Water requirement for two floor = 2 × 15000 *gal*

= 30000 *gal*

= 113562.36 *L*= 113563 L

Total water requirement for one-day storage = (113563 + 173000)

= 286563 *L*

There are two underground water tanks,

∴ Water requirement for tank = $\frac{286563}{2}$ = 143282 L=143.28 m^3

= 143.28× 3.28^3 ft^3

= 5056 ft^3

<u>Tank dimension</u>

Let,

Inside width dimension, B = 20 *ft* = 6.09m

Height = 7.5 *ft*

Free board = 0.5 *ft*

Final height = 7.5+0.5 = 8 *ft = 2.44m*

So inside length dimension, $L = \frac{5056}{20 \times 7.5}$ = 33.71*ft* = 35*ft*

Hence the dimension of the tank compartment will be 20 ft wide and 35 ft long.

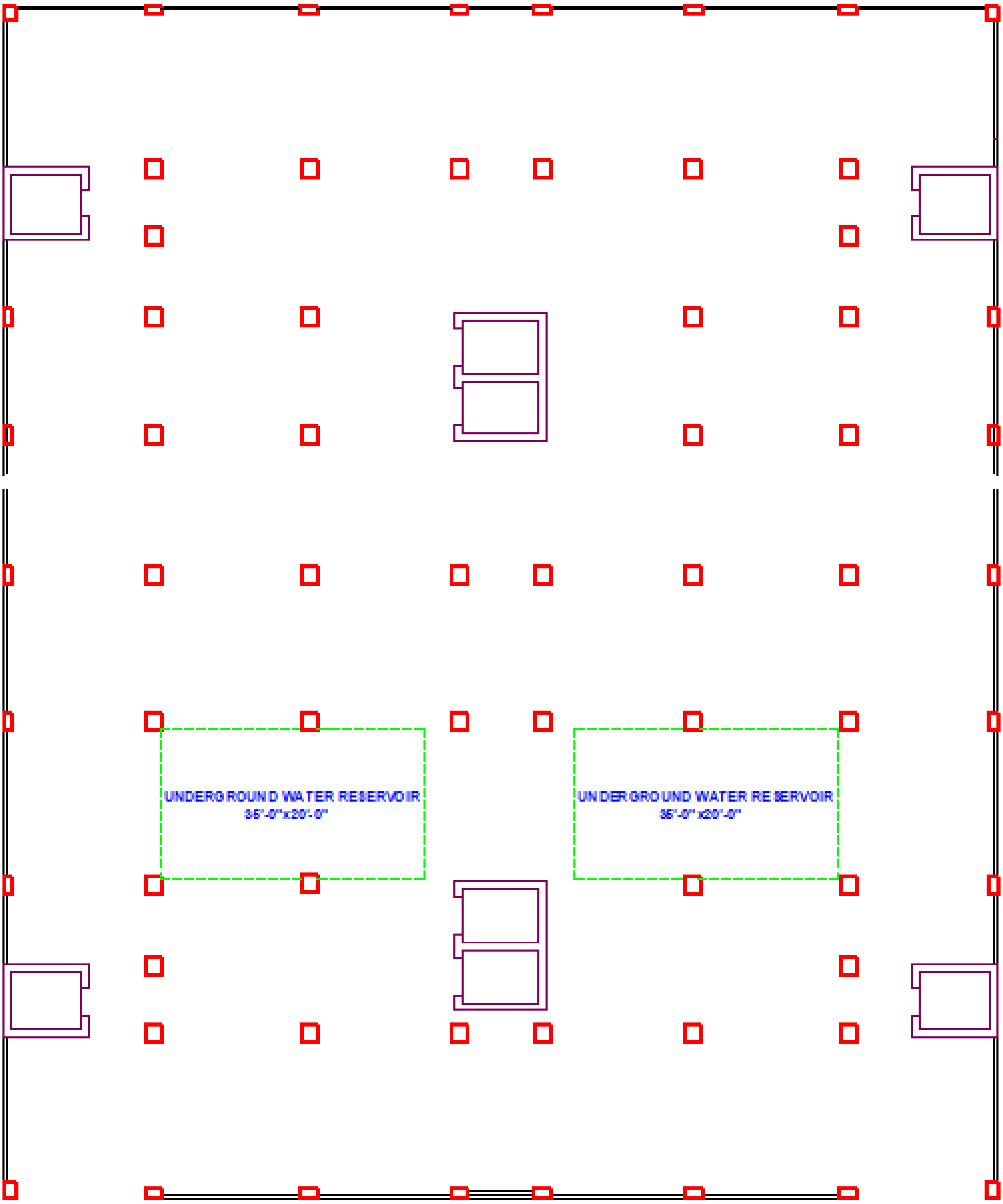


Figure 3.32: Underground water reservoir

A. *Design of long walls*

Both long walls will be designed considering empty condition.

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

$$=20 \times 2.44 \times \frac{1-\sin30}{1+\sin30} = 16.27kN/m^2$$

$$\therefore p = 16.27kN/m^2$$

Thickness of the wall:

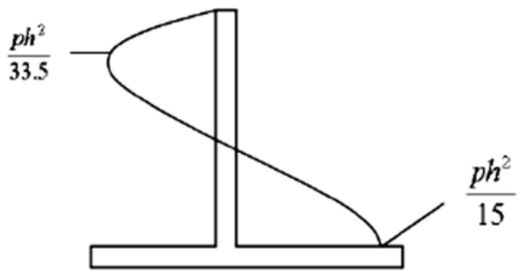
Moment at outer face of long wall

$$=\frac{ph^2}{33.5} = \frac{16.27\times2.44^2}{33.5} = 2.89kN\text{-}m/m$$

$$= \frac{2.89 \times 1000}{4.448 \times 0.3048} \times \frac{12}{1000}$$

$$= 25.57k - in$$

(Per meter run)



So, tension per feet run= 25.57 ×0.3048 = 7.80 *k-in/ft*

Moment at inner face of long wall,

$$M_{\max} = \frac{ph^2}{15} = \frac{16.27x2.44^2}{15} = 6.46\ kN - m/m$$

= 57.17 *k-in* (per meter run) = 17.43 *k-in* (per ft run)

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

D = total thickness of tank wall,

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

$$\therefore\ D^2 = \frac{6x17.43}{0.410x12}$$

$D = 4.61" \equiv 10.0"$ $(preferable$ min$imum\ \ thickness)$

[Here $f_{ct} = (6 \rightarrow 8)\sqrt{f_{c'}}$

Let, $f_{ct} = 7.5\sqrt{f_{c'}} = 7.5 \times \sqrt{3000} = 410.79psi$]

$\therefore$ 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 = 17.43 *k- in* (per ft run)

Steel requirement, $A_S = \frac{M}{f_S jd} = \frac{17.43*1000}{30000*0.904*8.5} = 0.076\ in^2/ft$

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

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

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

$S_{max} = 3h = 3\times10'' = 30''$

$S_{max} = 18''$

So, use ϕ12mm @ 6.5"c/c.

Vertical Reinforcement for outer face of wall

M = 7.80 *kip-in* (per ft run).

$A_S = \frac{M}{f_s jd} = \frac{7.80*1000}{30000*0.904*8.5} = 0.034\ in^2/ft$

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

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 =16.27 kN/m^2

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

$$L = 20 + \frac{10}{12} = 20.83\ ft = 6.35\ m$$

$$M = \frac{16.27x6.35^2}{12} = 54.67\ \ k - in/meter = 16.67\ k - in/ft$$

Now check '*d*',

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

$$\therefore d = \sqrt{\frac{2x16.67}{1.35x0.288x0.904x12}} = 2.81" < provided\ d = 8.5"\ ok.$$

Vertical reinforcement:

M = 16.67 *kip-in* (per ft run).

$A_S = \frac{M}{f_s jd} = \frac{16.67*1000}{30000*0.904*8.5} = 0.072\ in^2/ft$

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

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

$\frac{L}{B} = \frac{35}{20} = 1.75 \leq 2$

So, it is a two- way slab.

Minimum thickness of the slab,

$h = \frac{2(20+35)}{180} x12 = 7.33"$ 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:

$\frac{a}{b}$ = 20/35= 0.57

Case 1

$M_{a,neg} = 0$

$M_{b,neg} = 0$

For positive moment, $C_a dl = 0.084$

$C_b dl = 0.009$

$M_{a.positive} = 0.084 \times 0.195 \times 20^2 = 78.62$ *k-in*

$M_{b.positive} = 0.009 \times 0.195 \times 35^2 = 25.80$ *k-in*

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

$$\therefore d = \sqrt{\frac{2x78.62}{1.35x0.288x0.904x12}} = 6.11" < provided\, d = 10 - 1 = 9" \;\; ok.$$

Reinforcement calculation:

- **Main steel (short direction)**

M = 78.62 *kip-in* (per ft. run).

$$A_S = \frac{M}{f_s jd} = \frac{78.62*1000}{30000*0.904*9} = 0.32\, in^2/ft$$

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

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 = 25.80 *kip-in* (per ft. run).

$$A_S = \frac{M}{f_s jd} = \frac{25.80*1000}{30000*0.904*9} = 0.105\ in^2/ft$$

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

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:

$\frac{L}{B} = \frac{35}{20} = 1.75 \leq 2$

So, it is a two- way slab.

Let the thickness of the slab is 20″.

Load calculation:

Water pressure	= 62.5×7.5	= 468.75 *psf.*
Self-weight of the slab	= $\frac{20}{12}$×150	= 250 *psf.*
Floor finish & LL (assume)		= 20.0 *psf.*

Total load, W = 738.75 *psf.* = 0.738 *ksf*

Check for depth *d*:

$\frac{a}{b}$ = 20/35= 0.57

Case 1

$M_{a,neg} = 0$

$M_{b,neg} = 0$

For positive moment, $C_a dl = 0.084$

$C_b dl = 0.009$

$M_{a.positive} = 0.084 \times 0.738 \times 20^2 = 297.56$ *k-in*

$M_{b.positive} = 0.009 \times 0.738 \times 35^2 = 97.64$ *k-in*

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

$$\therefore d = \sqrt{\frac{2x297.56}{1.35x0.288x0.904x12}} = 11.88" < provided\ d = 20 - 1 = 19"\ ok.$$

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, $P_u = \gamma h \times B \times L$

  = (62.5×8) × (20+1.67) × (35+1.67) = 397.319 *k*

- Weight of 10″ thick long and short walls

  = 0.83 (2×21.67+2×36.67) × 8 ×150 = 116.213*k*

  Weight of 10″ top slab = 0.83×21.67×36.67 ×150 = 98.932 *k*

  Weight of 20″ base slab = 1.67 ×21.67×41.67×150 = 199.06 *k*

  Total downward weight = 414.205 *k*

This is greater than floatation force 397.319 *k*, so reservoir is safe in design.

Reinforcement calculation:

- **Main steel (short direction)**

M = 297.56 *kip-in* (per ft. run).

$A_S = \frac{M}{f_s jd} = \frac{297.56*1000}{30000*0.904*19} = 0.58\ in^2/ft$

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

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 = 97.64 *kip-in* (per ft. run).

$$A_S \;= \frac{M}{f_s jd} = \frac{97.64*1000}{30000*0.904*19} = 0.189\; in^2/ft$$

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

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 3.33~3.37.

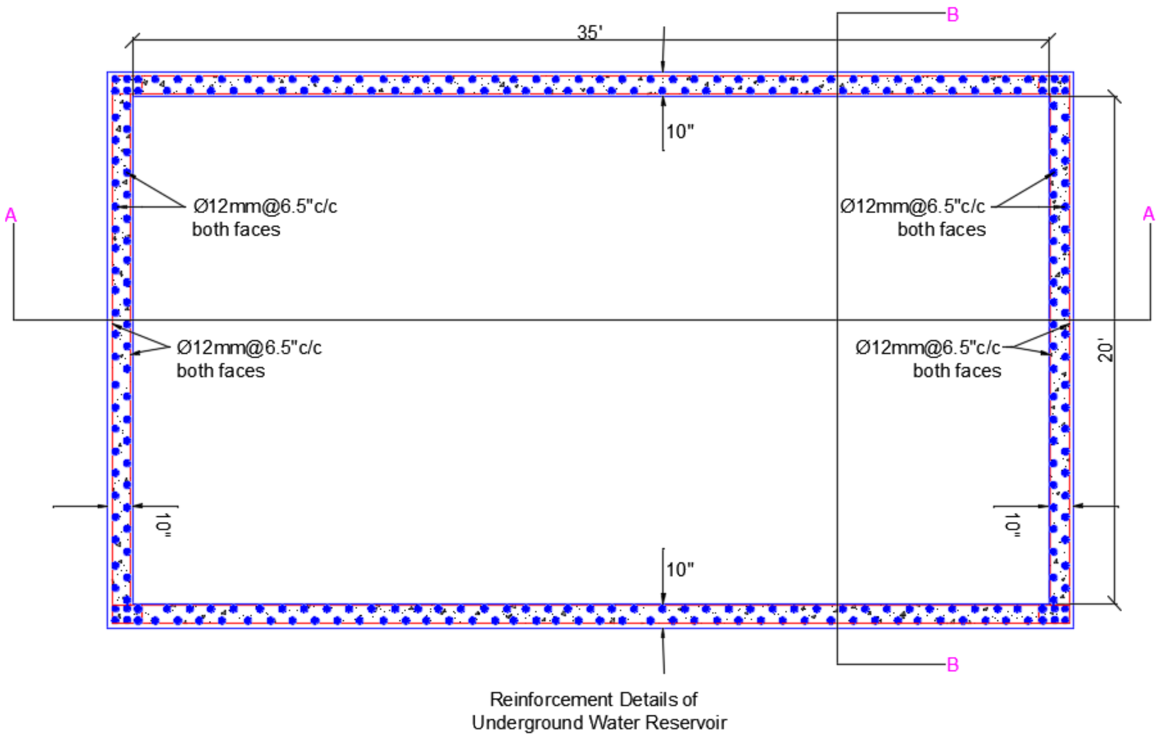


Figure 3.33: Details of reinforcement arrangement of Wall of the underground water reservoir.

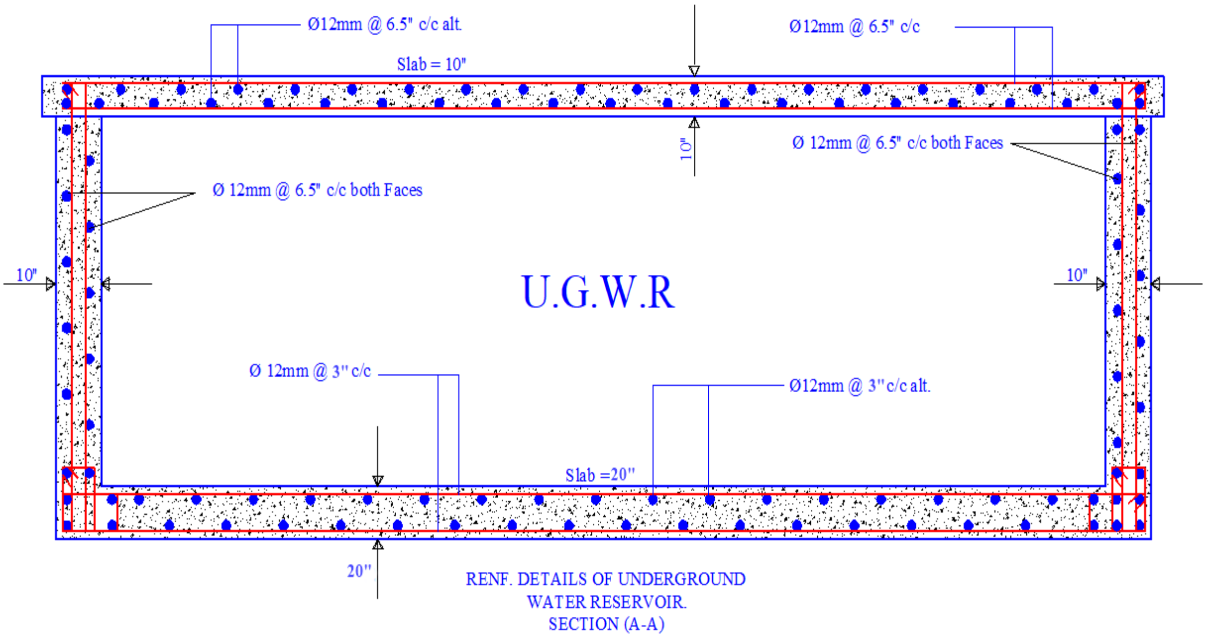


Figure 3.34: Details of section A-A of the underground water reservoir.

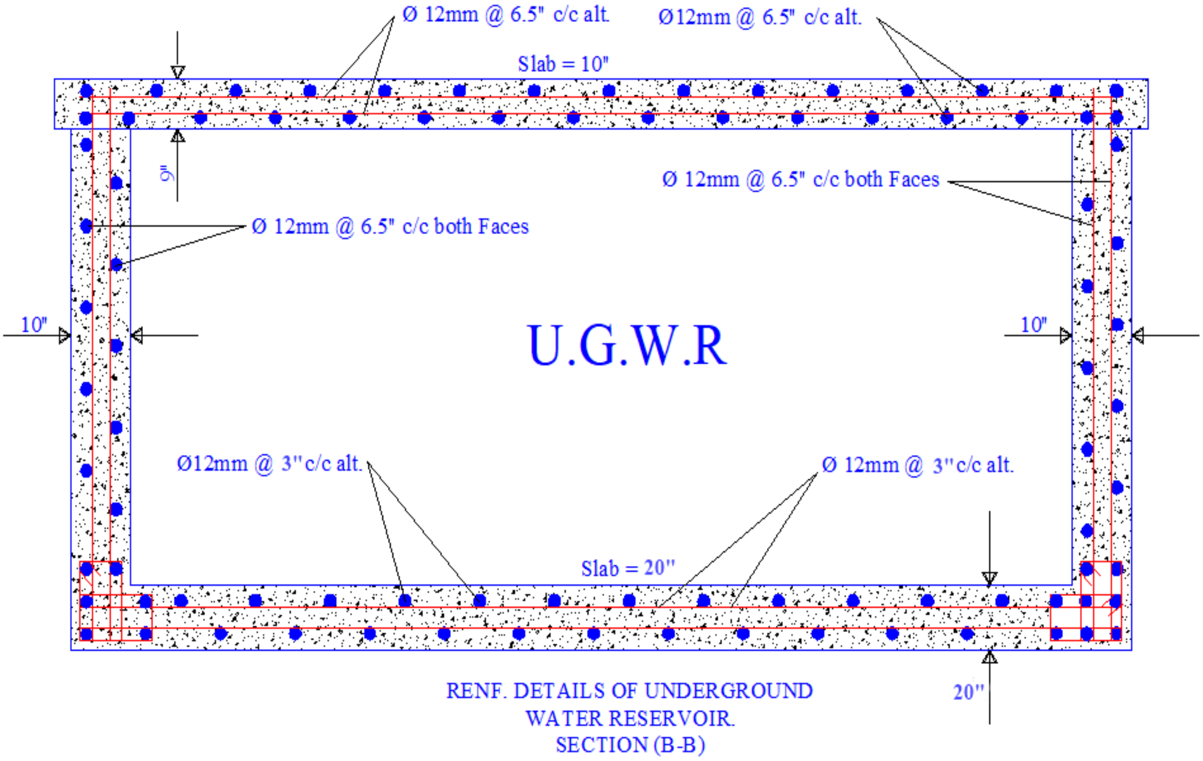


Figure 3.35: Details of section B-B of the underground water reservoir.

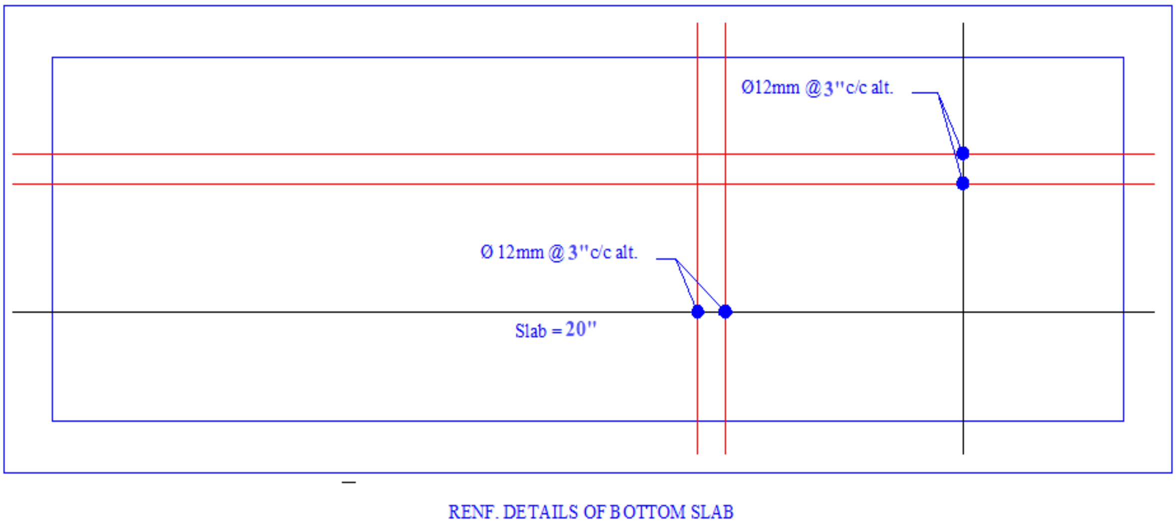


Figure 3.36: Reinforcement details of bottom slab of the underground water reservoir.

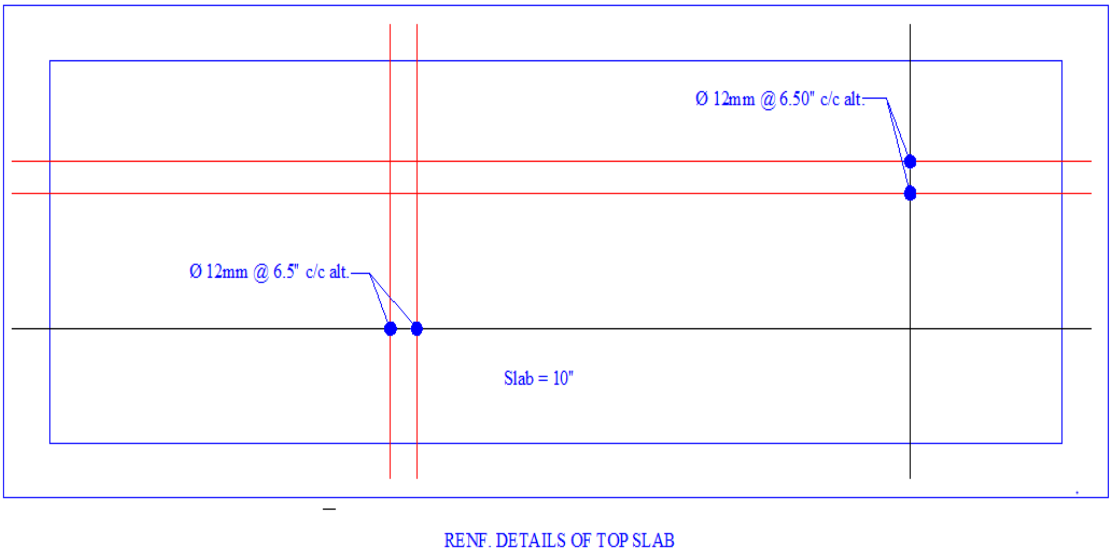


Figure 3.37: Reinforcement details of top slab of the underground water reservoir.

CHAPTER 4

CONCLUSIONS & RECOMMENDATIONS

4.1 Conclusions

From the study, it is observed that:

- Preparation of commercial Floor plans requires comparatively more efforts, attention and considerations according to the need of rentable spaces, owner desires, aesthetics, cost, safety and comfort of the visitors.
- Proper knowledge on software is essential for analysis of high rise structure.
- Selection of loadings, materials & their properties should meet the requirement of building codes properly**.**

4.2 Recommendations

Based on the objectives, scopes and limitations of the study (stated in *Chapter 1*), few *recommendations* can be proposed for further studies:

- Column-beam joints are the important design consideration in a high rise structure which were not done in this study is highly recommended in further studies.
- This study was conducted based on edge supported floor system, further analyses considering other floor system such as flat plate or flat slab can be considered to see the change in moment, shear, axial forces etc. in different building elements and also their cross-sectional dimensions as well reinforcement requirements.
- Sway and deflection control may be considered in analysis and design.

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