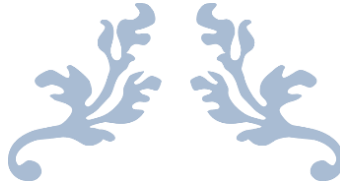




Prince Mohammad Bin Fahd University
College of Engineering
Department of Civil Engineering



**Design of 60 Storey RCC & CLT Connected Twin
Towers in Al-Khobar**

Course Title: Learning Outcome Assessment III (senior design project)

Course Number: ASSE 4311 Fall 2018/2019



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Abstract:

To catch up with the urban renaissance that has been happening in Saudi Arabia, and as an effective element of the vision of 2030, Saudi Arabia has built up a major economical, educational, engineering, and commercial projects that will help making Saudi Arabia all the more an advanced, stable, and more secure condition to be more pulled in by investors and tourists. In this manner, we pick the eastern area to set up our project, where the city has been receiving numerous conservative and designing activities over the most recent couple of years.

We choose King Fahd road because it's one of the most important roads in the eastern province as it connects Dammam and AL-Khobar together, and a second reason that lots of the big Saudi and foreign companies are in that area along with a couple of international hotels, which makes that area the perfect spot for our project.

Our design project is the design of a 60 storey Connected Twin Tower from the top and bottom using two different slab floor systems which are; Roller Compacted Concrete (RCC), and Cross Laminated Timber (CLT). The Twin Towers are 40 m apart and the total area of the whole building is $120\text{m} \times 60\text{m}$. The Twin Towers are 60 storeys each, one is a residential tower and the other one is the commercial tower. One of the main objectives of the senior design project is designing a unique 4 storey connection between the two towers. The bottom storey of the connection part has a glass slab pathway and that what makes the building a tourists' attraction place.

Chapter 1: introduction

1.1 General

In line with the great vision of the kingdom of Saudi Arabia (KSA), vision 2030 and along with its urbanization growth the eastern province is one of the largest business centers in (KSA). There will be lots of business opportunities for foreign investors to invest their money that will eventually improve the economy of KSA to be aligned with the vision; through high-rise building with multiple purposes such as offices, hotels, malls, cinema and more. Therefore, High-rise buildings are needed in the eastern province, where both Dammam and Al-Khobar cities are the biggest business cities in the eastern province. In addition, there is no better place to establish this huge project rather than this location which is on King Fahad highway; where all the businesses headquarters are located. The senior design project is a high-rise building consists of two towers connected from the top and bottom. One tower is residential and the other one is commercial with multiple purposes.

1.2 Project description:

The senior design project is Design of 60 Storey RCC &CLT Connected Twin Towers in AL-Khobar. The Twin Towers are 40 m apart. The Twin Towers are 60 storeys each, one is a residential tower and the other one is the commercial tower. The total area of the whole building is $120\text{m} \times 60\text{m}$. To be more detailed, the area of each tower is $40\text{m} \times 60\text{m}$ and the storey height is 4m. The top part area is $40\text{m} \times 60\text{m}$ as well. The main challenge is designing the hanging part between the two towers where the deflection limit is the dominated factor. The Twin Towers are to be designed twice using two different slab floor systems Roller Compacted Concrete (RCC) and Cross Laminated Timber (CLT). The hanging part between the two towers is 4 storey where the girders and sub beams are steel with (RCC) slab floor system.

The commercial tower has offices, hotels, malls, gym, restaurants, and other entertainment facilities. The location of the project is on King Fahd Dammam Al-Khobar highway.

1.3 CLT Background:

BRIEF HISTORY

Cross-laminated timber (CLT), a new generation of engineered wood product developed initially in Europe, has been gaining popularity in residential and non-residential applications in several countries. Numerous impressive low- and mid-rise buildings built around the world using CLT showcase the many advantages that this product can offer to the construction sector.

It is a potentially cost-competitive wood-based solution that complements the existing light frame and heavy timber options, and is a suitable candidate for some applications which currently use concrete, masonry and steel. CLT is an innovative wood product that was introduced in the early 1990s in Austria and Germany and has been gaining popularity in residential and non-residential applications in Europe.

After several slow years, construction in CLT increased significantly in the early 2000s, partially driven by the green building movement but also due to better efficiencies, product approvals, and improved marketing and distribution channels.

DEFINITION

CLT panels consist of several layers of structural lumber boards stacked crosswise (typically at 90 degrees) and glued together on their wide faces and, sometimes, on the narrow faces as well. In special configurations, consecutive layers may be placed in the same direction, giving a double layer to obtain specific structural capacities. CLT products are usually fabricated with three to seven layers and even more in some cases.

Introduction to CLT (Cross Laminated Timber):

CLT is an engineered wood panel typically consisting of three, five, or seven layers of dimension lumber oriented at right angles to one another and then glued to form structural panels with exceptional strength, dimensional stability, and rigidity. .

Because of CLT's structural properties and dimensional stability, this mass timber product is

well suited to floors, walls and roofs used in mid-rise construction. The wall and floor panels may be left exposed in the interior, which provides additional aesthetic attributes. The panels are used as prefabricated building components, which can speed up construction practices or allow for off-site construction.

CLT is manufactured to customized dimensions; panel sizes vary by manufacturer. While length is usually limited by transportation restrictions, longer panels can be manufactured. CLT panels are typically installed like plywood in terms of grain orientation. Wall panels are oriented with the grain of the outside layers' parallel to the vertical loads of the application. Floor and roof applications have the grain of their exterior layers oriented parallel to the span direction.

Openings within panels can be pre-cut in the factory to any dimension and shape, including openings for doors, windows, stairs, service channels and ducts. The resulting product is load bearing, stable and can act as a diaphragm or shear wall. (19 October 2015 by [Sofianna Teriaki](#))

Planks in alternating directions

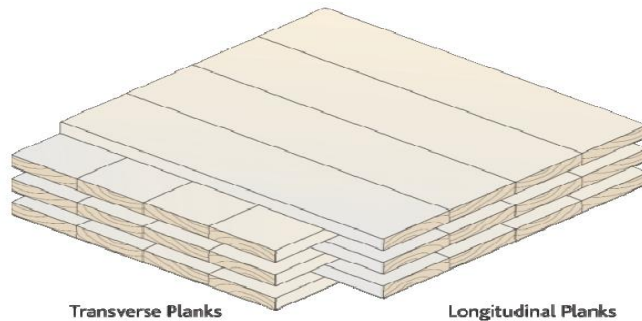


Fig. 1.3.1: Scott Breneman, PhD, PE, SE

1.4 Project objectives:

One of the main objectives of the senior design project is designing a unique 4 storey connection between the two towers. The bottom storey of the connection part has a glass slab pathway and that what makes the building a tourists' attraction place. The other objectives, the building is to be designed twice using two different slab floor systems (RCC) & (CLT). Afterward, the analysis of (RCC) & (CLT) slabs floor systems is to be performed to compare the advantages and disadvantages between both systems.

1.5 Project Methodology:

The project's methodology goes through several stages; these stages will be discussed in this part:

The first stage:

The original layout images of the structure are taken from a reliable source which is the Pinterest website. The original layout is for a building located in Makati Central Business District in the Philippines [Fig. 1]. The layout is redrawn with slight changes to serve our idea for the senior design project. Our layout done by duplicate the original layout after the slight changes [Fig. 2] so the twin towers and the hanging part can have one complete layout. The idea of doing that is designing a unique 4 storey connection between the two towers.

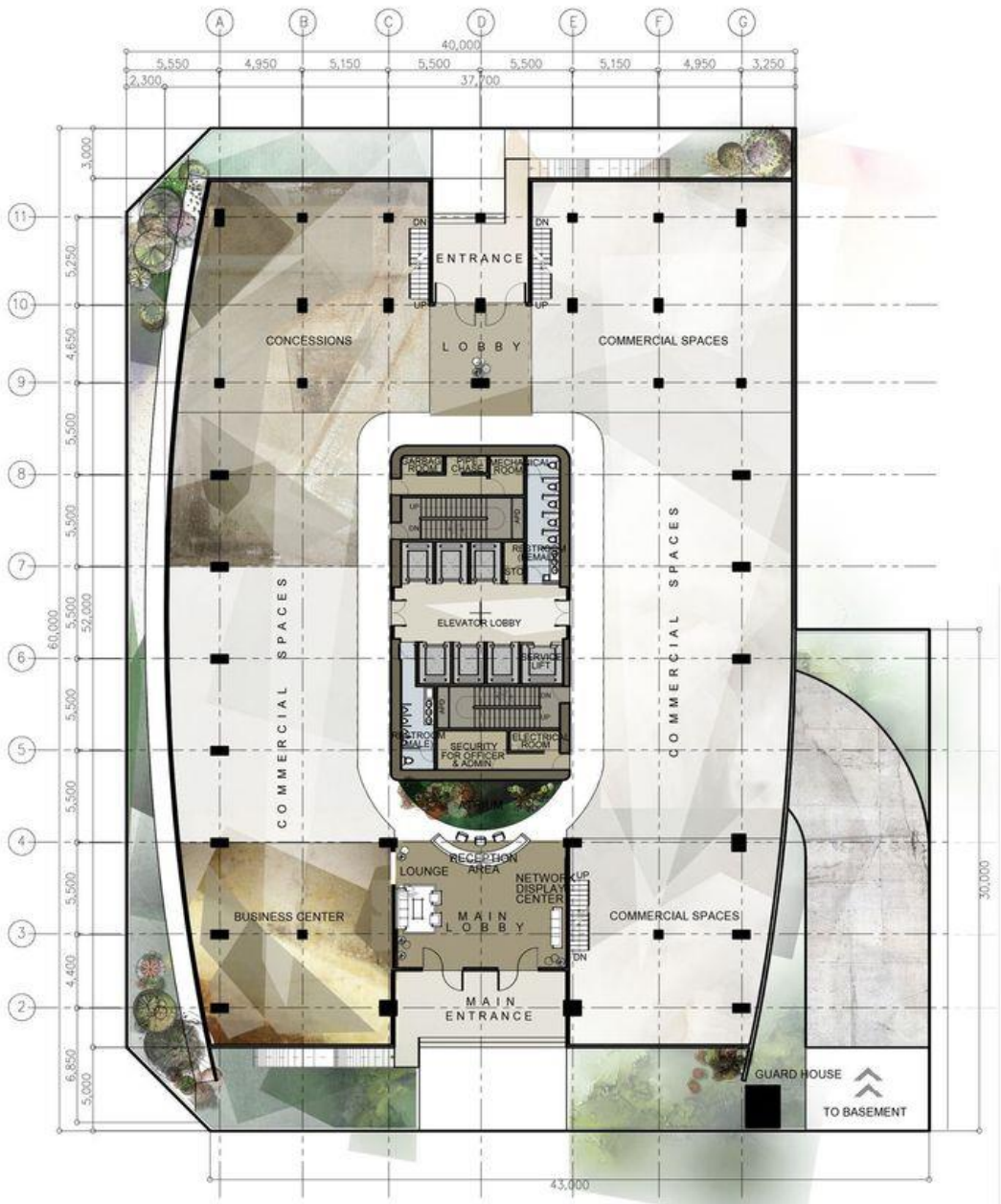


Fig. 1: Original Layout

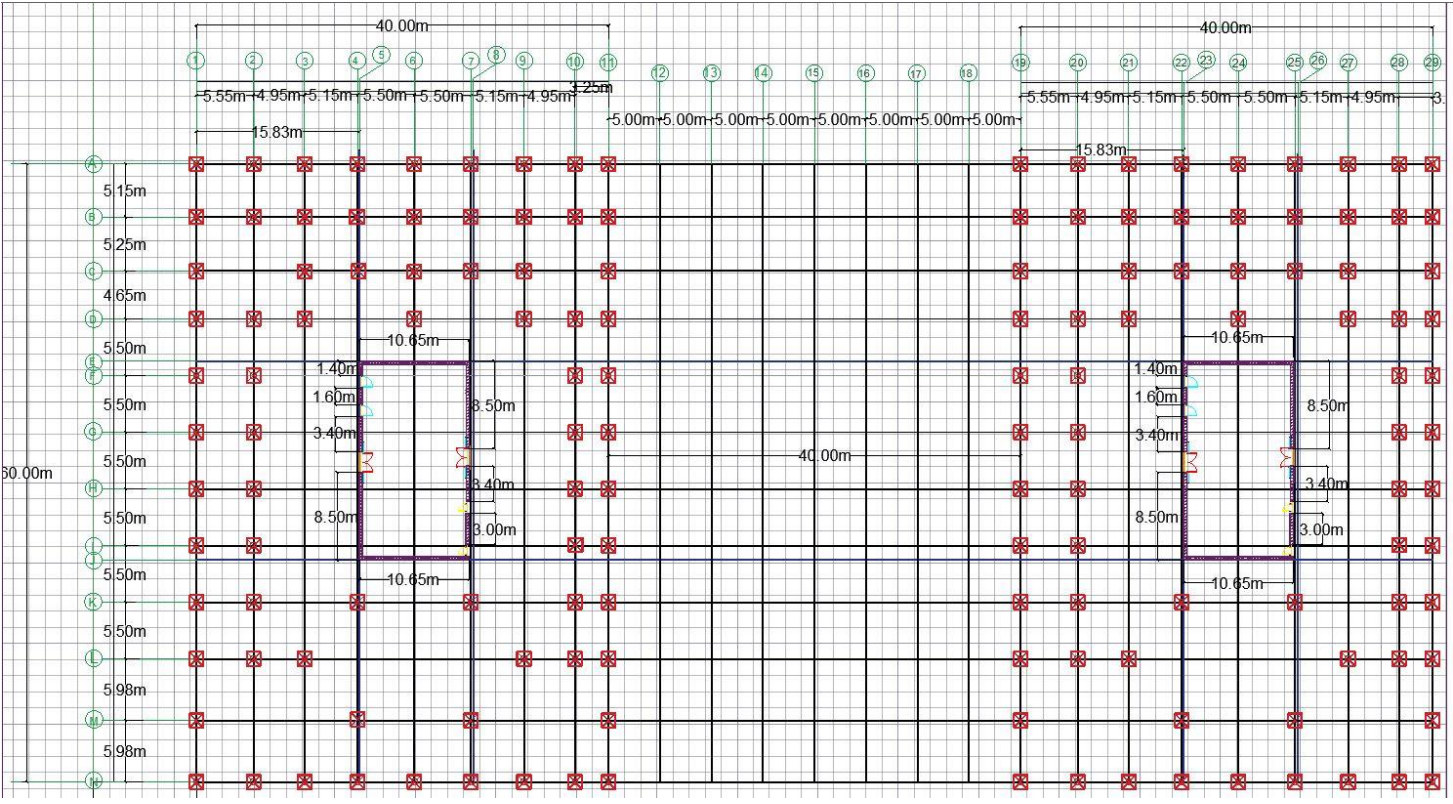


Fig. 2: Our Project Layout

The second stage:

As mentioned earlier that the whole building consists of twin towers one is commercial and the other is residential. The second stage is dividing the towers level, each tower has 60 levels and these levels are divided to serve multiple purposes. Based on the storey purpose; the live load and the dead load are obtained from the Saudi Building Code (SBC).

The following tables provide more details about the service that each storey give:

Residential tower			Connected top part			commercial tower		
Storey	Storey Name	purpose	Storey	Storey Name	purpose	Storey	Storey Name	purpose
43-60	38th Floor - 55th Floor	residential Suite, Super Deluxe Room	59-60	54th Floor - 55th Floor	Wedding hall	58-60	53th Floor - 55th Floor	VIP Suites
42	37th Floor	Female gym and swimming pool	58	53th Floor	Museum	57	52th Floor	Sky lobby
41	36th Floor	Male gym and swimming pool	57	52th Floor	Sky lobby & coffeshop	56	51th Floor	Emergency clinic
34-40	29th Floor - 35th Floor	Apartments	1-4	B1-B4	Parking	54-55	49th Floor - 50th Floor	Control rooms
32-33	27th Floor - 28th Floor	Emergency clinic & pharmacy				53	48th Floor	Female boutique
31	26th Floor	Mechanical layer				52	47th Floor	Male boutique
11-30	6th Floor - 25th Floor	Apartments				51	46th Floor	Mosque
10	5th Floor	Restaurants, & Coffee-shop				50	45th Floor	Library
8-9	3rd Floor - 4th Floor	Supermarket, Barber shop, laundry, services.				49	44th Floor	Female spa and sport center
7	2nd Floor	Mechanical layer				48	43th Floor	Male spa and sport
5-6	GF - 1st Floor	lobby & Management				47	42th Floor	Sky lobby
1-4	B1-B4	Parking				46	41th Floor	Mechanical layer
						36-45	31th Floor - 40th Floor	Hotel
						35	30th Floor	Mechanical layer
						34	29th Floor	Conference hall
						17-33	12th Floor - 28th Floor	Offices
						16	11th Floor	Mechanical layer
						13-15	8th Floor - 10th Floor	Mall (stores, restaurants, & Coffee-shops)
						11-12	6th Floor - 7th Floor	Entertainment
						8-10	3rd Floor - 5th Floor	Parking
						7	2nd Floor	Mechanical layer
						5-6	GF- 1sr Floor	lobby & Management
						1-4	B1-B4	Parking

The third stage:

The third stage is the hand calculation process. Firstly, the top part that connects the twin towers is to be designed. As known that the span between the two towers of the top part is 40m. The girders and sub-beams of the top part are going to be designed based on slab self-weight, finishing and live load. Then the whole structure is to be designed twice manually using two different slab floor systems which are RCC and CLT. Secondly, the twin tower design is to be designed twice using two different slab floor systems RCC & CLT. For RCC, the girders and joists are to be designed based on slab self-weight, finishing and live load. The columns design to consider the load that is transferred from the girders to columns as a point load. For CLT, the design process is the same as RCC design except CLT has the different unit weight which will produce much less weight on the structure. So, it is expected that the columns size will be less than in the RCC design which eventually will affect the foundation.

The fourth stage:

The fourth stage is modeling the whole structure on ETABS software. The analysis is to be run to compare the hand calculation with ETABS's calculation. In addition, the complicated forces such as wind load and seismic forces are to be calculated on ETABS as well as the required rebar for slab, beam, column, and the shear wall. The deflection limit and the structural drift is to be calculated using ETABS and compare them with our hand calculation.



Fig. 3: 3D View

The fifth stage:

The fifth stage is the geotechnical design of foundation system. RCC & CLT, each one of them will have different foundation dimensions. Concrete density is higher than timber density where the timber produces less loading on the structure. So, that is why the foundation will be designed twice.

The sixth stage:

The sixth stage is the project constraints and cost estimation. The project constraints as follow are:

1. Geotechnical design: Must know the type of soil that the structure sets on to design a proper foundation and not to exceed the settlement limit.
2. Building regulation and restrictions: Set of rules that specify the standards for designed and constructed buildings.
3. Environmental constraint: Air pollution and soil contamination.

1.6 The project scope:

All calculations, analysis, and design procedures are covered in five main chapters in this report as follows:

- **Chapter1:** General information, objectives, methodology, project description, scopes of the project and CLT background.
- **Chapter2:** Constraints and Design Codes.
- **Chapter3:** Preliminary design.
- **Chapter4:** steel reinforcement (girders, beams, column & concrete slab)
- **Chapter5:** Geotechnical design of the appropriate foundation system: geotechnical report & soil profile.
- **Chapter6:** used software.
- **Chapter7:** Comparison between RCC & CLT.
- **Apendix1:**
- **Apendix2:**
- **Apendix3:** shear & moment diagrams for CLT model strip1.
- **Apendix4:** shear & moment diagrams for CLT model strip2.
- **Apendix5:** shear & moment diagrams for RCC model strip1.
- **Apendix6:** shear & moment diagrams for RCC model strip2.
-

CHAPTER 2: Constraints and Design Codes

2.1 Introduction:

The main goal of Assessment III project is to Expose students and provide them with the opportunity to apply concepts, rules, methods and techniques learned in their undergraduate education to a real project. It is a comprehensive and meaningful effort, focusing on a major area of Civil Engineering and integrating design skills that students acquire in major areas of Civil Engineering in the curriculum.

There are many approaches to engineering design, choosing an adequate design develops creativity, use of modern design theory and methodology, formulation of design problem statements and specifications, and consideration of alternative solutions. Include a variety of realistic constrains such as serviceability, economic factors, health and safety, and environment considerations

The best designs are often a compromise of multiple competing criteria. Discussion of these design considerations is required in the project report.

2.2 Design codes:

Loads and regulations are taken from international codes: Saudi Building Code (SBC), American Concrete Institute (ACI), and American Institute for Steel Construction (AISC) to insure project safety and quality.

2.2.1 Structural Design

2.2.1.1 *Strength and Serviceability Requirements for Concrete Structures*

- As required from SBC 304, the structural members shall be designed with strengths that must be at least equal to the required strengths calculated for the factored loads and forces.
- Members should also meet the requirements of SBC 304, to ensure the adequate performance at service load levels.

2.2.2 Design of Members subject to flexure or axial loads

- Concrete members column, beams, girders are designed based SBC & AISC. Where the steel beams and girders are designed based on AISC.

2.2.3. Design for Shear Strength

- Design for the cross sections subjected to the shear wall
- For the detailed calculations, it would be made based on sectionwhile the rest of the design shall be made based on sections

2.2.4 Design of slab

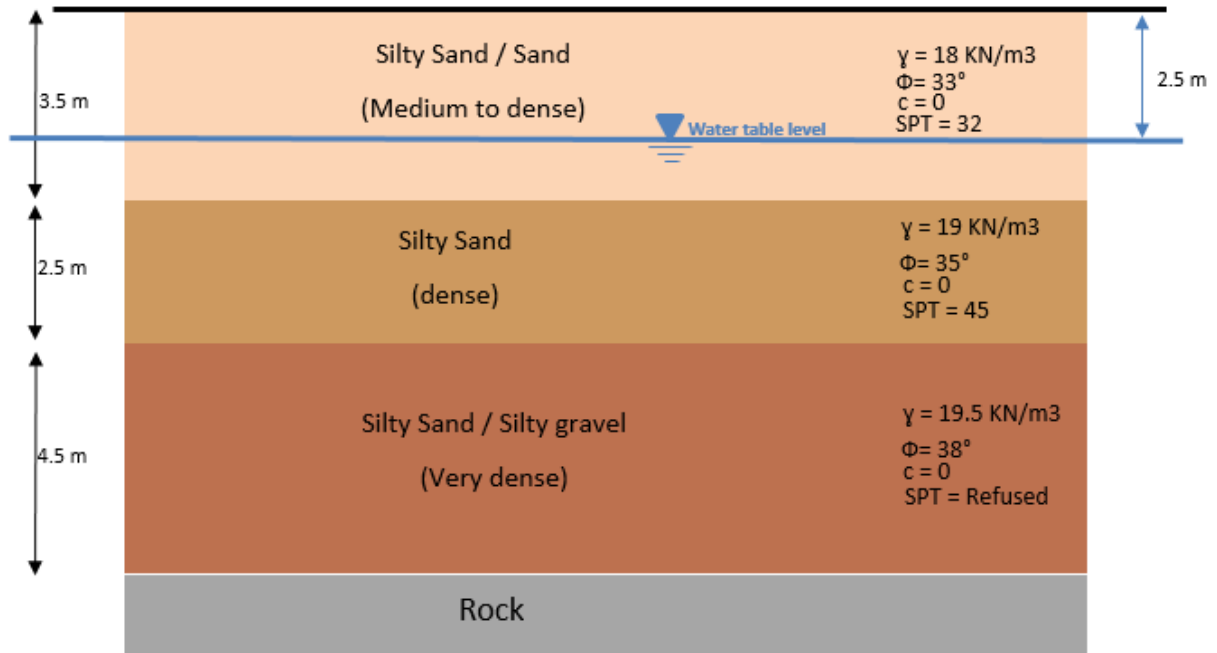
- **Concrete slab design equation is obtained from SBC.**
- **CLT slab design, used the doubled RCC slab thickness.**

2.3 Constraints:

Constraints		Solution
Geotechnical	1. Water table at 2.5 m	Dewatering
		Permanent drainage system
	2. Rock	Excavation of 4.5 m
Material	1. Sulfates	Type 5 sulfate resistance Portland cement
	2. Chlorides	Coating the foundation
Environmental	1. contaminated water	Transporting the contaminated water to the right disposal place
	2. Contaminated soil	Dispose the contaminated soil into hazardous landfill
Structural	1. Deflection limits & drift	Considering all possible and reasonable load combination cases to ensure safety and stability of the building.
	2. Load combination cases	
Cost	1. The Solution of the constraints affect the cost	

2.3.1 Geotechnical Constraints:

The constraints are the water table level and the bedrock. Our building is 60 storeys including basement. The basement is 4 levels. Each level is 4m high. Therefore, at least we have to excavate 16m. The average water table level is 2.5m. This requires dewatering the site to be able to construct the foundation. It is inevitable to excavate 5.5m of the rock strata.



2.3.2 Material Constraints:

According to the geotechnical report, the water and the soil have a certain amount of harmful chemical concentration higher than the allowable . The chemical are sulfates and chlorides. The sulfates concentration affects the foundation unless we use the type IV sulfate resistance Portland cement. Chlorides may harm the foundation. We are suggesting using MAT foundation supported by bored pile. The soil is contaminated with chlorides. Chloride is the enemy of steel reinforcements. Therefore, the solution is to coat the steel rebar and the concrete mix after it gets hardened.

Sample Type	Chemical concentration	
	Sulfate	Chloride
Soil	0.048% > 0.372%	0.061 > 0.137%
	0.039% > 0.386%	0.059% > 0.112%
Water	600ppm -665ppm	1670ppm –1790ppm

2.3.3 Environmental Constraints:

We have to put the environment in our consideration. The excavated soil is contaminated. Therefore, the contaminated soil must be disposed into hazardous landfill. The dewatered water is contaminated is well. It has to be transported to the right disposal place or transported to waste water treatment plant.

2.3.4 Structural Constraints:

To ensure the safety, stability and durability we developed load combination cases in ETABS model to put the building under multiple crucial scenarios. The design will be based on the worst scenario. There are many uncertainties regarding the nature. We cannot exactly predict the magnitude of an earthquake, unusual storm happens for example once each 100 years or extreme floods, etc. we look at project location environmental history and we considered all possible and reasonable load combination cases to ensure safety, stiffness and stability of the building.

The top part that is hanged between the twin towers is by itself a constraint due its long span. It was a great challenge to design the girders of the top, many times we designed sections they fulfill the safety, but they failed in the deflection limit. The constraint is designing a section that its deflection less than 1 inches and its depth accepted so it is not affect the storey height. The solution is that we build-up a section and we increased the storey height 1m for the top four storeys.

2.3.5 Cost Constraints:

All of the previous constraints geotechnical, material, environmental, and structural are affecting the project cost. The constraints solutions are costly specially the geotechnical constraint solutions. Transporting the contaminated water is costly. Dispose the contaminated soil into engineering hazardous landfill. It is must to design a hazardous landfill where in KSA there is no hazardous landfill is fulfilling the international environmental standards. Lastly, the steel girders of the top part are huge sections. This may affect the project total cost.

CHAPTER 3: Preliminary Design

3.1 Design the top part between the two towers:

The top part consists of girders, sub-beams and slab. Girders and sub-beam are steel members. The slab is roller compacted concrete (RCC). First, calculating the slab thickness to know exactly its weight on sub-beams. The weight and load coming from the slab is transferred to sub-beams and the sub-beams transfer the load including sub-beams self-weight to the girders as well. The hand calculation is considering the gravity load only including live load, dead load and member self-weight. The slab thickness that we get may change later on and the steel members as well. The slab reinforcement is might be the reason behind changing the slab thickness. ETABS calculation will consider multiple load combination cases including wind load and seismic forces along X & Y directions. The initial slab thickness including the cover is to be defined on ETABS to run the analysis. After running the analysis, ETABS will provide the required area of steel to reinforce the slab. Same story for the steel members. In our hand calculation regarding sub-beams and girders design, the self-weight, live load and dead load only are considered to get an idea about what will happen later with ETABS analysis. We will assign preliminary sections for girders and sub-beams on ETABS to run the analysis.

3.1.1 Design two-way slab (RCC):

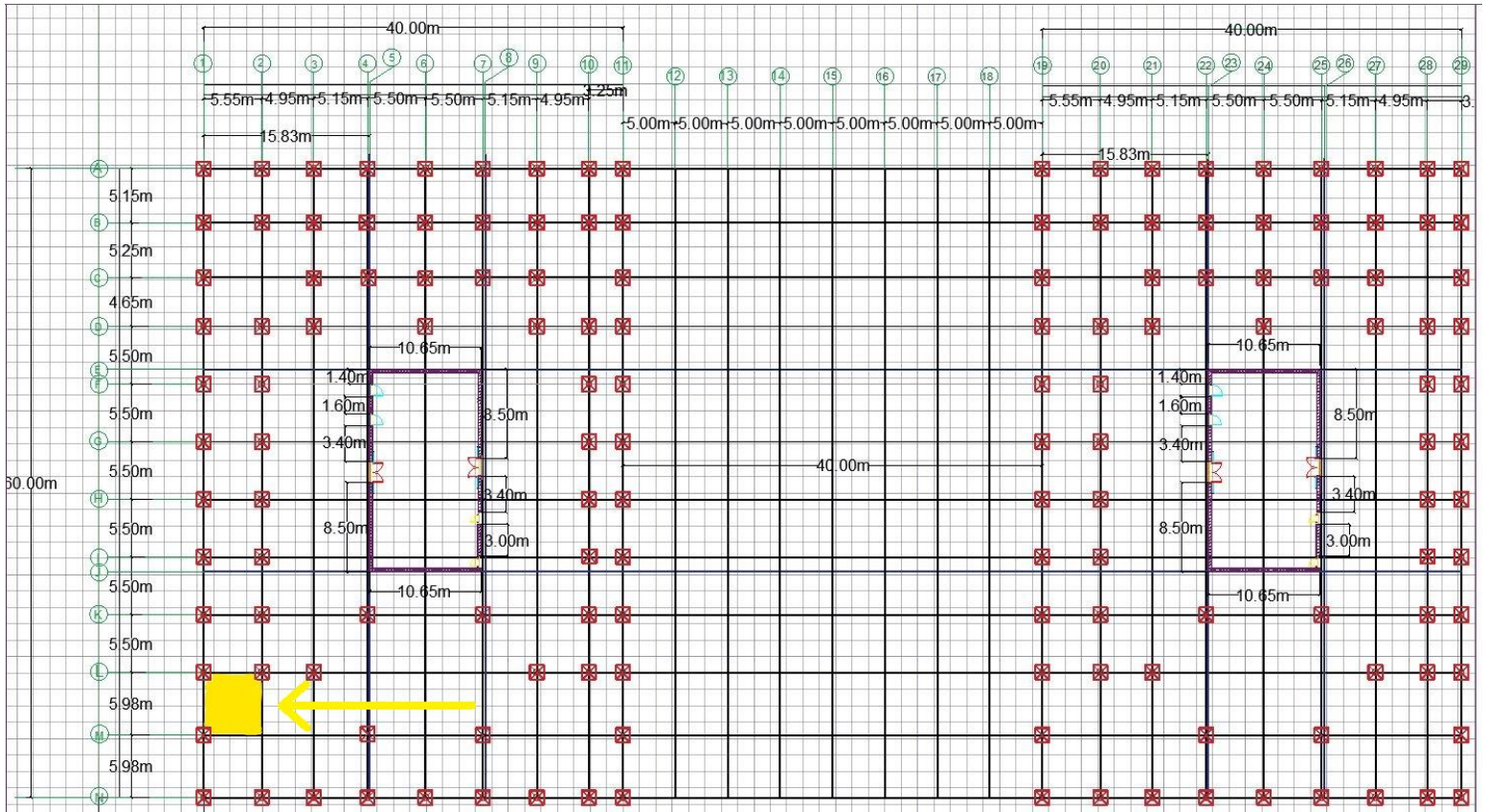
The slab thickness is obtained based on the critical situation where the bigger slab area is carried by the sub beam which is 5.55m x 5.975m as shown in figure3.1.1. Since the ratio of the longer beam to the shorter beam is 1.08. so, it is two-way slab.

$$r = \frac{\text{long span}}{\text{short span}} = \frac{5.975}{5.55} = 1.08$$

There are two equation obtained from (ACI) code to get the slab thickness:

1. slab thickness = $\frac{\text{shorten span}}{35} = \frac{218.52}{35} = 6.243''$
2. slab thickness = $\frac{\text{peimeter}}{180} = \frac{2(235.25+218.52)}{180} = 5.042'' = 128.067 \text{ mm}$

The slab thickness obtained from the second equation, the second equation is considered because it gave the smaller slab thickness. Also, the concrete cover is $2'' = 50.80 \text{ mm}$. The final slab thickness is $128.067 + 50.80 = 178.867 \text{ mm}$ and this thickness is not practical. Therefore, the final slab thickness is 180mm applied for the whole building since it is the biggest slab area. The required steel reinforcement for the slab is obtained automatically by ETABS after assigning the slab dimension. After getting the area of steel required from ETABS, we will choose the number of bar and the quantities.



3.1.2 Dead load:

Dead load can be considered as constant loads that cannot be changed over time. In other words, it is structure self-weight. This includes all of:

CONCRETE SLAB											
Residential Tower				Connected part				Commercial Tower			
Materials	Unit weight (KN/m ³)	Thickness (mm)	W (KN/m ²)	Materials	Unit weight (KN/m ³)	Thickness (mm)	W (KN/m ²)	Materials	Unit weight (KN/m ³)	Thickness (mm)	W (KN/m ²)
tiles	28.5	20	0.57	tiles	28.5	20	0.57	tiles	28.5	20	0.57
mortar	20.4	25	0.51	mortar	20.4	25	0.51	mortar	20.4	25	0.51
concrete slab	24	178.87	4.29288	concrete slab	24	178.87	4.29288	concrete slab	24	178.87	4.29288
plaster			0.25	plaster			0.25	plaster			0.25
profile metal deck	10.33	75	0.77475	profile metal deck	10.33	75	0.77475	profile metal deck	10.33	75	0.77475
E & M			0.5	E & M			0.5	E & M			0.5
Ceiling			1.8	Ceiling			2.41	Ceiling			1.8
swimming pool			1.635	swimming pool			9.30763	swimming pool			1.635
total			10.33263	total				total			10.33263

Loading: (dead load + live load)

Slab Self-Weight:

Thickness x concrete unit weight (γ_c)

$$= \frac{5.042}{12} \text{ ft} \times 150 \text{ lb/ft}^3 = 63.03 \text{ lb/ft}^2$$

Concrete cover (2''):

$$= \frac{2}{12} \text{ ft} \times 150 \text{ lb/ft}^3 = 25 \text{ lb/ft}^2$$

Finishing:

Tile: $0.57 \text{ kN/m}^2 = 11.91 \text{ lb/ft}^2$

Mortar: $0.51 \text{ kN/m}^2 = 10.65 \text{ lb/ft}^2$

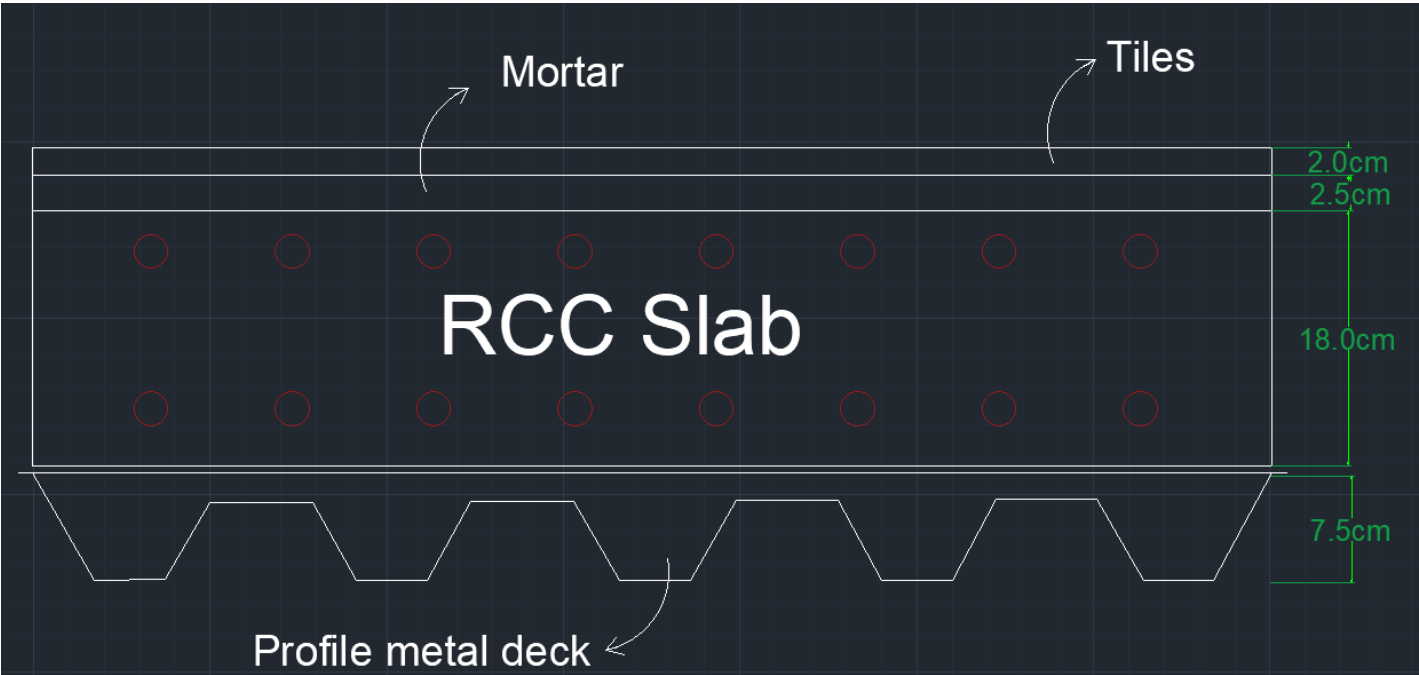
Plaster: $0.25 \text{ kN/m}^2 = 5.22 \text{ lb/ft}^2$

PMD: $0.775 \text{ kN/m}^2 = 16.186 \text{ lb/ft}^2$

E&M: $0.5 \text{ kN/m}^2 = 10.443 \text{ lb/ft}^2$

Ceiling: $2.41 \text{ kN/m}^2 = 50.334 \text{ lb/ft}^2$

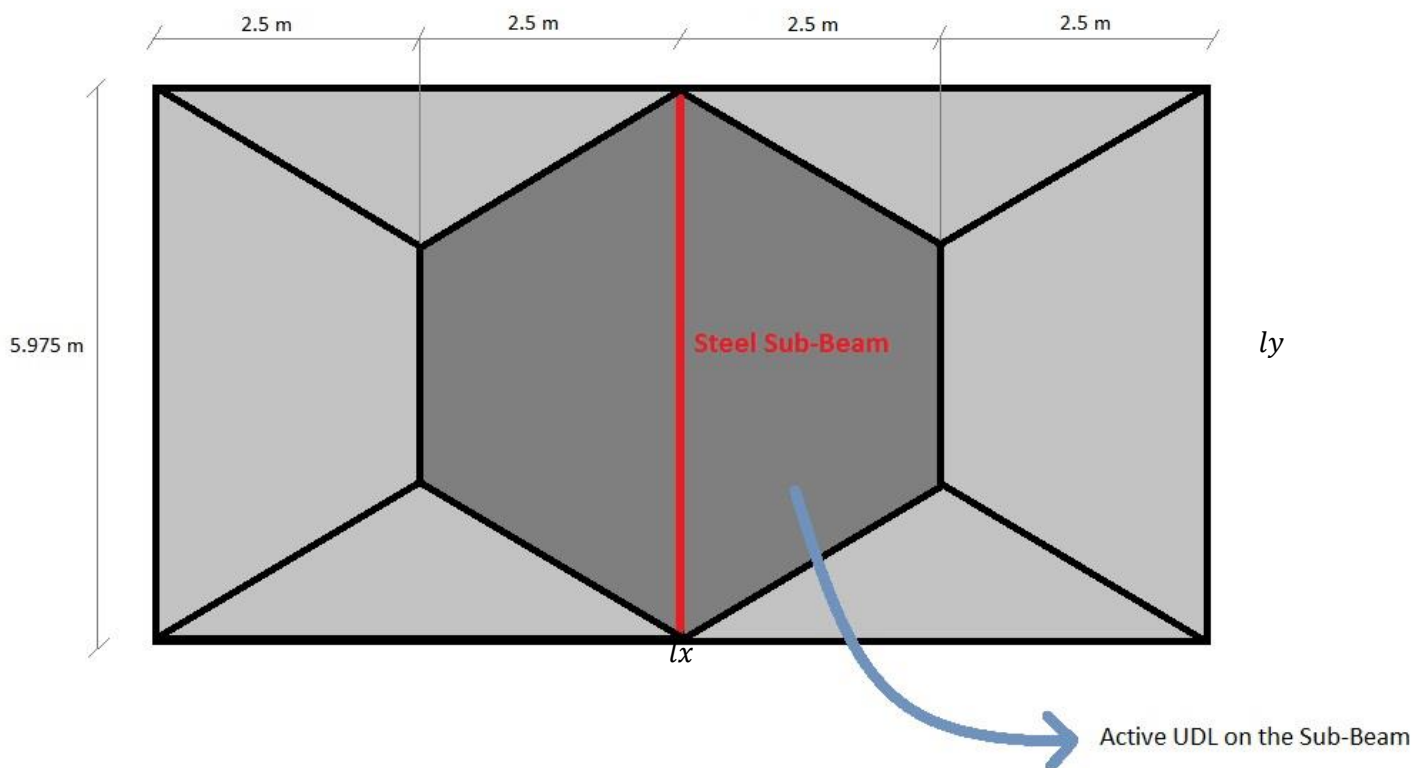
RCC slab cross section:



3.1.3 Design steel sub-beam for the top part:

The dark shaded hexagon area is the uniformly distributed load (UDL) acting on the steel sub-beam. UDL is coming from slab self-weight, finishing and live load. The finishing (UDL) is coming from Tile, Mortar, Plaster, Profile Metal deck (PMD), Electrical & Mechanical (E&M), and Ceiling. The sub-beam is carrying the UDL from the slab self-weight and the finishing, and live load. The live load used in our calculation for the top part is 10kN/m^2 . As mentioned earlier that the top part is 4 storeys. The live load of two of them was 10kN/m^2 and the other two storeys' live load was less. So, that is why 10kN/m^2 is considered for all of them.

Initial sub-beam design could be changed after including multiple load combination cases in ETABS analysis as results.

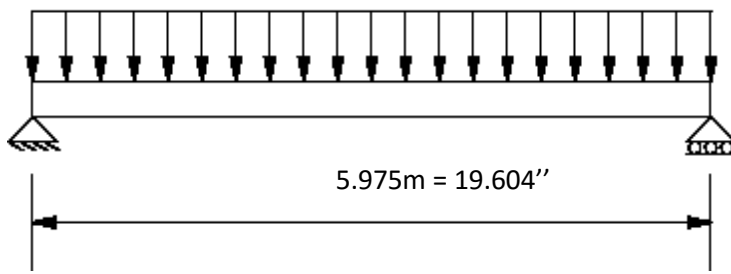


Total dead load and live load including safety factors:

$$\begin{aligned}
 W_u &= 1.2 W_D + 1.6 W_L \\
 &= 1.2(63.03 + 25 + 11.91 + 10.65 + 5.22 + 16.186 + 10.443 + 50.334) + 1.6 (209) \\
 &= 565.728 \text{ lb/ft}^2 = 27.087 \text{ kN/m}^2
 \end{aligned}$$

The sub-beam is designed based on the critical situation, which is maximum loading to satisfy the safety and quality of the structure. The design of the steel sub-beams members are following the AISC Code regulations.

This formula $\{I_y = \frac{W_u Lx}{6} (3 - (\frac{lx}{ly})^2)\}$ is used to transfer the uniformly distributed load from the turbidity area to be active on the sub-beam that is under its stress.



$$I_y = \frac{27.087 \times 5}{6} \left(3 - \left(\frac{5}{5.975} \right)^2 \right) = 51.91 \text{ kN/m} = 3557.025 \text{ lb/ft}$$

$$\delta_{\text{all}} = \frac{L}{240} = \frac{5.975 \times 1000}{240} = 24.9 \text{ mm} = 0.98 \text{ in}$$

$$\delta_{\text{all}} = \frac{ML^2}{C I_x}$$

$$M = \frac{WL^2}{8} = \frac{1}{8} (351.91)(5.975)^2 = 370.644 \text{ kN-m} = 273.373 \text{ kip-ft}$$

$$\delta_{\text{all}} = \frac{ML^2}{C I_x}$$

$$0.98 = \frac{273.373(19.604)^2}{161(I_x)}$$

$$I_x = 665.876 \text{ in}^4$$

Choosing the sub-beam section based on the moment of inertia $I_x = 795 \text{ in}^4$:

The selected member is **W14 x 74**.

Checking the deflection of the sub-beam in addition to its self-weight:

$$W_{U+D} = 3.557025 + 0.074 = 3.631 \text{ kip/ft}$$

$$M = \frac{WL^2}{8} = \frac{1}{8} (3.631)(19.604)^2 = 174.432 \text{ kip-ft}$$

$$\delta_{\text{all}} = \frac{ML^2}{C I_x} = \frac{174.432(19.604)^2}{161 (795)} = 0.524'' < 0.98''$$

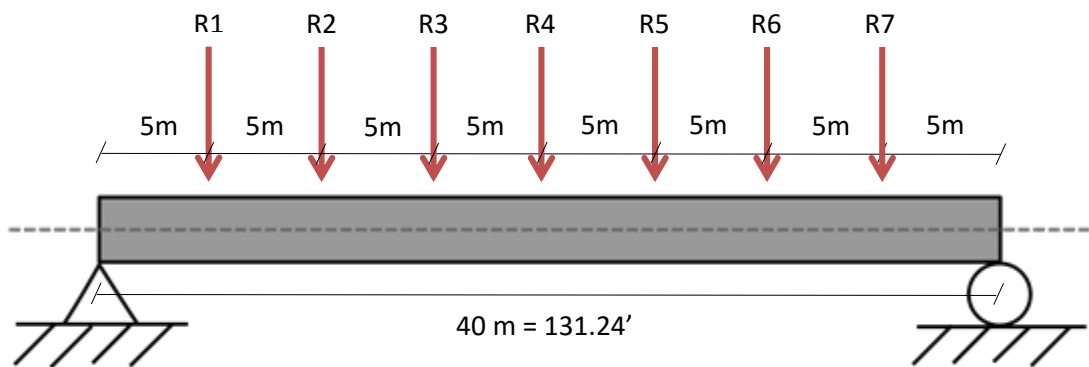
Checking the shear force (v_u) and the moment (M_u):

$$V_u = \frac{1}{2} W_u L = \frac{1}{2} (3.631)(19.604) = 35.59 \text{ kip} < 192 \text{ kip}$$

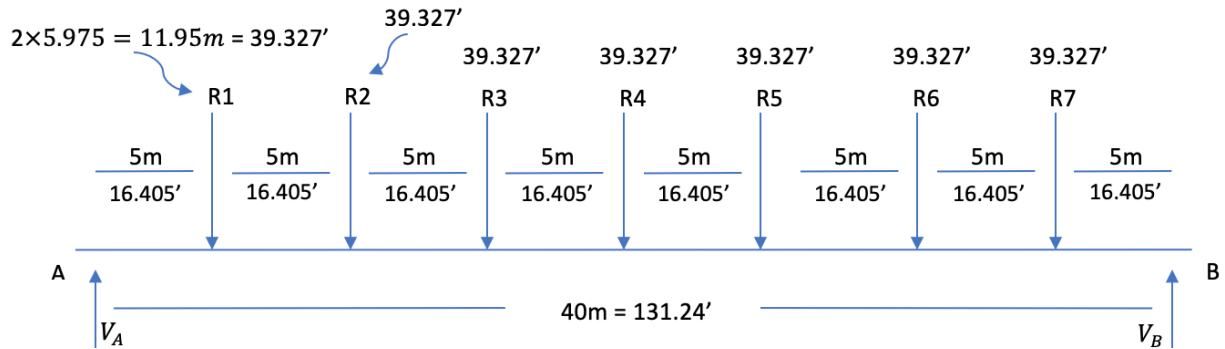
$$M_u = \frac{WL^2}{8} = \frac{3.631 (19.604)^2}{8} = 174.432 \text{ kip.ft} < 473 \text{ kip.ft}$$

3.1.4 Design steel girder for tee top part:

As mentioned earlier, all load coming from slab and sub-beam are transferred to the girders. The girder span is 40m so, the deflection limit is the governing factor in our design and the deflection has to be less 1 inch. The following is girder design process. Sub-beams are designed based on the critical situation. In fact, the sub-beams are resting on the girders and the load is transfer from sub-beam to the girders as point load. The designed section is to be assigned in ETABS to be able to run the analysis with the consideration of the possible load combination cases.



Girder Design:



$$R = \frac{1}{2} WL$$

$$R_1 = \frac{1}{2} (3.631) (39.327) = 74.4 \text{ kips}$$

$$R_1 = R_2 = R_3 = R_4 = R_5 = R_6 = R_7$$

$$\delta_{all} = \frac{L}{240} = \frac{131.24 \times 12}{240} = 6.562''$$

To get the reactions:

$$\begin{aligned} \curvearrowright \Sigma M_A = 0 : & 71.4 (16.405) + 71.4 (32.81) + 71.4 (49.215) + 71.4 (65.62) + 71.4 (82.025) \\ & + 71.4 (98.43) + 71.4 (114.835) - V_B (131.24) = 0 \end{aligned}$$

$$V_B = 249.9 \text{ K}$$

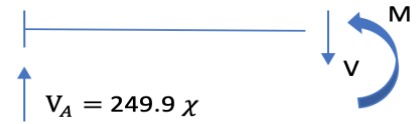
$$V_A = 249.9 \text{ K}$$

Just checking:

$$\frac{71.4 \times 7}{2} = 249.9 \text{ Kips}$$

Now we will calculate Shear and Moment on every point to find Maximum Shear and Maximum Moment.

1. At $x = 16.405'$

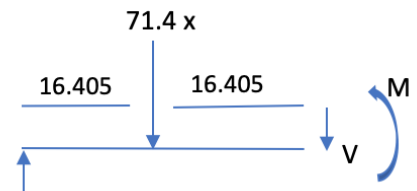


$$\overset{+}{\curvearrowright} \Sigma M = -249.9 (32.81) + 71.4 (16.405) + M = 0$$

$$M = 4099.61 \text{ K-ft}$$

$$V = 249.9 \text{ K}$$

2. At $x = 32.81'$



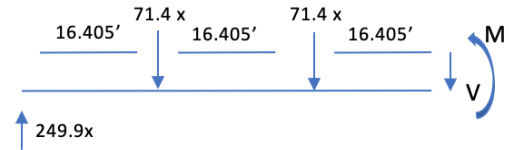
$$\overset{+}{\curvearrowright} \Sigma M = 0 : -249.9 (32.81) + 71.4 (16.405) + M = 0$$

$$M = 70.27.902 \text{ K-ft}$$

$$\overset{+}{\downarrow} \Sigma V = 0 : V + 71.4 - 249.9 = 0$$

$$V = 178.5 \text{ K}$$

3. At $\chi = 49.215'$



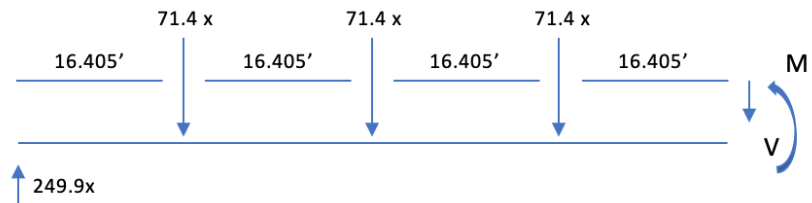
$$+\curvearrowright \Sigma M = 0: -249.9 (49.215) + 71.4 (32.81) + 71.4 (16.405) + M = 0$$

$$M = 8784.878 \text{ K-ft}$$

$$+\downarrow \Sigma V = 0: -249.9 + 71.4 + 71.4 + V = 0$$

$$V = 107.1x$$

4. At $\chi = 65.62'$



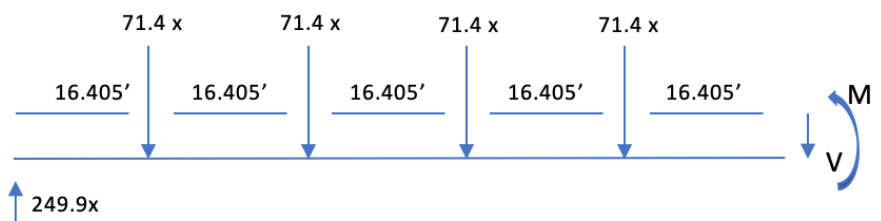
$$+\curvearrowright \Sigma M = 0: -249.9 (65.62) + 71.4 (49.215) + 71.4 (32.81) + 71.4 (16.405) + M = 0$$

$$M = 9370.536 \text{ K-ft}$$

$$+\downarrow \Sigma V = 0: -249.9 + 71.4 + 71.4 + 71.4 + V = 0$$

$$V = 35.7 \text{ K}$$

5. At $\chi = 82.025'$



$$\Sigma M = 0: -249.9 (82.025) + 71.4 (65.62) + 71.4 (49.215) + 71.4 (32.81) + 71.4 (16.405) +$$

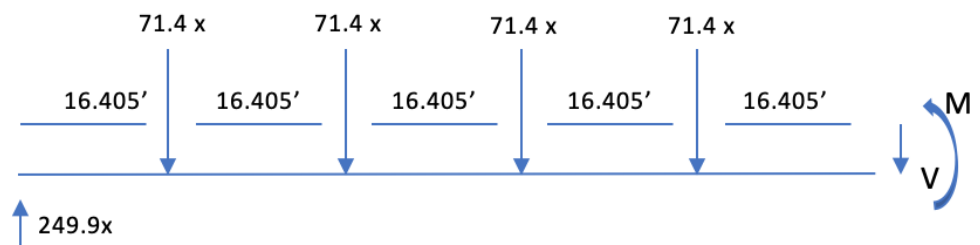
$$+ \curvearrowright M = 0$$

$$M = 8784.878 \text{ K-ft}$$

$$+ \downarrow V = 0: -249.9 + 71.4 + 71.4 + 71.4 + 71.4 + V = 0$$

$$V = -35.7 \text{ K}$$

6. At $x = 98.43'$



$$+ \curvearrowright \Sigma M = 0: -249.9 (98.43) + 71.4 (82.025) + 71.4 (65.62) + 71.4 (49.215) + 71.4 (32.81)$$

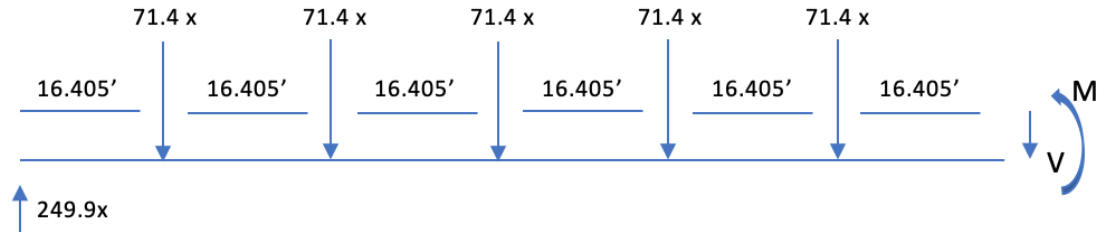
$$+ 71.4 (16.405) + M = 0$$

$$M = 7027.902 \text{ K-ft}$$

$$\Sigma V \quad + \downarrow = 0: -249.9 + 5 (71.4) + V = 0$$

$$V = -107.1 \text{ K}$$

7. At $x = 114.835'$



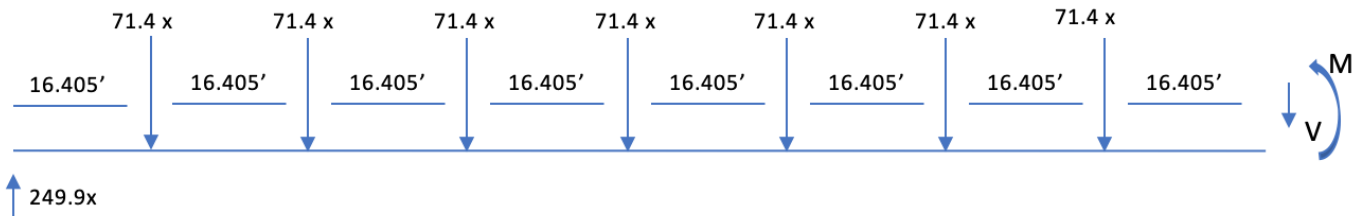
$$\begin{aligned} \curvearrowright \Sigma M = 0: & -249.9 (114.835) + 71.4 (98.43) + 71.4 (82.025) + 71.4 (65.62) + 71.4 (49.215) \\ & + 71.4 (32.81) + 71.4 (16.405) + M = 0 \end{aligned}$$

$$M = 4099.61 \text{ K-ft}$$

$$+\downarrow \Sigma V = 0: -249.9 + 6 (71.4) + V = 0$$

$$V = -178.5x$$

8. At $131.24'$



$$\begin{aligned} \curvearrowright \Sigma M = 0: & -249.9 (131.24) + 71.4 (114.835) + 71.4 (98.43) + 71.4 (82.025) + 71.4 (65.62) \\ & + 71.4 (49.215) + 71.4 (32.81) + 71.4 (16.405) + M = 0 \end{aligned}$$

$$M = 0$$

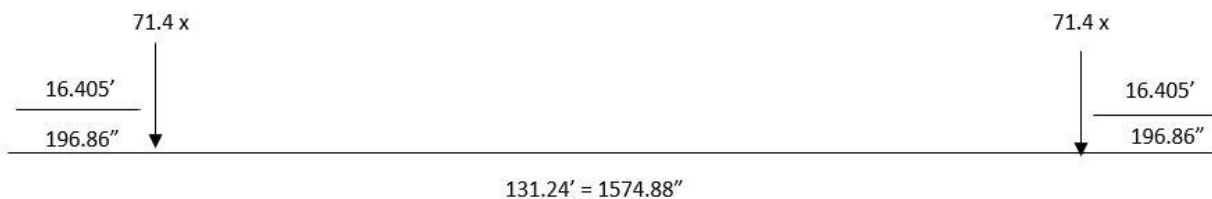
$$+\downarrow \Sigma V = 0: -249.9 + 7 (71.4) + V = 0$$

$$V = -249.9x$$

Girder Deflection:

Now we will calculate to find the deflection of the girder by using the formulas from (AISC), and we want to make the deflection equal to the allowable deflection to know the moment of inertia needed for the built-up section and should be higher than the one from the allowable deflection.

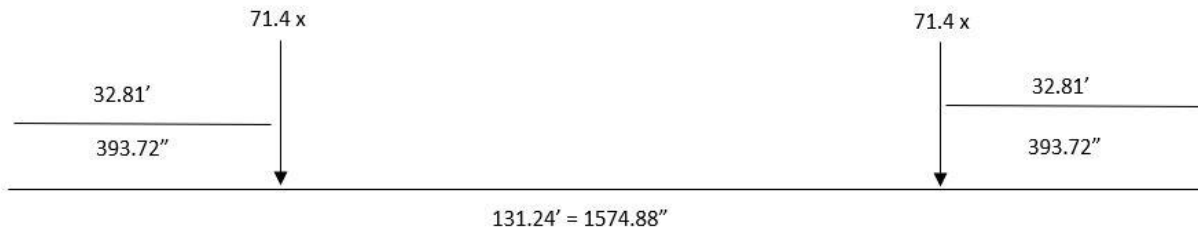
$$\delta_{max} = \frac{Pa}{24EI} (3l^3 - 4a^2)$$



$$\delta_{max} = \frac{71.4 (196.86)}{24(29000)I} [3(1574.88)^2 - 4(196.86)^2]$$

$$\delta_{max} = \frac{147136.11}{I}$$

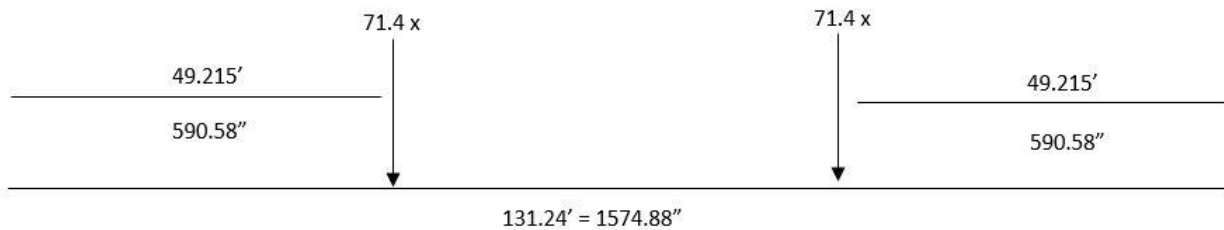
$$\delta_{max} = \frac{Pa}{24EI} (3l^3 - 4a^2)$$



$$\delta_{max} = \frac{71.4 (393.72)}{24 (29000)} [3(1574.88)^2 - 4(393.72)^2]$$

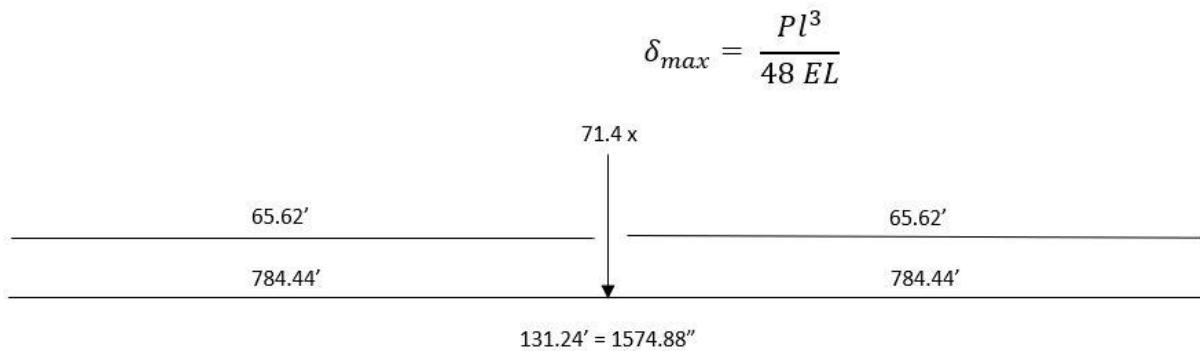
$$\delta_{max} = \frac{275488.883}{I}$$

$$|\delta_{max} = \frac{Pa}{24EI} (3l^3 - 4a^2)$$



$$\delta_{max} = \frac{71.4 (590.58)}{24 (29000)} [3(1574.88)^2 - 4(590.58)^2]$$

$$\delta_{max} = \frac{366274.992}{I}$$



$$\delta_{max} = \frac{Pl^3}{48 EL}$$

$$\delta_{max} = \frac{71.4 (1574.88)^3}{48 (29000)I}$$

$$\delta_{max} = \frac{200355.55}{I}$$

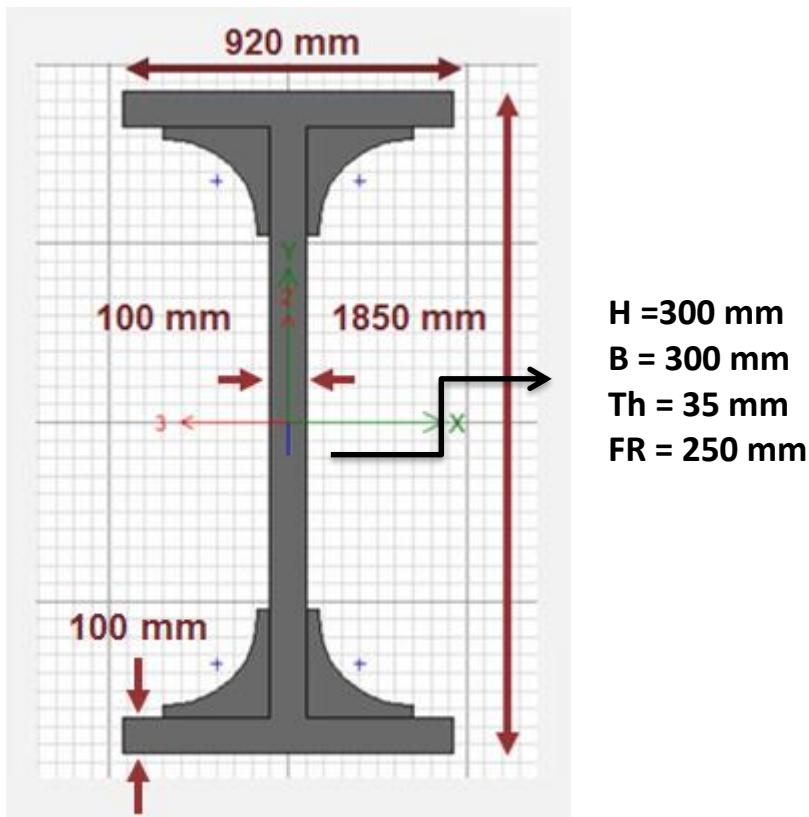
$$\text{So, } \delta_{all} = \frac{\Sigma \delta_{max}}{I} \rightarrow 6.562 = \frac{989255.535}{I}$$

$$I = 150755.187 \text{ in}^4$$

Girder built-up section:

After getting the required moment of inertia to design safe section, we build-up a section without considering wind load and seismic forces and this is the build-up section W 36 X 652.

After ETABS analysis; this section (w 36 x 652) has failed due to load combination case, which is wind load along Y direction and dead load. Then we increased the web thickness and the depth of the section in many trails to get it safe, stable and economized. We could build-up a huge section without considering the exaggerated depth and economy. We considered in our built-up section dimensions three things. The first thing is the depth of the section should be not too long to affect the storey height. Secondly, the section dimensions should be logical and applicable in the construction phase. The last concern was the economy. We tried to minimize the section until it fulfills safety and stability requirements. Therefore, the final built-up section is W36 X 652 with L 8X8X1/2 in the four interior angles under the flanges.



3.2 Design concrete beams for the tow tower:

In this part is the concrete beam calculation for the towers. The towers beams have designed for only two different spans, one with 51.94 ft span and the other one with 70.71 ft span. Since the layout of the building is symmetrical. We are designing only two beam cross sections and apply them for the whole towers. We are using equations from Saudi Building Code (SBD) to find the cross section of the beam and then to be assigned in ETABS to get the required steel reinforcement area. Then we will choose the number of bar and the quantities.

Concrete beams design:

$$h = \frac{l}{16} = \frac{51.94 \cdot 12}{16} = 38.955'' = 40''$$

$$d = h - 3 = 40 - 3 = 37''$$

$$b = \frac{d}{1.5} = \frac{37}{1.5} = 24.667''$$

$$\text{Beam self-weight} = 1.2(\gamma_c * b * h)$$

$$= 150 * \frac{24.66}{12} * \frac{40}{12}$$

$$= 1233.35 \text{ lb/ft} = 1.6722 \text{ KN/m}$$

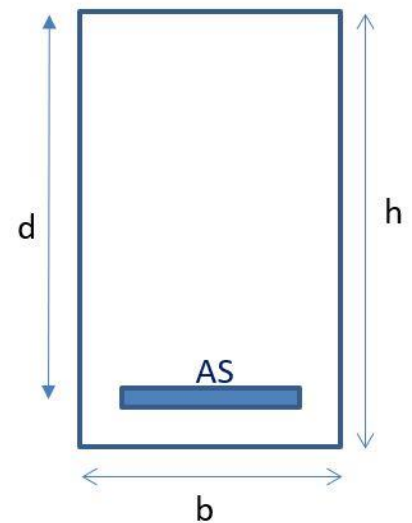
Beam cross section:

Area of steel required (As):

$$h = 40 \text{ in} = 1020 \text{ mm}$$

$$d = 37 \text{ in} = 939.8 \text{ mm}$$

$$b = 24.667 \text{ in} = 630 \text{ mm}$$



Same process for the other beam and the only difference is the span length:

$$h = \frac{l}{16} = \frac{70.71 \cdot 12}{16} = 53.033''$$

$$d = h - 3 = 53.033 - 3 = 50.033''$$

$$b = \frac{d}{1.5} = \frac{50.033}{1.5} = 33.355''$$

$$\text{Beam self-weight} = 1.2(\gamma_c * b * h)$$

$$= 150 * \frac{33.055}{12} * \frac{53.033}{12}$$

$$= 2191.257 \text{ lb/ft} = 2.971 \text{ KN/m}$$

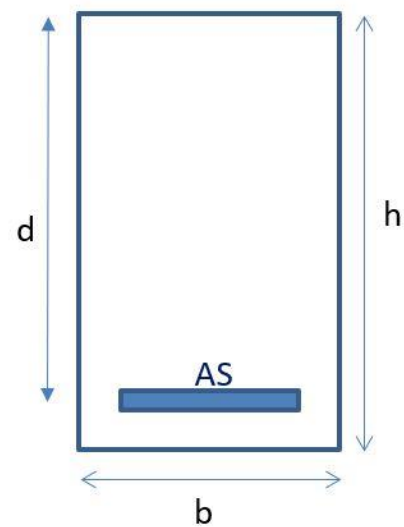
Beam cross section:

Area of steel required (A_s)

$$h = 40 \text{ in} = 1020 \text{ mm}$$

$$d = 50.033 \text{ in} = 1450 \text{ mm}$$

$$b = 33.355 \text{ in} = 900 \text{ mm}$$



Loading (dead load for both towers):

Self-Weight:

Thickness x concrete unit weight (γ_c)

$$= \frac{5.042}{12} \text{ ft} \times 150 \text{ lb/ft}^3$$

$$= 63.03 \text{ lb/ft}^2 = 3.018 \text{ KN/m}^2$$

Concrete cover (2''):

$$= \frac{2}{12} \text{ ft} \times 150 \text{ lb/ft}^3 = 25 \text{ lb/ft}^2 = 1.197 \text{ KN/m}^2$$

Total slab thickness including the cover is 7.042''

Finishing:

Tile: 0.57 kN/m² = 11.91 lb/ft²

Mortar: 0.51 kN/m² = 10.65 lb/ft²

Plaster: 0.25 kN/m² = 5.22 lb/ft²

PMD: 0.775 kN/m² = 16.186 lb/ft²

E&M: 0.5 kN/m² = 10.443 lb/ft²

Ceiling: 1.8 kN/m² = 37.595 lb/ft²

Floors with Swimming pool:

Level 41 & 42 the only two having swimming pool. These two levels are the only once having swimming pool. Since SBC doesn't provide us with exact swimming pool dead load. So, we assumed the volume of the swimming pool to be able to convert the water volume into liters. Thus 1 liter equals 1 Kg, then we converted the water mass into weight by multiplying it by 0.00981 m/s to get it in kN.

Swimming pool (L=20m, b= 10m, h= 2m):

$$20 * 10 * 2 = 400 \text{ m}^3$$

$$400 \text{ m}^3 = 400,000 \text{ liters} = 400,000 \text{ Kg}$$

$$400,000 \text{ Kg} * 0.00981 = 3924 \text{ KN}$$

$$\frac{3924}{60(40)} = 1.635 \frac{\text{kN}}{\text{m}^2}$$

The total dead load = 10.255 kN/m²

The dead load without slab load 6.04 kN/m²

Considered Live Load:

The live load is obtained from Saudi Building Code (SBC). There are only three live load considered in our hand calculation 5, 7 and 10kN/m² the live depend on the activities that each floor provide.

These are the floors and their live load. The floors having live load 2 - 5 kN/m² all of them considered as 5 kN/m² the floors having live load 6-7.5 kN/m² all of them are considered as 7.5 kN/m² the floors having more than 7.5-10 kN/m² live load are considered as 10kN/m². The following table expresses some examples about the live load.

Floor Purpose	Live Load (KN\m²)
Mechanical layer	10
Parking	10
Wedding hall	10
Library	7.5
Museum	6.5
Spa, fitness center	5
Restaurants, & Coffeehouse	5
lobby & Management	5
Emergency clinic & pharmacy	2
Apartments	2

$$W_u (5 \text{ kN/m}^2) = 1.2 (8.62) + 1.6(5) = 18.344 \text{ kN/m}^2$$

$$W_u (7.5 \text{ kN/m}^2) = 1.2 (8.62) + 1.6(7.5) = 28.344 \text{ kN/m}^2$$

$$W_u (10 \text{ kN/m}^2) = 1.2 (8.62) + 1.6(10) = 26.344 \text{ kN/m}^2$$

For floor 41 & 42 (swimming pool):

$$W_u (10 \text{ kN/m}^2) = 1.2 (10.255) + 1.6(10) = 28.306 \text{ kN/m}^2$$

Note: Dead load without swimming pool = 4.405 kN/m²

Modified concrete beam designed:

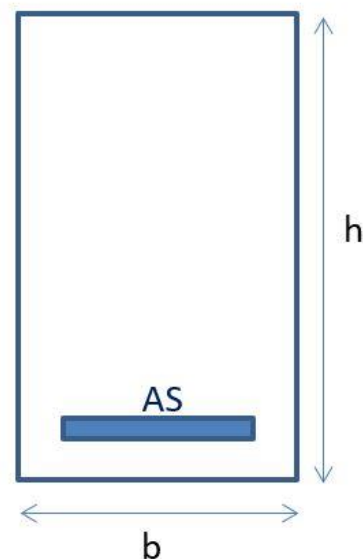
After assigning, the concrete beams dimensions and properties in ETABS. These two beams have failed the analysis because we did not consider the wind and seismic load in our hand calculation to get an initial beam cross sectional dimension. We came up with new concrete beams to fulfill safety and stability requirements. After assigning the wind, seismic loads and other load combinations in ETABS model. As results, we designed bigger sections to fulfill safety and stability requirements. The following figures show the modified concrete beams.

Beam cross section:

Area of steel required (A_s)

$$h = 51.020 \text{ in} = 1250 \text{ mm}$$

$$b = 40.816 \text{ in} = 1000 \text{ mm}$$

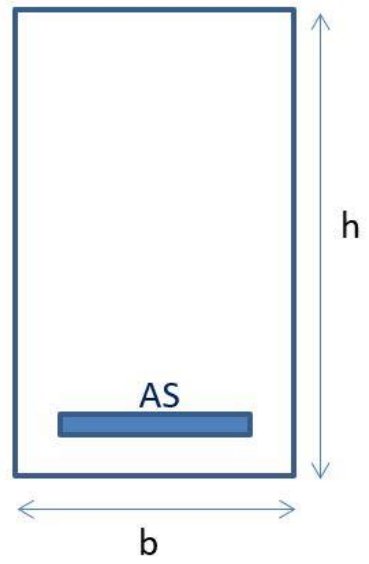


Beam cross section:

Area of steel required (A_s)

$h = 57.143 \text{ in} = 1400 \text{ mm}$

$b = 48.980 \text{ in} = 1200 \text{ mm}$



3.3 Columns Design:

The column design is done based on the critical situations, which are in columns M11 and M7. M11 is located on the edge of the residential tower. It is carrying the load coming from the tower itself (turbidity area of slab) and in addition to the load coming from top part that connects the two towers up to 20m since the span is 40m. The remaining 20m is carried by the other column right across column M11 in the commercial tower. This scenario is only for the top 4 storeys. M7 location is critical as well. Since this column is carrying the bigger slab area in addition to the other live loads of the purposes coming from different storey levels and this scenario is for the whole tower (60 storeys).

	Dead load (kN/m ²)	Live load (kN/m ²)	1.2DL+1.6LL (kN/m ²)
Tower UDL	8.62	5	18.344
Over hanged (top part UDL)	9.23	10	27.087

Column Design: (M11): - (For the top 4 storeys)

$$P_u \text{ slab (5 kN/m}^2 \text{ \& 10 kN/m}^2) = W_u b L$$

$$= 18.344 (5.975)(6.675) + 27.087 (20) \left(\frac{5.975}{2} + \frac{5.975}{2} \right) = 3968.513 \text{ KN}$$

$$R = \frac{1}{2} WL$$

$$R = \frac{1}{2} (0.06)(11.94)$$

$$= 0.36 \text{ kN}$$

$$\sum F_y = 3 \times \frac{0.36}{2} = 1.26 \text{ kN}$$

P_u (beams):

$$P_{u(B)} = 1.2(\gamma_c l b h)$$

$$= 4[1.2(23.6 * 2.9875 * 0.63 * 102)]$$

$$+ 1.2(23.6 * 6.675 * 0.63 * 102)]$$

$$+ 1.2(76.9729 * 20 * 0.48399) = 1223.045 + 1.26 = 1234.305 \text{ KN}$$

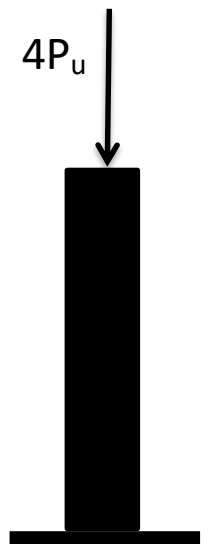
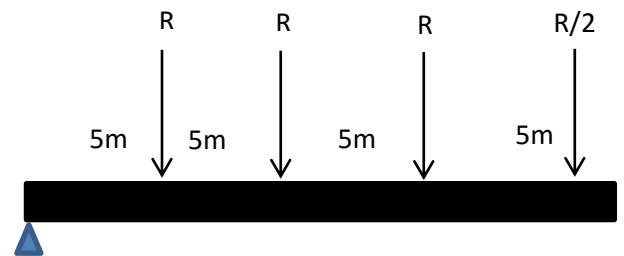
$$P_u = P_{u(s)} + P_{u(b)}$$

$$= 3968.513 + 1234.305$$

$$= 5202.818 \text{ kN}$$

$$4P_u = 4 * 5202.818$$

$$= 20811.272 \text{ kN}$$



Still column design for (M11): (the rest storeys):

$$P_{u(s)} = 18.344 (32.883) = 731.614 \text{ kN}$$

$$P_{u(s)} = 2[1.2(1/2 * 1.6722 * 11.95)] + 1.2 (23.6 * 6.675 * 0.63 * 1.02)$$
$$= 145.454 \text{ kN}$$

$$P_u = 731.614 + 145.454 = 876.068 \text{ kN} = 196.948 \text{ KIPS}$$

$$P_{u(s)} = 22.344 (39.883) = 891.145 \text{ kN}$$

$$P_{u(s)} = 145.454 \text{ kN}$$

$$P_u = 891.145 + 145.454 = 1036.6 \text{ kN} = 233.037 \text{ KIPS}$$

$$P_{u(s)} = 26.344 (39.883) = 1050.678 \text{ kN}$$

$$P_{u(B)} = 145.454 \text{ kN}$$

$$P_u = 1050.678 + 145.454 = 1196.132 \text{ kN} = 268.901 \text{ KIPS}$$

$$P_u = 28.306(39.883) + 145.454 = 1274.382 \text{ kN} = 286.4925 \text{ KIPS}$$

Column Design: (M7): - (the whole storeys)

$$P_{u(s)} = 18.344 ((5.275 * 5.975) + (6.9 * 8.9625)) = 1712.584 \text{ kN}$$

Find the R_s on the girder: -

Sub. Beam on girder(M9):

$$R_1 = \frac{1}{2} (1.6722) (11.95) = 9.99 \text{ kN}$$

Sub- beam on girder (L7):

$$R_2 = \frac{\frac{1}{2}(3.1777)(6.9)}{2} = 5.482 \text{ kN}$$

Sub beam on girder (M6)

$$R_3 = \frac{1/2 (1.6722) (8.9625)}{2} = 3.747 \text{ kN}$$

Total point load on girders= 9.99 + 5.482+ 3.74

$$= 19.219 \text{ kN}$$

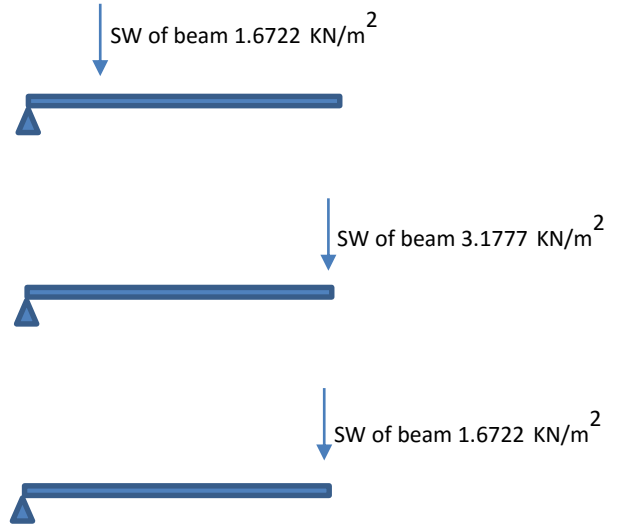
$$P_{u(B)} = 1.2(23.6 * 6.675 * 0.63 * 1.02)$$

$$+(23.6 * 2.9875 * 0.63 * 1.02)$$

$$+(23.6 * 5.975 * 0.63 * 1.02)$$

$$+(23.6 * 5.5 * 0.63 * 1.02)$$

$$= 384.67 \text{ kN}$$



$$P_u (5 \text{ kN/m}^2) = 1712.584 + 19.219 + 384.67 = 2116.473 \text{ kN}$$

$$P_u (7.5 \text{ kN/m}^2) = 2086.022 + 19.219 + 384.67 = 2489.911 \text{ kN}$$

$$P_u (10 \text{ kN/m}^2) = 1712.584 + 19.219 + 384.67 = 2863.349 \text{ kN}$$

$$P_{u(\text{For Floor 41-42})} = 2642.63 + 19.219 + 384.67 = 3046.519 \text{ kN}$$

Required column gross section area:

$$P_u = 0.8(0.65)[0.85(4)(A_g - 0.02A_g) + 60(0.02)A_g]$$

$$A_g = \frac{P_u}{2.3566}$$

- The columns dimension for each group of storey levels [APPENDIX 1]:

Model with RCC slab floor system	
Storey levels	Columns Dimensions
1-4	3.2 m X 3.2 m
5-12	3.1 m X 3.1 m
13-20	2.8 m X 2.8 m
21-28	2.6 m X 2.6 m
29-36	2.4 m X 2.4 m
37-44	2.1 m X 2.1 m
45-52	1.8 m X 1.8 m
53-56	1.3 m X 1.3 m
57-60	0.7 m X 0.7 m

3.5 Shear wall design

The way or method used of calculating the shear wall is the same as the column design process.

Shear wall thickness:

$$\begin{aligned} P_{u(s)} &= 1.2(\gamma_c l b h) \\ &= 18.344[2(10.65 * 2.75) + 2(19.3 * 5.05)] \\ &= 4650.3 \text{ kN} \end{aligned}$$

$$\begin{aligned} P_{u(B)} &= 1.2(\gamma_c l b h) \\ &= 1.2(\gamma_c l b h) \\ &= 8[1.2(23.6 * 5.05 * 0.9 * 1.3)] \\ &= 2[1.2(23.6 * 2.75 * 0.9 * 1.3)] \\ &= 1520.87 \text{ kN} \end{aligned}$$

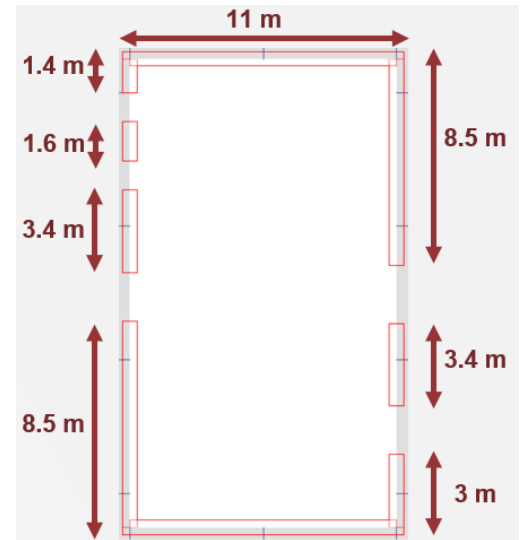
$$P_{u(5 \text{ kN/m}^2)} = 4650.3 + 1520.87 = 6171.17 \text{ kN}$$

$$P_{u(7.5 \text{ kN/m}^2)} = 5664.32 + 1520.87 = 7185.19 \text{ kN}$$

$$P_{u(10 \text{ kN/m}^2)} = 6678.34 + 1520.87 = 8199.19 \text{ kN}$$

$$P_{u(\text{swimming pool})} = 7175.71 + 1520.87 = 8696.58 \text{ kN}$$

- The calculation of shear wall thickness is 0.6m [APPENDIX 2].



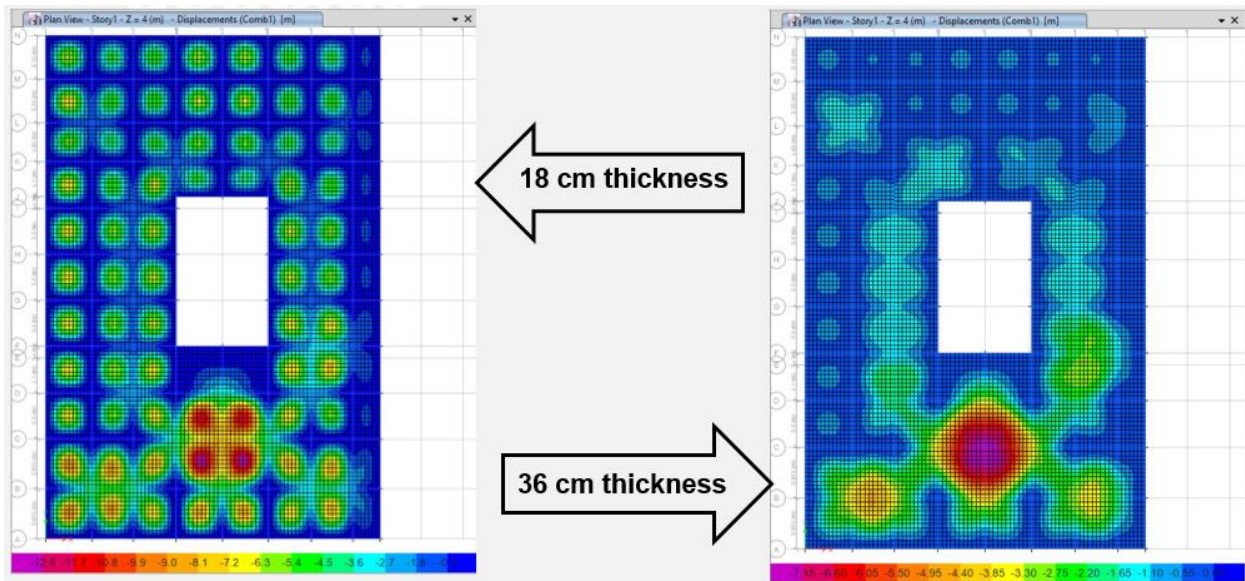
3.4 CLT preliminary design:

3.4.1 CLT slab design:

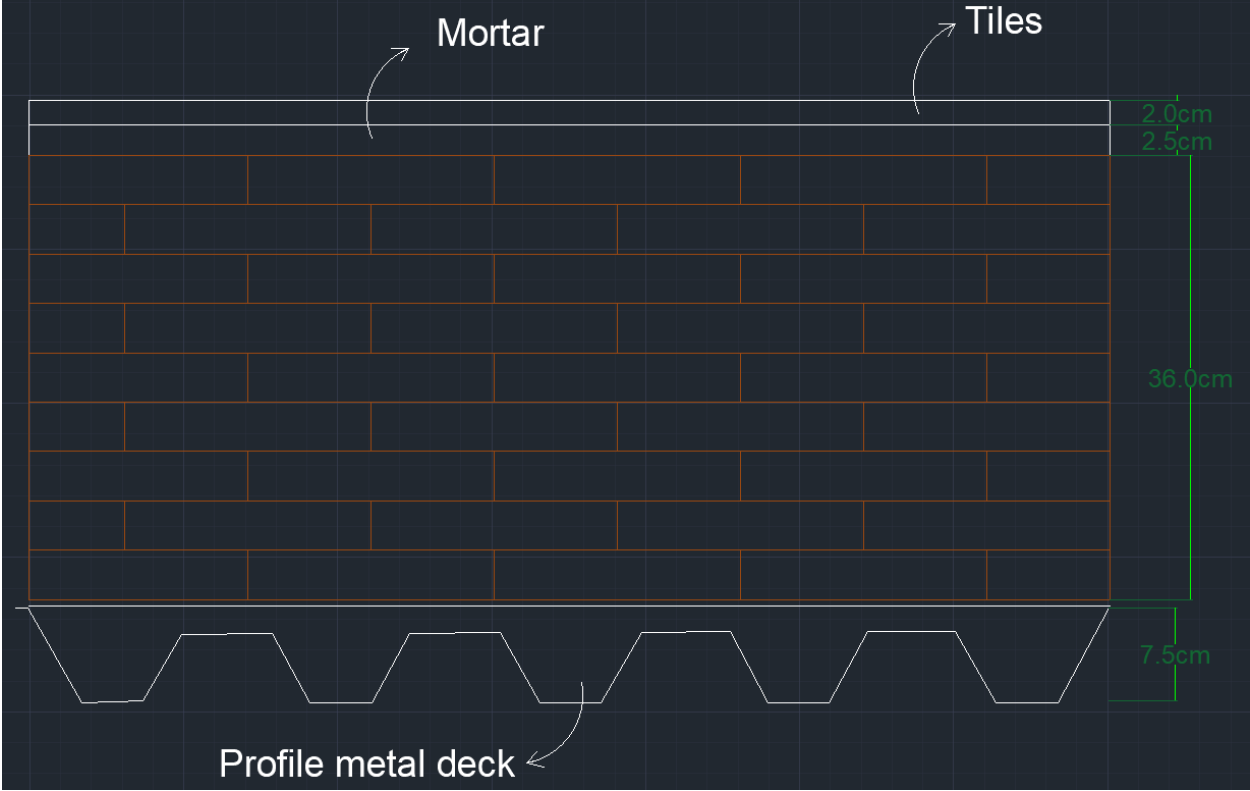
The way of dealing with cross-laminated timber (CLT) floor slab system is the same as dealing with the roller compacted concrete (RCC). However, the only difference is the self-weight of slab because of the density. Timber density is 4 kN/m^3 while the concrete density is 23.6 kN/m^3 .

Since we calculated the RCC slab thickness, which is 18 cm, we considered the same thickness for CLT slab at the first time we ran ETABS analysis. After ETABS analysis, we decide to double the thickness, which is 36 cm because of the deflection. When the thickness was 18 cm, each slab portion of the floor deflected locally. When the slab doubled, the whole floor deflected as one unit. Moreover, there is another practical reason, which was the connection. It is easier and more practical to use bigger nails to anchor the CLT slab.

The slab used for the parking area is (RCC) because of two reasons. It is not practical to use timber in the parking area due some uncertainties. When we ran ETABS analysis the two beams in the basement (parking) have failed. Even when used huge sections. However, when replace the CLT slab with RCC the analysis is passed. The other reason is that we have to maintain the timber dry as possible to protect it, but with the presence of cars, it might be impossible.



CLT slab cross section:



3.4.2 Concrete beams:

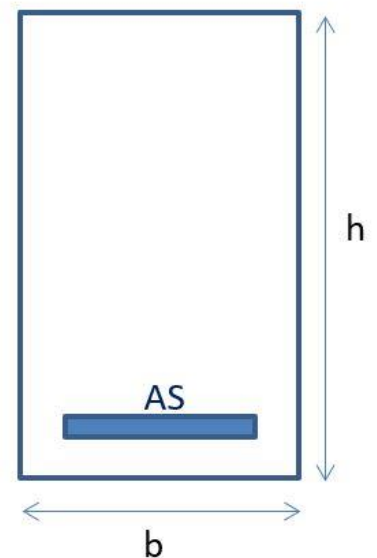
In this part is the concrete beam calculation for the towers. The towers beams have designed for only two different spans, one with 51.94 ft (beam) span and the other one with 70.71 ft (girder) span. Since the layout of the building is symmetrical. We are designing only two beam cross sections and apply them for the whole towers. We are using equations from Saudi Building Code (SBD) to find the cross section of the beam and then to be assigned in ETABS to get the required steel reinforcement area. Then we will choose the number of bar and the quantities.

The designed section did not differ that much comparing with the designed section in RCC slab model. The only difference is with girders. Girder cross section in CLT design is smaller than the girder in RCC design.

Girder cross section:

$$h = 1350 \text{ mm}$$

$$b = 1100 \text{ mm}$$



CLT SLAB								
Residential Tower				Connected part	Commercial Tower			
Materials	Unit weight (KN/m ³)	Thikness (mm)	W (KN/m ²)		Materials	Unit weight (KN/m ³)	Thikness (mm)	W (KN/m ²)
tiles	28.5	20	0.57		tiles	28.5	20	0.57
mortar	20.4	25	0.51		mortar	20.4	25	0.51
CLT slab	4	360	1.44		CLT slab	4	178.87	0.71548
plaster			0.25		plaster			0.25
profile metal deck	10.33	75	0.77475		profile metal deck	10.33	75	0.77475
E & M			0.5		E & M			0.5
Ceiling			1.8		Ceiling			1.8
swimming pool			1.635		swimming pool			1.635
total			7.47975		total			6.75523

- Loading: (Dead load) : (Both Towers) :

1. Self-weight: thickness x γ_T

= 0.18 m x 4 kN/m³

= 0.72 kN/m² = 15.04 lb/ft²

2. Finishing: tile = 0.57 kN/m² = 11.91 lb/ft²

Mortar = 0.51 kN/m² = 10.65 lb/ft²

Plaster = 0.25 kN/m² = 5.22 lb/ft²

PMD = 0.775 kN/m² = 16.186 lb/ft²

E&M = 0.5 kN/m² = 10.443 lb/ft²

3. Ceiling:

= 1.8 kN/m² = 37.595 lb/ft²

3. Swimming pool: (20m x 10m x 2m =400

$$400 \text{ m}^3 = 400000 \text{ liter} = 400000 \text{ kg}$$

$$400000 \text{ kg} \times 0.00981 = 3924 \text{ kN}$$

$$\frac{3924}{60 \times 40} = 1.635 \text{ kN/m}^2 = 34.148$$

$$\text{Total dead load with swimming pool} = 6.76 \text{ kN/m}^2$$

$$\text{Total dead load without swimming pool} = 5.125 \text{ kN/m}^2$$

- Loading: (Dead load & live load): (Both Towers)

We will design for three live loads, firstly with 5 kN/m^2 , and then with 7.5 kN/m^2 , and finally with 10 kN/m^2 .

$$w_{u(5 \text{ kN/m}^2)} = 1.2(5.125) + 1.6(5) = 14.15 \text{ kN/m}^2$$

$$w_{u(7.5 \text{ kN/m}^2)} = 1.2(5.125) + 1.6(7.5) = 18.15 \text{ kN/m}^2$$

$$w_{u(10 \text{ kN/m}^2)} = 1.2(5.125) + 1.6(10) = 22.15 \text{ kN/m}^2$$

$$\text{Floor 41-42} \leftarrow w_{u(10 \text{ kN/m}^2)} = 1.2(6.76) + 1.6(10) = 24.112 \text{ kN/m}^2$$

(With swimming pool)

3.4.3 Column design:

Since the structure is designed twice with two different slab floor systems RCC & CLT. The same procedure of designing the columns is going to be the same in both designs. It is expected to have smaller column dimensions than the column in RCC, because CLT slab is ultra-lightweight comparing the concrete weight.

Column Design Calculations: (M11. For the top four storeys)

$$- p_u \text{ of slab : } p_{u(s)} = w_u \text{ b L}$$

$$= 14.15 (5.975 \times 6.675) + 27.087(20 \times (\frac{5.975}{2} + \frac{5.975}{2})) = 3801.243 \text{ kN}$$

$$\begin{array}{c} \downarrow \\ \text{DL + LL of both towers} = 5 \text{ kN/m}^2 \end{array}$$

$$\begin{array}{c} \downarrow \\ \text{DL+ LL of top part} = 10 \text{ kN/m}^2 \end{array}$$

$$- p_u \text{ of beams : } p_{u(8)} = 1234.305 \text{ kN/m}^2$$

$$p_u = p_{u(5)} + p_{u(8)} = 3801.243 + 1234.305 = 5035.548 \text{ kN/m}^2$$

Column Design Calculations (M11. The rest storeys):

$$\text{LL} = 5 \text{ kN/m}^2 \leftarrow p_{u(s)} = 14.15(39.883) = 564.344 \text{ KN}$$

$$p_{u(B)} = 145.454 \text{ KN}$$

$$p_u = 709.8 \text{ KN}$$

$$LL = 7.5 \text{ kN/m}^2 \leftarrow p_{u(s)} = 18.15 (39.883) = 723.877 \text{ kN}$$

$$p_{u(B)} = 145.454 \text{ kN}$$

$$p_u = 1028.863 \text{ kN}$$

$$\text{Floor 41-42} \leftarrow p_u = 24.112(39.883) + 145.454 = 1107.113 \text{ kN}$$

(With swimming pool)

Column Design (N7. The whole storeys)

$$LL = 5 \text{ kN/m}^2 \leftarrow p_{u(5)} = 14.15 ((5.275 \times 5.975) + (6.9 \times 8.9825)) = 1321.035 \text{ kN}$$

Total point load on girders = 19.219 kN

$$p_{u(B)} = 384.67 \text{ kN}$$

$$LL = 5 \text{ kN/m}^2 \leftarrow p_u \leftarrow p_u = 1321.035 + 19.219 + 384.67 = 1724.924 \text{ kN}$$

$$LL = 7.5 \text{ kN/m}^2 \leftarrow p_u \leftarrow p_u = 1694.473 + 19.219 + 384.67 = 2098.362 \text{ kN}$$

$$LL = 10 \text{ kN/m}^2 \leftarrow p_u = 2067.91 + 19.219 + 384.67 = 2471.8 \text{ kN}$$

$$\text{Floor 41-42} \leftarrow p_u = 2251.08 + 12.219 + 384.67 = 2654.97 \text{ kN}$$

3.4.4 Shear wall design:

The way of calculating the shear wall is the same as the column design process.

Shear wall Calculation Design:

$$p_{u(s)} = 14.15 [2 (10.65 \times 2.75) + 2 (19.3 \times 5.05)] = 3587.1 \text{ kN}$$

$$p_{u(B)} = 1520.87 \text{ kN}$$

$$\text{LL} = 5 \text{ kN/m}^2 \leftarrow p_u = 3587.1 + 1520.87 = 5107.97 \text{ kN}$$

$$\text{LL} = 7.5 \text{ kN/m}^2 \leftarrow p_u = 4601.12 + 1520.87 = 6121.99 \text{ kN}$$

$$\text{LL} = 10 \text{ kN/m}^2 \leftarrow p_u = 5615.14 + 1520.87 = 7136.01 \text{ kN}$$

$$\text{Floor 41- 42} \leftarrow p_u = 6112.51 + 1520.87 = 7633.38 \text{ kN}$$

Chapter 4: Steel Reinforcements

4.2 RCC Slab Steel Reinforcements:

The layout is divided based on two layers as strips. The first layer, the strips are along X direction as shown in figure 4.1.1. The second layer, the strips are along Y direction as shown in figure 4.1.2. The strips are the layout grid line. Then we ran ETABS analysis to check the maximum moments of both strips as shown in figure 4.1.3 in each floor as per group. We have eight groups. Therefore, we have nine RCC slab cross sections with same thickness, but different area of steel. We could design one slab with the same area of steel and apply it for the whole building, but it is not economical. That is the reason behind dividing the storeys into groups who have similar bending moments.

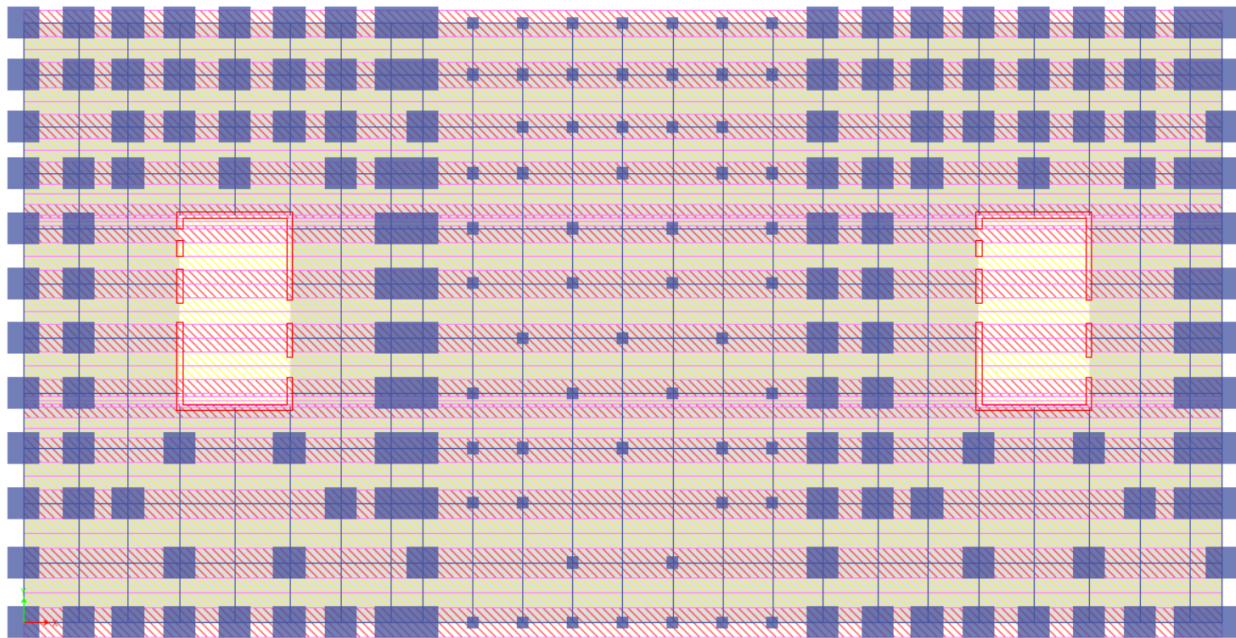


Figure 4.1.1

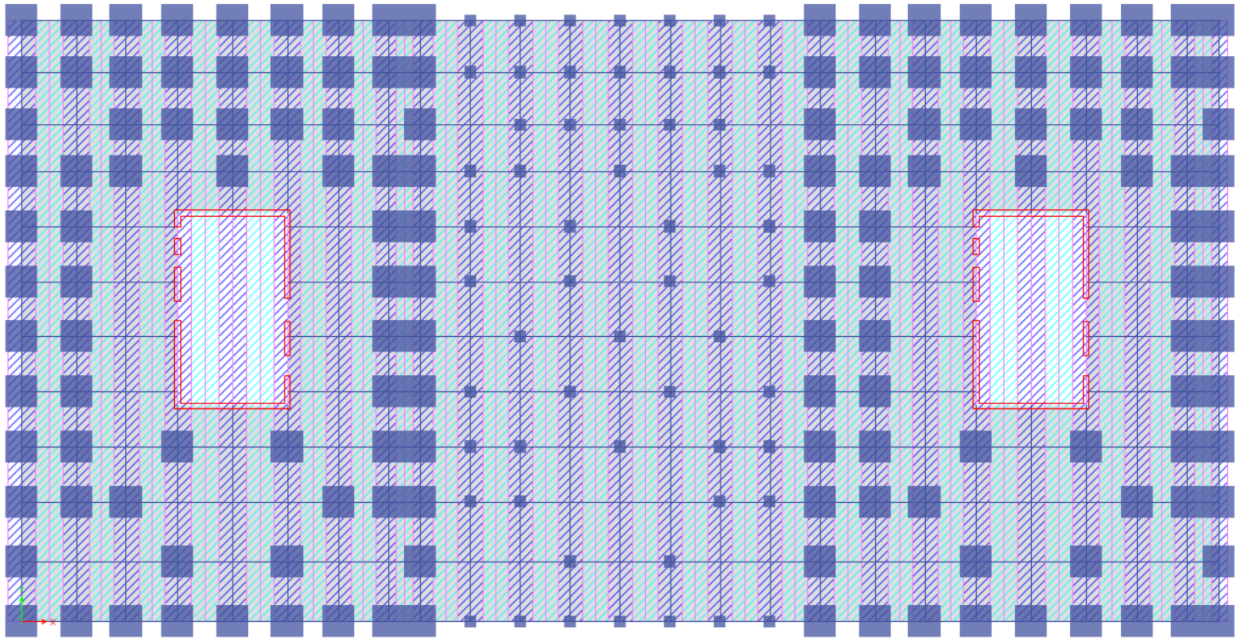


Figure 4.1.2

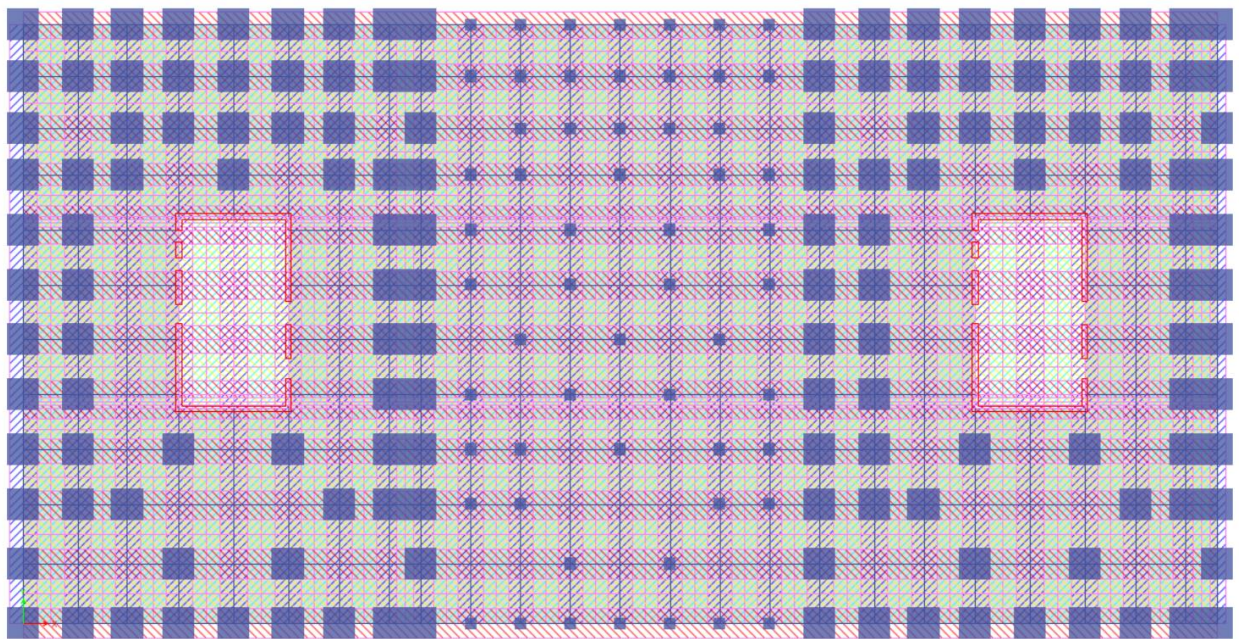


Figure 4.1.3

Group1. Floor 1- 4:

Slab Reinforcement calculations:

$$M_{\max} = 171.37 \text{ kN.m} , f_c' = 69 \text{ Mpa} , f_y = 345 \text{ Mpa}$$

$$R_u = \frac{M_{\max}}{0.9 b d^2} = \frac{171.37(10^6)}{0.9(1000)(165^2)} = 6.994 \text{ N/mm}^2$$

$$\rho = \frac{0.85}{f_y} f_c' \left(1 - \sqrt{1 - \frac{2 R_u}{0.85 f_c'}} \right) = \frac{0.85 (69)}{345} \left(1 - \sqrt{1 - \frac{2 (6.994)}{0.85 (69)}} \right) = 0.0217$$

$$A_s = \rho b d = 0.0217(1000)(165) = 3580.5 \text{ mm}^2$$

$$\text{Using } 12 \# 20 = 3770 \text{ mm}^2$$

$$\rho_{\text{Min}} = \frac{3 \sqrt{f_c'}}{f_y} = \frac{3\sqrt{69000}}{345000} = 0.0023 \gg A_{\text{steel Min}} = 380 \text{ mm}^2$$

$$\rho_{\text{Max}} = 0.75 \left(\frac{0.85 \beta f_c'}{f_y} \right) \left(\frac{600}{600+f_y} \right) = 0.75 \left(\frac{0.85 (0.65)(69)}{345} \right) \left(\frac{600}{600+345} \right) = 0.0526$$

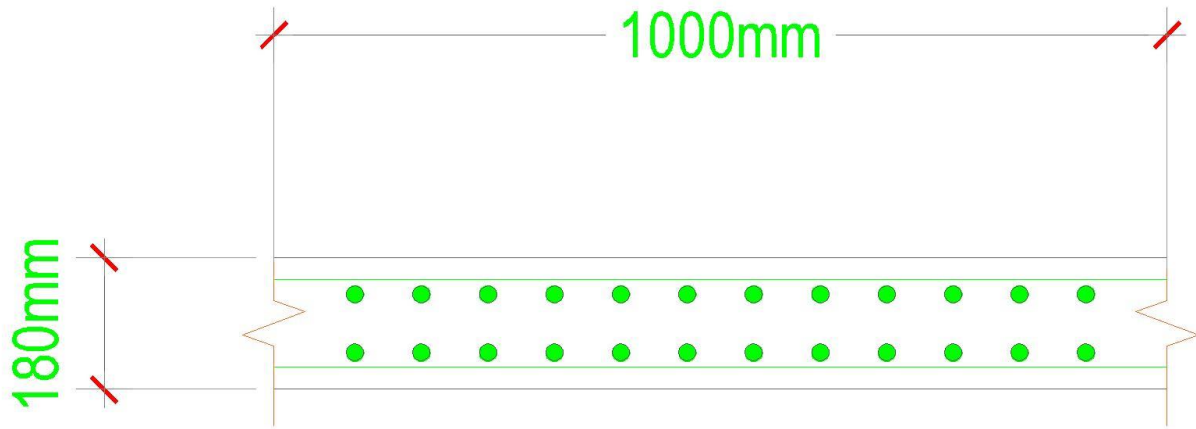
$$A_{\text{steel Max}} = 0.0526 (1000)(165) = 8679 \text{ mm}^2 \gg A_{\text{steel Min}} < A_{\text{steel}} < A_{\text{steel Max}}$$

$$S_{\min} \leq A_{\text{steel (1 bar)}} \times \frac{b}{A_{\text{(required steel area)}}} \rightarrow S \leq 341.16 \times \frac{1000}{3580.5}$$

$$S \leq 95.28$$

$$S = 85$$

$$S_{\min} \leq 85 \text{ mm} < S_{\text{Max}}$$



12#20 @ 85mm C/C
Cover = 15mm

Group2. Floor 5- 12:

Slab Reinforcement calculations:

$$M_{\max} = 160.02 \text{ kN.m}$$

$$R_u = \frac{M_{\max}}{0.9 b d^2} = \frac{160.02 (10^6)}{0.9(1000)(165^2)} = 6.53 \text{ N/mm}^2$$

$$\rho = \frac{0.85 (69)}{345} \left(1 - \sqrt{1 - \frac{2 (6.53)}{0.85 (69)}} \right) = 0.0201$$

$$A_s = 0.020 (1000)(165) = 3316.5 \text{ mm}^2$$

$$\text{Using } 12 \# 19 \ A_s = 3402 \text{ mm}^2$$

$$A_{s \text{ Min}} = (0.0023)(1000)(165) = 380 \text{ mm}^2$$

$$A_{s \text{ Max}} = (0.0526)(1000)(165) = 8679 \text{ mm}^2$$

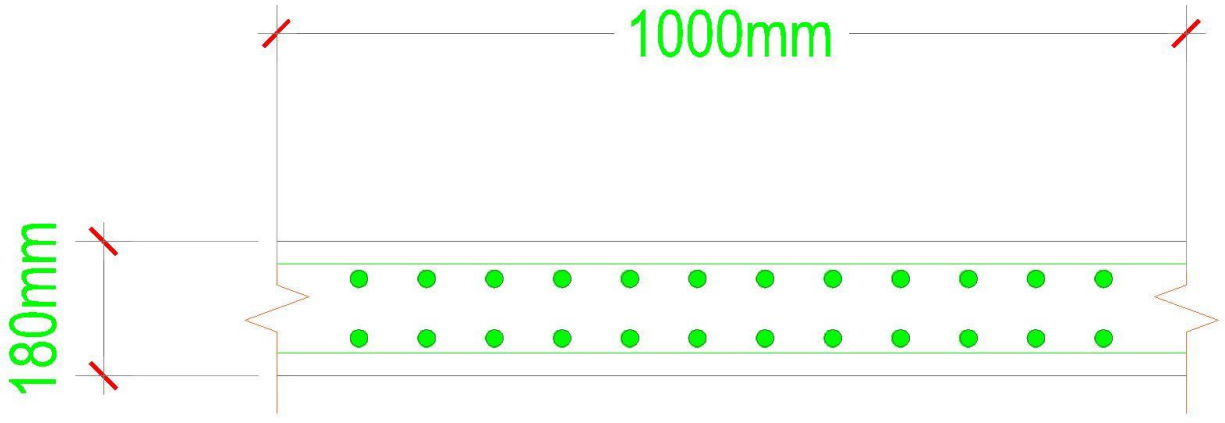
$$A_{s \text{ Min}} < A_s < A_{s \text{ Max}}$$

$$S_{\min} \leq 283.5 \left(\frac{1000}{3316.5} \right)$$

$$S \leq 85.5$$

$$S = 85$$

$$S_{\min} \leq 85 \text{ mm} < S_{\text{Max}}$$



12#19 @ 85mm C/C
Cover = 15mm

Group3. Floor 13- 20:

Slab Reinforcement calculations:

$$M_{\max} = 135.16 \text{ kN.m}$$

$$R_u = \frac{M_{\max}}{0.9 b d^2} = \frac{135.16 (10^6)}{0.9(1000)(165^2)} = 5.516 \text{ N/mm}^2$$

$$\rho = \frac{0.85 (69)}{345} \left(1 - \sqrt{1 - \frac{2 (5.516)}{0.85 (69)}} \right) = 0.0168$$

$$A_{s \text{ required}} = 0.0168 (1000)(165) = 2772 \text{ mm}^2$$

$$\text{Using 11 \# 18 } A_s = 2799.2 \text{ mm}^2$$

$$A_{s \text{ Min}} = 380 \text{ mm}^2$$

$$A_{s \text{ Max}} = 8679 \text{ mm}^2$$

$$A_{s \text{ Min}} < A_s < A_{s \text{ Max}}$$

$$S_{\min} \leq 254.5 \left(\frac{1000}{2772} \right)$$

$$S \leq 97.81$$

$$S = 95$$

$$S \leq 95 \text{ mm} < S_{\text{Max}}$$



11#18 @ 95mm C/C
Cover = 15mm

Group4. Floor 21- 28:

Slab Reinforcement calculations:

$$M_{\max} = 182.08 \text{ kN.m}$$

$$R_u = \frac{M_{\max}}{0.9 b d^2} = \frac{182.08 (10^6)}{0.9(1000)(165^2)} = 7.43 \text{ N/mm}^2$$

$$\rho = \frac{0.85 (69)}{345} \left(1 - \sqrt{1 - \frac{2 (7.43)}{0.85 (69)}} \right) = 0.023$$

$$A_{s \text{ required}} = 0.023 (1000)(165) = 3795 \text{ mm}^2$$

$$\text{Using } 10 \# 22 A_s = 3801.3 \text{ mm}^2$$

$$A_{s \text{ Min}} = 380 \text{ mm}^2$$

$$A_{s \text{ Max}} = 8679 \text{ mm}^2$$

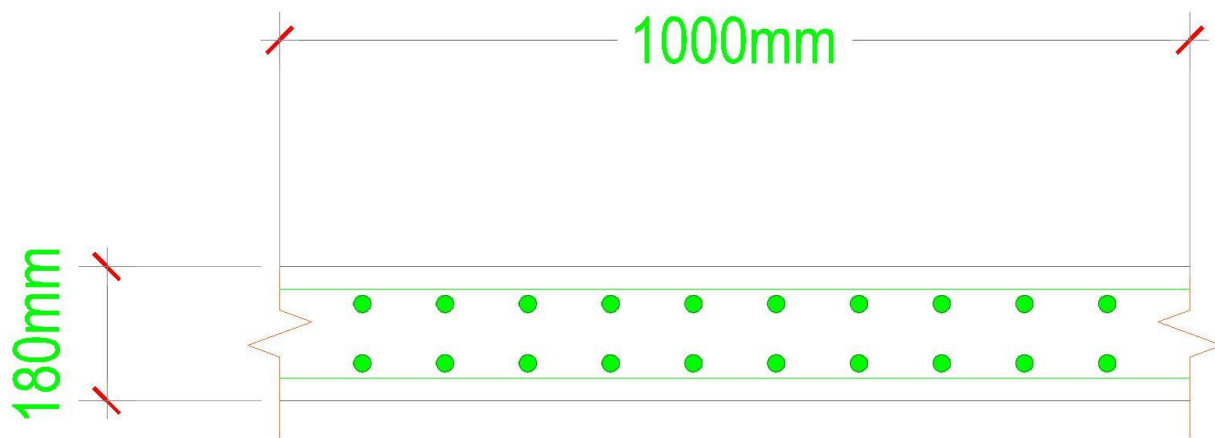
$$A_{s \text{ Min}} < A_s < A_{s \text{ Max}}$$

$$S_{\min} \leq 380.1 \left(\frac{1000}{3795} \right)$$

$$S \leq 106.9$$

$$S = 105$$

$$S \leq 105 \text{ mm} < S_{\text{Max}}$$



10#22 @ 105mm C/C

Cover = 15mm

Group5. Floor 29- 36 & Floor 45- 52:

Slab Reinforcement calculations:

$$M_{\max} = 146.36 \text{ kN.m}$$

$$R_u = \frac{M_{\max}}{0.9 b d^2} = \frac{146.36 (10^6)}{0.9(1000)(165^2)} = 5.97 \text{ N/mm}^2$$

$$\rho = \frac{0.85 (69)}{345} \left(1 - \sqrt{1 - \frac{2 (5.97)}{0.85 (69)}} \right) = 0.018$$

$$A_{s \text{ required}} = 0.018(1000)(165) = 2970 \text{ mm}^2$$

$$\text{Using } 12 \# 18 A_s = 3054 \text{ mm}^2$$

$$A_{s \text{ Min}} = 380 \text{ mm}^2$$

$$A_{s \text{ Max}} = 8679 \text{ mm}^2$$

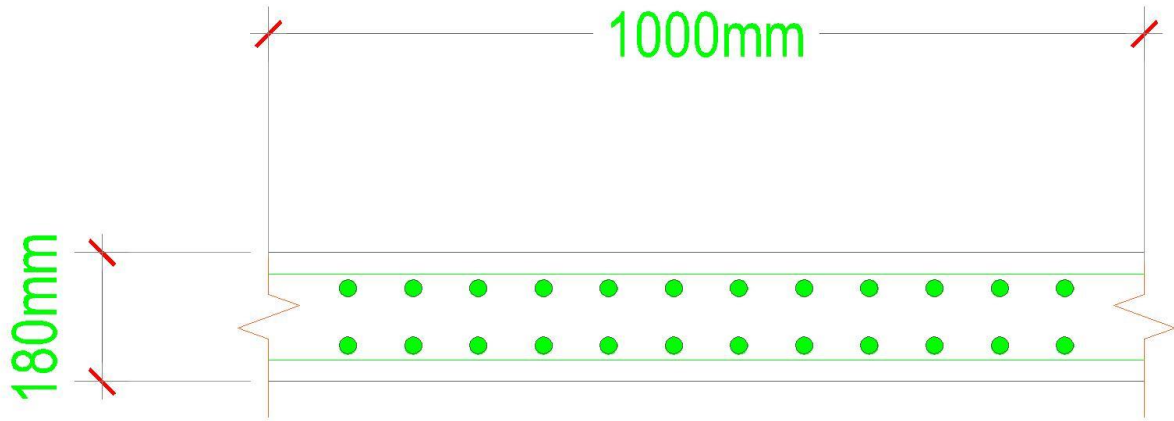
$$A_{s \text{ Min}} < A_s < A_{s \text{ Max}}$$

$$S_{\min} \leq 254.5 \left(\frac{1000}{2970} \right)$$

$$S \leq 85.7$$

$$S = 85$$

$$S \leq 85 \text{ mm} < S_{\text{Max}}$$



12#18 @ 85mm C/C
Cover = 15mm

Group6. Floor 37- 44:

Slab Reinforcement calculations:

$$M_{\max} = 120.42 \text{ kN.m}$$

$$R_u = \frac{M_{\max}}{0.9 b d^2} = \frac{120.42 (10^6)}{0.9(1000)(165^2)} = 4.91 \text{ N/mm}^2$$

$$\rho = \frac{0.85 (69)}{345} \left(1 - \sqrt{1 - \frac{2 (4.91)}{0.85 (69)}} \right) = 0.015$$

$$A_{s \text{ required}} = 0.015(1000)(165) = 2475 \text{ mm}^2$$

$$\text{Using } 10 \# 18 A_s = 2545 \text{ mm}^2$$

$$A_{s \text{ Min}} = 380 \text{ mm}^2$$

$$A_{s \text{ Max}} = 8679 \text{ mm}^2$$

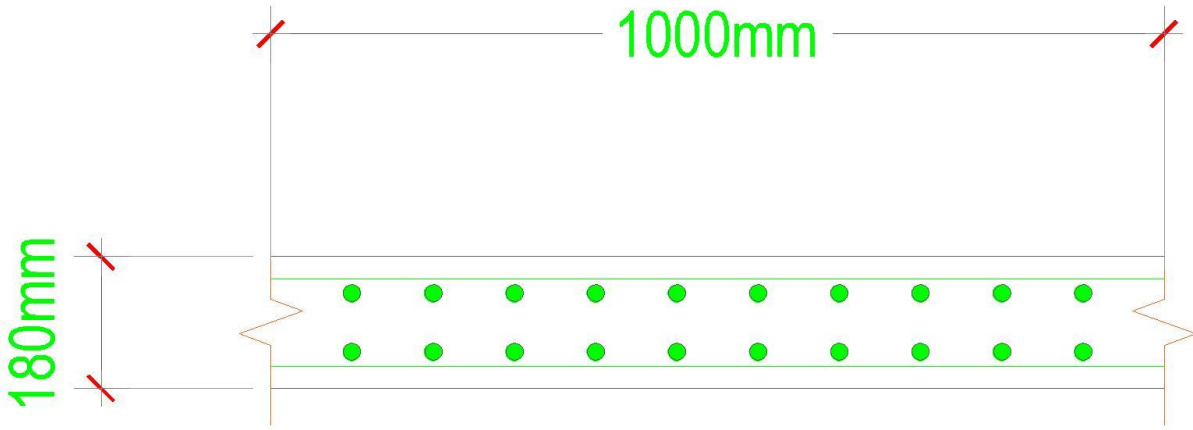
$$A_{s \text{ Min}} < A_s < A_{s \text{ Max}}$$

$$S_{\min} \leq 254.5 \left(\frac{1000}{2475} \right)$$

$$S \leq 106.7$$

$$S = 105$$

$$S \leq 105 \text{ mm} < S_{\text{Max}}$$



10#18 @ 105mm C/C
Cover = 15mm

Group7. Floor 53- 56:

Slab Reinforcement calculations:

$$M_{\max} = 111.18 \text{ kN.m}$$

$$R_u = \frac{M_{\max}}{0.9 b d^2} = \frac{111.18 (10^6)}{0.9(1000)(165^2)} = 4.54 \text{ N/mm}^2$$

$$\rho = \frac{0.85 (69)}{345} \left(1 - \sqrt{1 - \frac{2 (4.54)}{0.85 (69)}} \right) = 0.014$$

$$A_{s \text{ required}} = 0.014(1000)(165) = 2310 \text{ mm}^2$$

$$\text{Using } 12 \# 16 A_s = 2413 \text{ mm}^2$$

$$A_{s \text{ Min}} = 380 \text{ mm}^2$$

$$A_{s \text{ Max}} = 8679 \text{ mm}^2$$

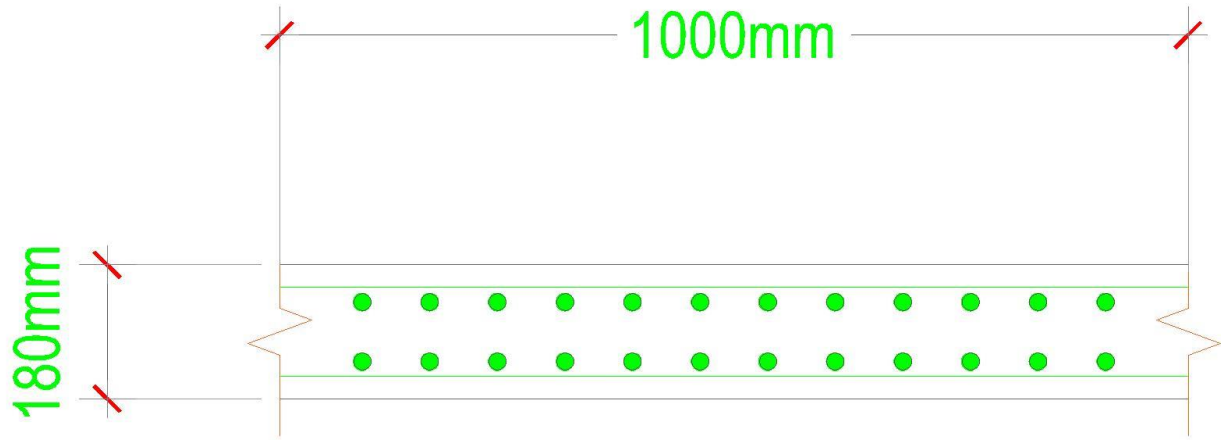
$$A_{s \text{ Min}} < A_s < A_{s \text{ Max}}$$

$$S_{\min} \leq 201.1 \left(\frac{1000}{2310} \right)$$

$$S \leq 87.06$$

$$S = 85$$

$$S_{\min} \leq 85 \text{ mm} < S_{\text{Max}}$$



12#16 @ 85mm C/C
Cover = 15mm

Group8. Floor 57- 60:

Slab Reinforcement calculations:

$$M_{\max} = 128.85 \text{ kN.m}$$

$$R_u = \frac{M_{\max}}{0.9 b d^2} = \frac{(128.85) (10^6)}{0.9(1000)(165^2)} = 5.26 \text{ N/mm}^2$$

$$\rho = \frac{0.85 (69)}{345} \left(1 - \sqrt{1 - \frac{2 (5.26)}{0.85 (69)}} \right) = 0.016$$

$$A_{s \text{ required}} = 0.016(1000)(165) = 2640 \text{ mm}^2$$

$$\text{Using } 10 \# 19 A_s = 2835.3 \text{ mm}^2$$

$$A_{s \text{ Min}} = 380 \text{ mm}^2$$

$$A_{s \text{ Max}} = 8679 \text{ mm}^2$$

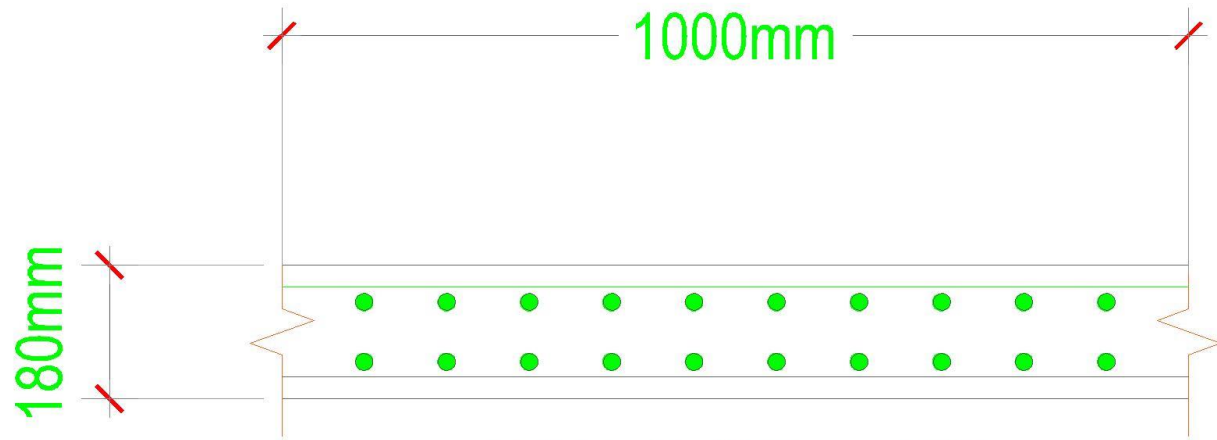
$$A_{s \text{ Min}} < A_s < A_{s \text{ Max}}$$

$$S_{\min} \leq 283.5 \left(\frac{1000}{2640} \right)$$

$$S \leq 107.4$$

$$S = 105$$

$$S_{\min} \leq 105 \text{ mm} < S_{\text{Max}}$$



10#19 @ 105mm C/C
Cover = 15mm

Model with RCC slab floor system

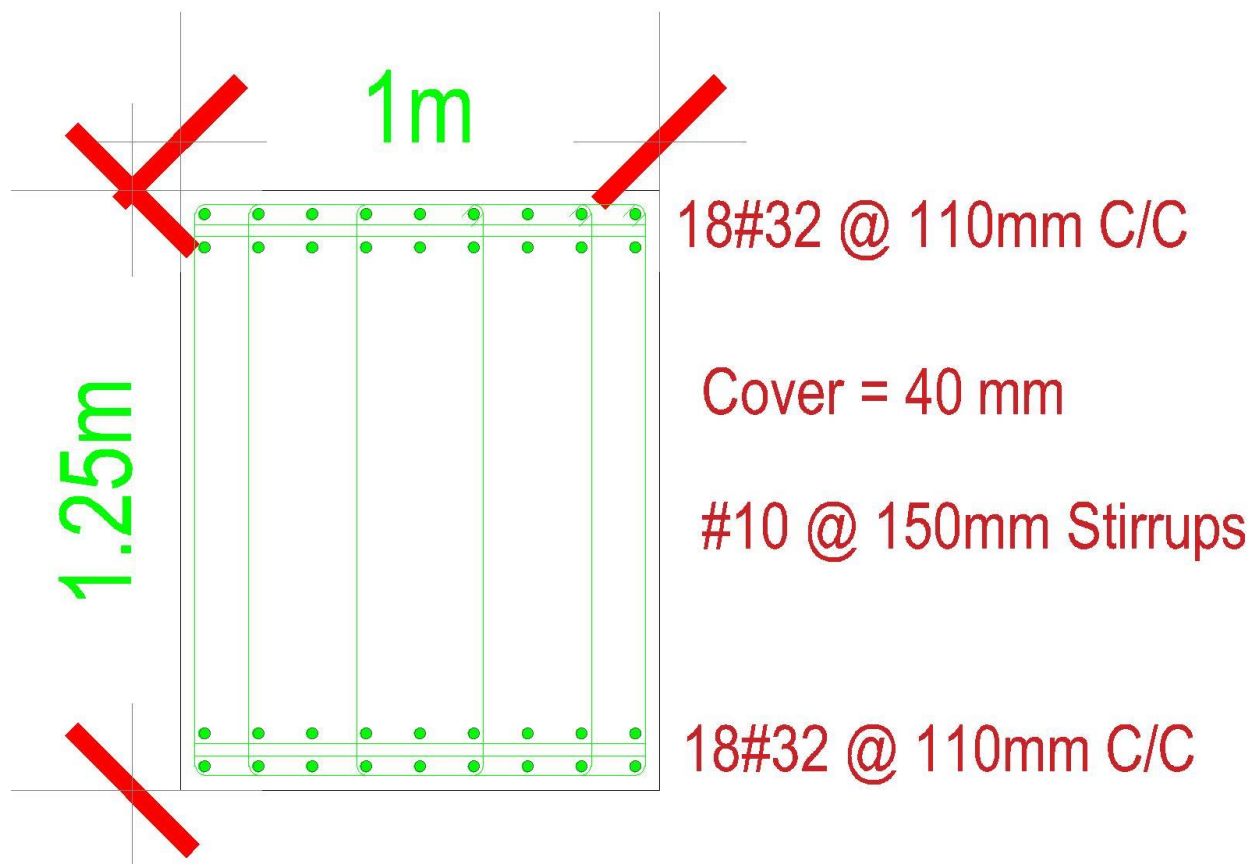
Storey levels	Slab Reinforcement
1-4	12 # 20 → $A_s = 3770 \text{ mm}^2$
5-12	12 # 19 → $A_s = 3402 \text{ mm}^2$
13-20	11 # 18 → $A_s = 2799 \text{ mm}^2$
21-28	10 # 22 → $A_s = 3801.3 \text{ mm}^2$
29-36	12 # 18 → $A_s = 3054 \text{ mm}^2$
37-44	10 # 18 → $A_s = 2545 \text{ mm}^2$
45-52	12 # 18 → $A_s = 3054 \text{ mm}^2$
53-56	12 # 16 → $A_s = 2413 \text{ mm}^2$
57-60	10 # 19 → $A_s = 2835 \text{ mm}^2$

4.2 Steel Reinforcement of Concrete Beams:

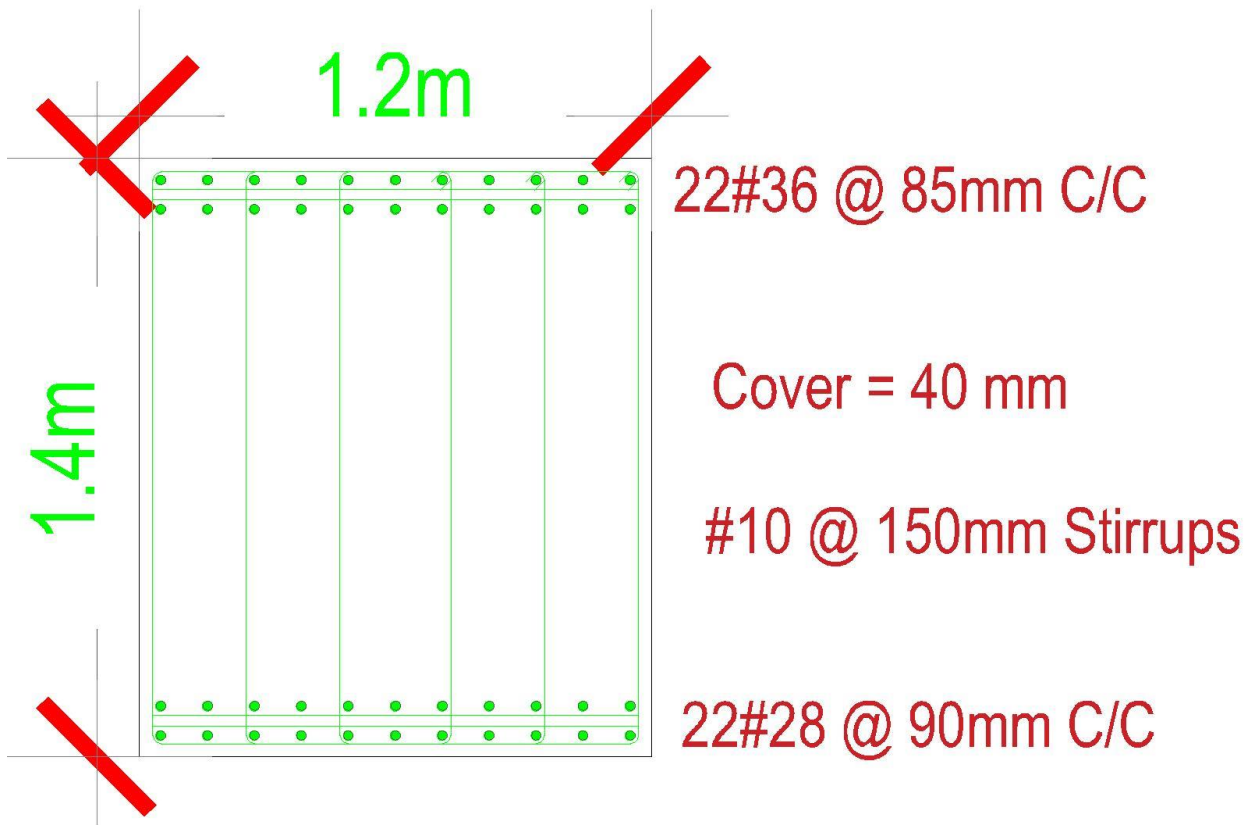
4.2.1 Steel Reinforcement of Concrete Beams for RCC model:

We designed the two cross section based on three live loads (5, 7.5, 10 kN/m^2). We selected four different spans as per live load to have an idea about the steel required in different span length. We select the shortest beam, the longest beam and two beams in between. EATBS analysis provide us with area of steel required. For each live load we have four spans. We took the maximum required steel reinforcements top and bottom from the four spans. then we create our own section for a specific live load. And then same process for the other two live loads.

- Floors with 10 kN/m^2 :
 - Cross section: 1m X 1.25m

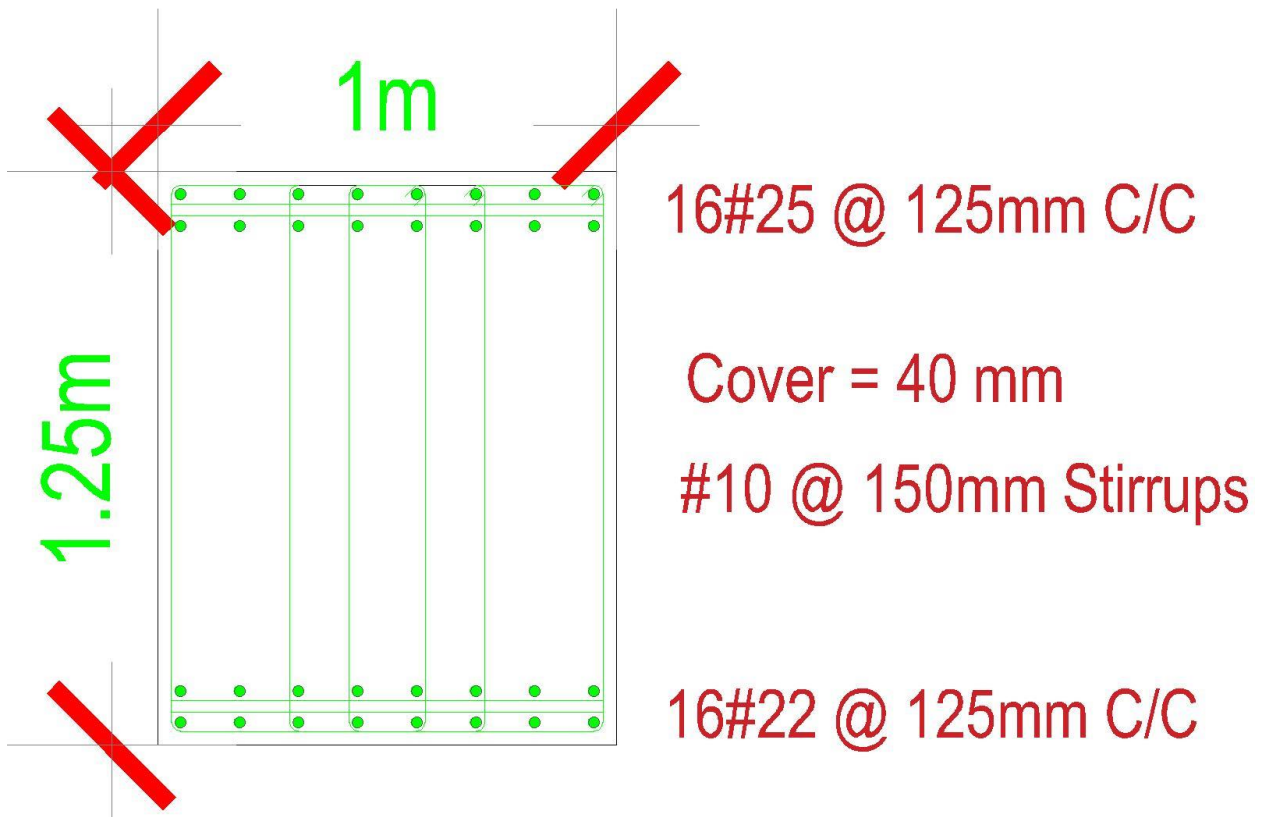


➤ Cross section: 1.2m X 1.4m

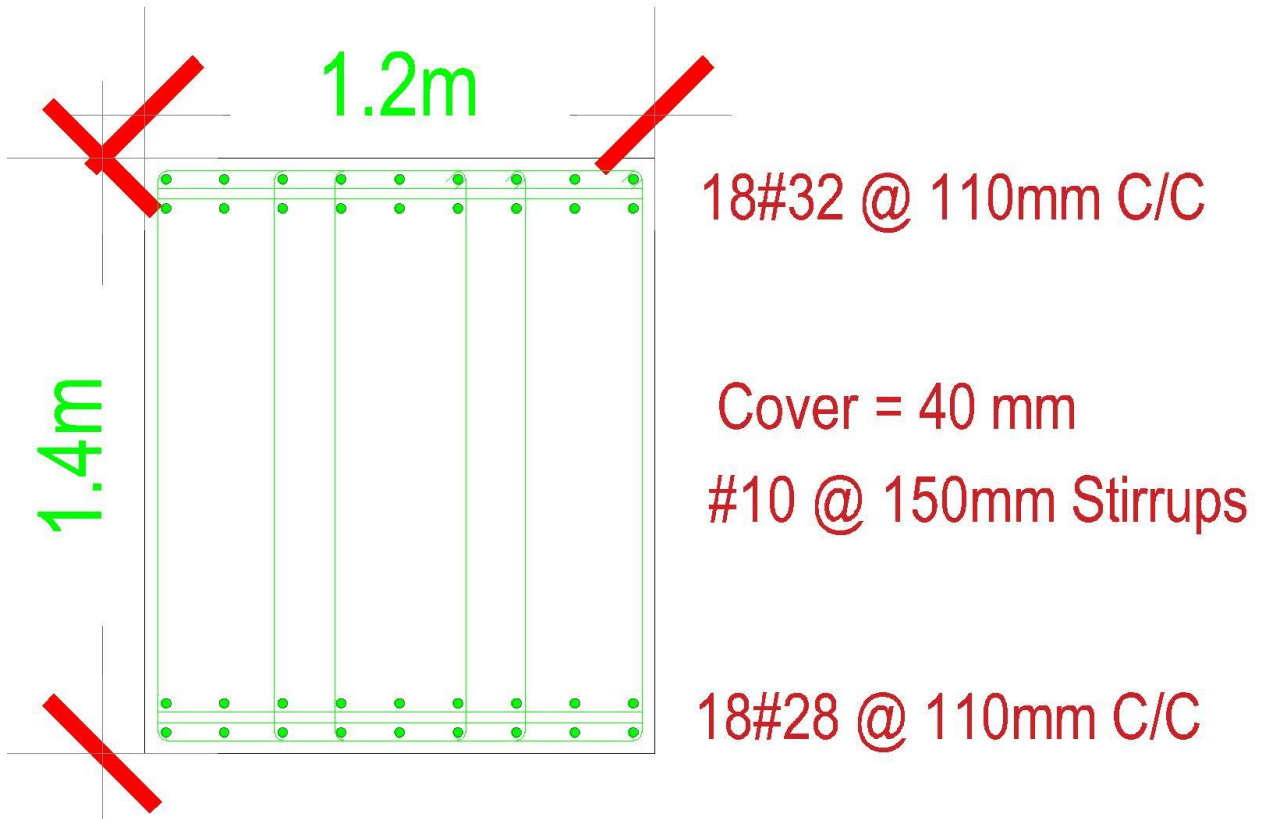


- Floors with 7.5 kN/m^2 :

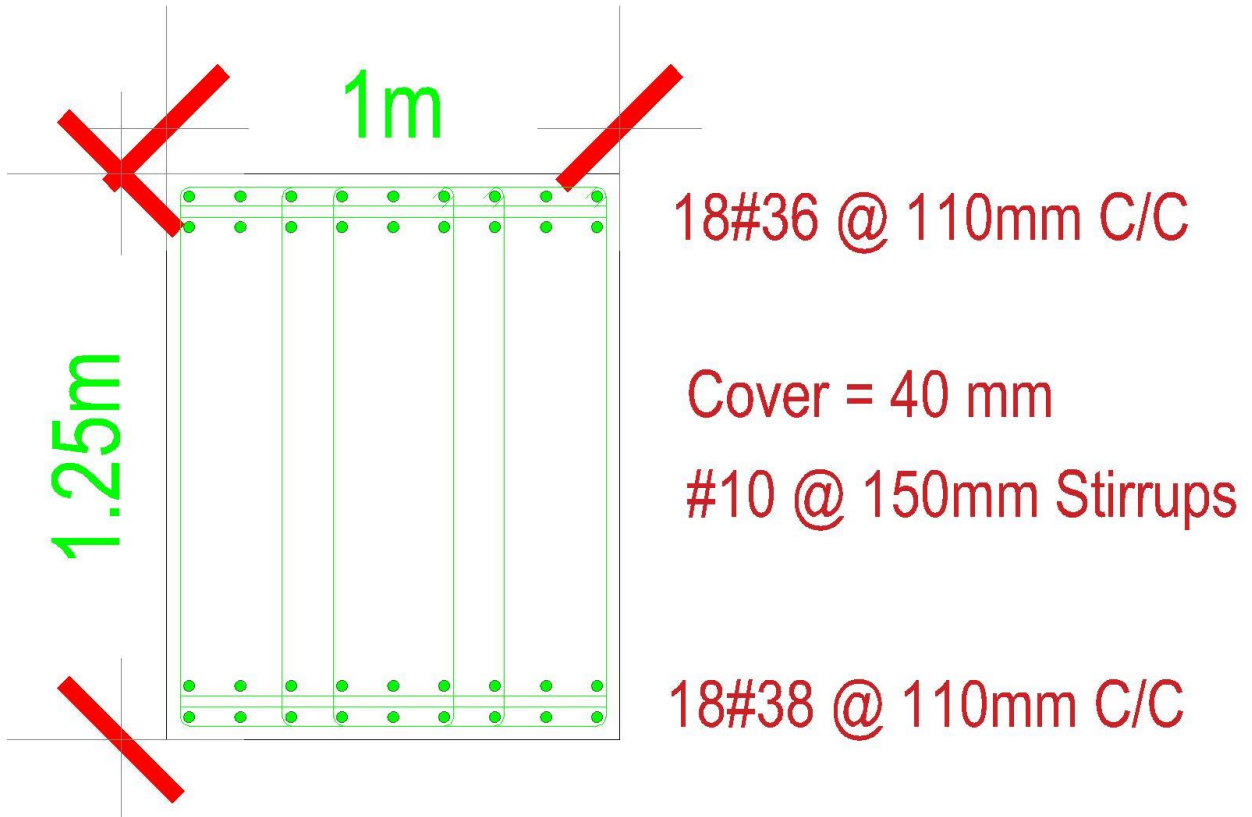
➤ Cross section: 1m X 1.25m



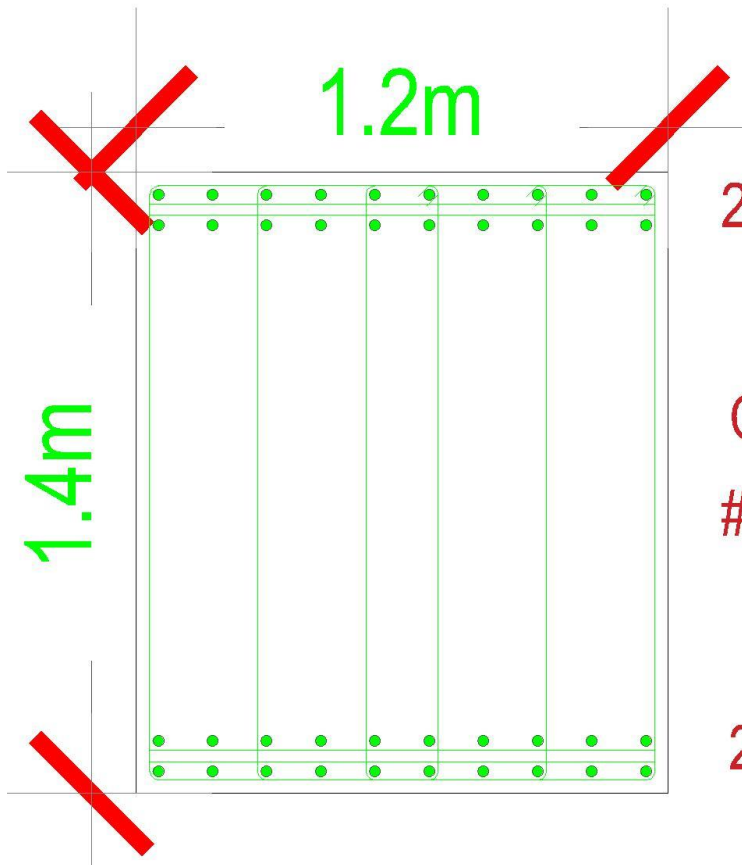
➤ Cross section: 1.2m X 1.4m



- Floors with 5 kN/m^2 :
 - Cross section: $1\text{m} \times 1.25\text{m}$



➤ Cross section: 1.2m X 1.4m



20#38 @ 95mm C/C

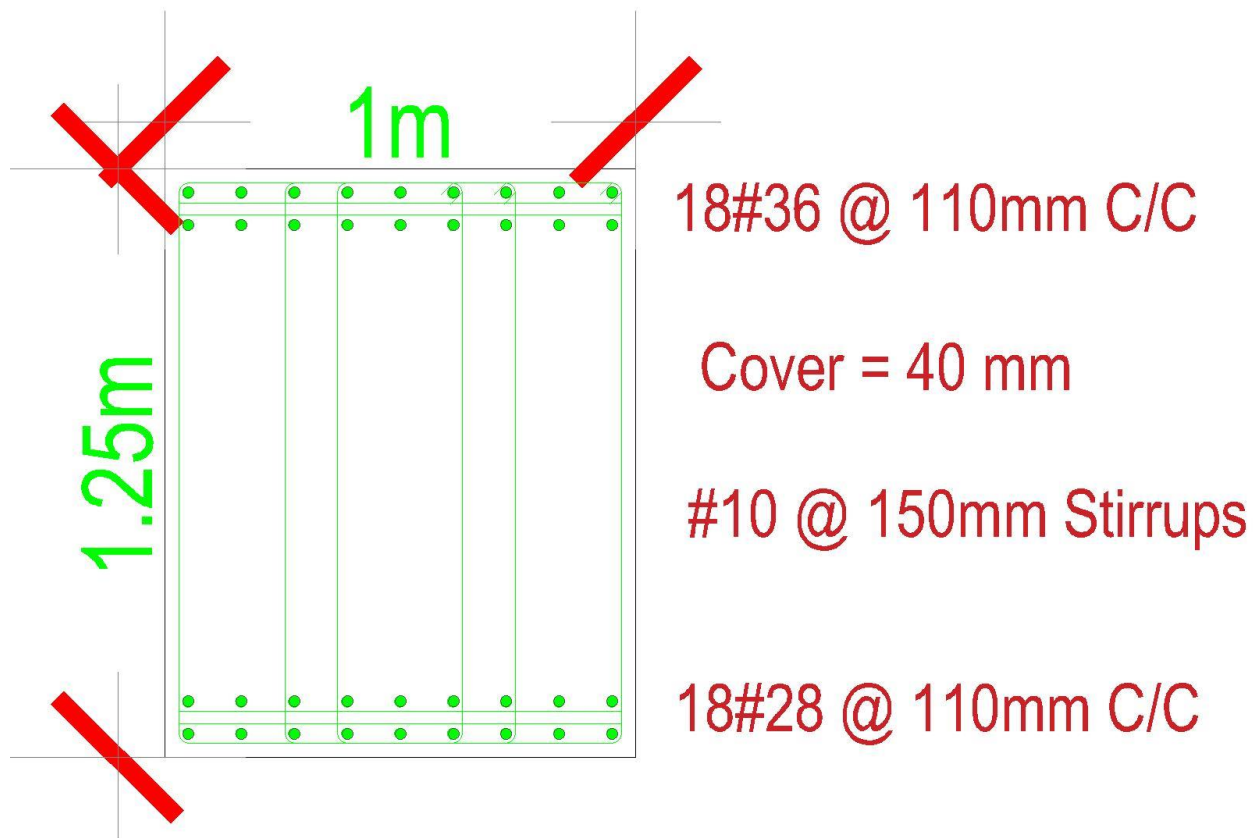
Cover = 40 mm

#10 @ 150mm Stirrups

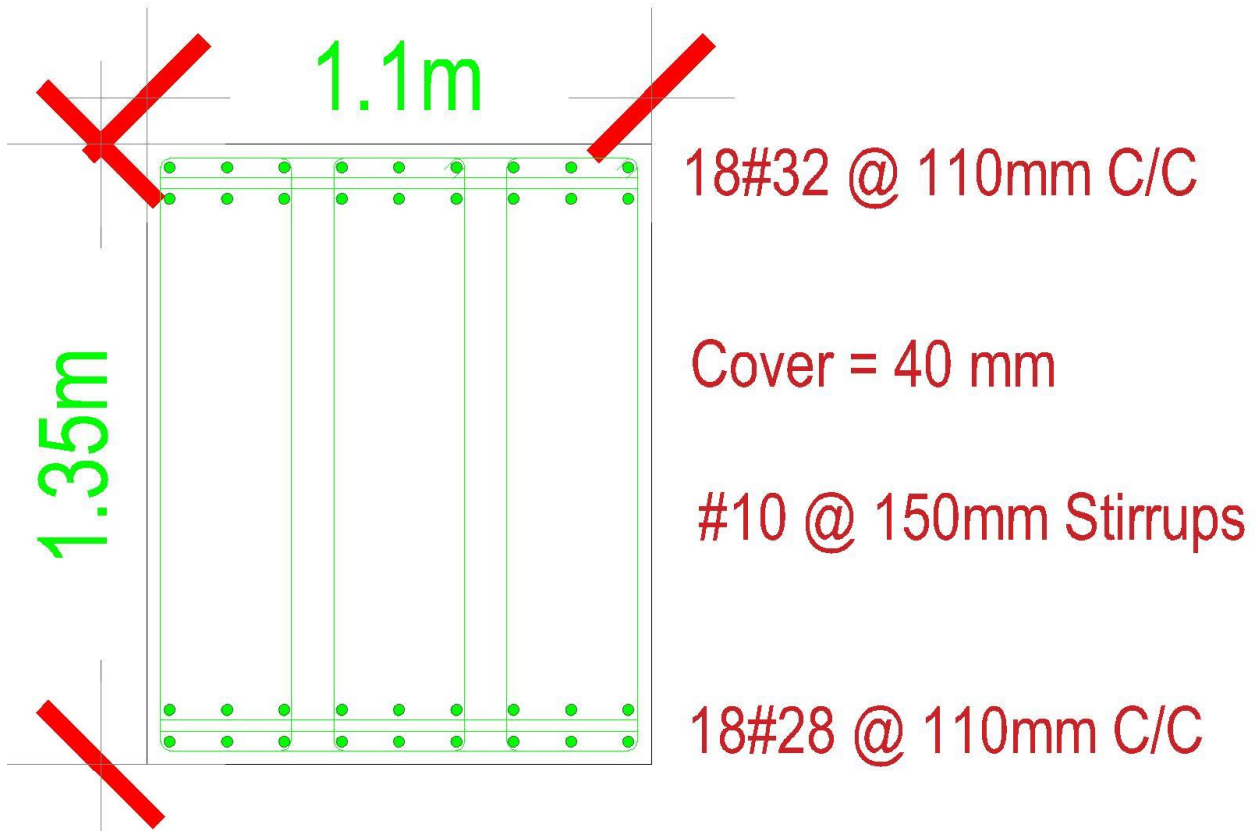
20#40 @ 95mm C/C

4.2.2 Steel Reinforcement of Concrete Beams for CLT model:

- Floors with 10 kN/m^2 :
 - Cross section: $1\text{m} \times 1.25\text{m}$

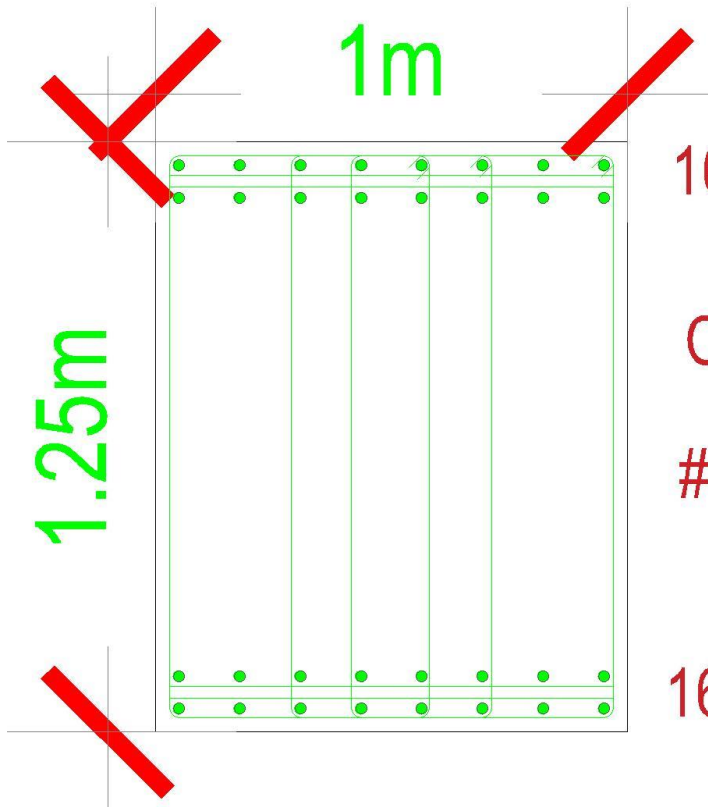


➤ Cross section: 1.1m X 1.35m



- Floors with 7.5 kN/m^2 :

➤ Cross section: $1\text{m} \times 1.25\text{m}$



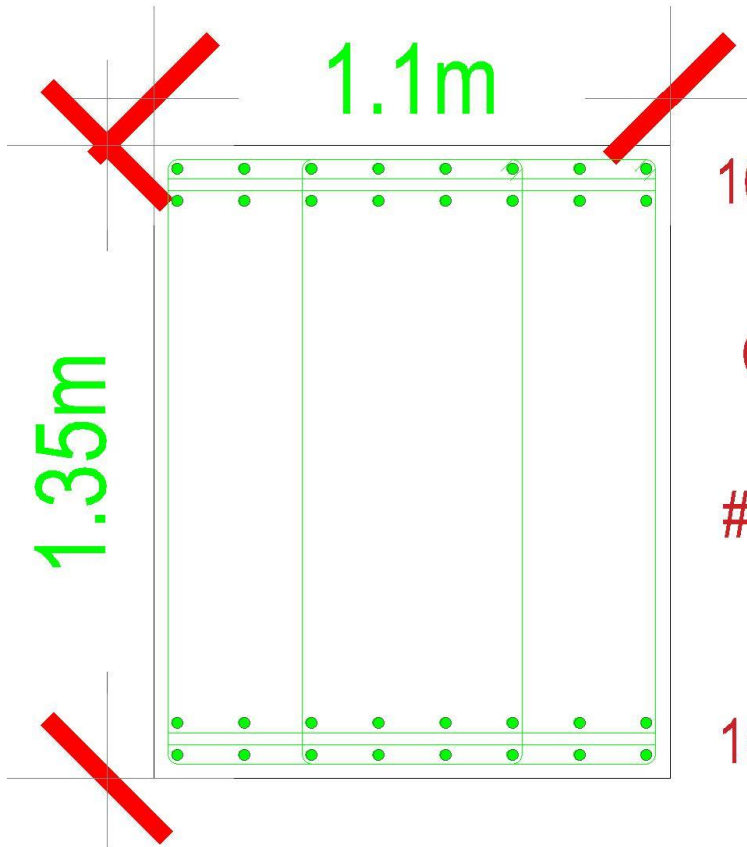
16#25 @ 125mm C/C

Cover = 40 mm

#10 @ 150mm Stirrups

16#20 @ 125mm C/C

➤ Cross section: 1.1m X 1.35m



16#28 @ 125mm C/C

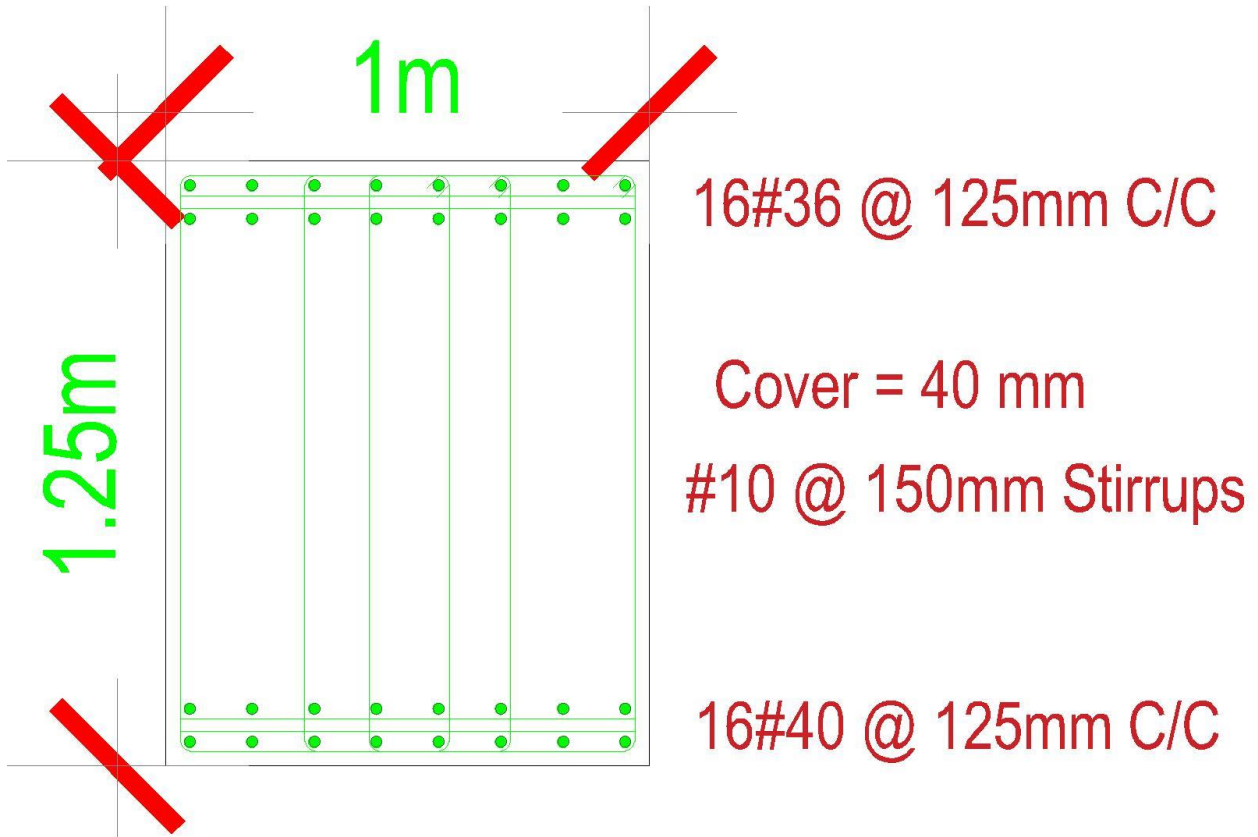
Cover = 40 mm

#10 @ 150mm Stirrups

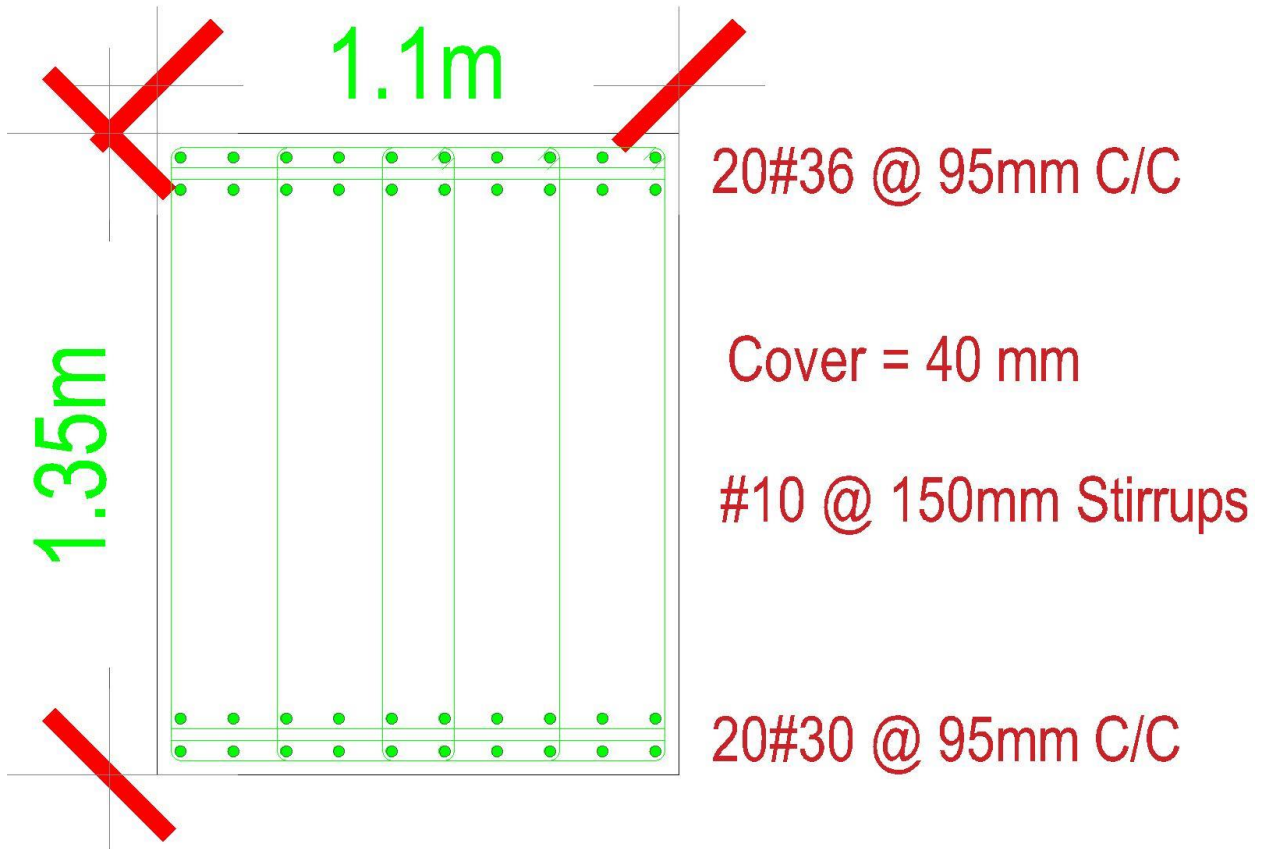
16#28 @ 125mm C/C

- Floors with 5 kN/m^2 :

➤ Cross section: $1\text{m} \times 1.25\text{m}$



➤ Cross section: 1.1m X 1.35m



4.3 Steel Reinforcement of columns:

4.3.1 Steel Reinforcement of columns for RCC model:

- Cross section: 3.2m x 3.2m

$$A_g = 3200 \times 3200 = 10240000 \text{ mm}^2$$

$$A_{s,req} = 102400 \text{ mm}^2 \text{ (from ETABS)}$$

Assumed quantity of bars: 31 bars in width.

29 bars in length.

$$\text{Total quantity} = 2(31) + 2(29) = 120 \text{ bars}$$

$$\text{Area of one bar required} = \frac{102400}{120} = 853.33 \text{ mm}^2$$

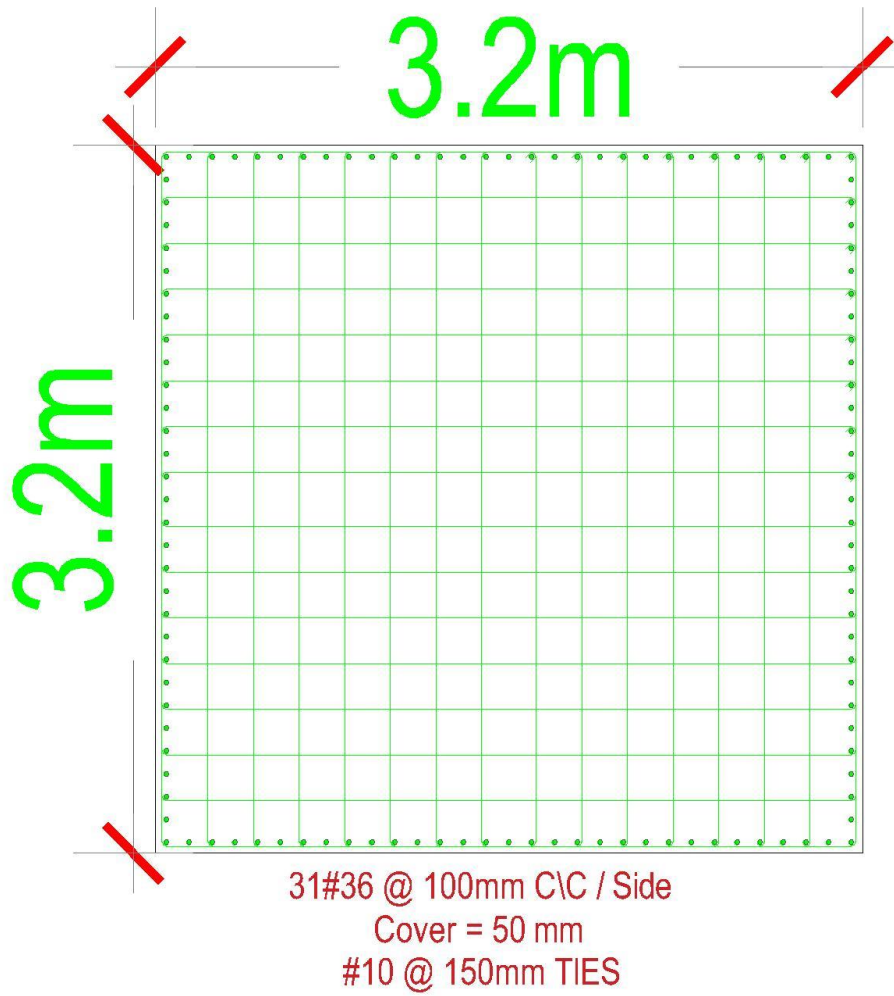
Choose #36 bar because it's area larger than 853.33 mm²

$$\frac{36^2 \pi}{4} = 1017.36 \text{ mm}^2 > 853.33 \text{ mm}^2$$

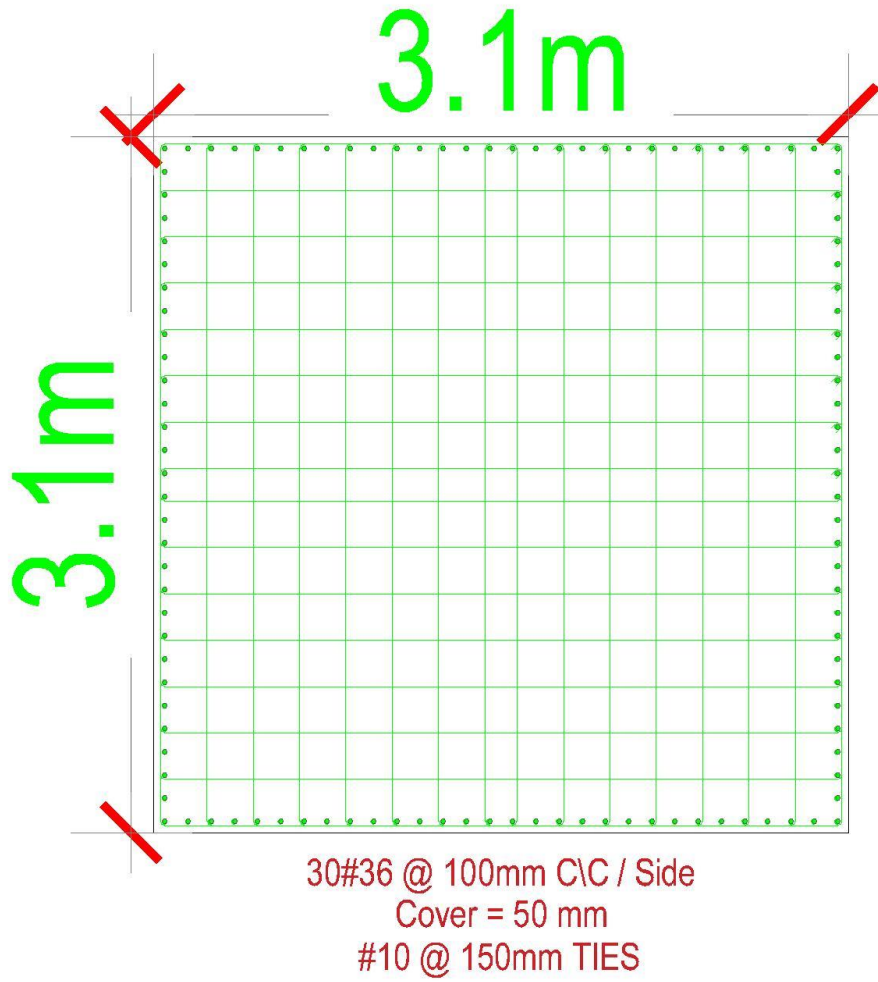
$$A_{s,provide} = 120(1017.36) = 122083.2 \text{ mm}^2 > 102400 \text{ mm}^2$$

$$P = \frac{122083.2}{10240000} = 0.0119 = 1.19\% \text{ (within the range 1-8\%)}$$

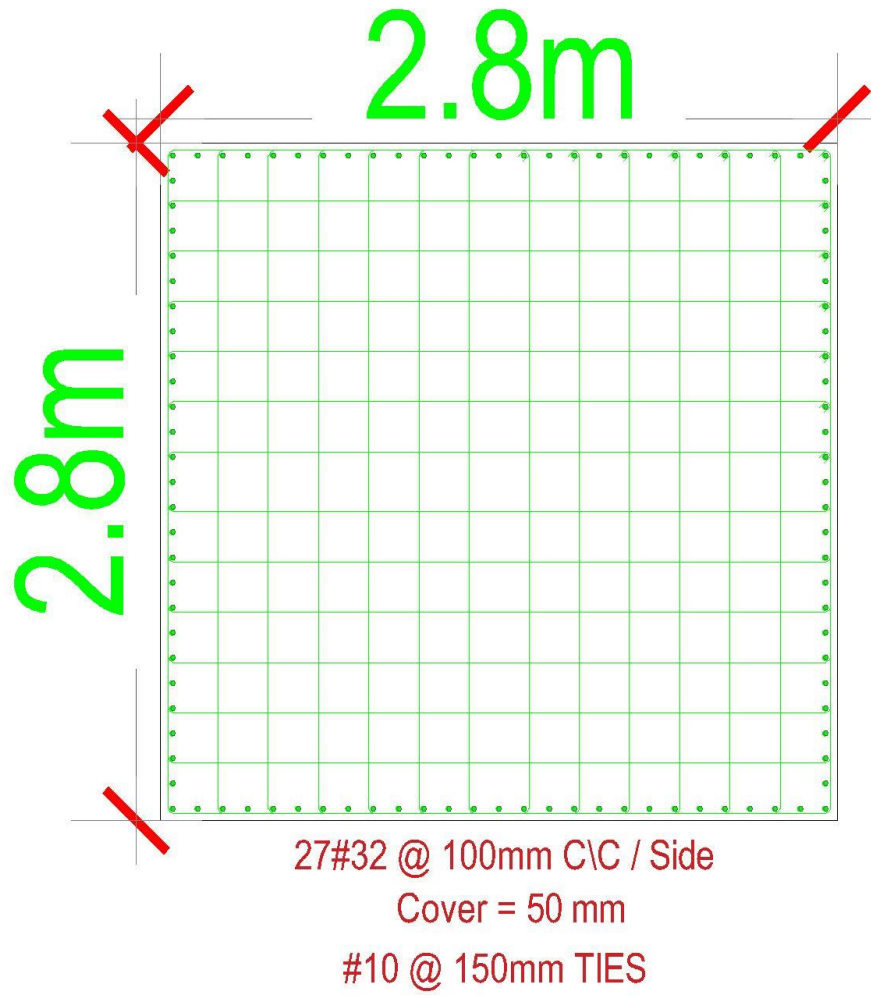
➤ Cross section 3.2m x 3.2m



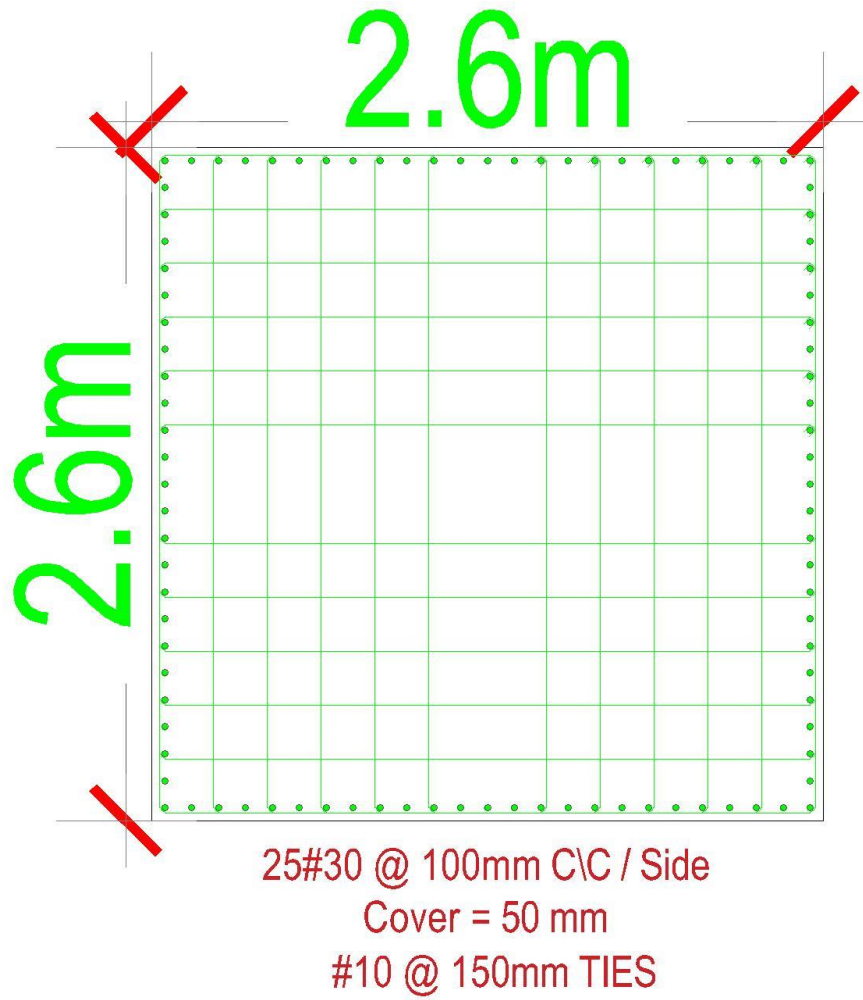
➤ Cross section 3.1m x 3.1m



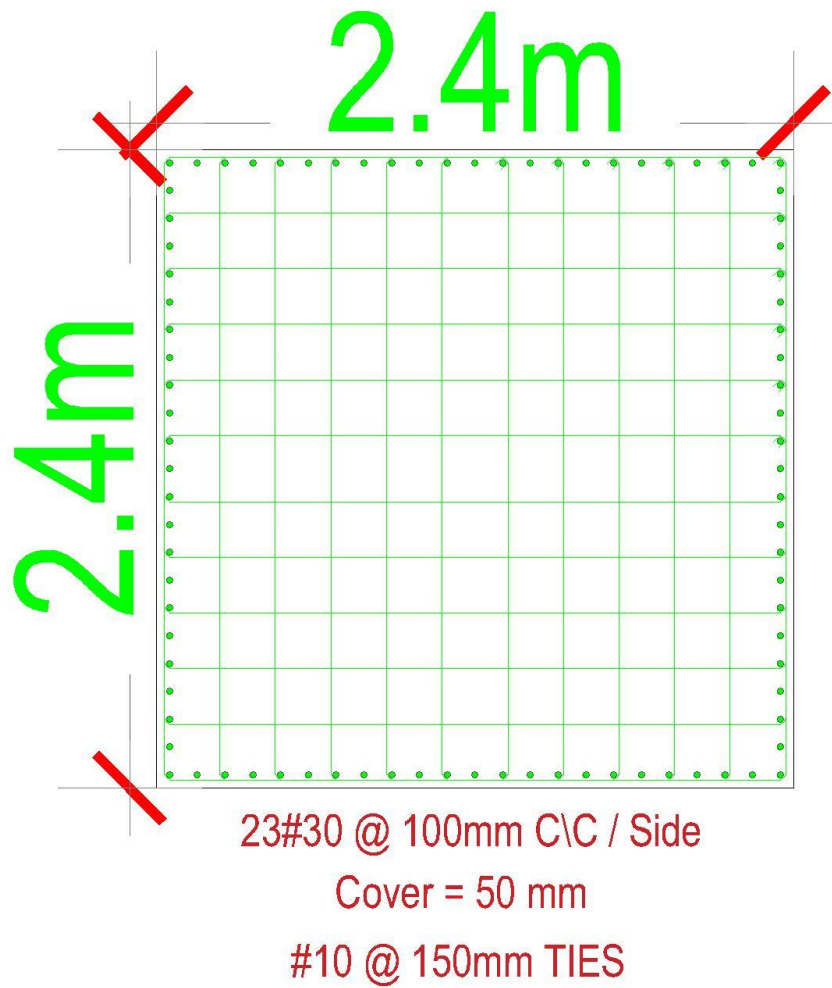
➤ Cross section 2.8m x 2.8m



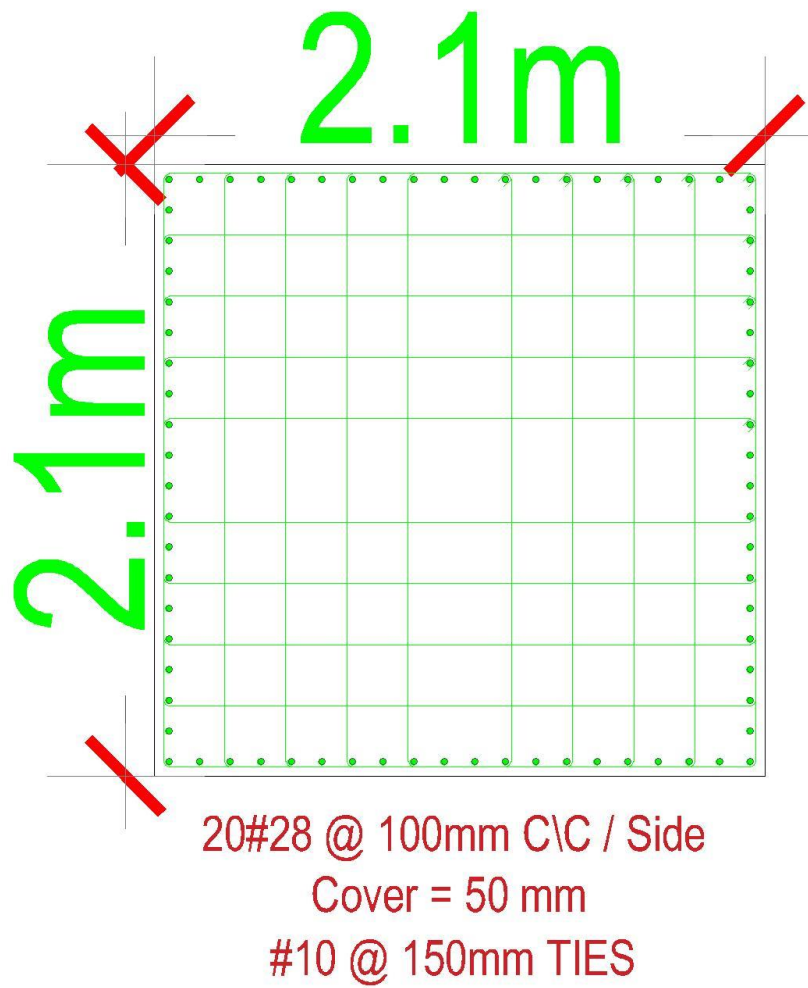
➤ Cross section 2.6m x 2.6m



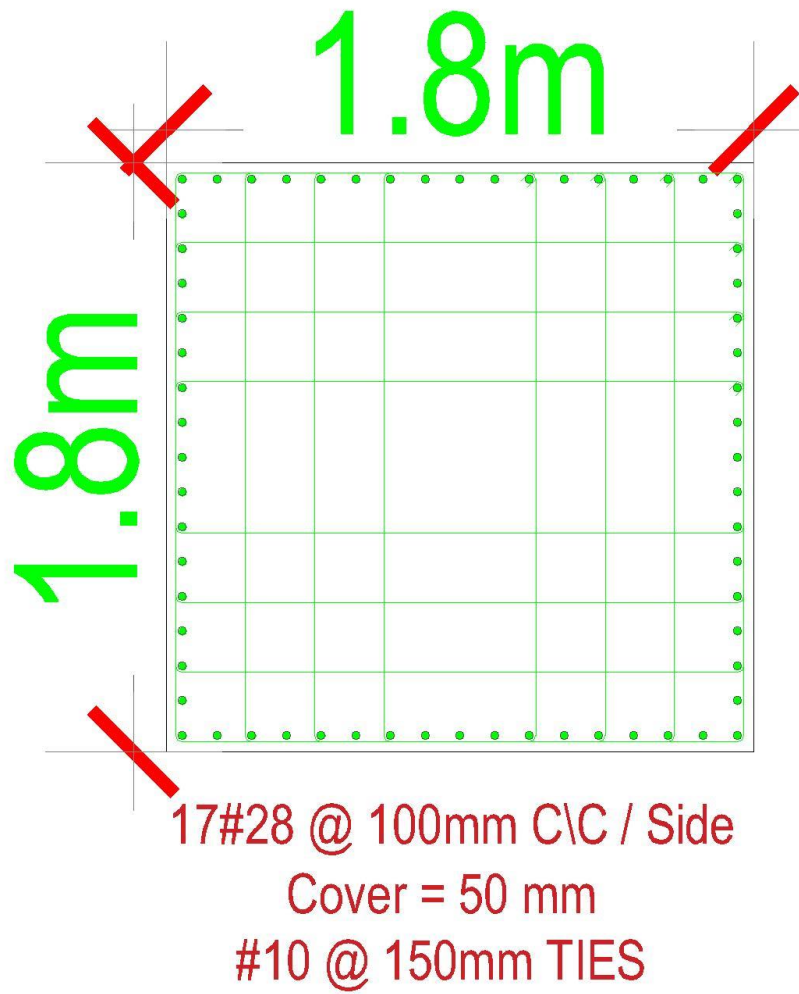
- Cross section 2.4m x 2.4m



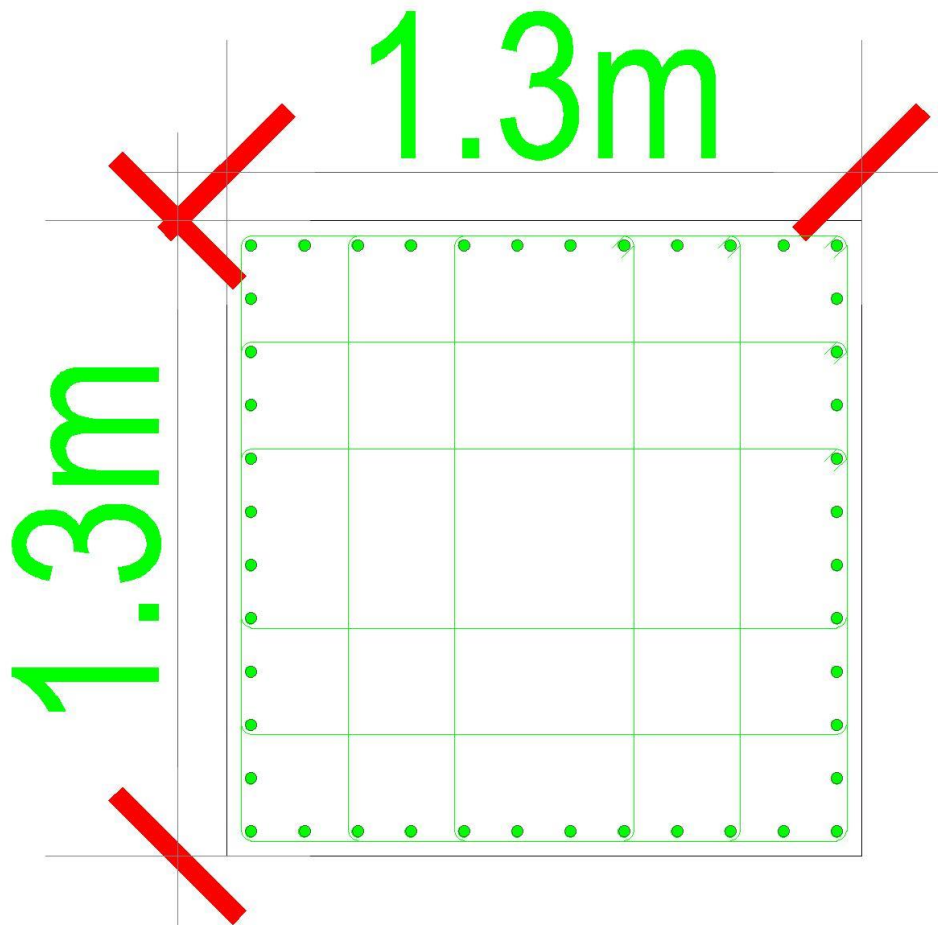
➤ Cross section 2.1m x 2.1m



➤ Cross section 1.8m x 1.8m



➤ Cross section 1.3m x 1.3m

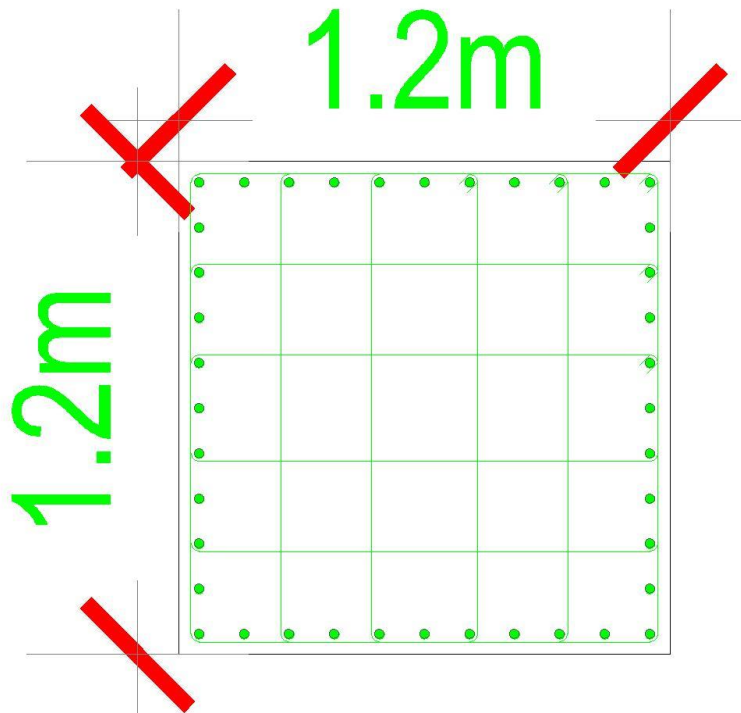


12#25 @ 100mm C\C / Side

Cover = 50 mm

#10 @ 150mm TIES

- Cross section 1.2m x 1.2m

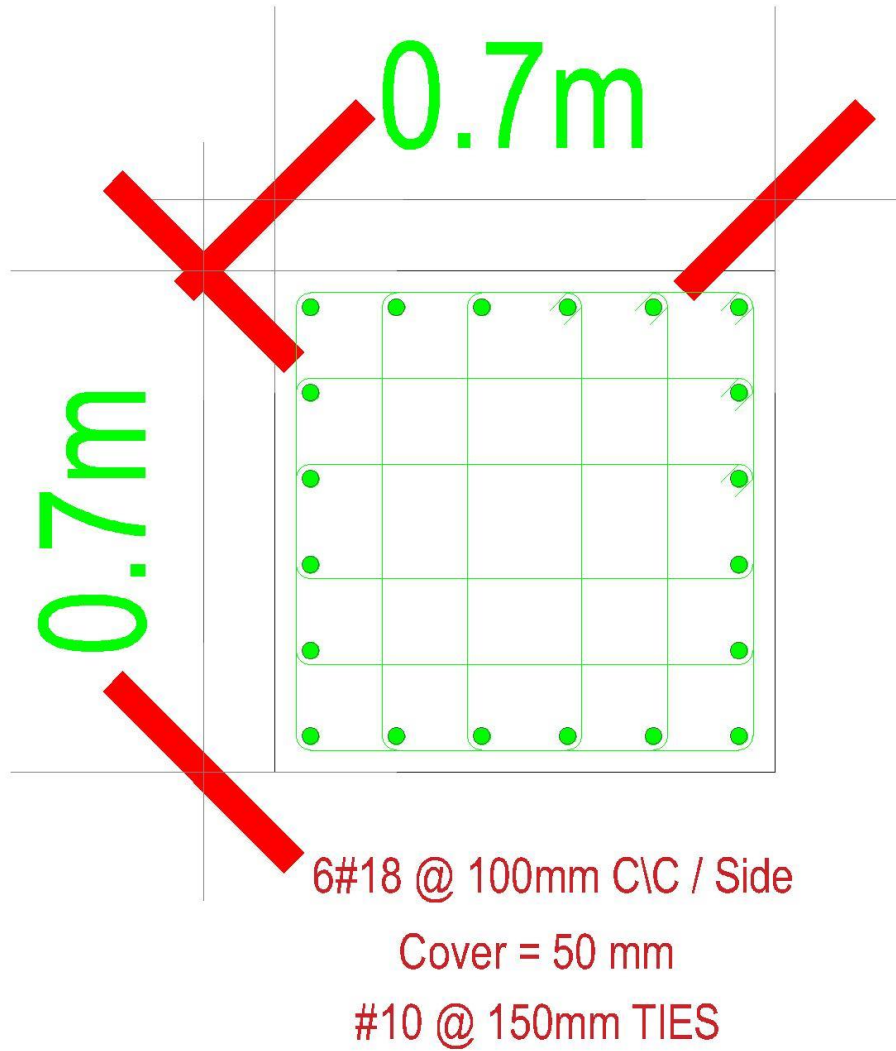


11#22 @ 100mm C\C / Side

Cover = 50 mm

#10 @ 150mm TIES

- Cross section 0.7m x 0.7m



4.3.2 Steel Reinforcement of columns for CLT model:

- Cross section: 2.9m x 2.9m

$$A_g = 2900 \times 2900 = 8410000 \text{ mm}^2$$

$$A_{s,req} = 84100 \text{ mm}^2 \text{ (From ETABS)}$$

Assumed quantity of bars: 28 bars in width.

26 bars in length.

$$\text{Total quantity} = 2(28) + 2(26) = 108 \text{ bars}$$

$$\text{Area of one bar required} = \frac{84100}{108} = 778.7 \text{ mm}^2$$

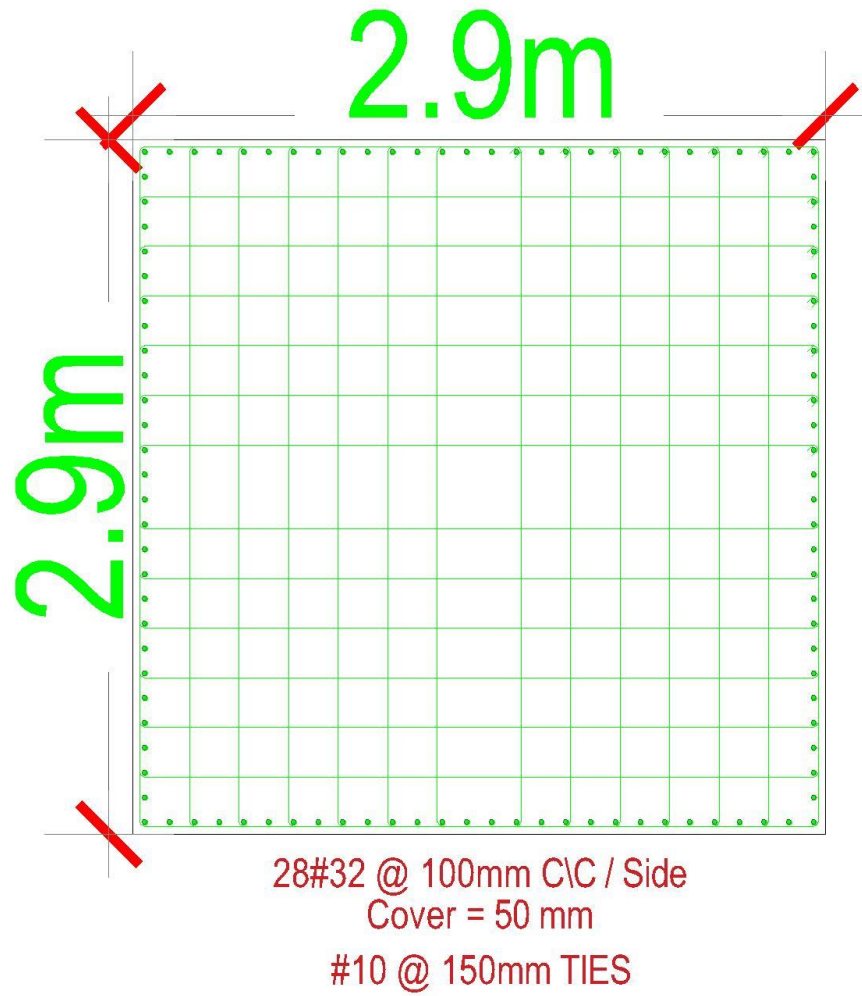
Choose #32 bar because it's area larger than 778.7 mm²

$$\frac{32^2 \pi}{4} = 803.84 \text{ mm}^2 > 778.7 \text{ mm}^2$$

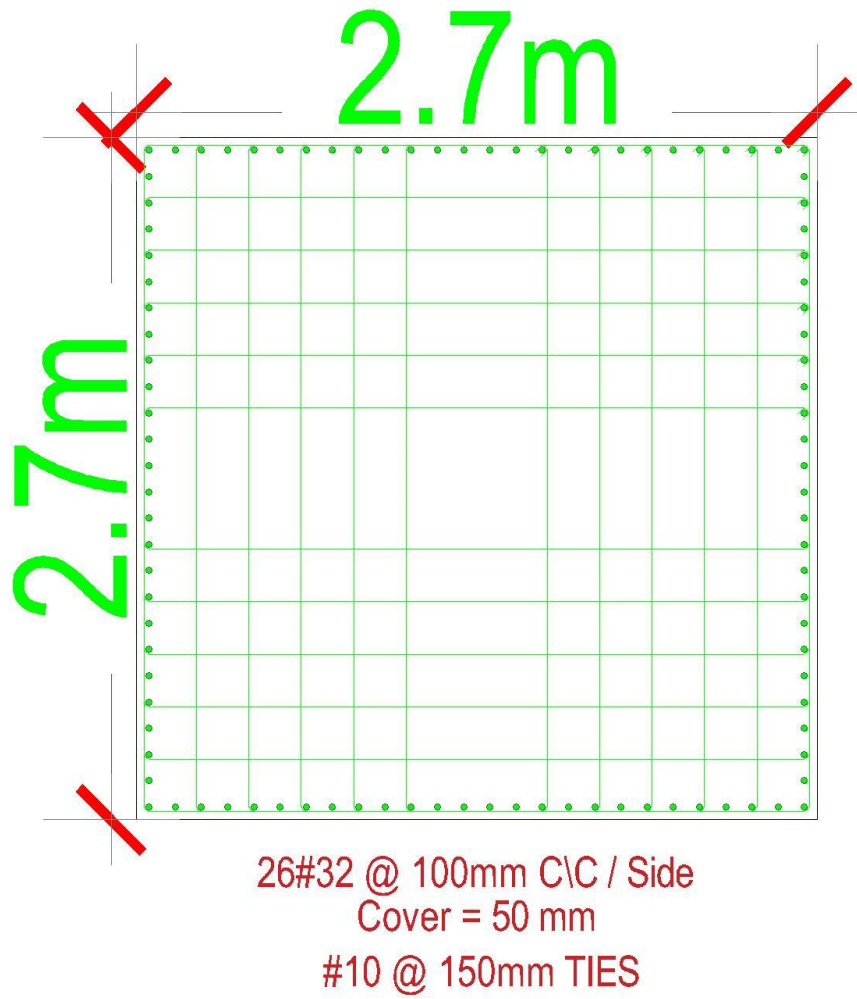
$$A_{s,provide} = 108(803.84) = 86814.72 \text{ mm}^2 > 84100 \text{ mm}^2$$

$$P = \frac{86814.72}{8410000} = 0.0119 = 1.19\% \text{ (within the range 1-8%)}$$

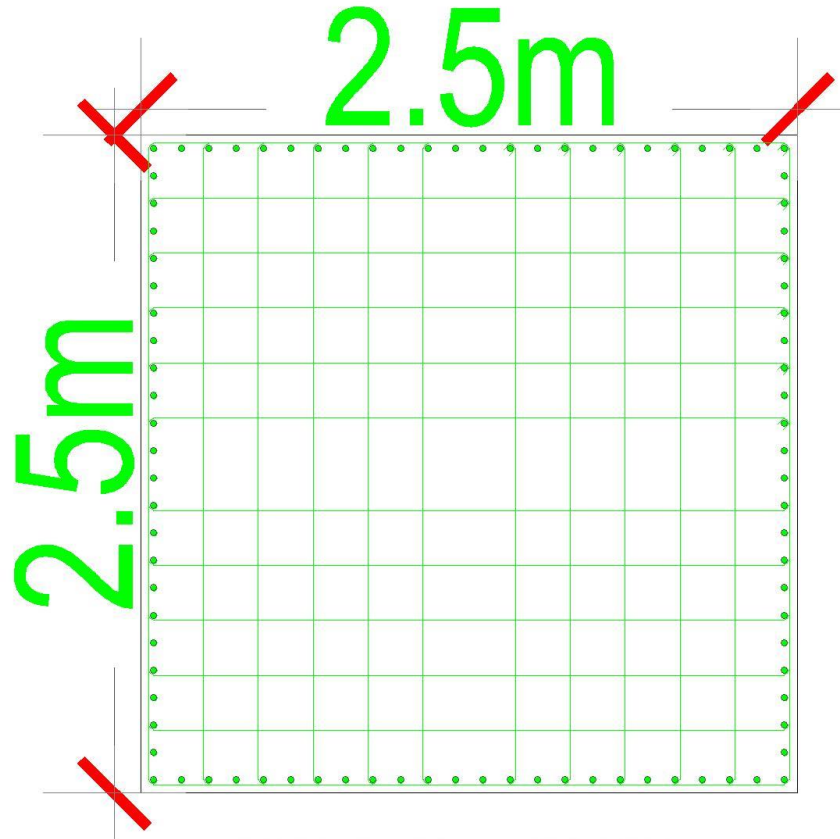
➤ Cross section 2.9m x 2.9m



➤ Cross section 2.7m x 2.7m

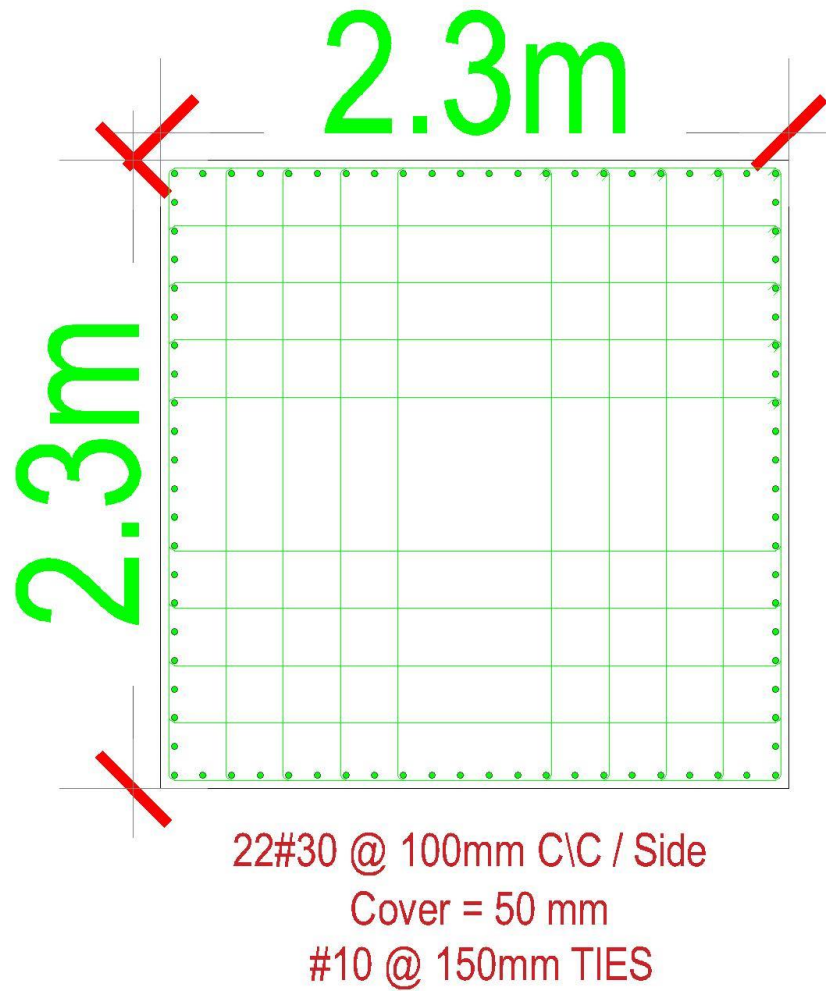


➤ Cross section 2.5m x 2.5m

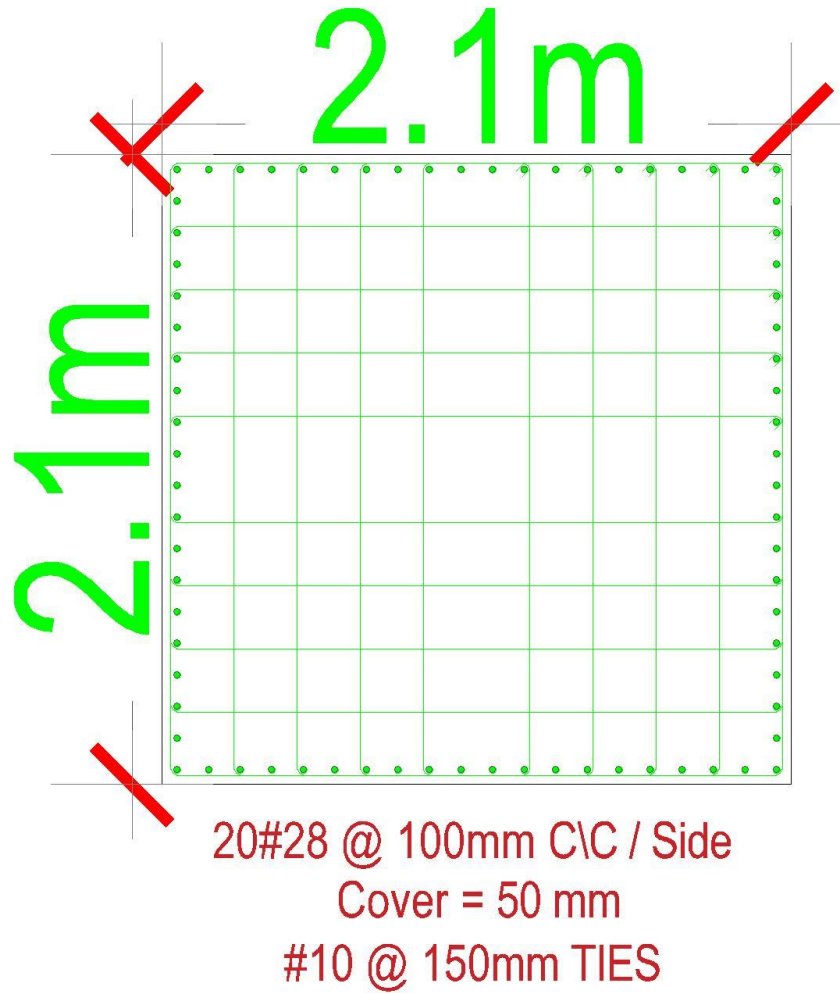


24#30 @ 100mm C\C / Side
Cover = 50 mm
#10 @ 150mm TIES

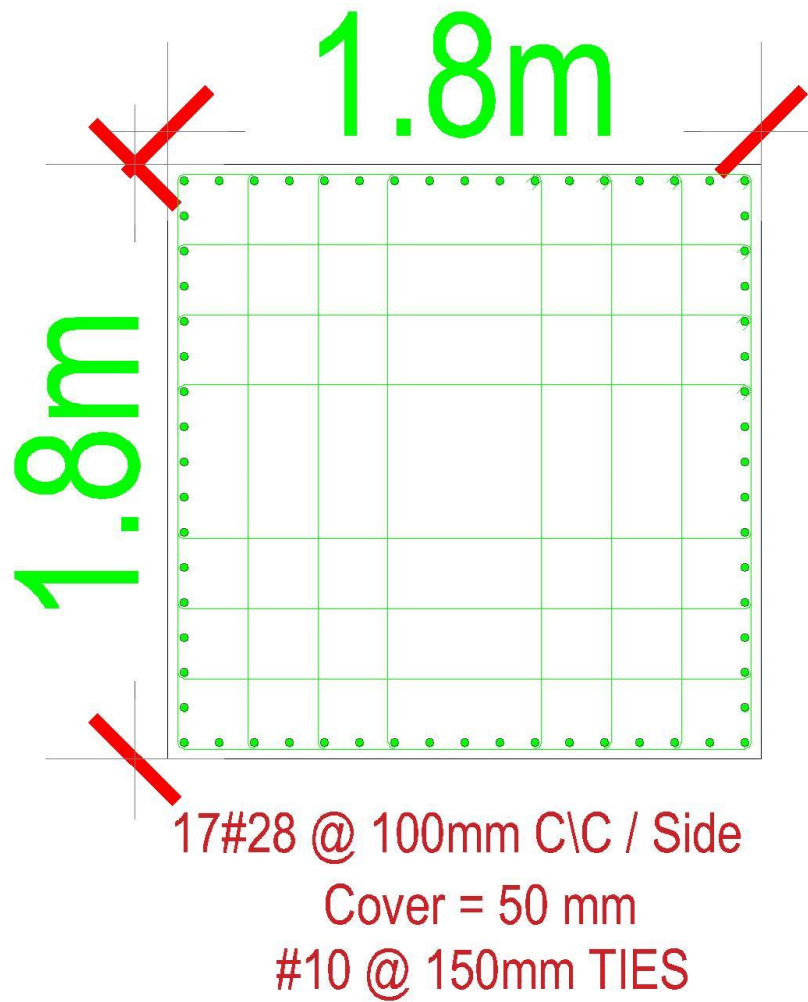
- Cross section 2.3m x 2.3m



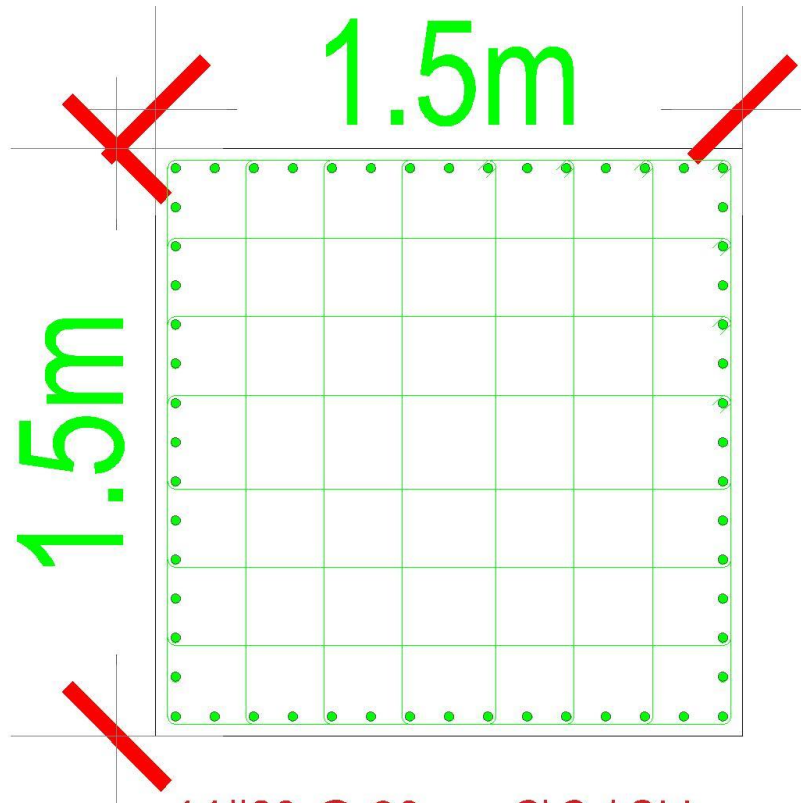
➤ Cross section 2.1m x 2.1m



➤ Cross section 1.8m x 1.8m



- Cross section 1.5m x 1.5m

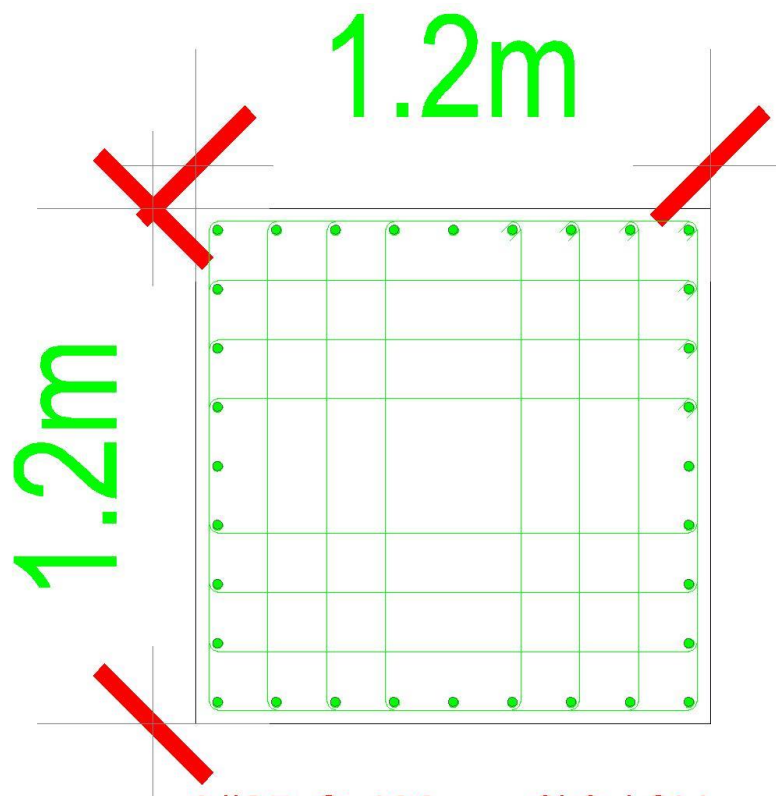


14#28 @ 90mm C\C / Side

Cover = 50 mm

#10 @ 150mm TIES

- Cross section 1.2m x 1.2m

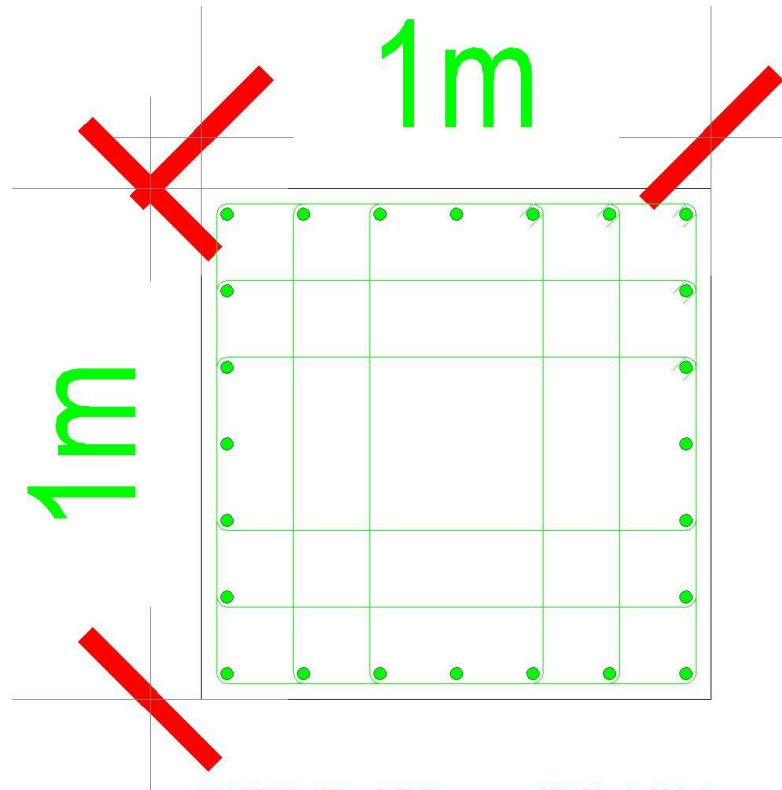


9#25 @ 100mm C\C / Side

Cover = 50 mm

#10 @ 150mm TIES

- Cross section 1m x 1m

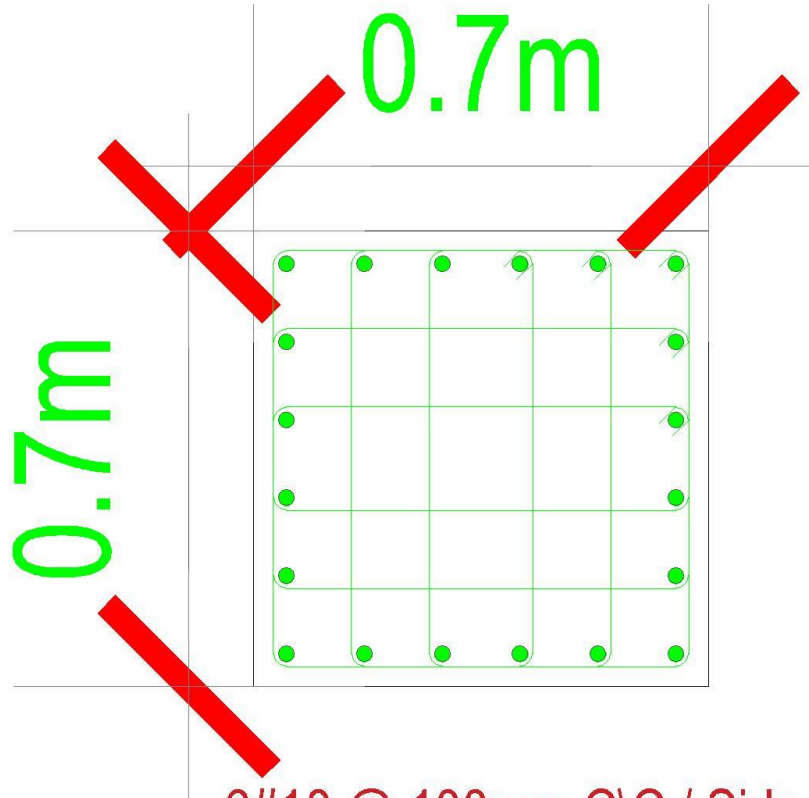


7#25 @ 100mm C\C / Side

Cover = 50 mm

#10 @ 150mm TIES

- Cross section 0.7m x 0.7m



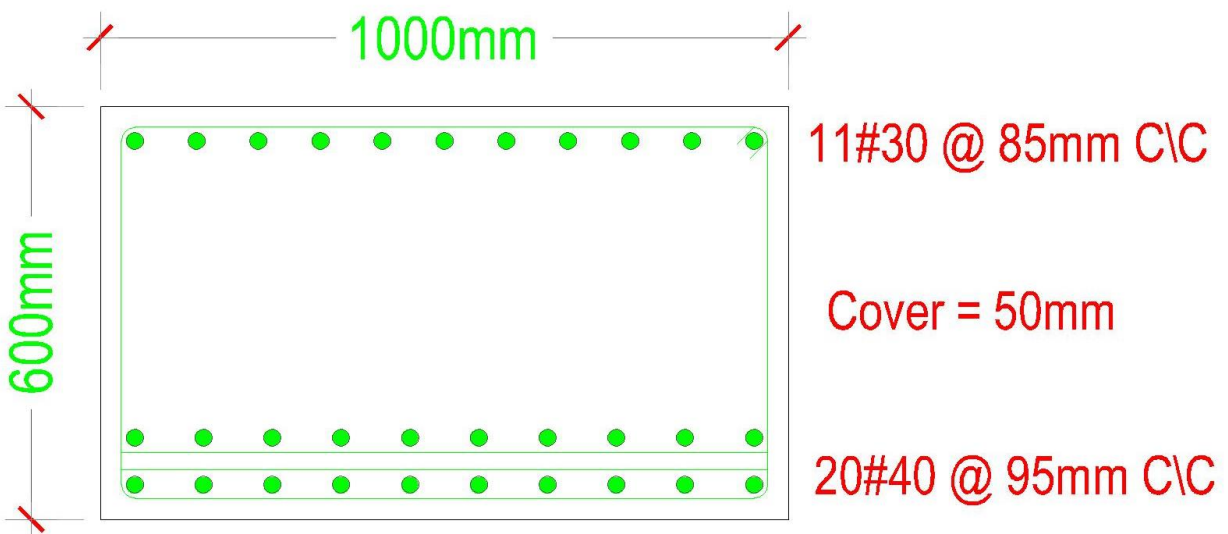
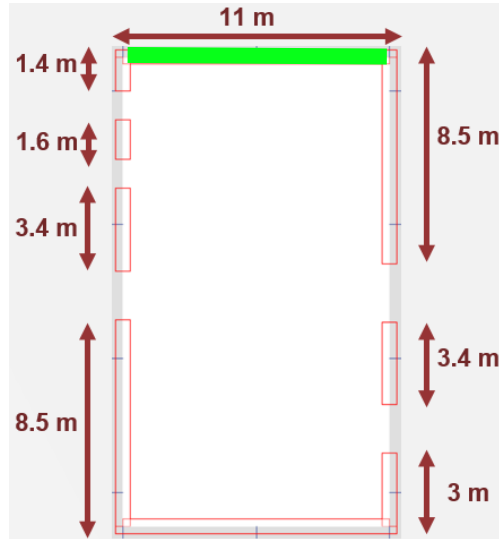
6#18 @ 100mm C\C / Side
Cover = 50 mm
#10 @ 150mm TIES

4.4 Steel Reinforcement of Shear Wall:

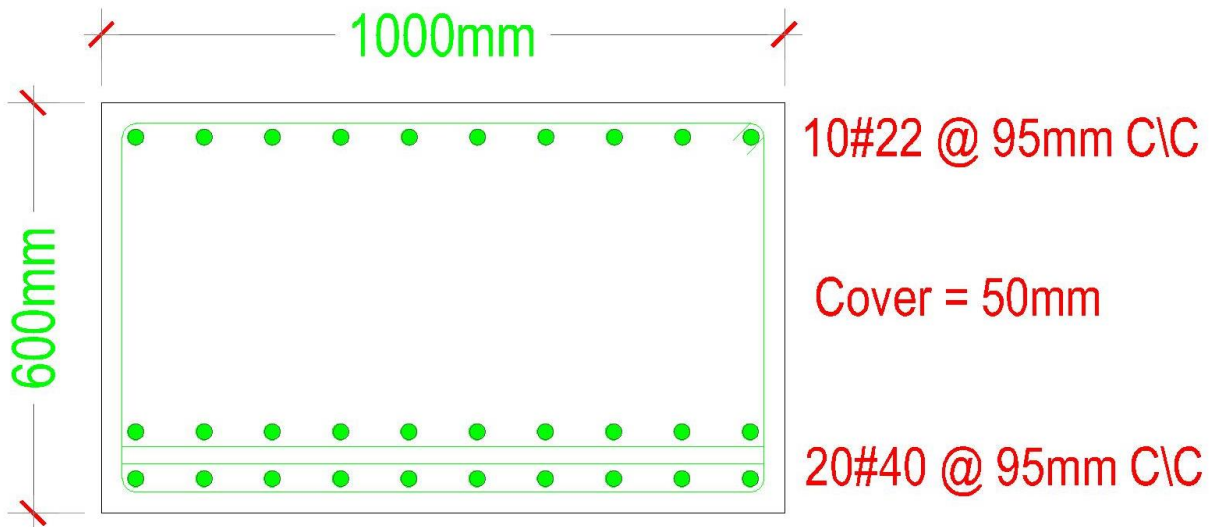
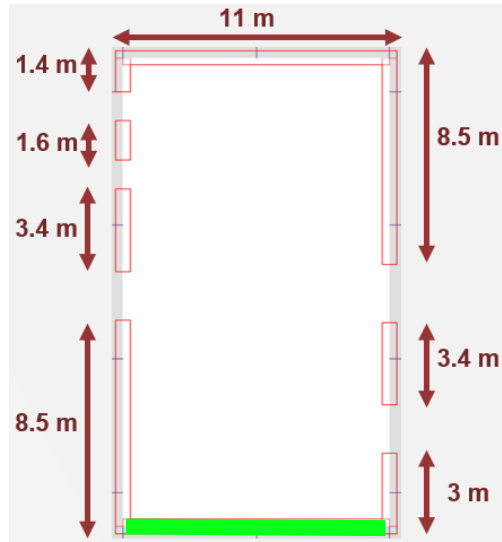
4.4.1 Steel Reinforcement of Shear Wall for RCC Model:

4.4.1.1 Shear wall of Residential Tower:

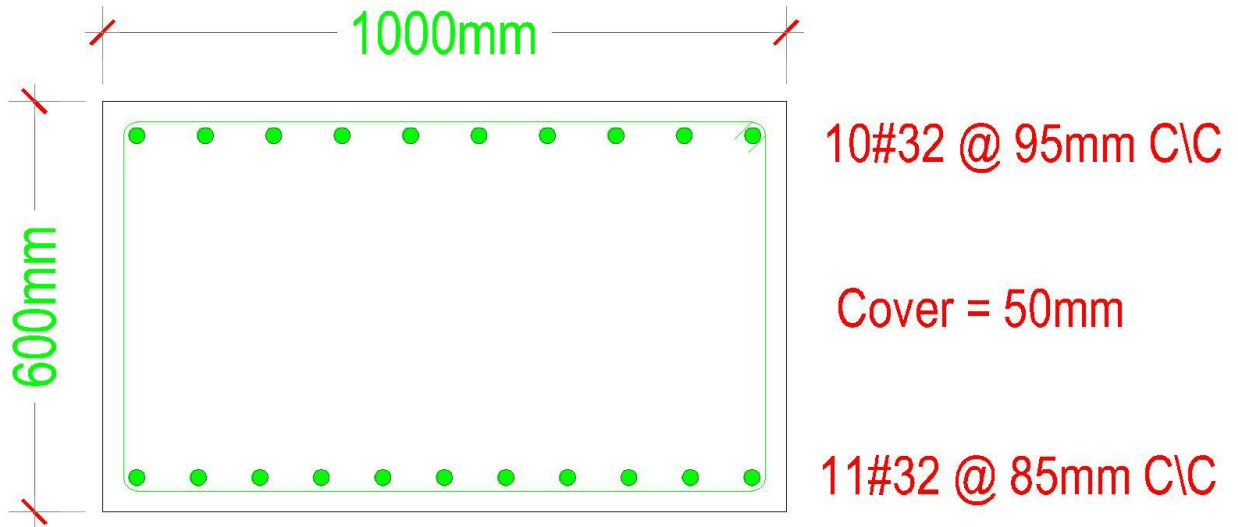
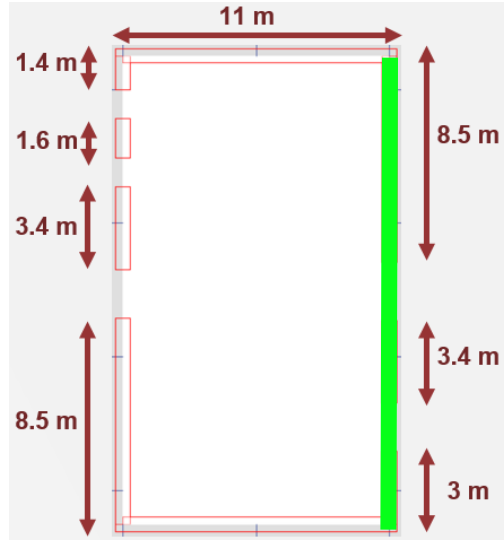
- Top



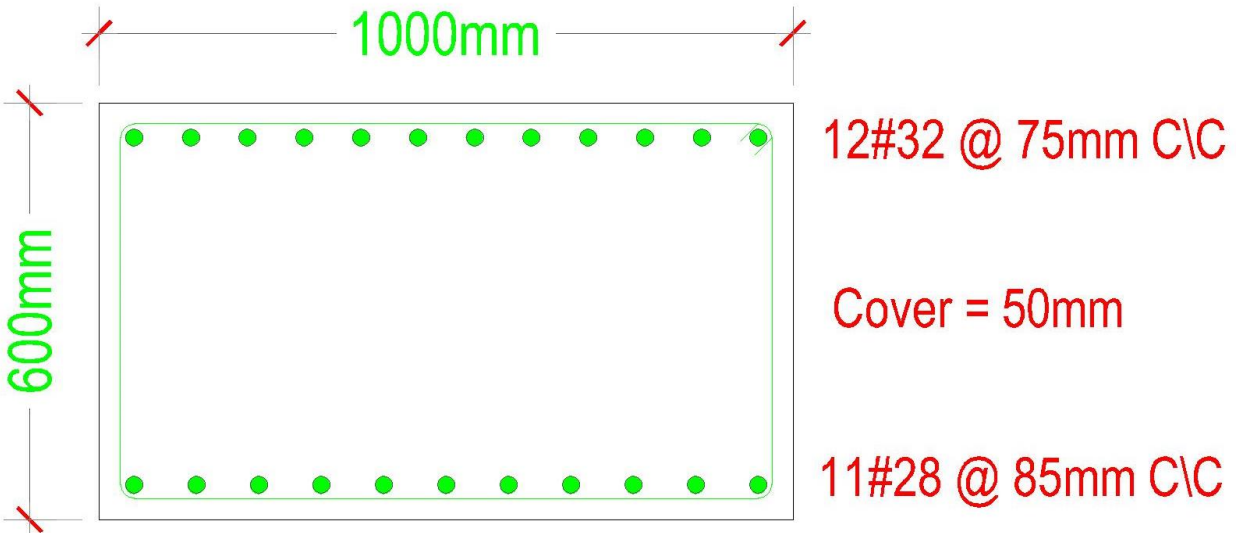
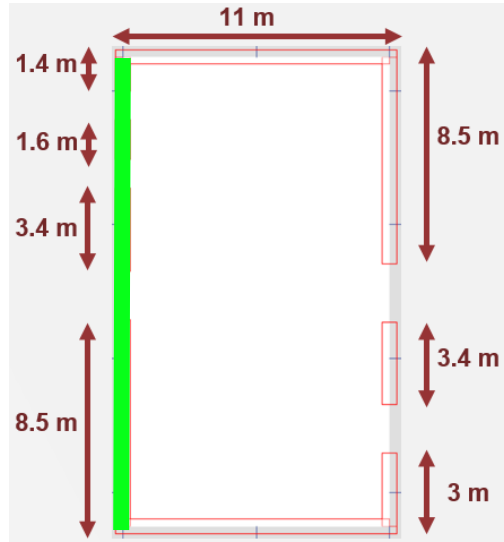
- Bottom



- Right Side

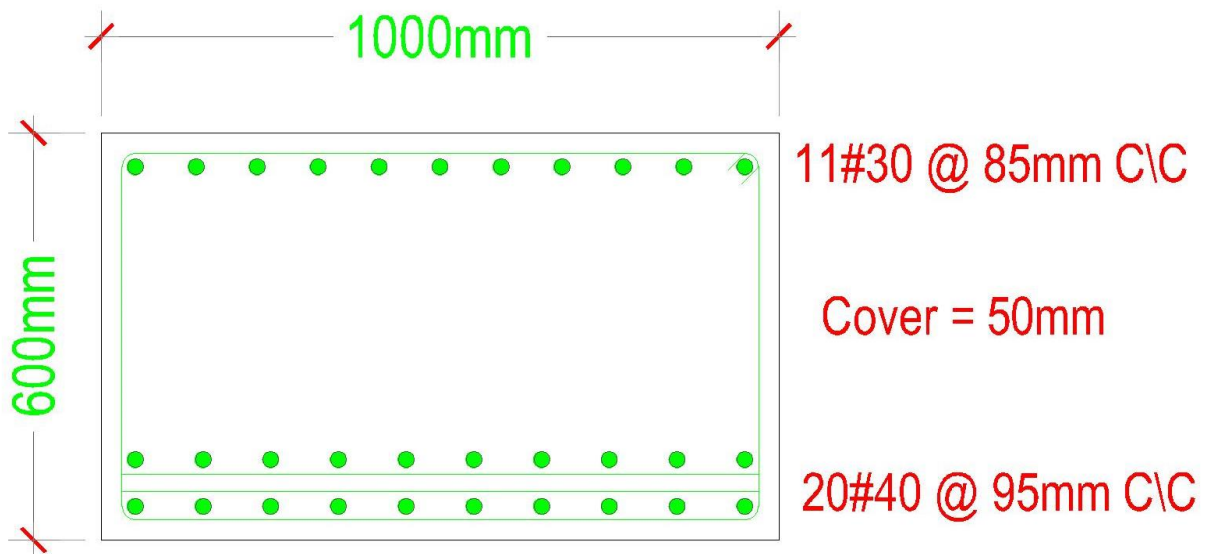
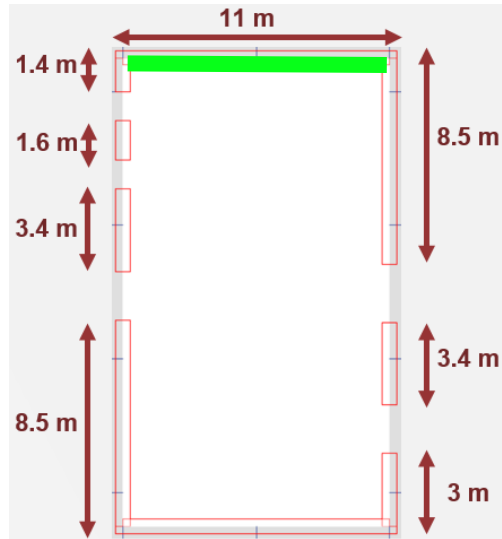


- Left Side

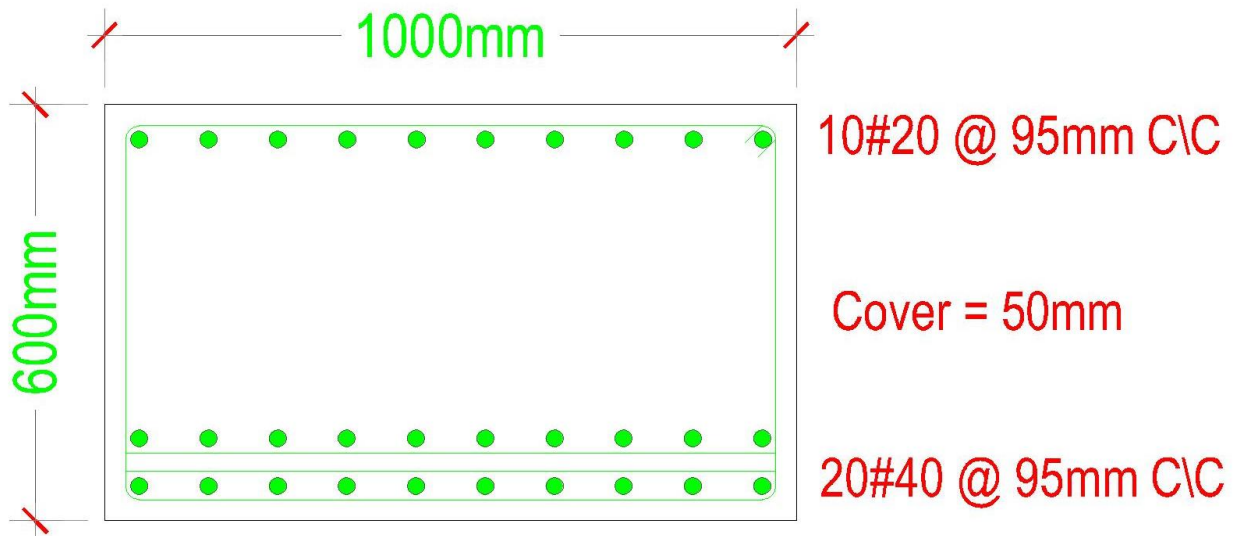
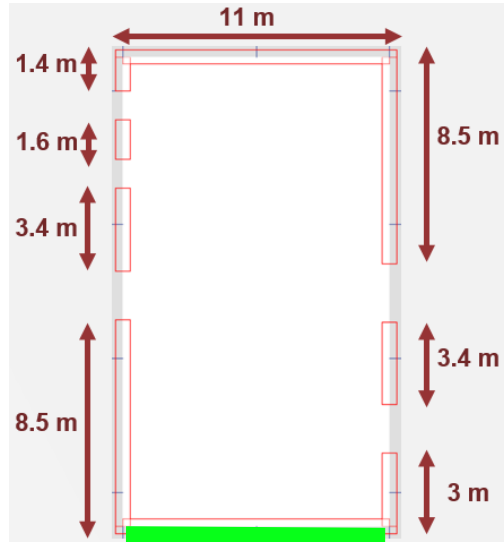


4.4.1.2 Shear Wall of Commercial Tower:

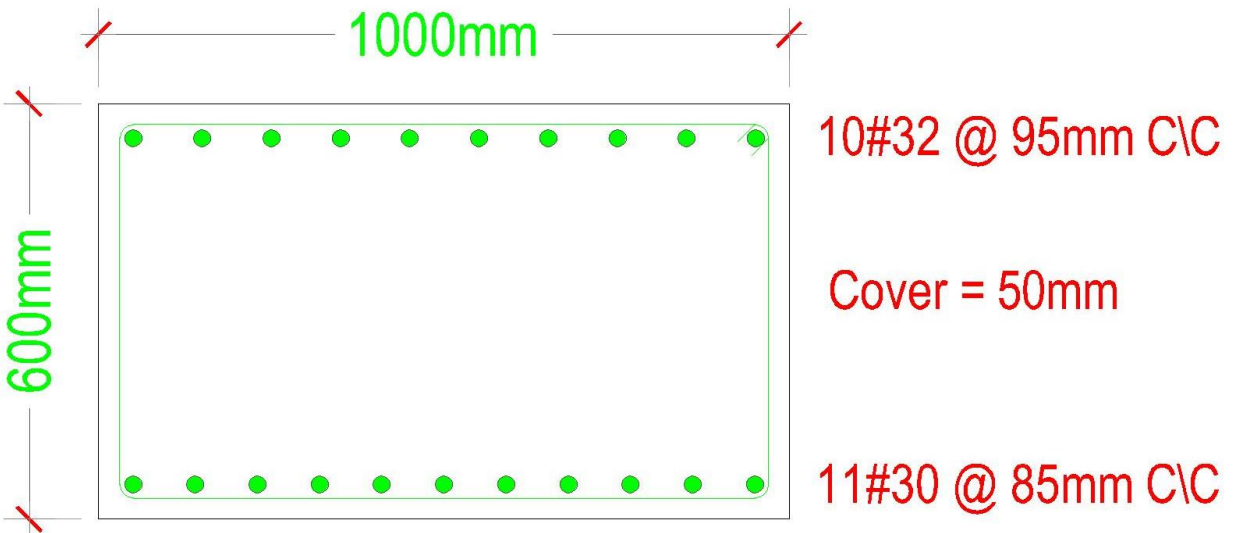
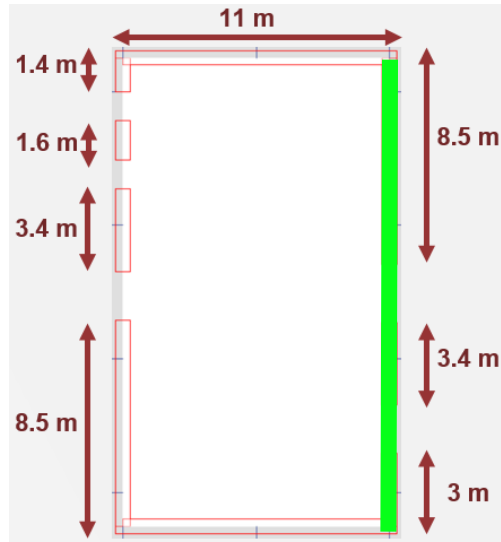
- Top



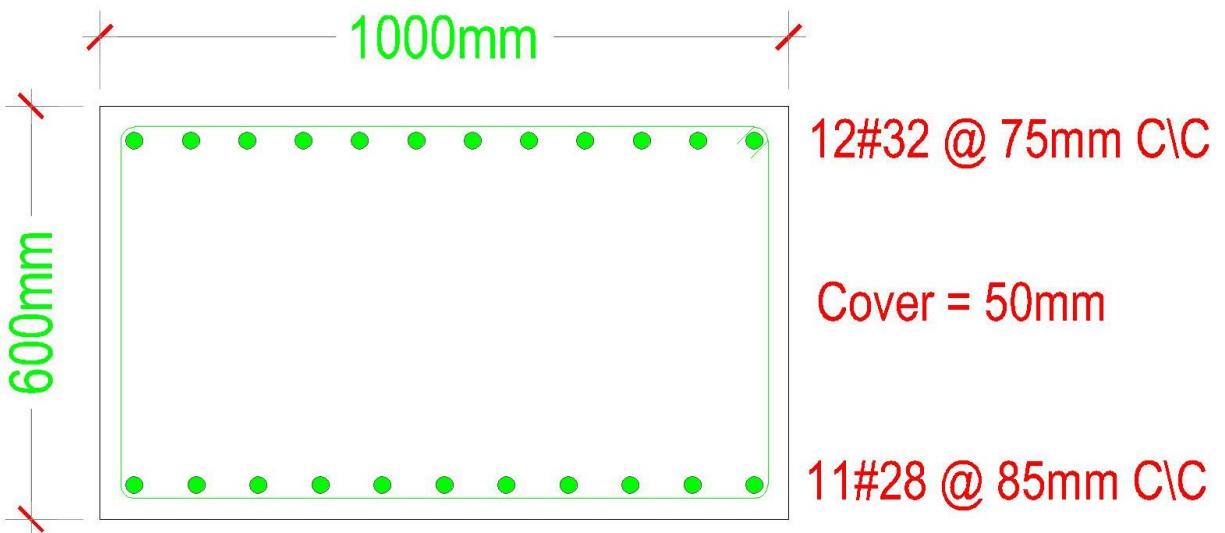
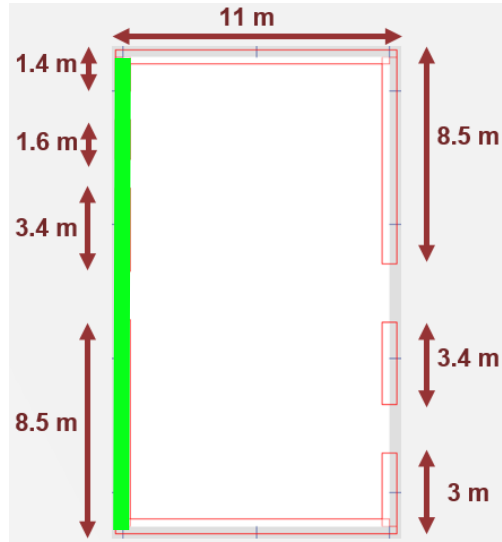
- Bottom



- Right Side



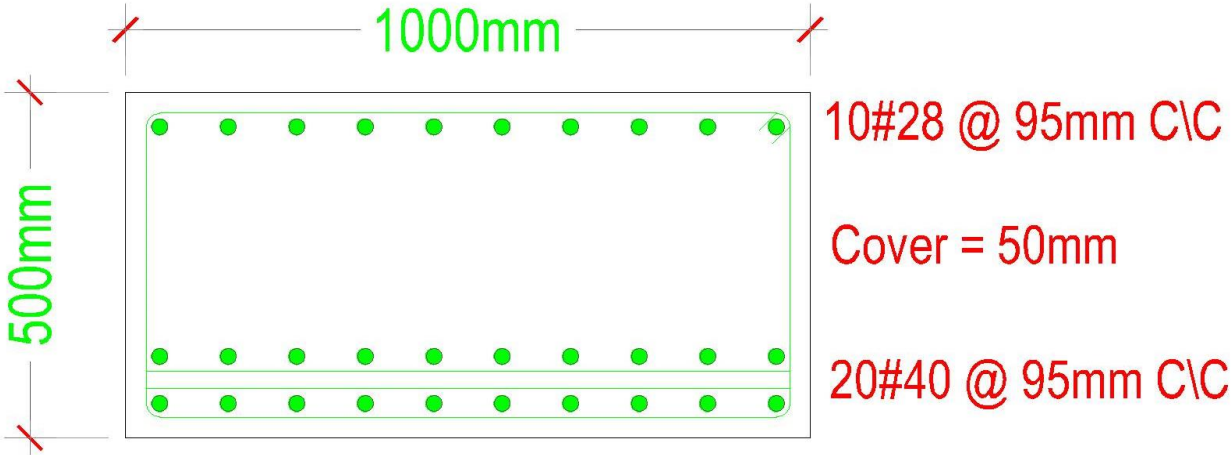
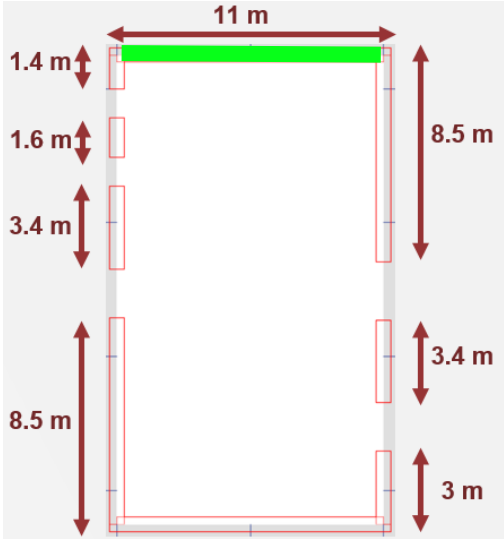
- Left Side



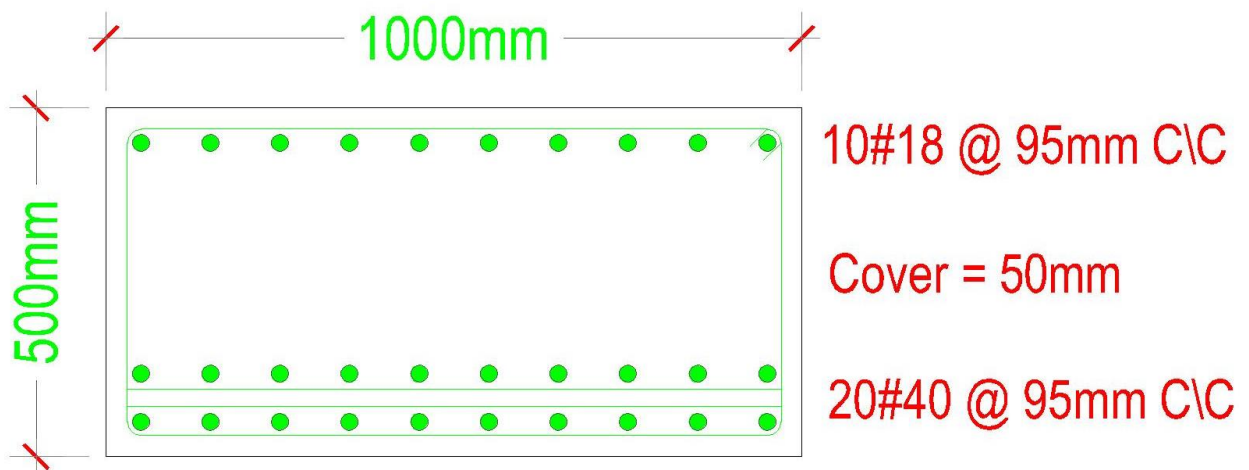
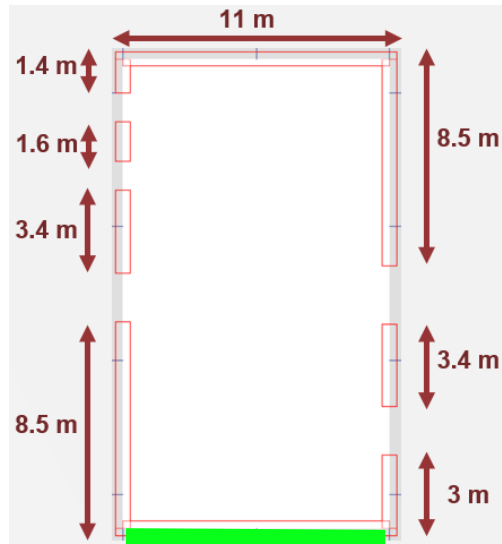
4.4.2 Steel Reinforcement of Shear Wall for CLT Model:

4.4.2.1 Shear wall of Residential Tower:

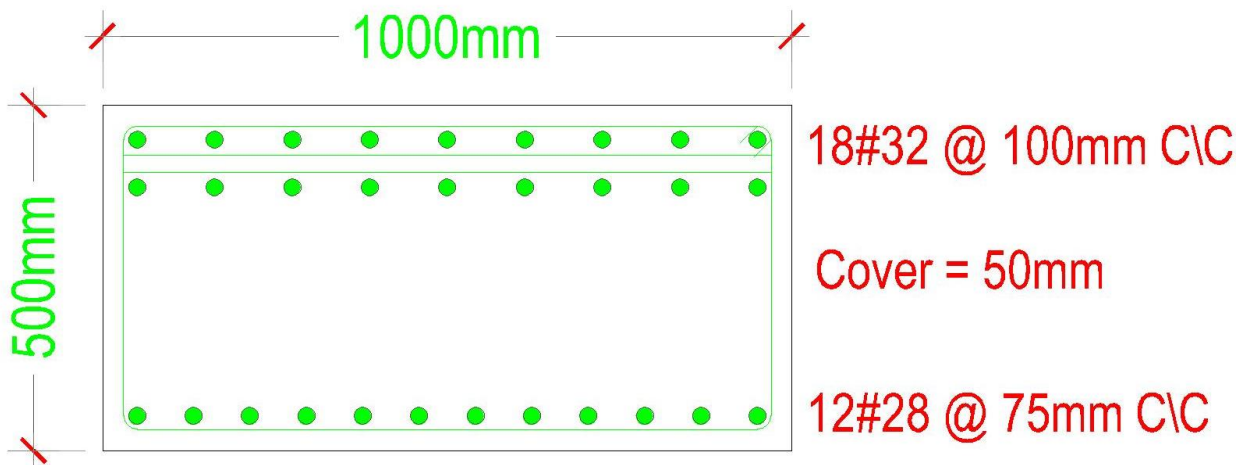
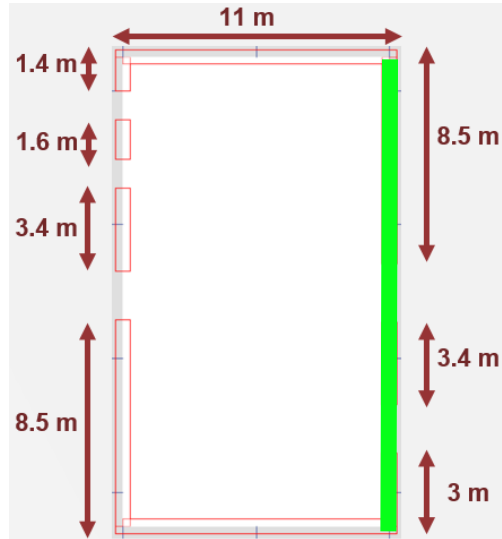
- Top



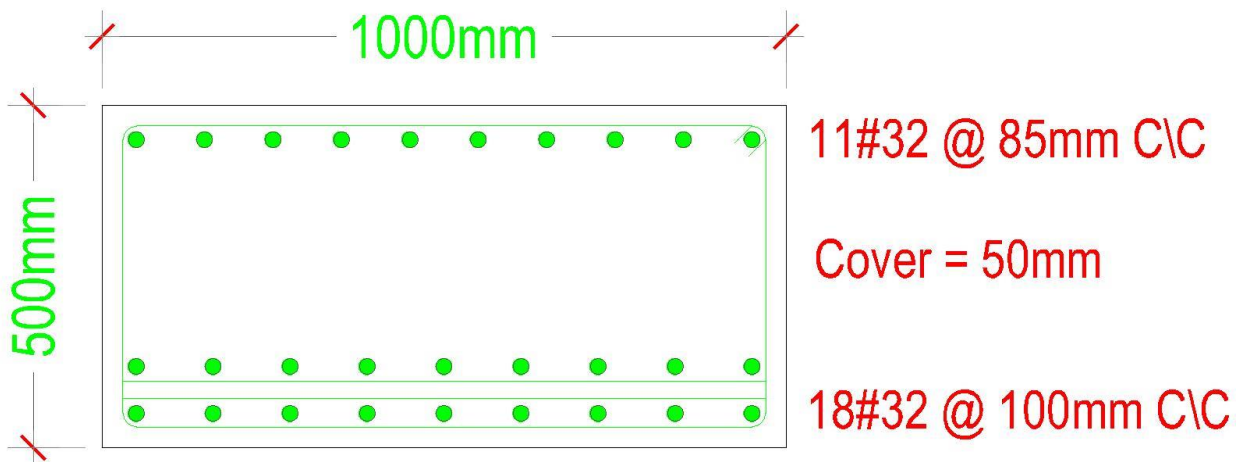
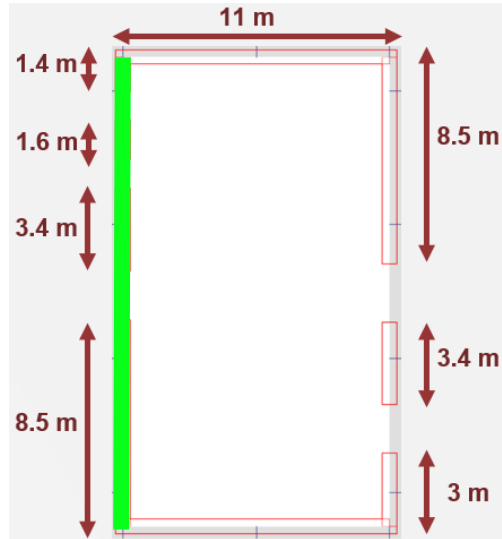
- Bottom



- Right Side

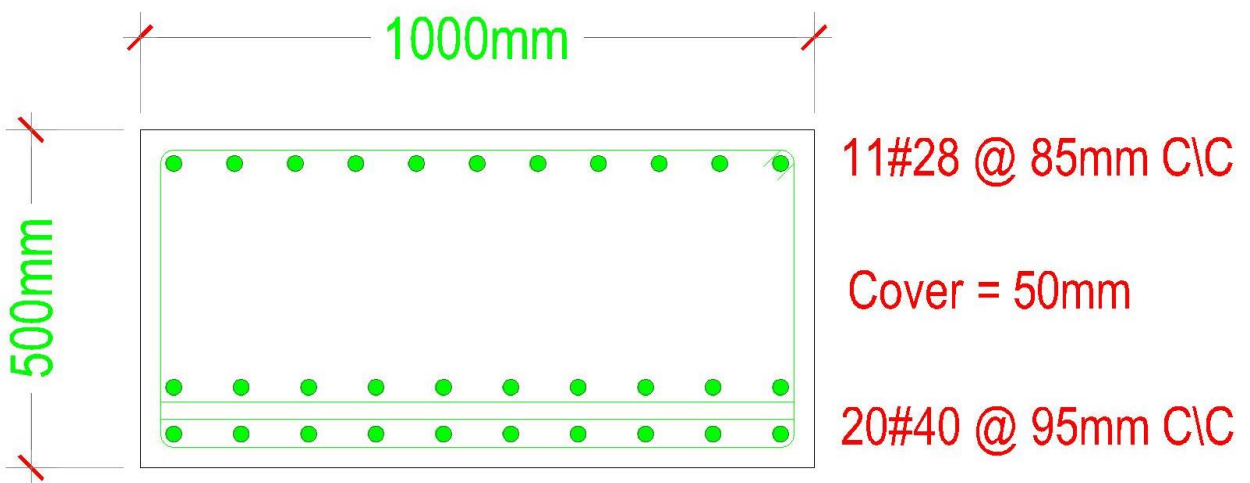
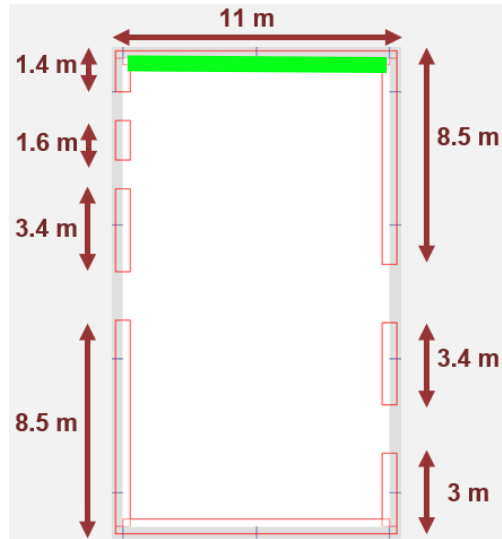


- Left Side

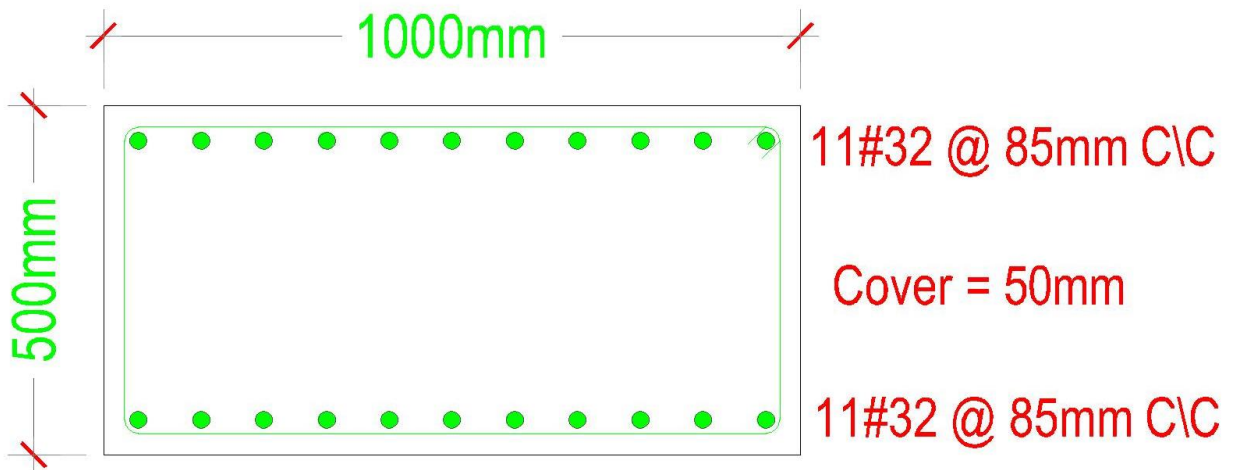
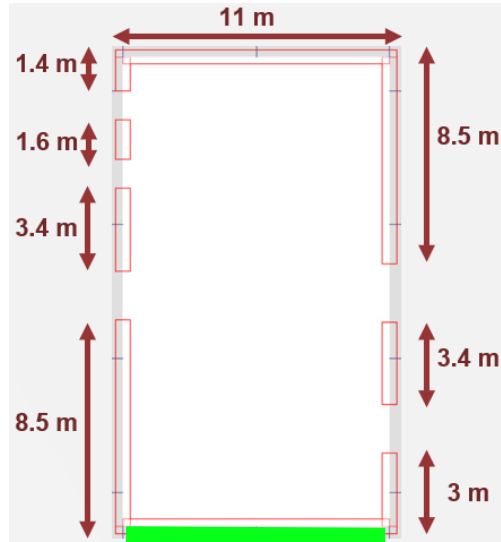


4.4.2.2 Shear Wall of Commercial Tower:

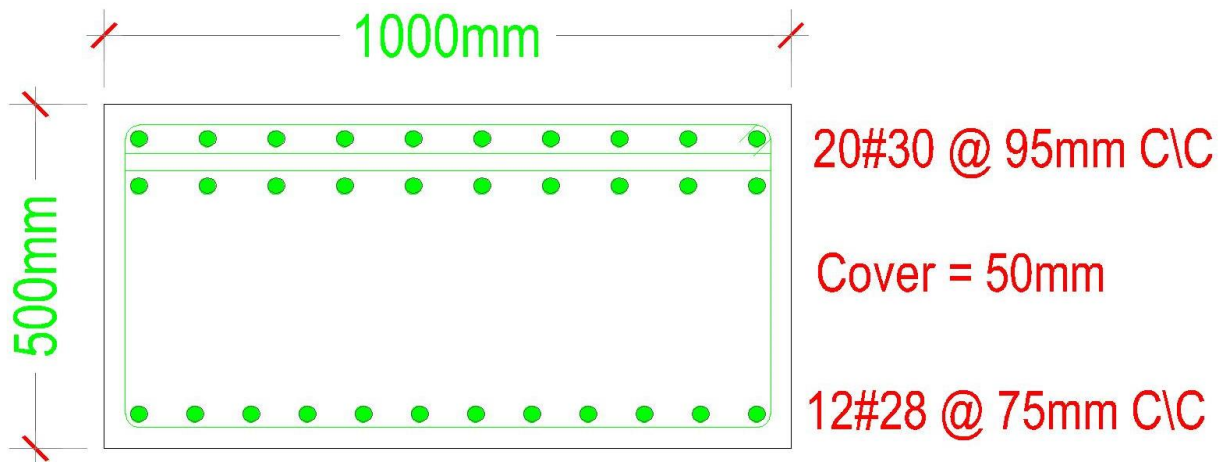
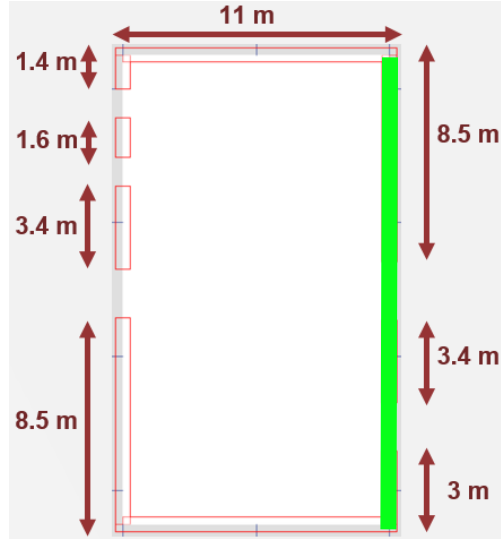
- Top



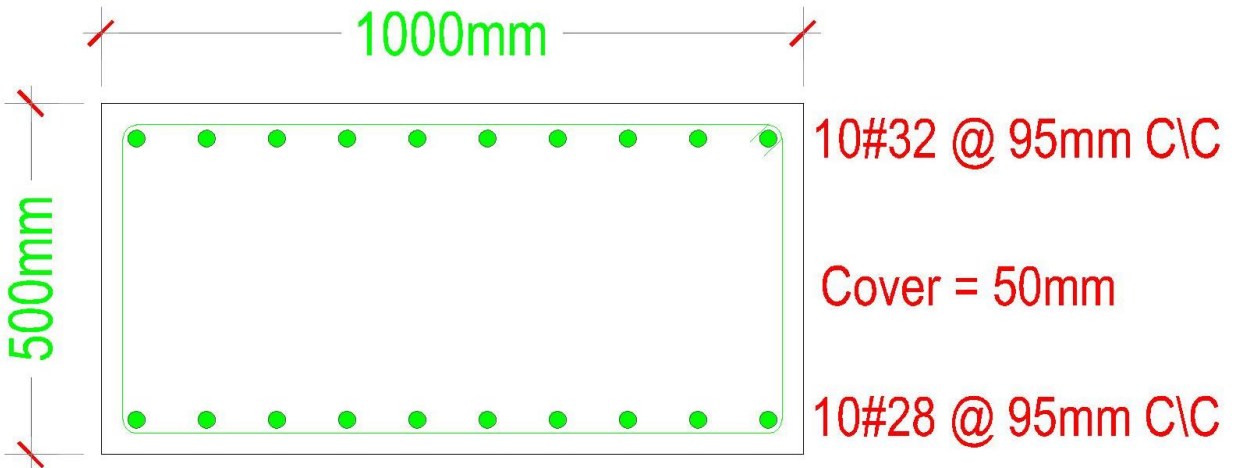
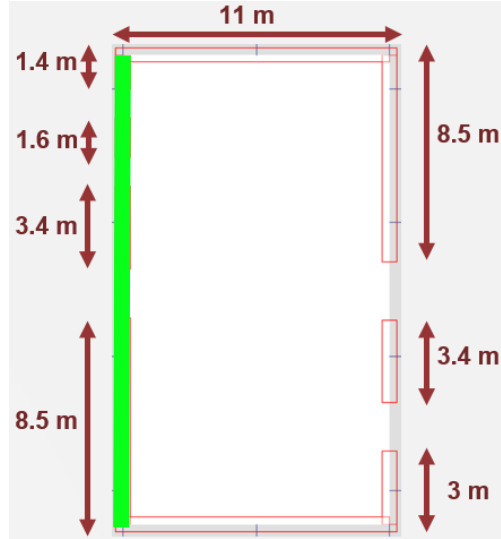
- Bottom



- Right Side



- Left Side



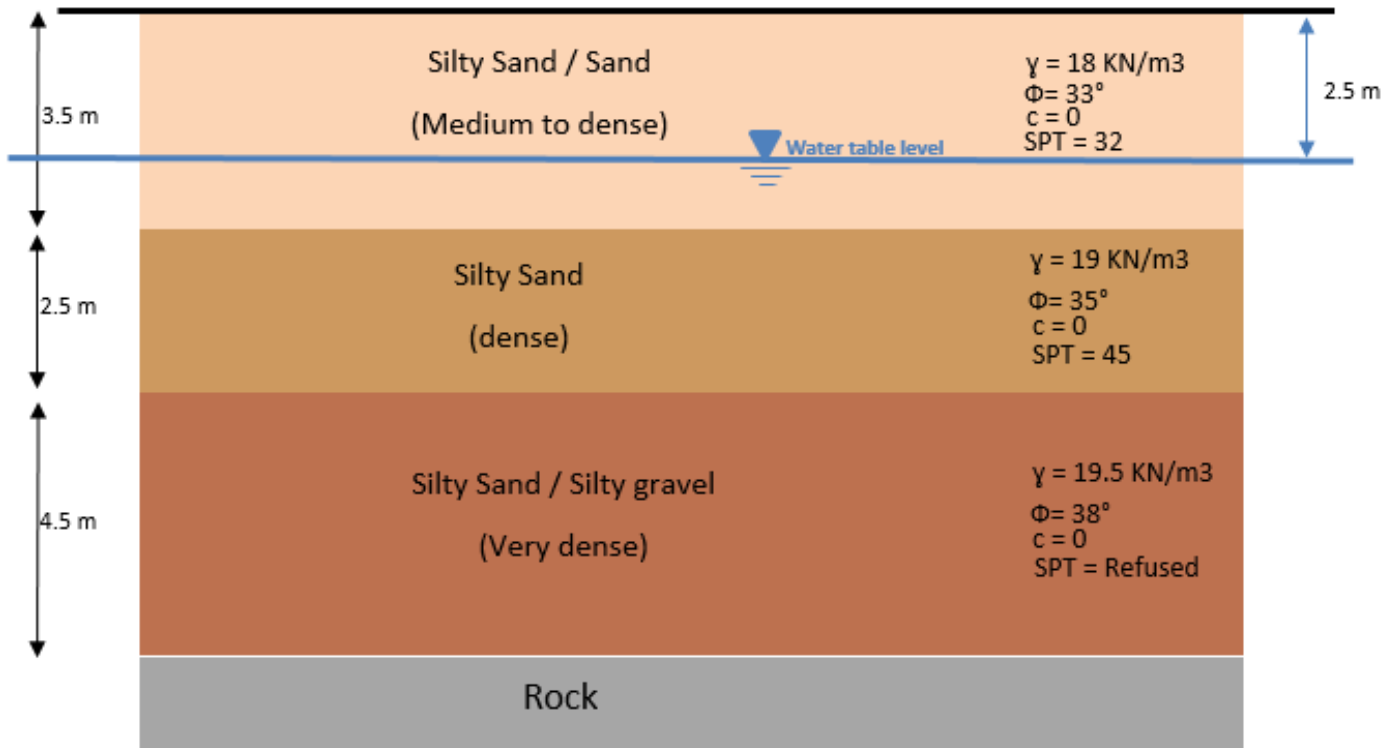
Chapter5: Geotechnical Design for Appropriate Foundation System

5.1 Geotechnical Report:

5.1.1 Soil profile:

The project land is located along Dammam – Khobar Highway, at “SAFCO” area of Dammam city. The soil has been tested by Gulf Consultant. From their geotechnical report we did some slight calculation to get our requirements. Those requirements are type of soil, water table level, angle of friction, cohesion, and the chemicals that contaminate the soil. Gulf Consultant did 26 boreholes(BH)to test the soil. We took the average of the water table level. see the following table.

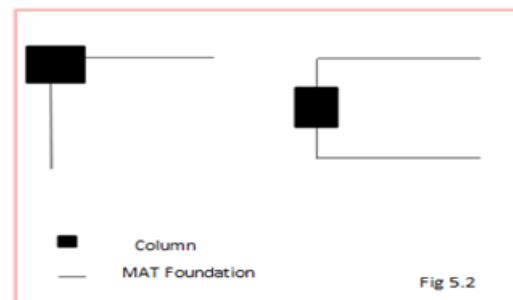
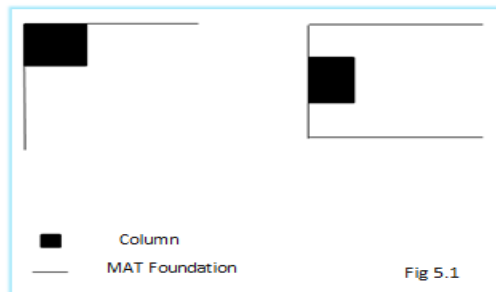
BH No.	Ground water level for each BH (m)	Average of water level (m)
1	2.67	2.531923077
2	1.9	
3	0	
4	0	
5	3.85	
6	4.7	
7	4.9	
8	5.2	
9	5.4	
10	3.9	
11	2.89	
12	2.75	
13	3.9	
14	2.75	
15	2.7	
16	3.9	
17	2.67	
18	0	
19	0	
20	0	
21	5.45	
22	3.4	
23	0	
24	2.9	
25	0	
26	0	
Σ	65.83	

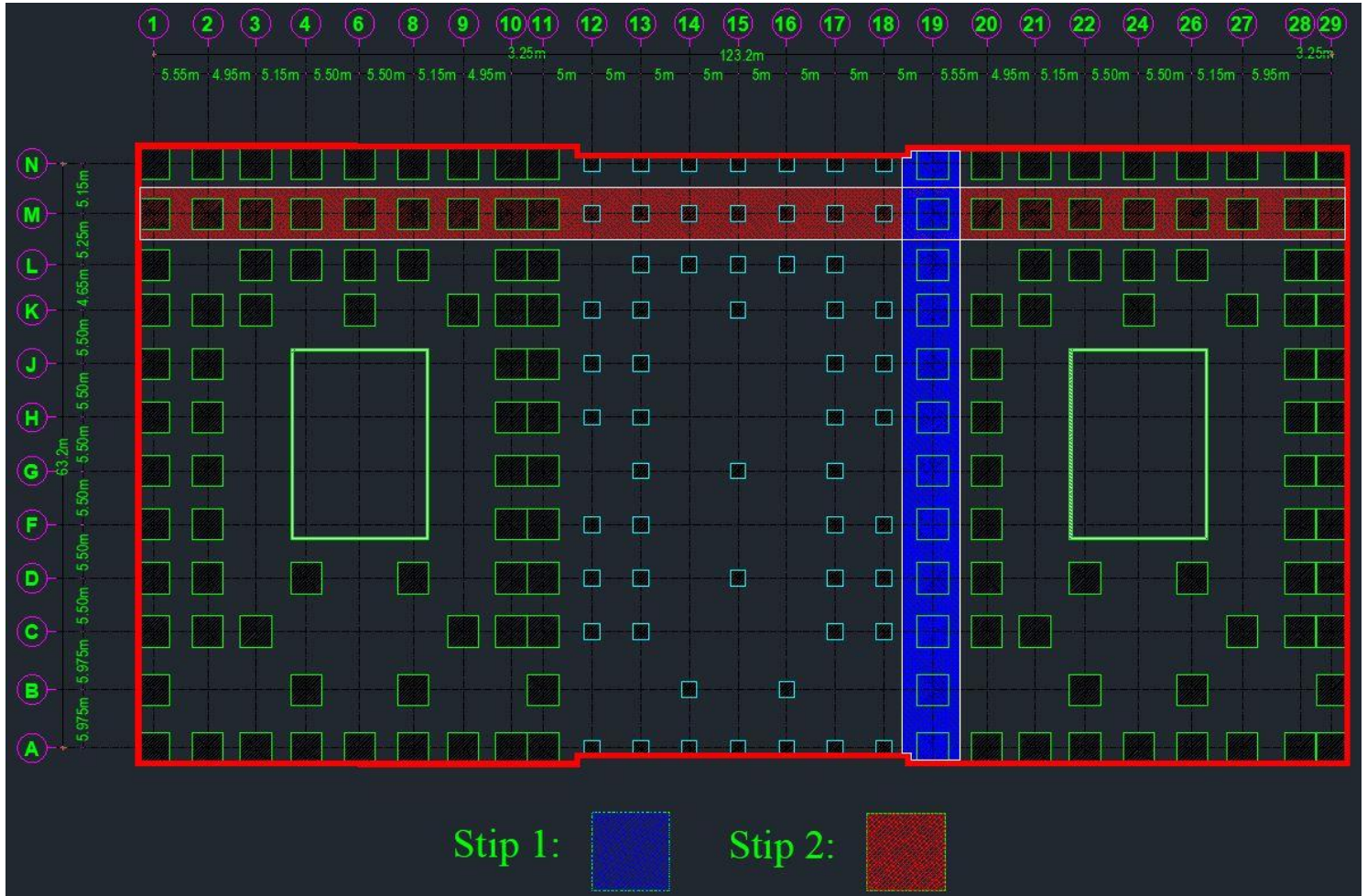


5.1.2 Geotechnical Design:

To design the geotechnical foundation system, it is must to know the reaction forces under each column caused by the critical load combination case. The critical load combination is dead load & live load. The reaction forces are obtained from ETABS analysis. We drew the MAT foundation layout including the column on a grid paper manually. The columns on the edge of the MAT foundation are inside the foundation perimeter. To clarify, the column footing located within the edge of the MAT foundation should be like in figure 5.1 and should not be like figure 5.2.

We specify the coordinates for each column from the origin point. We develop excel sheet for all columns coordinate. Then we shall calculate the eccentricity to overcome the MAT foundation if it resists up-left, rotation or over turning. We divide the foundation layout into 2 strips, one along Y direction and the other one along X direction. . The MAT foundation design will be based on the critical columns that are located in selected strips that having. These strips are having the higher number of columns and located in three different situations. The situations are corner, edge and the majority of the columns somewhere inside the MAT foundation and away from the edges and corners. See figure 5.3





5.1.3 Geotechnical Design Methodology:

1. Calculating the soil bearing capacity. The pressure under each column must be less than the soil bearing capacity.
2. Calculating MAT foundation area.
3. Summing the reactions coming from the columns.
4. Calculating the moment of inertia along X-axis & along axis to be able to calculate the pressure (q) under each column.
5. Computing the moment on X-axis & Y- axis.
6. Checking the pressure under each column if it is less than the soil bearing capacity. The pressure under each column is higher than the soil bearing capacity and then we decide to design bored pile to support the foundation.
7. All column pressures are higher than the soil bearing capacity.
8. We are providing supportive piles to increase the soil bearing capacity by providing bored piles.
9. Calculating soil reaction which is the net soil bearing capacity that pushes the foundation upward.
10. Using SAP2000 to compute the shear and moment diagrams for each strip along X and Y directions.
11. Designing the MAT foundation depth based on the maximum shear value.
12. Getting the maximum and minimum moment for each strip from SAP2000 to calculate the required area of steel.

This procedure is done twice, once for RCC model and once for CLT. The only difference was reaction forces acting on the columns, because CLT slab is lighter than RCC slab. This seems to be little difference, but in fact it affected the whole calculation and foundation thickness and required steel rebar.

5.2 Geotechnical Design Calculation for RCC Model:

Bearing Capacity of the soil:

From the geotechnical report, we extracted the compressive strength of the rock (R_c) and Rock Quality Designation (RQD). RQD is a measure of quality of rock core taken from a borehole. RQD signifies the degree of jointing or fracture in a rock mass measured in percentage, where RQD of 75% or more shows good quality hard rock and less than 50% show low quality weathered rocks.

$$R_c = 11474 \text{ /m}^2, \text{ RQD} = 54\%$$

$$Q_{\text{all}} = \frac{R_c \times 0.8}{10} \times RQD = \frac{11474 \times 0.8}{10} \times 0.54 = 495.7 \text{ kN/m}^2$$

Mat foundation design: (RCC Slab Model):

$$q = \frac{Q}{A} \pm \frac{M_x}{I_x} y \pm \frac{M_y}{I_y} x$$

$$A = 2 [44.6 (63.2)] + 34(61.2) = 7718.24 \text{ m}^2$$

$$I_x = 2 \left[\frac{1}{12} (44.6)(63.2)^3 \right] + \frac{1}{12} (34) (61.2)^3 = 2525900 \text{ m}^4$$

$$I_y = 2 \left[\frac{1}{12} (63.2)(44.6)^3 \right] + \frac{1}{12} (61.2) (34)^3 = 1134931.25 \text{ m}^4$$

$$\text{Excel} \leftarrow Q = 16175882.4 \text{ KN} , x' = 62.0565 , y' = 32.6158 \text{ m}$$

$$e_x = x' - \frac{B}{2} = 62.0565 - \frac{123.2}{2} = 0.4565 \text{ m}$$

$$e_y = y' - \frac{l}{2} = 32.6158 - \frac{63.2}{2} = 1.0158 \text{ m}$$

$$M_y = Q_{ex} = 16175882.4 (0.4565) = 7384290.32 \text{ KN.m}$$

$$M_x = Q_{ey} = 16175882.4 (1.0158) = 16431461.34 \text{ KN.m}$$

$$\text{So, } q = \frac{16175882.4}{7786.24} \pm \frac{16431461.34 y}{2525900} \pm \frac{7384290.32 x}{1134931.25}$$

$$q = 2077.5 \pm 6.34 y \pm 6.47 x$$

Strip 1:-

$$\text{At A19: } q = 2077.5 - 6.34(30) + 6.47(20) = 2016.7 \text{ KN/m}^2$$

$$\text{At B19: } q = 2077.5 - 6.34(24.025) + 6.47(20) = 2054.6 \text{ KN/m}^2$$

$$\text{At C19: } q = 2077.5 - 6.34(18.05) + 6.47(20) = 2092.5 \text{ KN/m}^2$$

$$\text{At D19: } q = 2077.5 - 6.34(12.55) + 6.47(20) = 2127.3 \text{ KN/m}^2$$

$$\text{At F19: } q = 2077.5 - 6.34(7.05) + 6.47(20) = 2162.2 \text{ KN/m}^2$$

$$\text{At G19: } q = 2077.5 - 6.34(1.55) + 6.47(20) = 2197.1 \text{ KN/m}^2$$

$$\text{At H19: } q = 2077.5 + 6.34(3.95) + 6.47(20) = 2231.9 \text{ KN/m}^2$$

$$\text{At I19: } q = 2077.5 + 6.34(9.45) + 6.47(20) = 2266.8 \text{ KN/m}^2$$

$$\text{At K19: } q = 2077.5 + 6.34(14.95) + 6.47(20) = 2301.7 \text{ KN/m}^2$$

$$\text{At L19: } q = 2077.5 + 6.34(19.6) + 6.47(20) = 2331.2 \text{ KN/m}^2$$

$$\text{At M19: } q = 2077.5 + 6.34(24.85) + 6.47(20) = 2364.5 \text{ KN/m}^2$$

$$\text{At N19: } q = 2077.5 + 6.34(30) + 6.47(20) = 2397.1 \text{ KN/m}^2$$

Strip 2:-

$$\text{At M1: } q = 2077.5 + 6.34(24.85) - 6.47(60) = 1846.9 \text{ KN/m}^2$$

$$\text{At M2: } q = 2077.5 + 6.34(24.85) - 6.47(54.45) = 1882.8 \text{ KN/m}^2$$

$$\text{At M3: } q = 2077.5 + 6.34(24.85) - 6.47(49.5) = 1914.8 \text{ KN/m}^2$$

$$\text{At M4: } q = 2077.5 + 6.34(24.85) - 6.47(44.35) = 1948.1 \text{ KN/m}^2$$

$$\text{At M6: } q = 2077.5 + 6.34(24.85) - 6.47(38.85) = 1983.7 \text{ KN/m}^2$$

$$\text{At M8: } q = 2077.5 + 6.34(24.85) - 6.47(33.35) = 2019.3 \text{ KN/m}^2$$

$$\text{At M9: } q = 2077.5 + 6.34(24.85) - 6.47(28.2) = 2052.6 \text{ KN/m}^2$$

$$\text{At M10: } q = 2077.5 + 6.34(24.85) - 6.47(23.25) = 2084.6 \text{ KN/m}^2$$

$$\text{At M11: } q = 2077.5 + 6.34(24.85) - 6.47(20) = 2105.6 \text{ KN/m}^2$$

$$\text{At M12: } q = 2077.5 + 6.34(24.85) - 6.47(15) = 2138 \text{ KN/m}^2$$

$$\text{At M13: } q = 2077.5 + 6.34(24.85) - 6.47(10) = 2170.3 \text{ KN/m}^2$$

$$\text{At M14: } q = 2077.5 + 6.34(24.85) - 6.47(5) = 2202.7 \text{ KN/m}^2$$

$$\text{At M15: } q = 2077.5 + 6.34(24.85) + 6.47(0) = 2235 \text{ KN/m}^2$$

$$\text{At M16: } q = 2077.5 + 6.34(24.85) + 6.47(5) = 2267.4 \text{ KN/m}^2$$

$$\text{At M17: } q = 2077.5 + 6.34(24.85) + 6.47(10) = 2299.7 \text{ KN/m}^2$$

$$\text{At M18: } q = 2077.5 + 6.34(24.85) + 6.47(15) = 2332.1 \text{ KN/m}^2$$

$$\text{At M19: } q = 2077.5 + 6.34(24.85) + 6.47(20) = 2364.4 \text{ KN/m}^2$$

$$\text{At M20: } q = 2077.5 + 6.34(24.85) + 6.47(25.55) = 2400.4 \text{ KN/m}^2$$

$$\text{At M21: } q = 2077.5 + 6.34(24.85) + 6.47(30.5) = 2432.4 \text{ KN/m}^2$$

$$\text{At M22: } q = 2077.5 + 6.34(24.85) + 6.47(35.65) = 2465.7 \text{ KN/m}^2$$

$$\text{At M24: } q = 2077.5 + 6.34(24.85) + 6.47(41.15) = 2501.3 \text{ KN/m}^2$$

$$\text{At M26: } q = 2077.5 + 6.34(24.85) + 6.47(46.65) = 2536.9 \text{ KN/m}^2$$

$$\text{At M27: } q = 2077.5 + 6.34(24.85) + 6.47(51.8) = 2570.2 \text{ KN/m}^2$$

$$\text{At M28: } q = 2077.5 + 6.34(24.85) + 6.47(56.75) = 2602.2 \text{ KN/m}^2$$

$$\text{At M29: } q = 2077.5 + 6.34(24.85) + 6.47(60) = 2623.3 \text{ KN/m}^2$$

All column pressures are more than the allowable soil bearing capacity (q_{all}) 495.7 kN/m^2 . Therefore, we are providing bored pile to support the allowable soil bearing capacity. So, each column pressure become less than the net bearing capacity. The designed bored piles are applicable for both model RCC & CLT. We developed excel sheet with different length and diameter of piles group. We concluded with efficient pile length and diameter.

Pile Design Calculation:

$$Q_i = \frac{Q_p + Q_s}{4}$$

$$Q_p = \frac{R_c \times 0.8}{10} \times RQD = \frac{11474(0.8)}{10} (0.54) = 495.7 \text{ kPa}$$

$$Q_s = P L f, \quad f = \alpha C_u$$

$$\alpha = 0.24$$

$$Q_s = (\pi \times 0.6)(3)(0.24 \times 250) = 339.9 \text{ kPa [for one pile]}$$

$$Q_i = \frac{495.7+339.3}{4} = 208.8 \text{ kPa [for one pile]}$$

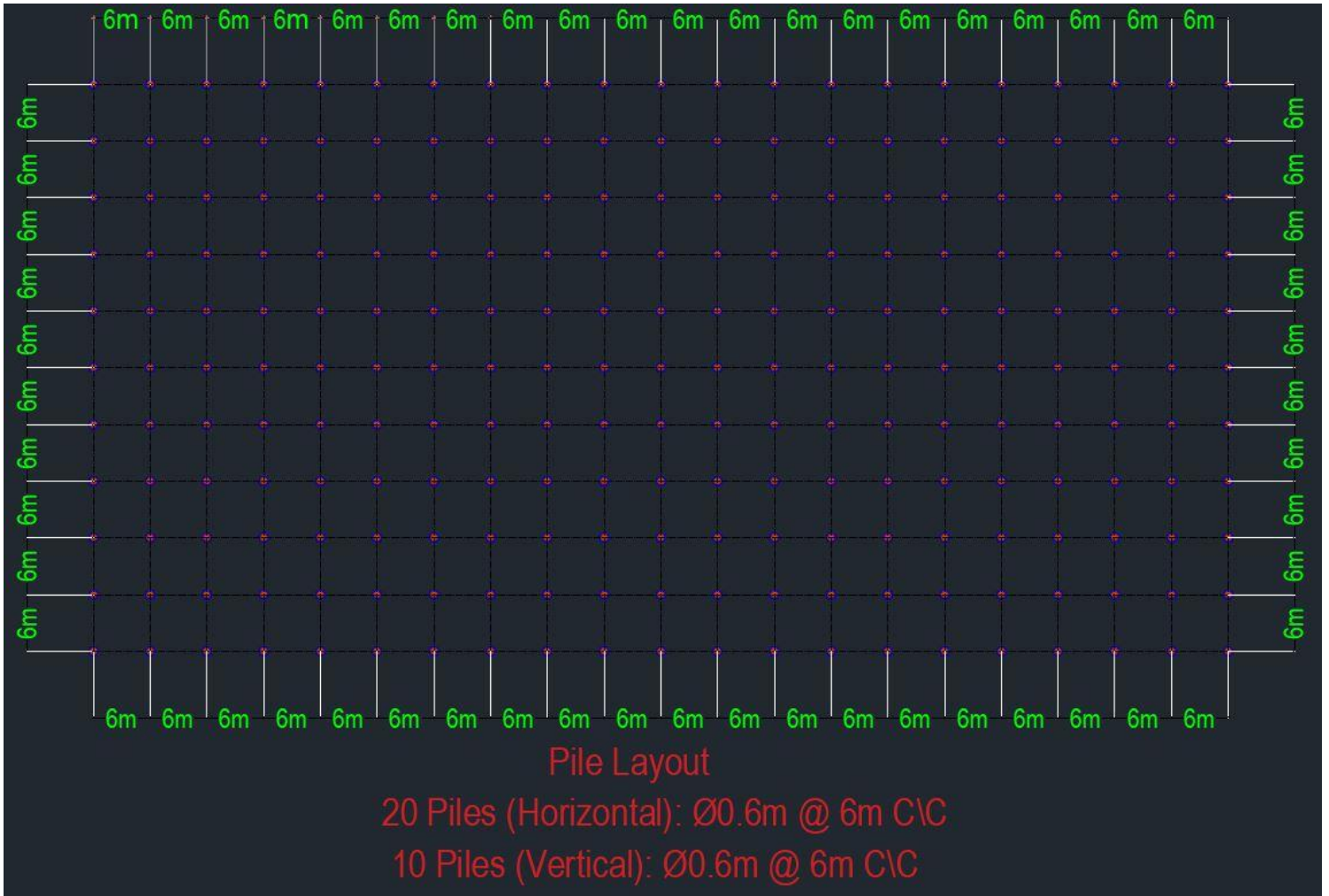
$$S = 10D = 10 (0.6) = 6 \text{ m}$$

$$\text{Number of piles}_{(x\text{-axis})} = \frac{123.2}{6} = 20 \text{ piles}$$

$$\text{Number of piles}_{(y\text{-axis})} = \frac{63.2}{6} = 10 \text{ piles}$$

$$\text{Number of piles under the MAT foundation} = 20(10) = 200 \text{ piles}$$

$$Q_{\text{allowable (total)}} = \frac{494.7+(200 \times 339.3)}{4} = 17080 \text{ kPa}$$





-Determination of Shear & Moment Diagram for “Strip 1”

$$\text{Average soil pressure: } q_{av} = \frac{q_{(At A19)} + q_{(N19)}}{2} = \frac{2016.7 + 2397.1}{2} = 2206.9 \text{ KN/m}^2$$

$$\text{Total soil reaction: } q_{av} B, L = 2206.9 \left[\left(\frac{5}{2}\right) + \left(\frac{5.55}{2}\right) \right] (63.2) = 735736.322 \text{ KN}$$

$$\text{Average load} = \frac{\text{load due to soil reaction} + \text{column loads}}{2}$$

$$= \frac{735736.322 + 1245988.3}{2} = 990862.311 \text{ KN}$$

$$\text{Modified average soil pressure: } q_{av(\text{modified})} = q_{av} \left(\frac{\text{Average load}}{\text{Total soil reaction}} \right)$$

$$q_{av(\text{modified})} = 2206.9 \left(\frac{990862.311}{735736.322} \right) = 2972.171 \text{ KN/m}^2$$

$$\text{Column loads Factor: } F = \frac{\text{Average load}}{\text{Column loads}} = \frac{990862.311}{1245988.3} = 0.7952421$$

$$\text{Load per unit length: } B q_{av(\text{modified})} = 5.275(2972.171) = 15678.2 \text{ KN}/m^2$$

-Determination of Shear & moment Diagram for “Strip 2”

$$\text{Average soil pressure: } q_{av} = \frac{q_{(At M1)} + q_{(At M29)}}{2} = \frac{1846.9 + 2623.3}{2} = 2235.1 \text{ KN}/m^2$$

$$\text{Total soil reaction: } q_{av} B, L = 2235.1 \left[\left(\frac{5.15}{2} \right) + \left(\frac{5.25}{2} \right) \right] (123.2) = 1431894.5 \text{ KN}$$

$$\begin{aligned} \text{Average load} &= \frac{\text{load due to soil reaction} + \text{column loads}}{2} \\ &= \frac{1431894.5 + 1709576.2}{2} = 1570735.4 \text{ KN} \end{aligned}$$

$$\text{Modified average soil pressure: } q_{av(\text{modified})} = q_{av} \left(\frac{\text{Average load}}{\text{Total soil reaction}} \right)$$

$$q_{av(\text{modified})} = 2235.1 \left(\frac{1570735.4}{1431894.5} \right) = 2451.82 \text{ KN}/m^2$$

$$\text{Column loads Factor: } F = \frac{\text{Average load}}{\text{Column loads}} = \frac{1570735.4}{1709576.2} = 0.91878$$

$$\text{Load per unit length: } B q_{av(\text{modified})} = 5.2(2451.82) = 12749.5 \text{ KN/m}^2$$

Mat Foundation Thickness for RCC Model:

The critical section for diagonal tension shear will be at the column B19.

Column B19 is carrying 11451.7 kN.

Determining the effective depth by the following equation from American Concrete Institute (ACI) code:

$$U = b_o d [\varphi(0.34)\sqrt{f'c'}]$$

$$U = \text{column load} \times \text{load factor}$$

$$U = 11451.7(1.7)$$

$$U = 194805.9 \text{ kN} = 194.81 \text{ MN}$$

$$b_o = 2(3.2 + d) + 2(3.2 + d)$$

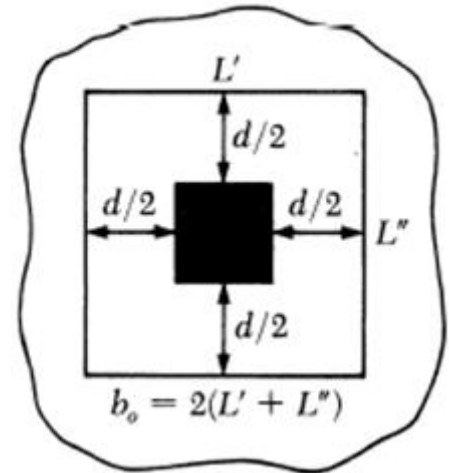
$$b_o = 6.4 + 2d + 6.4 + 2d$$

$$b_o = 12.8 + 4d$$

$$U = b_o d [\phi(0.34)\sqrt{f'c'}]$$

$$194.81 = (12.8 + 4d)d [0.85(0.34)\sqrt{69}]$$

$$d = 3.18 \text{ m}$$



Steel Reinforcement:

$$M_{\max (\text{strip } 1)} = 42940.03 \text{ kN.m}$$

$$M_{\min (\text{strip } 1)} = -27259.7 \text{ kN.m}$$

$$M_{\max (\text{strip } 2)} = 33976.7 \text{ kN.m}$$

$$M_{\min (\text{strip } 2)} = -18557.2 \text{ kN.m}$$

NOTE:

Strip 1: along Y direction

Strip 2: along X direction

- Strip 1:

Calculating the required steel reinforcement for the maximum positive moment using the following equation from ACI:

$$M' = \frac{M_{max}}{\beta_1}$$

$$M' = \frac{42940.03}{5.275}$$

$$M' = 8140.29 \frac{kN.m}{m}$$

$$M_u = M' (\text{load factor}) = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$8140.29(1.7) = 0.9 A_s (345000) \left(3.18 - \frac{a}{2}\right)$$

$$a = \frac{A_s(f_y)}{0.85f_c'b} = \frac{A_s(345)}{0.85(69)(1)} = 5.882 A_s \rightarrow A_s = 0.17 a$$

$$8140.29(1.7) = 0.9 A_s (345000)(2.99 - \frac{a}{2})$$

$$8140.29 (1.7) = 0.9 (0.17 a) (345000)(2.99 - \frac{a}{2})$$

$$a = 0.083539 \text{ m}$$

$$A_s = 0.17 (0.083539)$$

$$= 0.0142 \text{ m}^2/\text{m}$$

$$= 14201.63 \text{ mm}^2/\text{m}$$

Using 12# 40 @ 80 mm c/c $\rightarrow A_s = 15080 \text{ mm}^2$

- Strip 1:

Calculating the required steel reinforcement for the maximum negative moment

$$M' = \frac{M \text{ min}}{\beta_1}$$

$$M' = \frac{27259.7}{5.275}$$

$$M' = 5167.7 \frac{\text{kN.m}}{\text{m}}$$

$$M_u = M' (\text{load factor}) = \phi A_s f_y (d - \frac{a}{2})$$

$$5167.7(1.7) = 0.9 A_s (345000)(2.99 - \frac{a}{2})$$

$$a = \frac{As(fy)}{0.85fc'b} = \frac{As(345)}{0.85(69)(1)} = 5.882 As \rightarrow As = 0.17 a$$

$$5167.7(1.7) = 0.9 As (345000)(2.99 - \frac{a}{2})$$

$$5167.7 (1.7) = 0.9 (0.17 a) (345000)(2.99 - \frac{a}{2})$$

$$a = 0.052774 m$$

$$As = 0.17 (0.052774)$$

$$= 0.008971 m^2/m$$

$$= 8971.6 mm^2/m$$

Using 12 # 32 @ 80mm c/c $\rightarrow As = 9651 mm^2$

- Strip 2:

Calculating the required steel reinforcement for the maximum positive moment using the following equation from ACI:

$$M' = \frac{M \max}{\beta 1}$$

$$M' = \frac{33976.7}{5.2}$$

$$M' = 6533.98 \frac{kN.m}{m}$$

$$M_u = M' (load \ factor) = \phi As fy(d - \frac{a}{2})$$

$$6533.98(1.7) = 0.9 As (345000)(2.99 - \frac{a}{2})$$

$$a = \frac{As(fy)}{0.85fc'b} = \frac{As(345)}{0.85(69)(1)} = 5.882 As \rightarrow As = 0.17 a$$

$$6533.98(1.7) = 0.9 As (345000)(2.99 - \frac{a}{2})$$

$$6533.98(1.7) = 0.9 (0.17 a) (345000)(2.99 - \frac{a}{2})$$

$$a = 0.0668774 m$$

$$As = 0.17 (0.0668774)$$

$$= 0.0113691 m^2/m$$

$$= 11369.2 mm^2/m$$

Using 12 # 36 @ 80 mm c/c $\rightarrow As = 12215 mm^2$

- Strip 2:

Calculating the required steel reinforcement for the maximum negative moment

$$M' = \frac{M_{min}}{\beta_1}$$

$$M' = \frac{18557.2}{5.2}$$

$$M' = 3568.69 \frac{kN.m}{m}$$

$$M_u = M' (load factor) = \phi As fy (d - \frac{a}{2})$$

$$3568.69(1.7) = 0.9 As (345000)(2.99 - \frac{a}{2})$$

$$a = \frac{A_s(f_y)}{0.85f_c'b} = \frac{A_s(345)}{0.85(69)(1)} = 5.882 A_s \rightarrow A_s = 0.17 a$$

$$3568.69(1.7) = 0.9 A_s (345000)(2.99 - \frac{a}{2})$$

$$3568.69(1.7) = 0.9 (0.17 a) (345000)(2.99 - \frac{a}{2})$$

$$a = 0.03635 \text{ m}$$

$$A_s = 0.17 (0.03635)$$

$$= 0.0061795 \text{ m}^2/\text{m}$$

$$= 6179.5 \text{ mm}^2/\text{m}$$

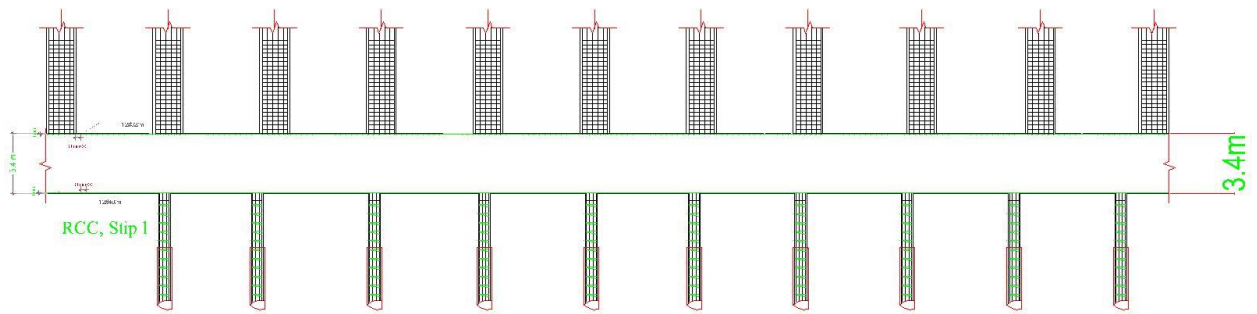
Using 10 # 30 @ 90 mm c/c $\rightarrow A_s = 7069 \text{ mm}^2$

The total depth of the MAT foundation:

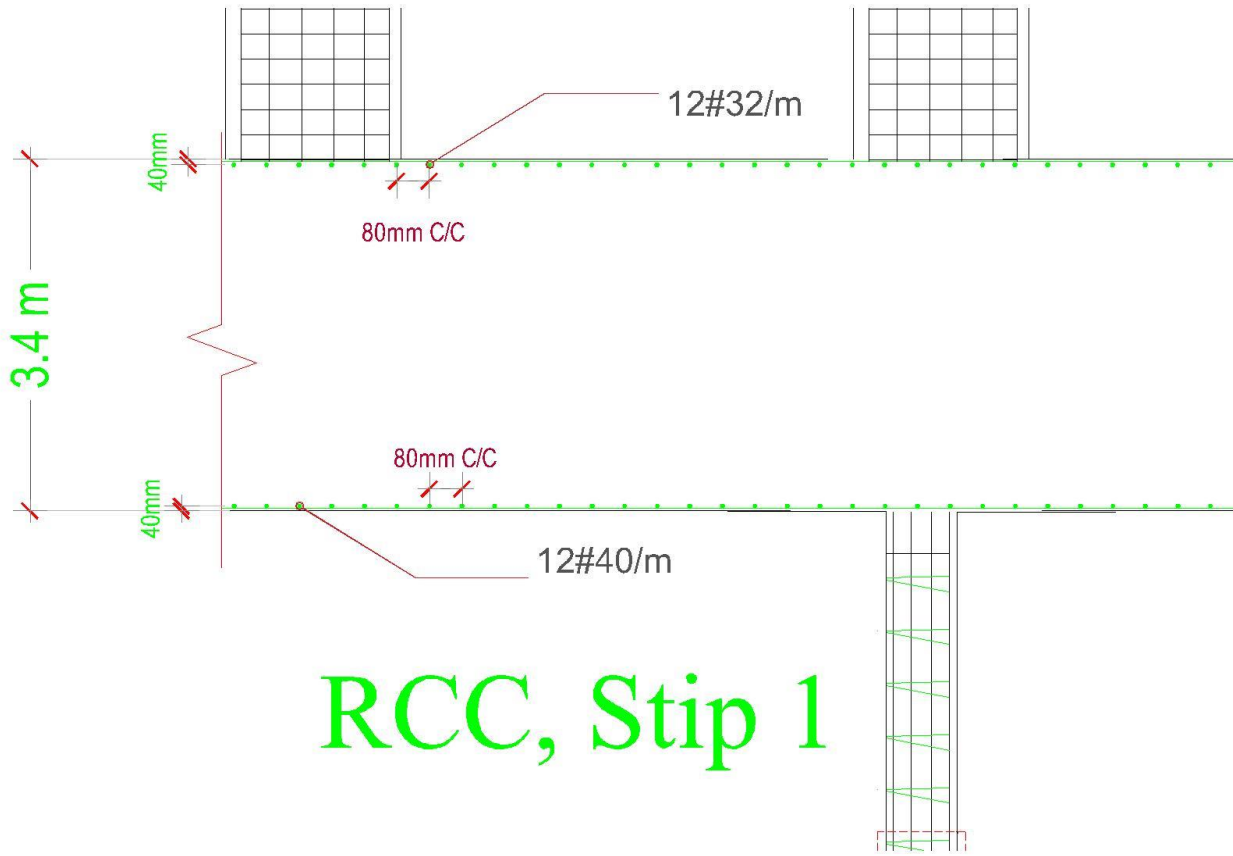
$$d = d_{\text{Concrete}} + \text{concrete cover} + \text{rebar diameter}$$

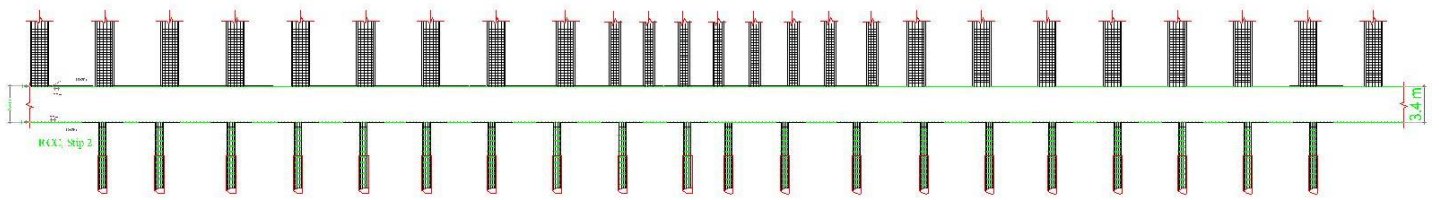
$$3.18 + 0.08 + 0.04 + 0.032 + 0.036 + 0.029 = 3.397\text{m. This is the theoretical depth.}$$

The practical depth is 3.4m

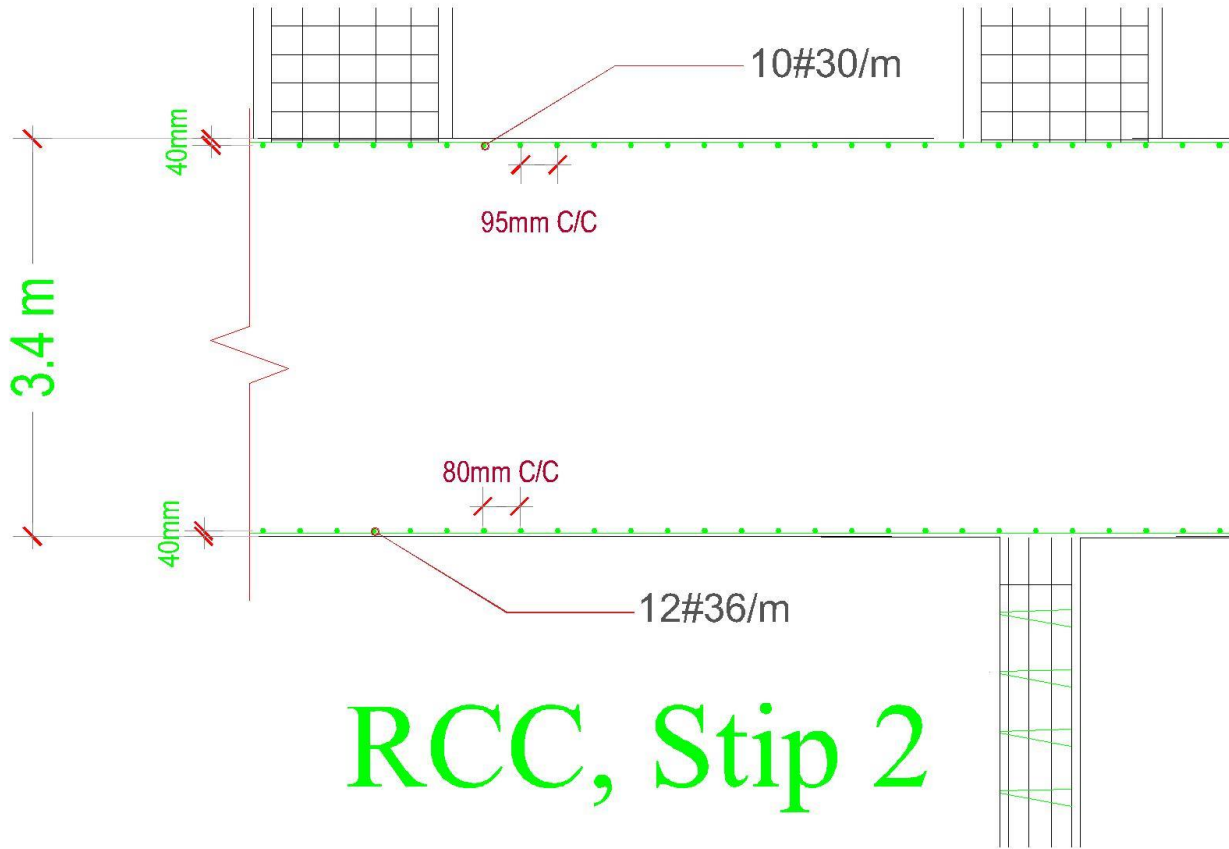


RCC, Stip 1





RCC, Stip 2



RCC, Stip 2

5.3 Geotechnical Design for CLT Model:

Bearing Capacity of the soil:

$$R_c = 11474 \text{ /m}^2, \text{ RQD} = 54\%$$

$$Q_{\text{all}} = \frac{R_c \times 0.8}{10} \times \text{RQD} = \frac{11747 \times 0.8}{10} \times 0.54 = 495.7 \text{ kN/m}^2$$

Mat foundation design :- (CLT slab)

$$q = \frac{Q}{A} \pm \frac{M_x}{I_x} y \pm \frac{M_y}{I_y} x$$

$$A = 2 [44.45(62.9)] + 34 (61.2) = 7692.61 \text{ m}^2$$

$$I_x = 2 \left[\frac{1}{12} (44.45)(62.9)^3 \right] + \frac{1}{12} (34) (61.2)^3 = 2493083.7 \text{ m}^4$$

$$I_y = 2 \left[\frac{1}{12} (62.9)(44.45)^3 \right] + \frac{1}{12} (61.2) (34)^3 = 1121143.08 \text{ m}^4$$

$$\text{Excel} \leftarrow Q = 14031955 \text{ kN}, \quad x = 61.9059 \text{ m}, \quad y^1 = 32.3349 \text{ m}$$

$$e_x = 61.9059 - \frac{122.9}{2} = 0.4559 \text{ m}$$

$$e_y = 32.3349 - \frac{62.9}{2} = 0.8849 \text{ m}$$

$$M_y = 14031955 (0.4559) = 6397168.29 \text{ kN.m}$$

$$M_x = 14031955 (0.8849) = 12416877 \text{ kN.m}$$

$$\text{So, } q = \frac{14031955}{7672.61} \pm \frac{12416877y}{2493083.7} \pm \frac{6397168.29x}{1121143.08}$$

$$q = 1815.2 \pm 4.87y \pm 5.68x$$

Strip 1:-

$$\text{At A19: } 1815.2 - 4.87(30) + 5.68(20) = 1782.7 \text{ KN/m}^2$$

$$\text{At B19: } 1815.2 - 4.87(24.025) + 5.68(20) = 1811.8 \text{ KN/m}^2$$

$$\text{At C19: } 1815.2 - 4.87(18.05) + 5.68(20) = 1840.9 \text{ KN/m}^2$$

$$\text{At D19: } 1815.2 - 4.87(12.55) + 5.68(20) = 181867.7 \text{ KN/m}^2$$

$$\text{At F19: } 1815.2 - 4.87(7.05) + 5.68(20) = 1894.5 \text{ KN/m}^2$$

$$\text{At G19: } 1815.2 - 4.87(1.55) + 5.68(20) = 1921.3 \text{ KN/m}^2$$

$$\text{At H19: } 1815.2 - 4.87(3.95) + 5.68(20) = 1948 \text{ KN/m}^2$$

$$\text{At I19: } 1815.2 - 4.87(9.45) + 5.68(20) = 1974.8 \text{ KN/m}^2$$

$$\text{At K19: } 1815.2 - 4.87(14.95) + 5.68(20) = 2001.6 \text{ KN/m}^2$$

$$\text{At L19: } 1815.2 - 4.87(19.6) + 5.68(20) = 2024.3 \text{ KN/m}^2$$

$$\text{At M19: } 1815.2 - 4.87(24.85) + 5.68(20) = 2049.8 \text{ KN/m}^2$$

$$\text{At N19: } 1815.2 - 4.87(30) + 5.68(20) = 2074.9 \text{ KN/m}^2$$

Strip 2:-

$$q = 1815.2 \pm 4.87y \pm 5.68x$$

$$\text{At M1: } 1815.2 + 4.87(24.85) - 5.68(60) = 1595.4 \text{ KN/m}^2$$

$$\text{At M2: } 1815.2 + 4.87(24.85) - 5.68(54.45) = 1626.9 \text{ KN/m}^2$$

$$\text{At M3: } 1815.2 + 4.87(24.85) - 5.68(49.5) = 1655.1 \text{ KN/m}^2$$

$$\text{At M4: } 1815.2 + 4.87(24.85) - 5.68(44.35) = 1684.3 \text{ KN/m}^2$$

$$\text{At M6: } 1815.2 + 4.87(24.85) - 5.68(38.85) = 1715.6 \text{ KN/m}^2$$

$$\text{At M8: } 1815.2 + 4.87(24.85) - 5.68(33.35) = 1746.8 \text{ KN/m}^2$$

$$\text{At M9: } 1815.2 + 4.87(24.85) - 5.68(28.2) = 1776 \text{ KN/m}^2$$

$$\text{At M10: } 1815.2 + 4.87(24.85) - 5.68(23.25) = 1804.2 \text{ KN/m}^2$$

$$\text{At M11: } 1815.2 + 4.87(24.85) - 5.68(20) = 1822.6 \text{ KN/m}^2$$

$$\text{At M12: } 1815.2 + 4.87(24.85) - 5.68(15) = 1851 \text{ KN/m}^2$$

$$\text{At M13: } 1815.2 + 4.87(24.85) - 5.68(10) = 1879.4 \text{ KN/m}^2$$

$$\text{At M14: } 1815.2 + 4.87(24.85) - 5.68(5) = 1907.8 \text{ KN/m}^2$$

$$\text{At M15: } 1815.2 + 4.87(24.85) - 5.68(0) = 1936.22 \text{ KN/m}^2$$

$$\text{At M16: } 1815.2 + 4.87(24.85) - 5.68(5) = 1964.6 \text{ KN/m}^2$$

$$\text{At M17: } 1815.2 + 4.87(24.85) - 5.68(19) = 1993 \text{ KN/m}^2$$

$$\text{At M18: } 1815.2 + 4.87(24.85) - 5.68(15) = 2021.4 \text{ KN/m}^2$$

$$\text{At M19: } 1815.2 + 4.87(24.85) - 5.68(20) = 2049.8 \text{ KN/m}^2$$

$$\text{At M20: } 1815.2 + 4.87(24.85) - 5.68(25.55) = 2081.3 \text{ KN/m}^2$$

$$\text{At M21: } 1815.2 + 4.87(24.85) - 5.68(30.5) = 2109.5 \text{ KN/m}^2$$

$$\text{At M22: } 1815.2 + 4.87(24.85) - 5.68(35.65) = 2138.7 \text{ KN/m}^2$$

$$\text{At M24: } 1815.2 + 4.87(24.85) - 5.68(41.15) = 2170 \text{ KN/m}^2$$

$$\text{At M26: } 1815.2 + 4.87(24.85) - 5.68(46.65) = 2201.2 \text{ KN/m}^2$$

$$\text{At M27: } 1815.2 + 4.87(24.85) - 5.68(51.8) = 2230.4 \text{ KN/m}^2$$

$$\text{At M28: } 1815.2 + 4.87(24.85) - 5.68(56.75) = 2258.6 \text{ KN/m}^2$$

$$\text{At M1: } 1815.2 + 4.87(24.85) - 5.68(60) = 2277 \text{ KN/m}^2$$

All column pressures are more than the allowable soil bearing capacity (q_{all}) 495.7 kN/m^2 . Therefore, we are providing bored pile to support the allowable soil bearing capacity. So, each column pressure become less than the net bearing capacity. The designed bored piles are applicable for both model RCC & CLT. We developed excel sheet with different length and diameter of piles group. We concluded with efficient pile length and diameter.

Pile Design Calculation:

$$Q_i = \frac{Q_p + Q_s}{4}$$

$$Q_p = \frac{R_c \times 0.8}{10} \times RQD = \frac{11474(0.8)}{10} (0.54) = 495.7 \text{ kPa}$$

$$Q_s = P L f, \quad f = \alpha C_u$$

$$\alpha = 0.24$$

$$Q_s = (\pi \times 0.6)(3)(0.24 \times 250) = 339.9 \text{ kPa [for one pile]}$$

$$Q_i = \frac{495.7+339.3}{4} = 208.8 \text{ kPa [for one pile]}$$

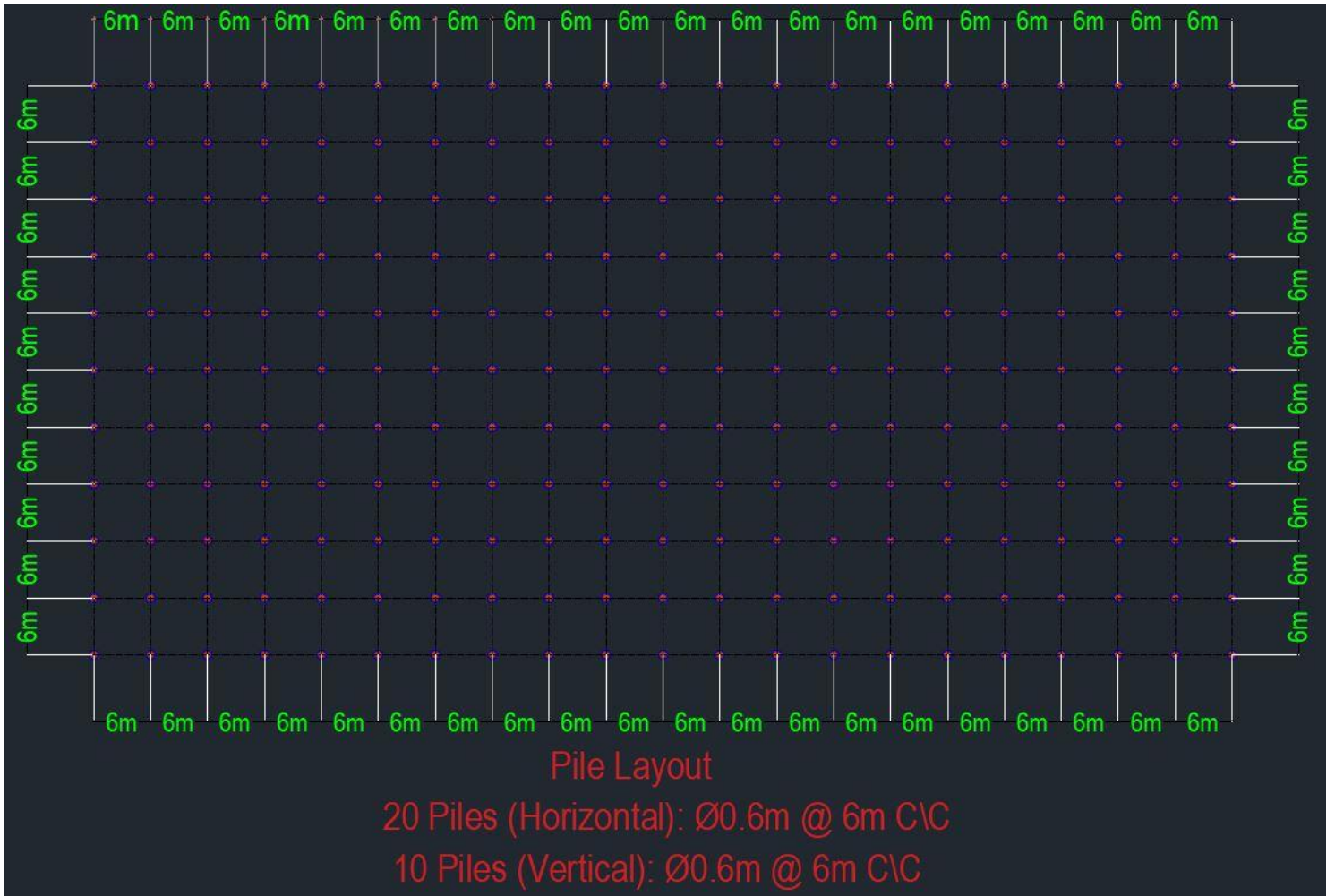
$$S = 10D = 10 (0.6) = 6 \text{ m}$$

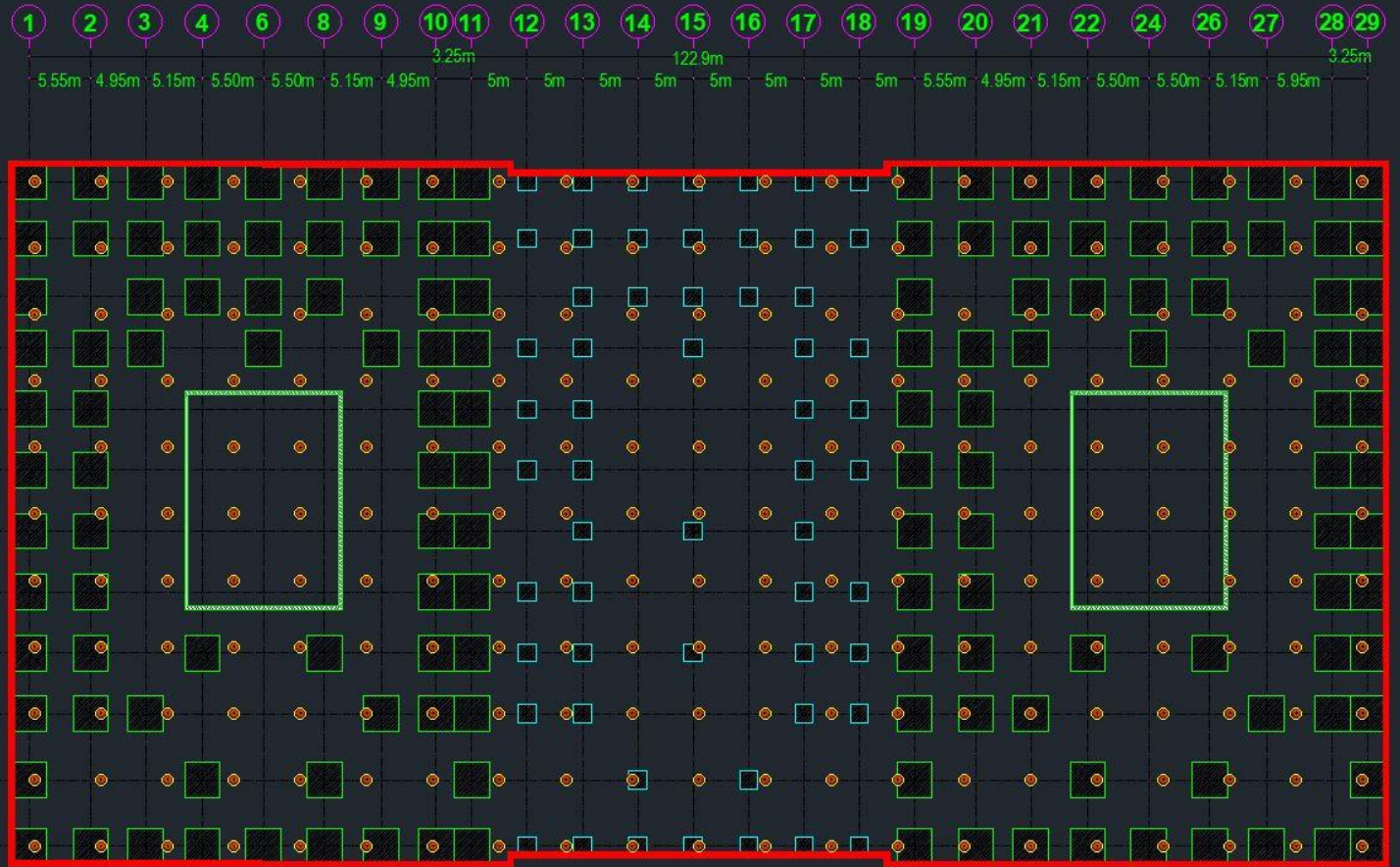
$$\text{Number of piles}_{(x\text{-axis})} = \frac{123.2}{6} = 20 \text{ piles}$$

$$\text{Number of piles}_{(y\text{-axis})} = \frac{63.2}{6} = 10 \text{ piles}$$

$$\text{Number of piles under the MAT foundation} = 20(10) = 200 \text{ piles}$$

$$Q_{\text{allowable (total)}} = \frac{494.7+(200 \times 339.3)}{4} = 17080 \text{ kPa}$$





Foundation Layout (CLT Model)

Determination of shear & moment Diagram for “Strip 1” :

$$q_{av} = \frac{1782.7+2074.9}{2} = 1928.8 \text{ KN}/m^2$$

Total soil reaction : $q_{av} B,L = 1928.8(5.275)(62.9) = 639971.02 \text{ KN}$

$$\text{Average load} = \frac{639971.02 + 1074464}{2} = 857217.86 \text{ KN}$$

$$q_{av(mod.)} = 1928.8 \left(\frac{857217.86}{1074464.7} \right) = 1538.81 \text{ KN}/m^2$$

$$\text{Column loads factor: } F = \frac{857217.86}{1074464.7} = 0.7978092$$

Load per unit length: B, $q_{av(mod.)} = 5.275(1538.81) = 8117.22 \text{ KN}/m$

Then show the result through SAP2000.

- Determination of shear & Moment diagram for “Strip 2”:

$$q_{av} = \frac{1595.4+2277}{2} = 1936.2 \text{ KN}/m^2$$

$$\text{Total soil reaction : } q_{av} B,L = 1936.2(5.2)(122.9) = 1237386.7$$

$$\text{Average load} = \frac{1237386.7 + 1462994.8}{2} = 1350190.75 \text{ KN}$$

$$q_{av(modified)} = 1936.2 \left(\frac{1350190.75}{1462994.8} \right) = 1786.91 \text{ KN}/m^2$$

$$\text{Column loads factor: } F = \frac{1350190.75}{1462994.8} = 0.9228951$$

$$\text{Load per unit length: } B, q_{av(mod.)} = 5.2(1786.91) = 9291.9 \text{ KN}/m^2$$

Mat Foundation Thickness for CLT Model:

The critical section for diagonal tension shear will be at the column B19.

Column B19 is carrying 99394 kN.

Determining the effective depth by the following equation from American Concrete Institute (ACI) code:

$$U = b_o d [\varphi(0.34)\sqrt{f'c'}]$$

$U =$ column load x load factor

$$U = 99394(1.7)$$

$$U = 168970 \text{ kN} = 168.97 \text{ MN}$$

$$b_o = 2(2.9 + d) + 2(2.9 + d)$$

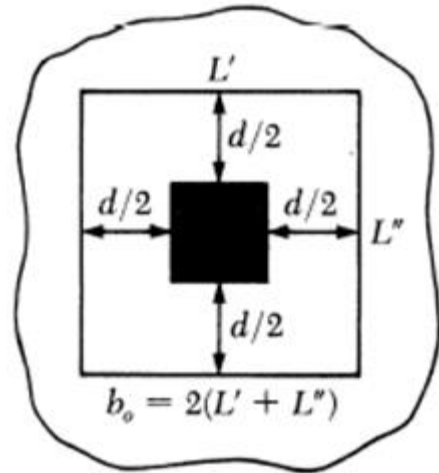
$$b_o = 5.8 + 2d + 5.8 + 2d$$

$$b_o = 11.6 + 4d$$

$$U = b_o d [\varphi(0.34)\sqrt{f'c'}]$$

$$168.97 = (11.6 + 4d)d [0.85(0.34)\sqrt{69}]$$

$$d = 2.99 \text{ m}$$



Steel Reinforcement:

$$M_{\max (\text{strip } 1)} = 26546.5 \text{ kN.m}$$

$$M_{\min (\text{strip } 1)} = -13710.2 \text{ kN.m}$$

$$M_{\max (\text{strip } 2)} = 24435.7 \text{ kN.m}$$

$$M_{\min (\text{strip } 2)} = -13542.5 \text{ kN.m}$$

NOTE:

Strip 1: along Y direction

Strip 2: along X direction

- Strip 1:

Calculating the required steel reinforcement for the maximum positive moment using the following equation from ACI:

$$M' = \frac{M_{max}}{\beta_1}$$

$$M' = \frac{26546.5}{5.275}$$

$$M' = 5032.5 \frac{kN.m}{m}$$

$$M_u = M' (\text{load factor}) = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$5032.5 (1.7) = 0.9 A_s (345000) \left(2.99 - \frac{a}{2}\right)$$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s (345)}{0.85 (69) (1)} = 5.882 A_s \rightarrow A_s = 0.17 a$$

$$5032.5 (1.7) = 0.9 A_s (345000) \left(2.99 - \frac{a}{2}\right)$$

$$5032.5 (1.7) = 0.9 (0.17 a) (345000) \left(2.99 - \frac{a}{2}\right)$$

$$a = 0.054707 m$$

$$A_s = 0.17 (0.054707)$$

$$= 0.0093 m^2/m$$

$$= 9300.2 mm^2/m$$

Using 10 # 36 @ 90 mm c/c $\rightarrow A_s = 10179 mm^2$

- Strip 1:

Calculating the required steel reinforcement for the maximum negative moment

$$M' = \frac{M \text{ min}}{\beta 1}$$

$$M' = \frac{137103.2}{5.275}$$

$$M' = 2599.1 \frac{kN.m}{m}$$

$$M_u = M' (\text{load factor}) = \phi A_s f_y (d - \frac{a}{2})$$

$$2599.1 (1.7) = 0.9 A_s (345000)(2.99 - \frac{a}{2})$$

$$a = \frac{A_s(f_y)}{0.85f_c'b} = \frac{A_s(345)}{0.85(69)(1)} = 5.882 A_s \rightarrow A_s = 0.17 a$$

$$2599.1 (1.7) = 0.9 A_s (345000)(2.99 - \frac{a}{2})$$

$$2599.1 (1.7) = 0.9 (0.17 a) (345000)(2.99 - \frac{a}{2})$$

$$a = 0.028128 m$$

$$A_s = 0.17 (0.028128)$$

$$= 0.004782 m^2/m$$

$$= 4781.8 mm^2/m$$

Using 10 # 25 @ 90mm c/c $\rightarrow A_s = 4909 mm^2$

- Strip 2:

Calculating the required steel reinforcement for the maximum positive moment using the following equation from ACI:

$$M' = \frac{M \max}{\beta 1}$$

$$M' = \frac{24435.7}{5.2}$$

$$M' = 4699.17 \frac{kN.m}{m}$$

$$M_u = M' (\text{load factor}) = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$4699.17 (1.7) = 0.9 A_s (345000) \left(2.99 - \frac{a}{2}\right)$$

$$a = \frac{A_s(f_y)}{0.85 f_c' b} = \frac{A_s(345)}{0.85(69)(1)} = 5.882 A_s \rightarrow A_s = 0.17 a$$

$$4699.17 (1.7) = 0.9 A_s (345000) \left(2.99 - \frac{a}{2}\right)$$

$$4699.17 (1.7) = 0.9 (0.17 a) (345000) \left(2.99 - \frac{a}{2}\right)$$

$$a = 0.051052 m$$

$$A_s = 0.17 (0.051052)$$

$$= 0.00867884 m^2/m$$

$$= 8678.84 mm^2/m$$

Using 11 # 32 @ 80 mm c/c $\rightarrow A_s = 8847 mm^2$

- Strip 2:

Calculating the required steel reinforcement for the maximum negative moment

$$M' = \frac{M \text{ min}}{\beta 1}$$

$$M' = \frac{13542.5}{5.2}$$

$$M' = 2604.3 \frac{kN.m}{m}$$

$$M_u = M' (\text{load factor}) = \phi A_s f_y (d - \frac{a}{2})$$

$$2604.3 (1.7) = 0.9 A_s (345000)(2.99 - \frac{a}{2})$$

$$a = \frac{A_s(f_y)}{0.85f_c'b} = \frac{A_s(345)}{0.85(69)(1)} = 5.882 A_s \rightarrow A_s = 0.17 a$$

$$2599.1 (1.7) = 0.9 A_s (345000)(2.99 - \frac{a}{2})$$

$$2599.1 (1.7) = 0.9 (0.17 a) (345000)(2.99 - \frac{a}{2})$$

$$a = 0.0281844 \text{ m}$$

$$A_s = 0.17 (0.0281844)$$

$$= 0.00479135 \text{ m}^2/\text{m}$$

$$= 4791.35 \text{ mm}^2/\text{m}$$

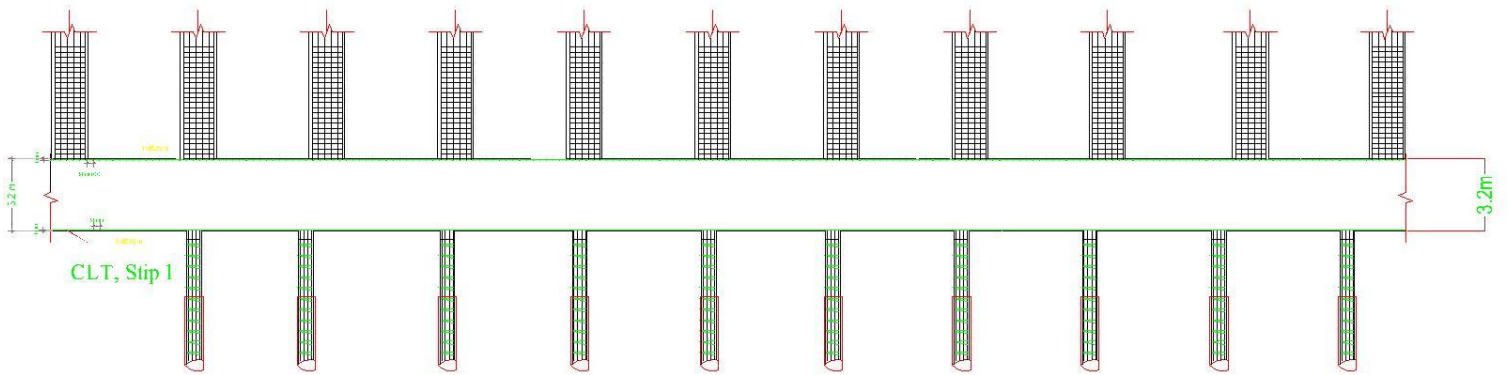
Using 10# 25 @ 90 mm c/c $\rightarrow A_s = 4909 \text{ mm}^2$

The depth of the MAT foundation:

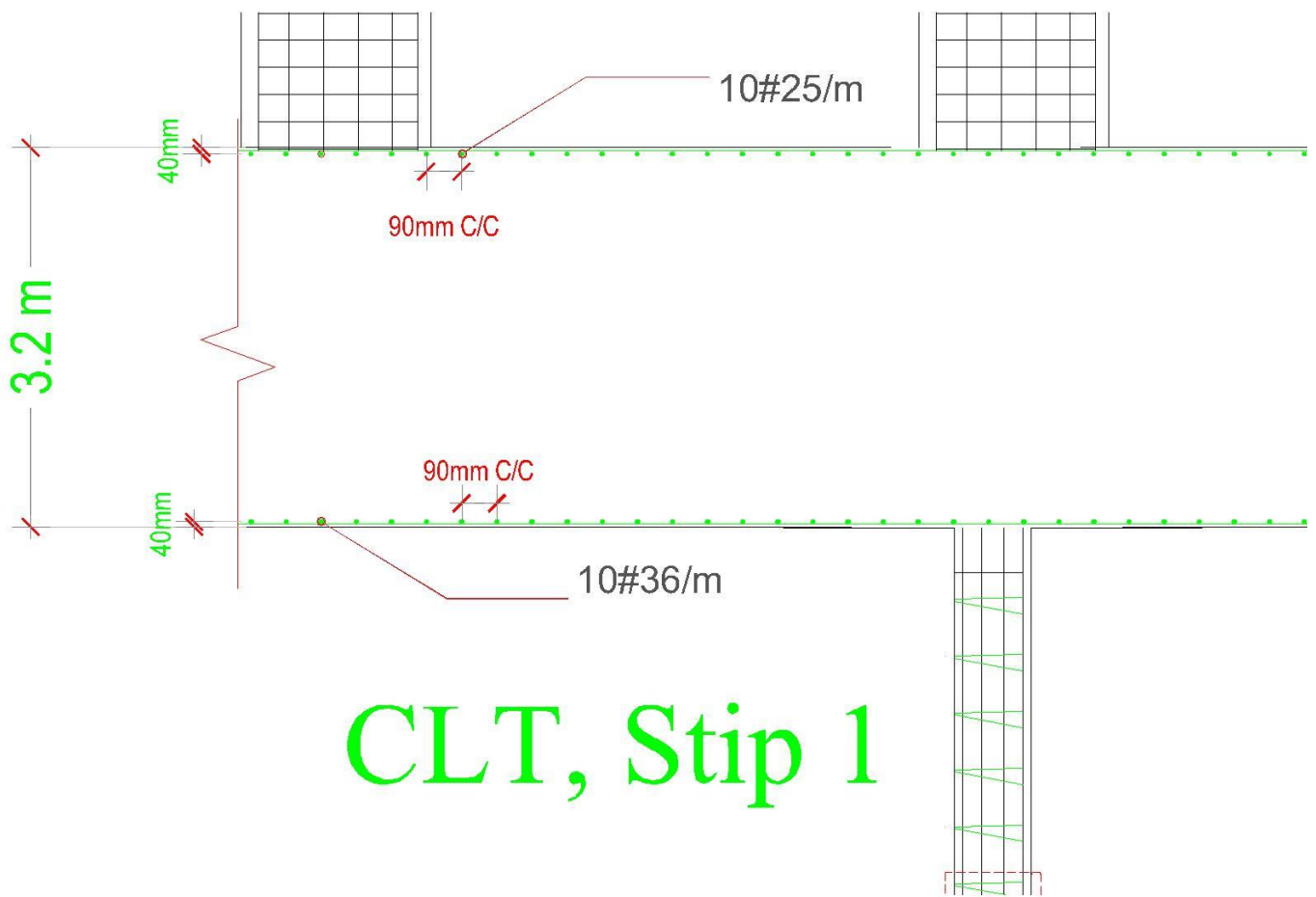
$$d = d_{\text{Concrete}} + \text{concrete cover} + \text{rebar diameter}$$

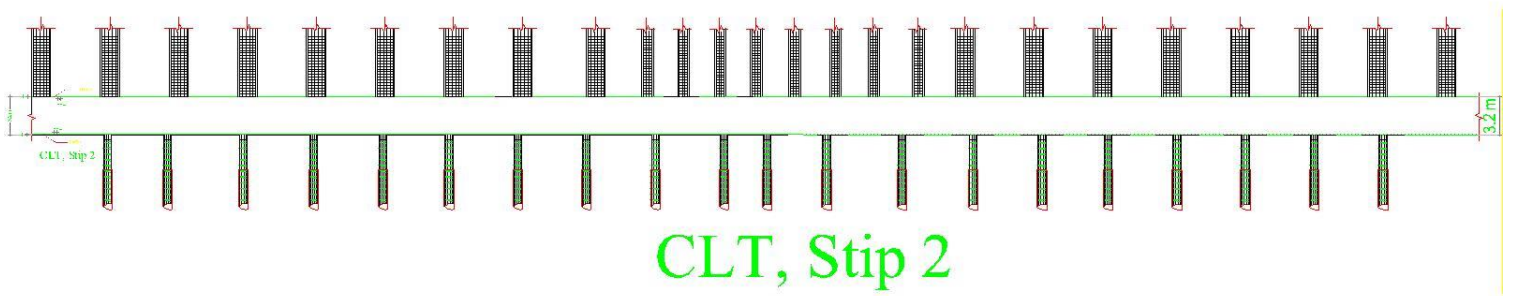
$2.99 + 0.08 + 0.036 + 0.025 + 0.32 + 0.025 = 3.188m$. This is the theoretical depth.

The practical depth is $3.2m$

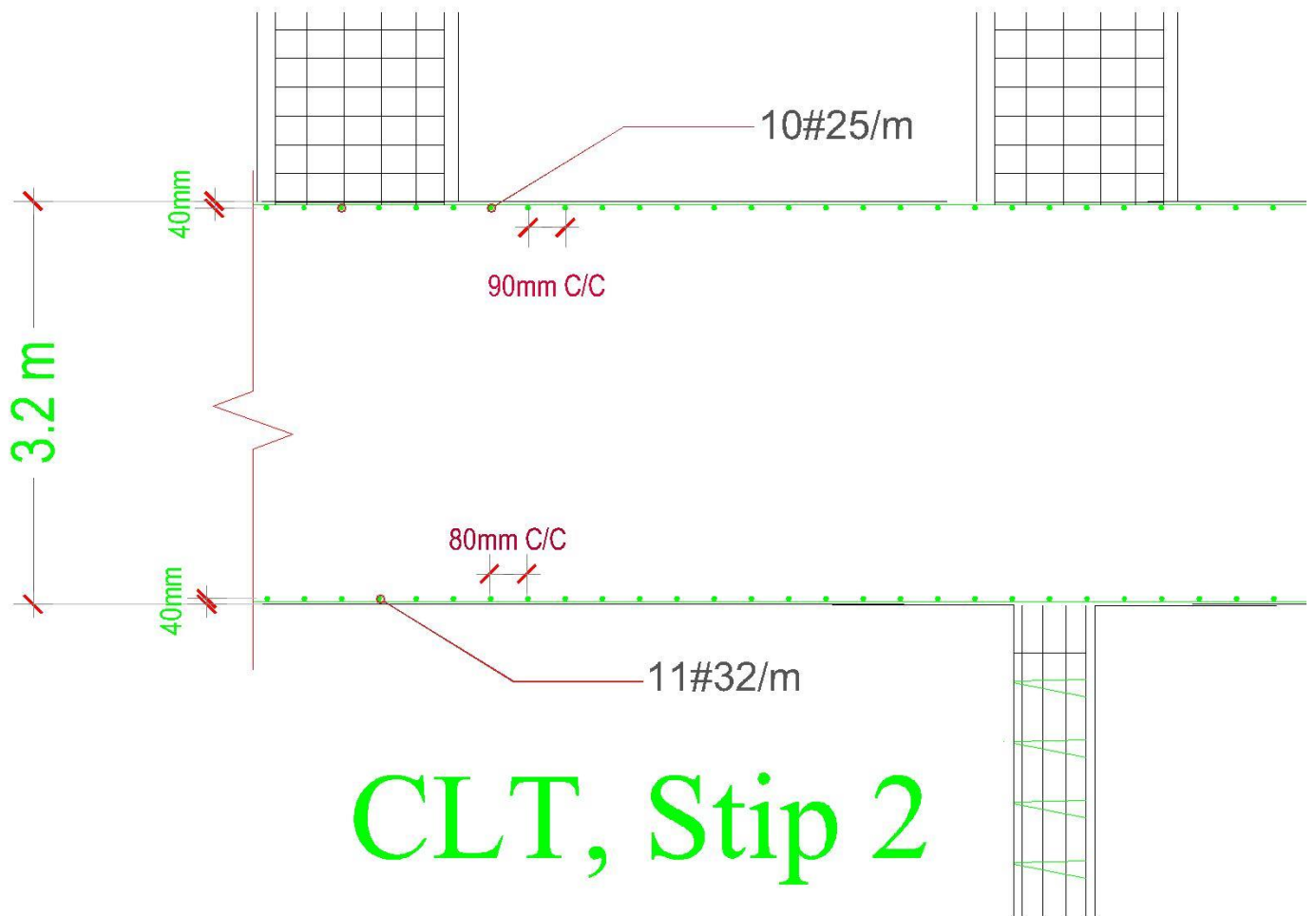


CLT, Stip 1





CLT, Stip 2



Chapter 6: Comparison between RCC & CLT models.

Introduction:

One of our senior design project objectives is to compare between CLT & RCC slab floor system. The comparison between both systems will be about structural members (beams, columns, shear wall), slab deflection, lateral displacement, foundation and cost. The numbers, figures and tables used are only representative and needed details will be presented in the appendixes.

6.1 Structural Members:

The structural members are beams, columns and shear wall. The comparison is going to be about member size or dimension and required reinforcements for beams and columns. Regarding the shear wall, the comparison is going to be about shear wall thickness and required reinforcements.

6.1.1 Beams:

The whole building has two different beam cross sections in both models RCC & CLT. However, the required reinforcements vary. The beams have the same cross section but different steel reinforcements due to the verity of the live load 5 kN/m^2 , 7.5 kN/m^2 & 10 kN/m^2 . Different live loads with different span generate different moment. Therefore, the required steel reinforcement quantity will be different but beam cross section stays the same.

6.1.2 Columns:

The loads carried by columns are accumulative. Since the concrete is heavier than timber, this made an impact on the columns size and required steel reinforcements.

Model with CLT slab floor system		Model with RCC slab floor system	
Storey levels	Columns Dimensions	Storey levels	Columns Dimensions
1-4	2.9 m X 2.9 m	1-4	3.2 m X 3.2 m
5-12	2.7 m X 2.7 m	5-12	3.1 m X 3.1 m
13-20	2.5 m X 2.5 m	13-20	2.8 m X 2.8 m
21-28	2.3 m X 2.3 m	21-28	2.6 m X 2.6 m
29-36	2.1 m X 2.1 m	29-36	2.4 m X 2.4 m
37-44	1.8 m X 1.8 m	37-44	2.1 m X 2.1 m
45-52	1.5 m X 1.5 m	45-52	1.8 m X 1.8 m
53-56	1.2 m X 1.2 m	53-56	1.3 m X 1.3 m
57-60	0.7 m X 0.7 m	57-60	0.7 m X 0.7 m

6.1.3 Shear Wall:

Many beams are resting on the shear wall and these beams are carrying the concrete slab in RCC model or the timber slab in the CLT model; the load carried by the shear wall is accumulative as well as columns case. Since the concrete is heavier than timber, this made an impact on the shear wall thickness and required steel reinforcements.

	Concrete	CLT
Shear wall thickness	0.6m	0.5m

6.2 Deflection:

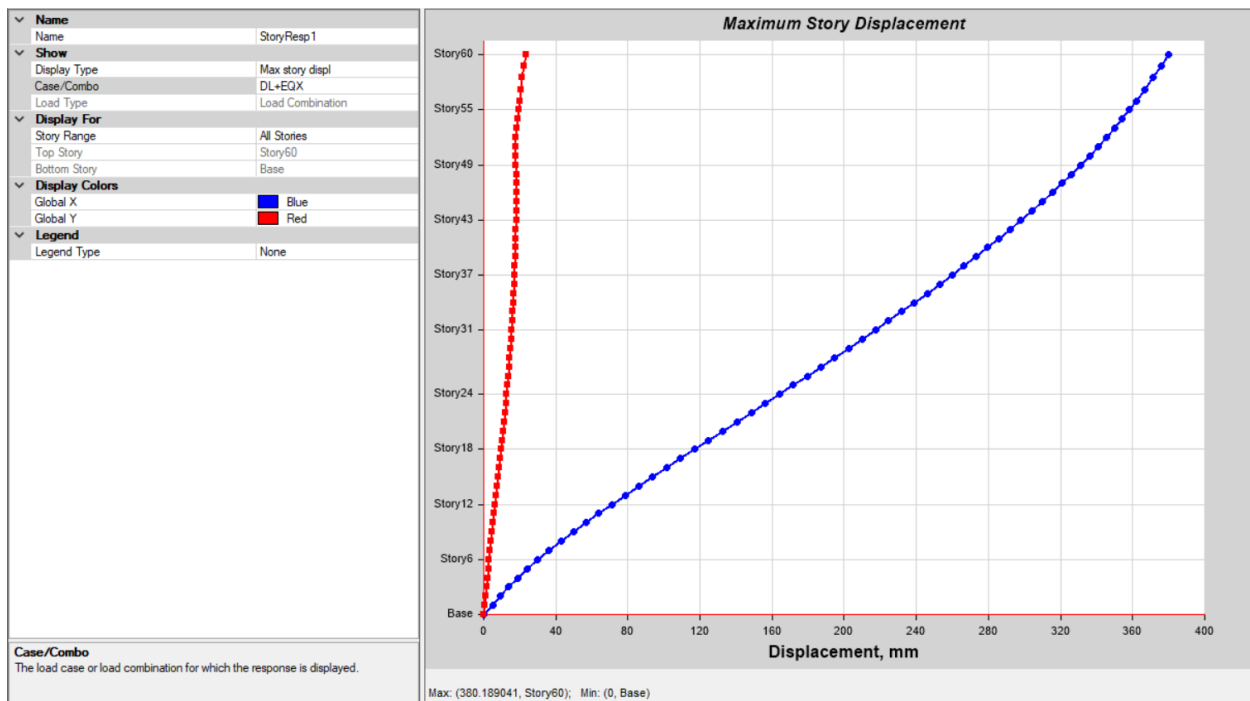
6.3 lateral Displacement:

According to SBC, the allowable drift of the building is 0.02% of the total height of the building. Wind load and seismic force including the existed active loads (live load & dead load) cause the lateral displacement. We found out that the concrete slab is rigid and timber slab is flexible. RCC model has less displacement than CLT model in load combination (dead load + wind load on X direction). On the other load combination (dead load + seismic force on X direction), RCC model has higher displacement than CLT model. Because of timber slab is flexible and can absorb seismic forces unlike concrete slab very rigid. The flexibility or rigidity of a material have advantages and dis advantages and this is what happened with both systems RCC and CLT. CLT is very efficient with earthquakes and RCC shows better results in resisting the wing load.

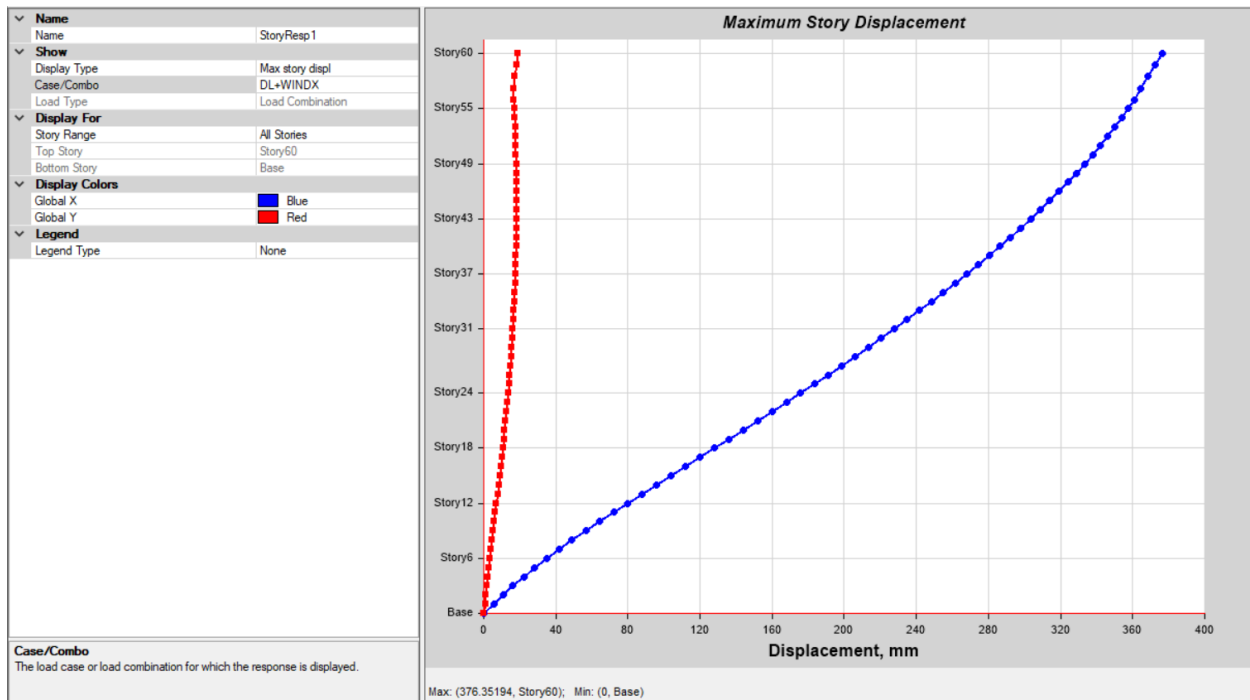
Maximum Lateral Displacement		
Load combination	RCC slab	CLT slab
1. Dead load + wind load	370 mm	400 mm
2. Dead load + seismic force	380 mm	340 mm

6.3.1 Displacement Graphs:

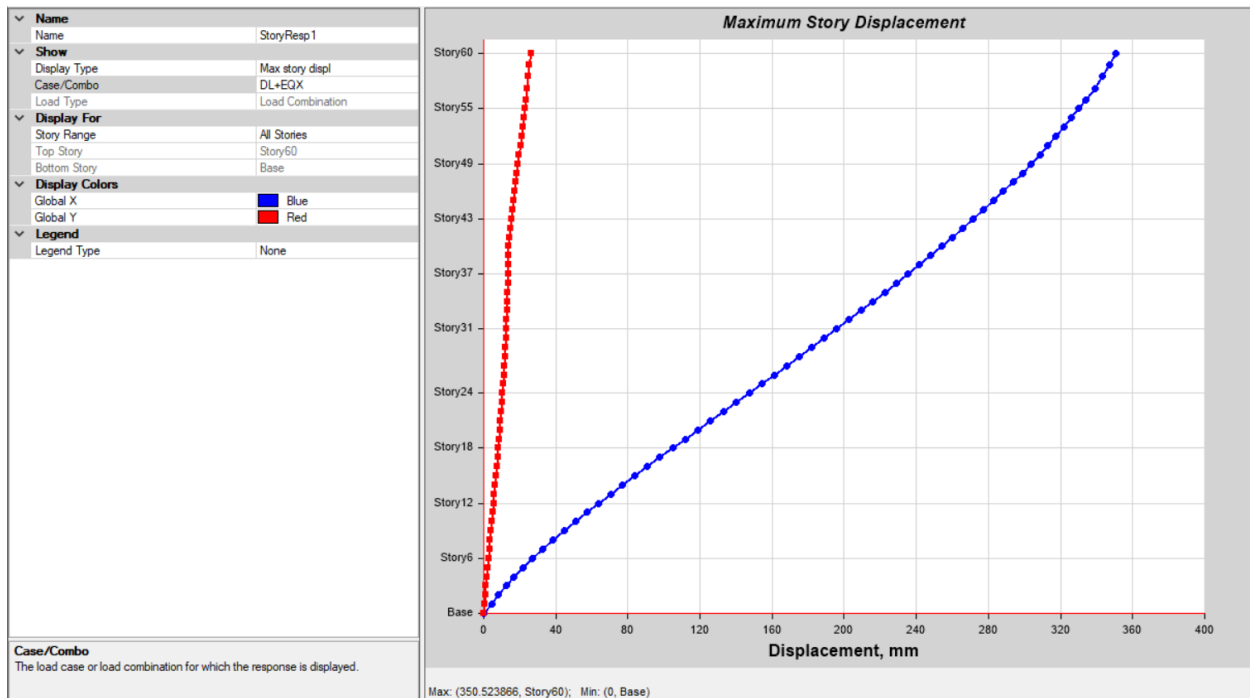
The following lateral displacement graphs are generated through ETABS software. There are many load combinations developed in our model. However, the following graph p the critical situation generated by two load combination. The load combinations are (dead load + wind load on X direction) & (dead load + seismic force on X direction).



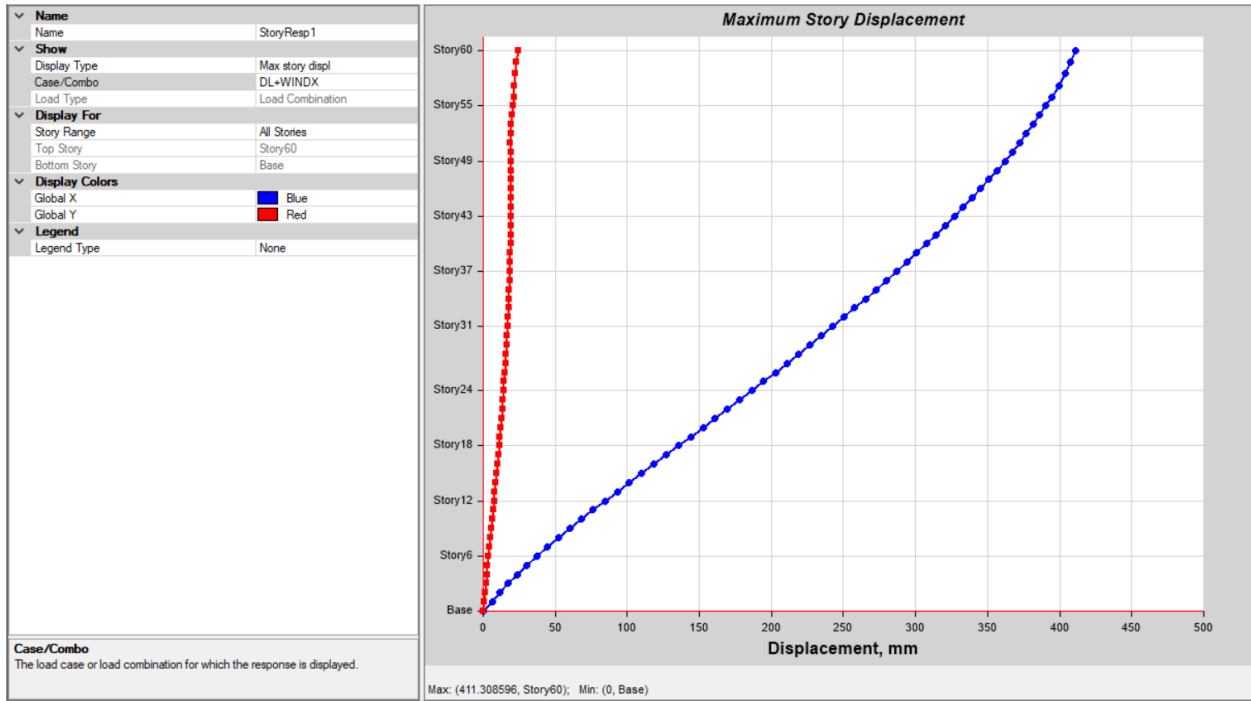
RCC Model Drift due to load combination 1 (seismic force + dead load)



RCC Model Drift due to load combination 2 (Wind load + dead load)



CLT Model Drift due to load combination 1 (seismic force + dead load)



CLT Model Drift due to load combination 2 (wind load + dead load)

Chapter 7: Cost Estimation

7.1 Introduction:

The cost of a project is one of the major calculations that should be made. Cost estimation is considered as one of the most important limitation for a project. In our project we will only compare the price of material between building with RCC slab and building with CLT slab; however, excluding the cost of construction.

Cost will be estimated by parametric estimation. The level of accuracy of this method will be between approximately between -10% to +30%. Cost of material was provided by Eastern trading and contracting company. Note that the prices provided exclude VAT.

- 69Mpa concrete type v price is 418 SR per m^3 .
- Cost of Steel is 2300 SR per ton.

Process:

The Calculation will be done on 2 sets. The first one will be the process of the RCC slab model and the second will be for the CLT slab model.

First process:

7.1.1 RCC Model cost estimation

Building (RCC Model)

Step 1: Volume of Columns

Step 2: Volume of Beams

Step 3: Volume of Shear wall

Step 4: Volume of Slab

Step 5: Volume of Foundation

Step 6: Volume of Piles

Step 7: Weight of reinforcement

Step 8: Total volume cost per cubic meter

Step 9: Total weight cost per unit ton

Step 10: Cost of finishing

Step 11: Find the Final Estimation

Step 1: Volume of Columns

Concrete Columns (RCC Model)

Floors	Number of floors	Area of column (m^2)	No. of columns in single floor	Height of columns (m)	Volume of column (m^3)
1-4	4	10.24	148	4	24248.32
1-4 (middle part)	4	1.44	57	4	1313.28
5-12	8	9.61	148	4	45512.96
13-20	8	7.84	148	4	3713.24
21-28	8	6.76	148	4	32015.36
29-36	8	5.76	148	4	27279.36
37-44	8	4.41	148	4	20885.76
45-52	8	3.24	148	4	15344.64
53-56	4	1.69	148	4	4001.92
57-60	4	0.49	148	5	1450.4
Total					140220.16

Step 2: Volume of Beams

Beam (RCC Model)

Number of floors	Beam Cross Section	Beam Height (m)	Beam Width (m)	Total Length (m)	Volume of Beams (m^3)
4	1.25m X 1m	1.25	1	2275	2843.75
56	1.25m X 1m	1.25	1	1525.9	1907.375
4	1.4m X 1.2m	1.4	1.2	398.2	668.976
56	1.4m X 1.2m	1.4	1.2	247.3	415.464
Total					5835.565

Step 3: Volume of Shear wall

Shear wall (RCC Model)					
Sides	Thickness	Length (m)	Height (m)	Number of stories	Volume of Shear wall (m³)
Side 1	0.6	11	4.06666666667	60	1610.4
Side 2	0.6	11	4.06666666667	60	1610.4
Side 3	0.6	14.9	4.06666666667	60	2181.36
Side 4	0.6	14.9	4.06666666667	60	2181.36
Total					7583.52
Total*2 (because there are two shear wall)					15167.04

Step 4: Volume of Slab

Slab (RCC Model)					
Number of floors	Slab Thickness (m)	Area of Floor (m²)	Shear wall area (m²)	Area of slab (m²)	Volume of Slab (m³)
8	0.18	7200	163.9	7036.1	10131.984
52	0.18	4800	163.9	4636.1	43393.896
Total					53525.88

Volume of slab = Slab thickness * (area of total floor – shear wall area)

Step 5: Volume of Foundation

Foundation (RCC Model)	
Area (m ²)	7718.24
Depth (m)	3.4
Volume (m ³)	26242.016

Step 6: Volume of Piles

Piles (RCC Model)	
Number of piles	200
Diameter (m)	0.6
Area (m ²)	0.282743
Depth (m)	3
Volume (m ³)	169.64

Step 7: Weight of reinforcement

Total volume of concrete = 241160.307 m³

Using ratio of 140 $\frac{Kg}{m^3}$ to determine weight of **Reinforcement** (As per the company instructions)

Reinforcement weight = Total volume of concrete * Ratio

$$\text{Reinforcement weight} = \frac{241160.307 * 140}{1000} = 33762.443 \text{ Tons}$$

Step 8: Total volume cost per cubic meter

Cost of concrete = Total volume of concrete * price per cubic meter

$$\text{Cost of concrete} = 241160.307 \text{ m}^3 * 418 \text{ SR}/\text{m}^3 = 100,805,008.3 \text{ SR}$$

Step 9: Total weight cost per unit ton

Cost of reinforcement = Weight of steel * price per ton

Cost of reinforcement = 33762.443 Tons * 2300 SR per ton = 77,653,618.9 SR

Step 10: Cost of finishing

Cost of finishing = 60% * Total cost

Cost of finishing = 60% * (100805008.3 + 77653618.9) = 107,075,176.3 SR

Step 11: Final Estimation

Total Cost of the building with RCC slab = cost of finishing + cost of reinforcement + cost of concrete

Cost = 107075176.3 + 77653618.9 + 100805008.3 = 285,533,803.5 SR

Second process:

7.1.2 CLT Model cost estimation

Building (CLT Model)
Step 1: Volume of Columns
Step 2: Volume of Beams
Step 3: Volume of Shear wall
Step 4: Volume of Slab
Step 5: Volume of Foundation
Step 6: Volume of Piles
Step 7: Weight of reinforcement
Step 8: Total volume cost per cubic meter
Step 9: Total weight cost per unit ton
Step 10: Cost of finishing
Step 11: Find the Final Estimation

Step 1: Volume of Columns

Concrete Columns (CLT Model)

Floors	Number of floors	Area of column (m ²)	No. of columns in single floor	Height of columns (m)	Volume of column (m ³)
1-4	4	8.41	148	4	19914.88
1-4 (middle part)	4	1	57	4	912
5-12	8	7.29	148	4	34525.44
13-20	8	6.25	148	4	29600
21-28	8	5.29	148	4	25053.44
29-36	8	4.41	148	4	20885.76
37-44	8	3.24	148	4	15344.64
45-52	8	2.25	148	4	10656
53-56	4	1.44	148	4	3409.92
57-60	4	0.49	148	5	1450.4
				Total	110005.76

Step 2: Volume of Beams

Beam (CLT Model)

Number of floors	Beam Cross Section	Beam Height (m)	Beam Width (m)	Total Length (m)	Volume of Beams (m ³)
4	1.25m X 1m	1.25	1	2275	2843.75
56	1.25m X 1m	1.25	1	1525.9	1907.375
4	1.35m X 1.1m	1.35	1.1	398.2	591.327
56	1.35m X 1.1m	1.35	1.1	247.3	367.2405
				Total	5709.6925

Step 3: Volume of Shear wall

Shear wall (CLT Model)

Sides	Thickness	Length (m)	Height (m)	Number of stories	Volume of Shear wall (m ³)
Side 1	0.5	11	4.06666666667	60	1342
Side 2	0.5	11	4.06666666667	60	1342
Side 3	0.5	14.9	4.06666666667	60	1817.8
Side 4	0.5	14.9	4.06666666667	60	1817.8
				Total	6319.6
				Total*2 (because there are two shear wall)	12639.2

Step 4: Volume of Slab

Slab (CLT Model)

Number of floors	Slab Thickness (m)	Area of Floor (m ²)	Shear wall area (m ²)	Area of slab (m ²)	Volume of Slab (m ³)
4	0.18	7200	163.9	7036.1	5065.992
52	0.36	4800	163.9	4636.1	86787.79
4	0.36	7200	163.9	7036.1	10131.984
				RCC Total	5065.992
				CLT Total	96919.776

Step 5: Volume of Foundation

Foundation (CLT Model)

Area (m ²)	7672.61
Depth (m)	3.2
Volume (m ³)	24552.352

Step 6: Volume of Piles

Piles (CLT Model)

Number of piles	200
Diameter (m)	0.6
Area (m ²)	0.282743
Depth (m)	3
Volume (m ³)	169.64

Step 7: Weight of reinforcement

Total volume of concrete = 158142.643 m³

Using ratio of 140 $\frac{Kg}{m^3}$ to determine weight of **Reinforcement** (As per the company instructions)

Reinforcement weight = Total volume of concrete * Ratio

$$\text{Reinforcement weight} = \frac{158142.643 * 140}{1000} = 22139.97 \text{ Tons}$$

Step 8: Total volume cost per cubic meter

Cost of concrete = Total volume of concrete * price per cubic meter

$$\text{Cost of concrete} = 158142.643 \text{ m}^3 * 418 \text{ SR}/\text{m}^3 = 66,103,624.77 \text{ SR}$$

Cost of Cross Laminated Timber = Total volume of CLT * price per cubic meter

$$\text{Cost of Cross Laminated Timber} = 96919.776 \text{ m}^3 * 1200 \text{ SR}/\text{m}^3 = 116,303,731.2 \text{ SR}$$

Step 9: Total weight cost per unit ton

Cost of reinforcement = Weight of steel * price per ton

$$\text{Cost of reinforcement} = 22139.97 \text{ Tons} * 2300 \text{ SR per ton} = 50,921,931 \text{ SR}$$

Step 10: Cost of finishing

Cost of finishing = 60% * Total cost

$$\text{Cost of finishing} = 60\% * (66103624 + 116303731.2 + 50921931) = 139,997,571.7 \text{ SR}$$

Step 11: Final Estimation

Total Cost of the building with CLT slab = cost of finishing + cost of reinforcement + cost of concrete
+ cost of cross laminated timber

$$\text{Cost} = 66103624 + 116303731.2 + 50921931 + 139997571.7 = 373,326,857.9 \text{ SR}$$

7.2 Summary:

These costs only include the material cost.

Building with RCC Slab	Building with CLT Slab
285,533,803.5 SR	373,326,857.9 SR

Chapter8: Used Software

8.1 AutoCAD:

AutoCAD stands for Automatic Computer Aided Design. During the 1980's, a group of engineers interested in simplifying how draftsmen, architects and engineers approach drawing projects, brain-stormed and came up with the idea of refining the difficult CAD processes that were popular in the 70's. To do this, internal graphics controllers were inbuilt into microcomputers which allowed designers simply draw diagrams at the front end while internal graphic controllers replicated these diagrams from the back end. And in the following decades, this innovative process would revolutionize the world of designs.

In simple words, AutoCAD is a commercial software application used to draft 2-dimensional and 3-dimensional models with the aid of a computer. [tutorial45.com]

8.1.1 AutoCAD Uses in our Project:

This incredible tool is very helpful. We used it many time during our project work. It is used in drawing the building's layout. In addition, it is used in drawing the slab cross section in both models RCC & CLT.

8.2 ETABS:

ETABS stands for Extended 3D Analysis of Building Systems. **ETABS** is an engineering software product that caters to multi-story building analysis and design. Modeling tools and templates, code-based load prescriptions, analysis methods and solution techniques, all coordinate with the grid-like geometry unique to this class of structure. Basic or advanced systems under static or dynamic conditions may be evaluated using ETABS. [Ondrej Kalny. May 15, 2013]

8.2.1 ETABS USES IN OUR PROJECT:

ETABS is used to redraw the building layout. We developed multiple load combination cases and run analysis. ETABS is our reference to check our analysis and design. We performed hand calculation to get rough estimation about the member dimension considering only gravity load. However, in ETABS analysis we consider many load combination cases. Then ETABS will tell which the critical load combination is. Based on ETABS analysis and the critical load combination we design our structural members' beams. Columns, and shear wall. Regarding the slab thickness, we calculate the slab thickness manually, assign it in ETABS, and run analysis to check its safety.

8.3 SAP2000:

SAP stands for Systems Applications and Products in Data Processing. SAP by definition is also ERP (Enterprise Resource Planning) software. Founded by Hopp, Hector, Plattner, Wellenreuther and Tschira in the year 1972, the SAP system features numerous fully integrated modules. As you continue reading, you will learn to define SAP and understand what SAP software is used for.

SAP 2000 is general-purpose civil-engineering software ideal for the analysis and design of any type of structural system. Basic and advanced systems, ranging from 2D to 3D, of simple geometry to complex, may be modeled, analyzed, designed, and optimized using a practical and

intuitive object-based modeling environment that simplifies and streamlines the engineering process. The SAPFire Analysis Engine integral to SAP2000 drives a sophisticated finite-element analysis procedure. Additional suites of advanced analysis features are available to users engaging state-of-the-art practice with nonlinear and dynamic consideration. Created by engineers for effective engineering, SAP2000 is the ideal software tool for users of any experience level, designing any structural system.

Integrated modeling templates, code-based loading assignments, advanced analysis options, design-optimization procedures, and customizable output reports all coordinate across a powerful platform to make SAP2000 especially useful for practicing professionals. [Rekha. 14 May 2011]

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8.3.1 SAP2000 Uses in our Project:

SAP 2000 is used to get the moment and shear diagrams for the MAT foundation. We model the foundation as simple supported beam. Moreover, we applied the load coming from column and soil reaction coming from the soil pressure and the bored pile supporting the soil bearing capacity. Based on the maximum shear we designed the depth of the foundation. Also, based on the maximum moment we designed the reinforcements.

Since we have one strip along X direction and one strip along Y direction, we modeled each strip as simple support beam to check the maximum positive moment and maximum negative moment. Positive and negative moment indicate the bending direction. Because of that, the tension and compression forces are existed at the top and bottom of the MAT foundation. Therefore, we designed two layers of steel reinforcements top and bottom along X & T direction to be sure, that the bending moment will not clack the MAT foundation concrete.

8.4 3D MAX:

Autodesk 3ds Max, formerly **3D Studio** and **3D Studio Max**, is a professional 3D computer graphics program for making 3D animations, models, games and images. It is developed and produced by Autodesk Media and Entertainment.

8.4.1 3D MAX Uses in Our Project:

It is used for visual aids purposes. Beam cross section with reinforcement and columns as well. We drew the building for our audience in the project proposal presentation to make our idea clear.

References:

- Scott Breneman, PhD, PE, SE Senior Technical Director – Project Resources and Solutions Division WoodWorks – Wood Products Council 2017.
- Introduction to CLT (cross laminated Timber) 19th October 2015 by Sofianna Teriaki
- Saudi building code (SBC)
- American Institute for Steel Construction (AISC)
- American Concrete Institute (ACI)
- SAAP2000.
- ETABS16.2.1.
- AutoCAD.

APPENDIX 1: Sections Extracted from Saudi Building Code

Appendix 1.1 Structural Design

Appendix 1.1.1 Strength and Serviceability Requirements for Concrete Structures

- Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in SBC 304.
- Members also shall meet all other requirements of SBC 304 to ensure adequate performance at service load levels.
- Required strength U shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-7). The effect of one or more loads not acting simultaneously shall be investigated.
- Design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of SBC 304, multiplied by the strength reduction factors ϕ in 9.3.2, 9.3.4, and 9.3.5.
- Designs shall not be based on yield strength of reinforcement $y f$ in excess of 550 MPa, except for prestressing steel.
- Reinforced concrete members subjected to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that adversely affect strength or serviceability of a structure.
- Minimum thickness stipulated in Table 9.5(a) shall apply for one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser thickness can be used without adverse effects.

TABLE 9.5(a)
MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR
ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED

	Minimum thickness, h			Cantilever
	Simply supported	One end continuous	Both ends continuous	
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	$l/20$	$l/24$	$l/28$	$l/10$
Beams or ribbed one-way slabs	$l/16$	$l/18.5$	$l/21$	$l/8$

Notes: Span length “l” is in mm.

- Deflection computed in accordance with 9.5.2.2 through 9.5.2.5 shall not exceed limits stipulated in Table 9.5(b).

TABLE 9.5(b)
MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$l/180^*$
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$l/360$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)**	$l/480\ddagger$
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$l/240\§$

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

** Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.3, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§ Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

- Section 9.5.3 shall govern the minimum thickness of slabs or other two-way construction designed in accordance with the provisions of Chapter 13 and conforming to the requirements of 13.6.1.2. The thickness of slabs without interior beams spanning between the supports on all sides shall satisfy the requirements of Section 9.5.3.2 or 9.5.3.4. The thickness of slabs with beams spanning between the supports on all sides shall satisfy requirements of Section 9.5.3.3 or 9.5.3.4.
- For slabs without interior beams spanning between the supports and having a ratio of long to short span not greater than 2, the minimum thickness shall be in accordance with the provisions of Table 9.5(c) and shall not be less than the following values:

(a) Slabs without drop panels as defined in Section 13.3.7.1 and 13.3.7.2 120 mm

(b) Slabs with drop panels as defined in Section 13.3.7.1 and 13.3.7.2 100 mm

TABLE 9.5(c)-MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS

Yield Strength f_y MPa*	Without drop panels†			With drop panels‡		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams‡		Without edge beams	With edge beams‡	
300	ln /33	ln /36	ln /36	ln /36	ln /40	ln /40
420	ln /30	ln /33	ln /33	ln /33	ln /36	ln /36
520	ln /28	ln /31	ln /31	ln /31	ln /34	ln /34

* For values of reinforcement yield strength between the values given in the table, minimum thickness shall be determined by linear interpolation.

† Drop panel is defined in 13.3.7.1 and 13.3.7.2

‡ Slabs with beams between columns along exterior edges. The value of α for the edge beam shall not be less than 0.8.

Appendix 1.1.2 Design of members subject to flexure or axial loads

- Strength design of members for flexure and axial loads shall be based on assumptions given in Sections 10.2.2 through 10.2.7, and on satisfaction of applicable conditions of equilibrium and compatibility of strains.
- Design of cross sections subject to flexure or axial loads, or to combined flexure and axial loads, shall be based on stress and strain compatibility using assumptions in 10.2.
- Design axial load strength ϕP_n of compression members shall not be taken greater than equation 10-1 and 10-2
- At every section of a flexural member where tensile reinforcement is required by analysis, except as provided in Section 10.5.2, and 10.5.3, the area A_s provided shall not be less than eq 10-3
- For structural slabs and footings of uniform thickness the minimum area of tensile reinforcement in the direction of the span shall be the same as that required by 7.12. Maximum spacing of this reinforcement shall not exceed three times the thickness, nor 300 mm.
- Distribution of flexural reinforcement in two-way slabs shall be as required by 13.3.
- Flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section as required by 10.6.4.
- Area of longitudinal reinforcement for non-composite compression members shall be not less than 0.01 nor more than 0.08 times gross area A_g of section.
- Minimum number of longitudinal bars in compression members shall be 4 for bars within rectangular or circular ties, 3 for bars within triangular ties, and 6 for bars enclosed by spirals conforming to Section 10.9.3.
- Axially loaded members supporting a slab system included within the scope of Section 13.1 shall be designed as provided in Chapter 10 and in accordance with the additional requirements of Chapter 13.

- When the specified compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, transmission of load through the floor system shall be provided by Section 10.15.1, 10.15.2, or 10.15.3.

Appendix 1.1.3 Design for Shear Strength

- Except for members designed in accordance with Appendix A, design of cross sections subject to shear shall be based on equation 11-1
- Shear strength shall be computed by provisions of Section 11.3.1.1 through 11.3.1.3, unless a more detailed calculation is made in accordance with Section 11.3.2.
- Types of shear reinforcement
Shear reinforcement consisting of the following shall be permitted:
 - (a) Stirrups perpendicular to axis of member;
 - (b) Welded wire fabric with wires located perpendicular to axis of member;
 - (c) Spirals, circular ties, or hoops.
- Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d / 2$ in nonprestressed members or $0.75h$ in prestressed members, nor 500 mm.
- Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45 deg line, extending toward the reaction from mid-depth of member $d / 2$ to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.
- A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where factored shear force V_u exceeds one-half the shear strength provided by concrete ϕV_c except:
 - (a) Slabs and footings;
 - (b) Concrete joist construction defined by 8.11;
 - (c) Beams with total depth not greater than 250 mm, 2.5 times thickness of flange, or 0.5 the width of web, whichever is greatest.
- Where shear reinforcement is required by Section 11.5.5.1 or for strength and where 11.6.1 allows torsion to be neglected, the minimum area of shear

reinforcement for prestressed (except as provided in 11.5.5.4) and nonprestressed members shall be computed by equation 11-13

- Design of shear reinforcement:
Where factored shear force V_u , exceeds shear strength ϕV_c , shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed in accordance with 11.5.6.2 through 11.5.6.9.

Appendix 1.11.4 Design of Two-Way Slab System:

- Provisions of Chapter 13 shall apply for design of slab systems reinforced for flexure in more than one direction, with or without beams between supports.
- A slab system shall be designed by any procedure satisfying conditions of equilibrium and geometric compatibility, if shown that the design strength at every section is at least equal to the required strength set forth in Section 9.2 and 9.3, and that all serviceability conditions, including limits on deflections, are met.
- Minimum thickness of slabs designed in accordance with Chapter 13 shall be as required by Section 9.5.3.
- Area of reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections, but shall not be less than required by 7.12.
- Spacing of reinforcement at critical sections shall not exceed two times the slab thickness, except for portions of slab area of cellular or ribbed construction. In the slab over cellular spaces, reinforcement shall be provided as required by 7.12.
- In addition to the other requirements of 13.3, reinforcement in slabs without beams shall have minimum extensions as prescribed in Fig. 13.3.8.

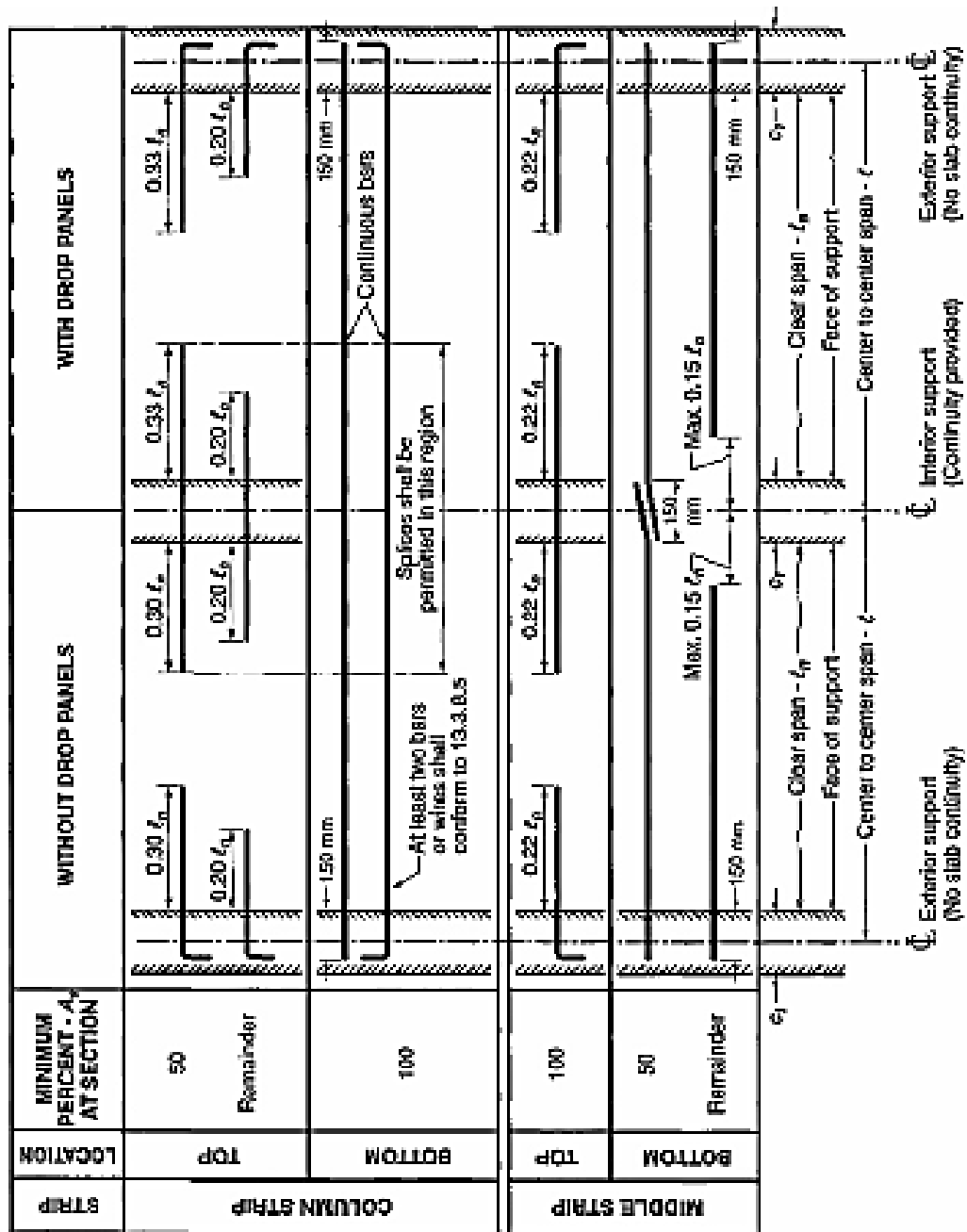


Fig. 13.3.8 - Minimum extensions for reinforcement in slabs without beams. (See 12.11.1 for reinforcement extension into supports)

Appendix 1.5 Design of Walls

- Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.
- Walls subject to axial loads shall be designed in accordance with Section 14.2, 14.3, and either 14.4, 14.5, or 14.8.
- Design for shear shall be in accordance with 11.10.
- Minimum vertical and horizontal reinforcement shall be in accordance with 14.3.2 and 14.3.3 unless a greater amount is required for shear by Section 11.10.8 and 11.10.9.
- Vertical and horizontal reinforcement shall not be spaced farther apart than two times the wall thickness, nor farther apart than 300 mm.
- Except as provided in 14.5, walls subject to axial load or combined flexure and axial load shall be designed as compression members in accordance with provisions of Section 10.2, 10.3, 10.10, 10.11, 10.12, 10.13, 10.14, 10.17, 14.2, and 14.3.

Appendix 1.2 Geotechnical Design

Appendix 1.2.1 Spread Footings

- Spread footings shall be designed and constructed in accordance with Sections 5.1 through 5.6. Spread footings shall be designed and constructed in accordance with Sections 5.4.1 through 5.4.4.
- The minimum depth of footing below the natural ground level shall not be less than 1.2 m for cohesionless soils, 1.5 m for silty and clay soils and 600 mm to 1200 mm for rocks depending on strength and integrity of the rock formations.
- Footings shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that total and differential settlements are tolerable. The minimum width of footings shall be 300 mm.

- Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings.
- Settlements shall be estimated based on methods of analysis approved by the building official. The least value found from Tables 5.1 and 5.2 shall be taken as the allowable differential settlement.

Table 5.1: Maximum Allowable Total Settlement (Saudi Building Code)

Footing Type	Total Settlement (mm)	
	Clay	Sand
Spread Footings	60	40
Mat Foundations	80	60

Table 5.2: Maximum Allowable Angular Torsion (Saudi Building Code)

Building Type	L/H	ρ/l
Multistory reinforced concrete structures founded on mat foundation	---	0.0015
Steel frame structure with side sway	---	0.008
Reinforced concrete or steel structure with interior or exterior glass or panel cladding	---	0.002-0.003
Reinforced concrete or steel structure with interior or exterior glass or panel cladding	≥ 5 ≤ 3	0.002 0.001
Slip and high structures as silos and water tanks founded on stiff mat foundations	---	0.002
Cylindrical steel tank with fixed cover and founded on flexible footing	---	0.008
Cylindrical steel tank with portable cover and founded on flexible footing	---	0.002-0.003
Rail for supporting hanged lift	---	0.003

L = Building length

l = Span between adjacent footings

H = Overall height of the structure

δ = Differential settlement

- Factor of safety shall not be less than 3 for permanent structures and 2 for temporary structures. Consideration shall be given to all possible circumstances including, but not limited to, flooding of foundation soil, removal of existing overburden by scour or excavation, and change in groundwater table level.

- The design, materials and construction of concrete footings shall comply with Sections 5.4.2.1 through 5.4.2.8 and the provisions of SBC 304 where applicable. Concrete in footings shall have a specified compressive strength (f_c) of not less than 20 MPa at 28 days.
- Concrete and masonry foundation walls shall be designed in accordance with SBC 304 or SBC 305. Foundation walls that are laterally supported at the top and bottom and within the parameters of Tables 6.1 through 6.3 are permitted to be designed and constructed in accordance with Sections 6.2 through 6.6.
- Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 13.4.2 and 13.4.3. Computations of lateral earth pressures shall comply with the provisions of Sections 7.2.1 through 7.2.6. The values set forth in Tables 7-2 and 7-3 shall be used in computations that include effects of wall friction.

Table 7.2: Ultimate Friction factors For Dissimilar Materials (Saudi Building Code)

Interface Materials	Friction Factor, $\tan\delta^*$
Clean sound rock	0.7
Clean gravel, gravel-sand mixtures, coarse sand	0.55 - 0.60
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 – 0.55
Clean fine sand, silty or clayey fine to medium sand	0.35 – 0.45
Fine sandy silt, nonplastic silt	0.30 – 0.35
Very stiff and hard residual or preconsolidated clay	0.40 - 0.50
Medium stiff and stiff clay and silty clay	0.30 – 0.35

*Values for δ shall not exceed one-half the angle of internal friction of the backfill soils for steel and precast concrete

And two-third the angle of internal friction of the backfill soils for cast-in place concrete

- The allowable soil pressure shall be determined in accordance with the provisions of Chapter 4. The factor of safety with respect to bearing capacity shall not be less than 3.

Appendix 1.2.2 Retaining Wall

- Retaining walls shall be designed to ensure stability against overturning, sliding, and stability of supporting ground. Stability analyses shall conform to the provisions of Sections 7.4.1 through 7.4.4.
- The retaining wall shall be proportioned so that the factor of safety against overturning is not less than 1.5. The value of angular distortion (settlement/length of structure) of retaining walls shall not exceed 0.002 radians.
- Where retaining walls are underlain by weak soils, the overall stability of the soil mass containing the retaining wall shall be checked with respect to the most critical surface of sliding. The stability analysis shall be made for after construction and for long-term conditions. The factor of safety for the overall stability of the soil mass containing the wall shall not be less than 2.
- Thickness of the upper part of the wall shall not be less than 300 mm, whereas thickness of the lower part of the wall shall be enough to resist shear without reinforcement. Depth of wall foundation shall be located below line of seasonal changes and shall be deep enough to provide adequate bearing capacity and soil sliding resistance. The wall foundation shall be proportioned such that the wall does not slide or overturn, the allowable bearing capacity of the soil is not exceeded, and that total and differential settlements are tolerable. The base and other dimensions shall be such that the resultant falls within the middle third of the base. Where additional front clearance is needed, it shall be permitted to construct counterfort retaining walls without a toe provided that the sliding and overturning stability requirements stated in Sections 7.4.1 and 7.4.2 are met.

Appendix 1.2.3 Combined Footing and Mat Foundation

- Analysis and design of combined footings and mats shall conform to all requirements of ACI 336.2R Suggested Analysis and Design Procedures for Combined Footings and Mats except as modified by Chapter 8. All provisions of SBC 303 not specifically excluded, and not in conflict with the provisions of Chapter 8 shall apply to combined footings and mats, where applicable. Design of combined or mat foundations shall be based on the Strength Design Method of SBC 304.
- Combined footings and mats shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in Section 4.8 SBC 301, are permitted to be used in

designing footings. Strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

- Soil contact pressure acting on a combined footing or mat and the internal stresses produced by them shall be determined from one of the load combinations given in Section 2.4 SBC 301, whichever produces the maximum value for the element under investigation.
- The maximum unfactored design contact pressures shall not exceed the allowable soil pressure as obtained from Chapter 4 or cause settlements that exceed the values set forth in Table 5.1 and 8.3. Where wind or earthquake forces form a part of the load combination, the allowable soil pressure may be increased as allowed by the Saudi Building Code or approved by the building official.
- Contact pressures at the base of combined footings and mats shall be determined in accordance with Sections 8.4.1.1 through 8.4.1.3.
- Settlements of combined footings and mats shall conform to the provisions of Sections 8.5.1 through 8.5.3. Total settlement of combined footings and mats shall not exceed the value set forth in Table 5.1. Differential settlements for combined footings shall not exceed the values set forth in Table 5.2. For mats the differential settlement shall be taken as three-fourths the total settlement if this is not more than 50 mm or determined based on relative stiffness, r_k , as shown in Table 8.3.
- Combined footings shall be designed and constructed in accordance with Sections 8.6.1 through 8.6.3.

Table 8.3: Maximum Allowable Differentials Settlement of Mats (Saudi Building Code)

k_r	Shape	Differential Settlement (mm)
0	Rectangular Base	$0.5 \times \Delta H^*$
	Square Base	$0.35 \times \Delta H$
0.5		$0.1 \times \Delta H$
>0.5	Rigid mat: no differential settlement	

* ΔH = Total settlement estimated based on approved methods of analysis but shall not exceed values in Table 5.1.

- Continuous foundations shall be designed and constructed in accordance with Sections 8.7.1 through 8.7.3.

- Mats shall be designed and constructed in accordance with Sections 8.9.1 through 8.9.3. Mats may be designed and analyzed as either rigid bodies or as flexible plates supported by an elastic foundation (the soil).
- A mat may be designed using the Strength Design Method of SBC 304. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. The mat plan shall be proportioned using unfactored loads and any overturning moments.
- The pressure diagram is considered linear and computed from Equation 8-2 and SBC 303 2007 8/11 shall be less than allowable load bearing. Loads shall include the effect of any column moments and any overturning moment due to wind or other effects. Any moments applied to the mat from columns or overturning, etc., shall be included when computing the eccentricity.
- The contact pressure shall not exceed the allowable load bearing determined from Chapter 4. The allowable soil pressure may be furnished as one or more values depending on long-term loading or including transient loads such as wind. The soil pressure furnished by the geotechnical engineer shall be factored to a pseudo “ultimate” value by multiplying the allowable pressure by the ratio of the sum of factored design loads to the sum of the unfactored design loads.
- The minimum mat thickness based on punching shear at critical columns shall be computed based on column load and shear perimeter. The depth of the mat shall be found without using shear reinforcement and determined on the basis of diagonal-tension shear as noted in SBC 304 Chapter 15.

Appendix 1.2.4 Underground Water-Retention Structures

- Walls or portions thereof that retain earth and enclose interior spaces and floors below grade, and underground water-retention structures shall be waterproofed and damp proofed in accordance with provisions of this Chapter, with the exception of those spaces containing groups other than residential and institutional where such omission is not detrimental to the building or occupancy. Ventilation for crawl spaces shall comply with Section 7.3.4 SBC 201.
- Underground water-retention structures shall meet the provisions of Sections 13.5.1 through 13.5.4.

- For design and constructions of underground water-retention structures, provisions of SBC 304 and ACI 350 Environmental Engineering Concrete Structures shall govern, where applicable.

Appendix 1.2.5 Deep Foundations (Piles)

- Piles are permitted to be designed in accordance with provisions for piers in Chapter 14 and Sections 17.3 through 17.10 where either of the following conditions exists, subject to the approval of the building official:
 - Group R-3 and U occupancies not exceeding two stories of light-frame construction, or
 - Where the surrounding foundation materials furnish adequate lateral support for the pile.
- The allowable axial and lateral loads on piers or piles shall be determined by an approved formula, load tests or method of analysis.
- The allowable compressive load on any pile where determined by the application of an approved driving formula shall not exceed 360 kN. For allowable loads above 360 kN, the wave equation method of analysis shall be used to estimate pile drivability of both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 14.8.3.
- Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 14.8.3 divided by a factor of safety of two.
- Piers, individual piles and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers.
- Any soil other than fluid soil shall be deemed to afford sufficient lateral support to the pier or pile to prevent buckling and to permit the design of the pier or pile in accordance with accepted engineering practice and the applicable provisions of this code.

- Where required by the design, the lateral load capacity of a pier, a single pile or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load.
- The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 25 mm at the ground surface.
- Allowable stresses greater than those specified for piers or for each pile type in Chapters 14 and 15 are permitted where supporting data justifying such higher stresses is filed with the building official.
- The settlement of piers, individual piles or groups of piles shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.
- The materials, reinforcement and installation of precast concrete piles shall conform to Sections 15.1.1.1 through 15.1.1.4.
- Piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.
- The minimum lateral dimension shall be 200 mm. Corners of square piles shall be chamfered.
- Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced not more than 100 mm apart, center to center, for a distance of 600 mm from the ends of the pile; and not more than 150 mm elsewhere except that at the ends of each pile, the first five ties or spirals shall be spaced 25 mm center to center.
- Precast nonprestressed concrete piles shall conform to Sections 15.1.2.1 through 15.1.2.5. Concrete shall have a 28-day specified compressive strength (f_c) of not less than 20 MPa.
- The minimum amount of longitudinal reinforcement shall be 0.8 percent of the concrete section and shall consist of at least four bars.
- The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f_c) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel (f_y) or a maximum of 210 MPa. The allowable tensile stress in the reinforcing steel shall

not exceed 50 percent of the yield strength of the steel (f_y) or a maximum of 165 MPa.

- Precast prestressed concrete piles shall conform to the requirements of Sections 15.1.3.1 through 15.1.3.5.
- Prestressing steel shall conform to ASTM A416. Concrete shall have a 28-day specified compressive strength (f_c) of not less than 35 MPa.
- Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 3 MPa for piles up to 9 meters in length, 4 MPa for piles up to 15 meters in length and 5 MPa for piles greater than 15 m in length. Effective prestress shall be based on an assumed loss of 210 MPa in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in SBC 304.
- Structural steel piles shall conform to the requirements of Sections 15.2.2 through 15.2.5.
- Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A36, ASTM A252, ASTM A283, ASTM A572, ASTM A588 or ASTM A913.
- The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength (f_y).
- Sections of H-piles shall comply with the following:
 - The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.
 - The nominal depth in the direction of the web shall not be less than 200 mm.
 - Flanges and web shall have a minimum nominal thickness of 10 mm.
- Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 200 mm. The pipe shall have a minimum of 220 mm² of steel in cross section to resist each 1360 N-m of pile hammer energy or the equivalent strength for steels having a yield strength greater than 240 MPa. Where pipe wall thickness less than 5 mm is driven open ended, a suitable cutting shoe shall be provided.
- The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 16.1.1 through 16.1.3.

- Concrete shall have a 28-day specified compressive strength (f_c) of not less than 20 MPa. Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 100 mm and not more than 150 mm. Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.
- Except for steel dowels embedded 1.5 m or less in the pile and as provided in Section 16.3.4, reinforcement where required shall be assembled and tied together and shall be placed in the pile as a unit before the reinforced portion of the pile is filled with concrete except in augered uncased cast-in-place piles. Tied reinforcement in augered uncased cast-in-place piles shall be placed after piles are concreted, while the concrete is still in a semifluid state.
- Enlarged base piles shall conform to the requirements of Sections 16.2.1 through 16.2.5.
- The maximum size for coarse aggregate for concrete shall be 20 mm. Concrete to be compacted shall have a zero slump.
- The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25% of the 28-day specified compressive strength (f_c). Where the concrete is placed in a permanent steel casing, the maximum allowable concrete stress shall be 33% of the 28-day specified compressive strength (f_c).
- Drilled or augered uncased piles shall conform to Sections 16.3.1 through 16.3.5.
- The allowable design stress in the concrete of drilled uncased piles shall not exceed 33 percent of the 28-day specified compressive strength (f_c). The allowable design stress in the concrete of augered cast-in-place piles shall not exceed 25 percent of the 28-day specified compressive strength (f_c). The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or (175 MPa).
- The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 300 mm.
- Driven uncased piles shall conform to Sections 16.4.1 through 16.4.4.
- The allowable design stress in the concrete shall not exceed 25 percent of the 28-day specified compressive strength (f_c) applied to a cross sectional area not greater than the inside area of the drive casing or mandrel.

- The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 300 mm.
- Steel-cased piles shall comply with the requirements of Sections 16.5.1 through 16.5.4.
- Pile shells or casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete. Steel shells shall have a sealed tip with a diameter of not less than 200 mm.
- The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f_c). The allowable concrete compressive stress shall be $0.40 (f_c)$ for that portion of the pile meeting the conditions specified in Sections 16.5.2.1 through 16.5.2.4.
- The thickness of the steel shell shall not be less than manufacturer's standard gage No. 14 gage 1.75 mm minimum.
- Concrete-filled steel pipe and tube piles shall conform to the requirements of Sections 16.6.1 through 16.6.5.
- Steel pipe and tube sections used for piles shall conform to ASTM A 252 or ASTM A 283. Concrete shall conform to Section 16.1.1. The maximum coarse aggregate size shall be 20 mm.
- The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f_c). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (f_y), provided f_y shall not be assumed greater than 250 MPa for computational purposes.
- Caisson piles shall conform to the requirements of Sections 16.7.1 through 16.7.6.
- Composite piles shall conform to the requirements of Sections 16.8.2 through 16.8.5.
- Isolated and multiple piers used as foundations shall conform to the requirements of Sections 17.2 through 17.10, as well as the applicable provisions.

Appendix 1.2.6 Slope Stability

- The minimum acceptable static factor of safety for cut, fill, and natural slopes is 1.3 in the absence of earthquake.
- The minimum dynamic factor of safety for cut, fill, and natural slopes is 1.1 for the case of an earthquake.
- Safety factor is defined as the quotient of the sum of forces tending to resist failure divided by the sum of forces tending to cause failure. 1.
- New buildings and additions to buildings may be constructed on or adjacent to a cut, fill, or natural slope provided that:
 - The slopes have an evaluated safety factor of at least 1.5 against deep-seated static failure.
 - The slopes ascending above proposed structures have an evaluated safety factor of at least 1.5 against surficial failure or adequately designed protective devices are recommended that will protect the construction from the hazard of mud and debris flow.
- Minor additions or alterations may be made to existing structures where acceptable devices are provided to mitigate potential damage from failure of adjacent slopes and where the hazard to life or property is not increased.

APPENDIX 2: Project Management

Appendix 2.1 Project Plan

Table B.1 Project Plan

Task	Assigned team members	Duration (weeks)
Setting Objectives	All	1
Collecting Data	All	4
Literature review	All	2
Feasibility study	All	1
Selecting Structural system	All	2
Preliminary design	All	6
ETABS modeling	All	8
Checking for Adjustments if any	All	2
Check soil capacity	All	1
Selecting foundation system	All	1
Design Foundation	All	2
Preparing Technical Report	All	16

Appendix 2.2 Contribution of Team Members

- **Abdulaziz Al-Mulhim**

- ✓ Setting Objectives
- ✓ Collecting Data
- ✓ Literature review
- ✓ Feasibility study
- ✓ Selecting Structural system
- ✓ Preliminary design
- ✓ ETABS modeling
- ✓ Checking for Adjustments if any
- ✓ Check soil capacity
- ✓ Selecting foundation system
- ✓ Design Foundation
- ✓ Preparing Technical Report

- **Ali Alabyadh**

- ✓ Setting Objectives
- ✓ Collecting Data
- ✓ Literature review
- ✓ Feasibility study
- ✓ Selecting Structural system
- ✓ Preliminary design
- ✓ ETABS modeling
- ✓ Checking for Adjustments if any
- ✓ Check soil capacity
- ✓ Selecting foundation system
- ✓ Design Foundation
- ✓ Preparing Technical Report

- **Saad Rabah**

- ✓ Setting Objectives
- ✓ Collecting Data
- ✓ Literature review
- ✓ Feasibility study
- ✓ Selecting Structural system
- ✓ Preliminary design
- ✓ ETABS modeling
- ✓ Checking for Adjustments if any
- ✓ Check soil capacity
- ✓ Selecting foundation system
- ✓ Design Foundation
- ✓ Preparing Technical Report

Appendix 2.3 Project Execution and Monitoring

- Meeting daily, to talk and continue the tasks that we have to do together
- 2 meetings a week with the advisors to assess our performance.

Appendix 2.3 Challenges and Decision Making

- Selecting an adequate structural system that satisfies all architectural and safety requirements in addition to keeping feasibility and cost in the reasonable range. In order to overcome this challenge, we performed several investigations regarding different structural members. We selected one-way solid slab because this can be an adequate design for the project due the large slab spans
- Selecting foundation system is also one of the challenges we faced. The required area of isolated footing was more than fifty percent. We decided to use piled-Mat foundation.

Appendix 2.4 Project Bill of Materials and Budget (if applicable)

This part cannot be performed because it requires the actual cost of materials, equipment; labors cost and number ...etc.

APPENDIX 3: Project Analysis

Appendix 3.1 Life-long Learning

- Time Management is an essential key point in our project. We learned how to divide our project into smaller activities and assign each activity time duration.
- Literature review plays an important role in such projects because you can learn about previous mistakes that have been done and how to prevent yourself from doing the same mistakes. Also, literature review can introduce you to a variety of different resources and connections.
- On such large-scale projects, important decisions are made on daily bases. We learned that we should include everything related into account, because these sequences of decisions will build the entire project.
- Organized work will flow smoothly. However, if you don't follow a well-organized plan you will mess up sooner or later.
- We had the opportunity to work on computer aided design & engineering Software such as ETABS, and AutoCAD.
- The completion of any inter-disciplinary project depends upon cooperation, coordination and combined efforts of several sources of knowledge.

Appendix 3.2 Impact of Engineering Solutions

There are many different approaches to figure out the structural & Foundation systems in construction. However, every different approach if not well analyzed will be problem-centered, and huge amount of money will be spent to maintain the problem-centered structure. In order to overcome such complications engineers came with solutions. We faced some of the engineering solutions in slab design, where we needed to add extra columns in each slab to become 20 columns for each floor. The reason of that is because of the frailer that ETABS shows. Because of the huge spans, without any supports.

APPENDIX 4

Appendix 4.1 Column Calculation (RCC Model)

FLOOR	LL used for the floor (KN/m2)	LOAD (Kips) per FLOOR	LOAD (Kips) Total	LOAD (KN) per FLOOR	Cumulative load (KN) Total	AREA OF COLUMNS (in ²)	Area of column (m ²)	SQRT of Area (m)	Column dimensions B*D (m ²)
56	5	475.8	475.8	2116.47	2116.47	216.27	0.1395	0.3735	
55	5	475.8	951.6	2116.47	4232.94	432.55	0.2791	0.5283	
54	5	475.8	1427.4	2116.47	6349.41	648.82	0.4186	0.647	
53	5	475.8	1903.2	2116.47	8465.89	865.09	0.5581	0.7471	0.7 X 0.7
52	7.5	559.75	2463	2489.91	10955.79	1448.8	0.9347	0.9668	
51	7.5	559.75	3022.7	2489.91	13445.70	1778.1	1.1471	1.071	
50	7.5	559.75	3582.5	2489.91	15935.61	2107.3	1.3596	1.166	
49	7.5	559.75	4142.2	2489.91	18425.52	2436.6	1.572	1.2538	1.3 X 1.3
48	10	643.71	4785.9	2863.35	21288.87	2518.9	1.6251	1.2748	
47	10	643.71	5429.6	2863.35	24152.22	2857.7	1.8437	1.3578	
46	10	643.71	6073.3	2863.35	27015.57	3196.5	2.0623	1.4361	
45	10	643.71	6717.1	2863.35	29878.92	3535.3	2.2808	1.5102	
44	10	643.71	7360.8	2863.35	32742.27	3874.1	2.4994	1.581	
43	10	643.71	8004.5	2863.35	35605.62	4212.9	2.718	1.6486	
42	10	684.88	8689.4	3046.52	38652.14	4573.3	2.9505	1.7177	
41	10	684.88	9374.2	3046.52	41698.66	4933.8	3.1831	1.7841	1.8 X 1.8
40	10	643.71	10018	2863.35	44562.01	4770.4	3.0777	1.7543	
39	10	643.71	10662	2863.35	47425.36	5077	3.2755	1.8098	
38	10	643.71	11305	2863.35	50288.71	5383.5	3.4732	1.8637	
37	10	643.71	11949	2863.35	53152.06	5690	3.671	1.916	
36	10	643.71	12593	2863.35	56015.41	5996.6	3.8687	1.9669	
35	10	643.71	13236	2863.35	58878.76	6303.1	4.0665	2.0166	
34	10	643.71	13880	2863.35	61742.11	6609.6	4.2643	2.065	
33	10	643.71	14524	2863.35	64605.46	6916.1	4.462	2.1123	2.1 X 2.1
32	5	475.8	15000	2116.47	66721.94	7142.7	4.6082	2.1467	
31	5	475.8	15475	2116.47	68838.41	7369.3	4.7544	2.1805	
30	5	475.8	15951	2116.47	70954.88	7595.9	4.9005	2.2137	
29	5	475.8	16427	2116.47	73071.35	7822.4	5.0467	2.2465	
28	5	475.8	16903	2116.47	75187.82	8049	5.1929	2.2788	
27	5	475.8	17379	2116.47	77304.30	8275.6	5.3391	2.3106	
26	5	475.8	17855	2116.47	79420.77	8502.1	5.4852	2.3421	
25	5	475.8	18330	2116.47	81537.24	8728.7	5.6314	2.3731	2.4 X 2.4
24	5	475.8	18806	2116.47	83653.71	8955.3	5.7776	2.4037	
23	5	475.8	19282	2116.47	85770.18	9181.9	5.9238	2.4339	
22	5	475.8	19758	2116.47	87886.66	9408.4	6.0699	2.4637	
21	5	475.8	20234	2116.47	90003.13	9635	6.2161	2.4932	

20	5	475.802	20709.3	2116.47	92119.60	9861.58	6.362297	2.522	
19	5	475.802	21185.1	2116.47	94236.07	10088.2	6.508472	2.551	
18	5	475.802	21660.9	2116.47	96352.54	10314.7	6.654648	2.58	
17	5	475.802	22136.7	2116.47	98469.02	10541.3	6.800823	2.608	2.6m X 2.6m
16	10	643.707	22780.4	2863.35	101332.37	10307.9	6.650237	2.579	
15	10	643.707	23424.1	2863.35	104195.72	10599.2	6.838152	2.615	
14	10	643.707	24067.8	2863.35	107059.07	10890.4	7.026068	2.651	
13	10	643.707	24711.6	2863.35	109922.42	11181.7	7.213984	2.686	
12	10	643.707	25355.3	2863.35	112785.77	11473	7.4019	2.721	
11	10	643.707	25999	2863.35	115649.12	11764.2	7.589816	2.755	
10	10	643.707	26642.7	2863.35	118512.47	12055.5	7.777732	2.789	
9	10	643.707	27286.4	2863.35	121375.82	12346.8	7.965647	2.822	2.8m X 2.8m
8	10	643.707	27930.1	2863.35	124239.17	12638	8.153563	2.855	
7	10	643.707	28573.8	2863.35	127102.52	12929.3	8.341479	2.888	
6	10	643.707	29217.5	2863.35	129965.87	13220.6	8.529395	2.921	
5	10	643.707	29861.2	2863.35	132829.22	13511.9	8.717311	2.953	
4	10	643.707	30504.9	2863.35	135692.57	13803.1	8.905227	2.984	
3	10	643.707	31148.6	2863.35	138555.92	14094.4	9.093142	3.015	
2	10	643.707	31792.3	2863.35	141419.27	14385.7	9.281058	3.046	
1	10	643.707	32436	2863.35	144282.62	14676.9	9.468974	3.077	3.1m X 3.1m
B1	10	643.707	33079.7	2863.35	147145.97	14968.2	9.65689	3.108	
B2	10	643.707	33723.4	2863.35	150009.32	15259.5	9.844806	3.138	
B3	10	643.707	34367.2	2863.35	152872.67	15550.7	10.03272	3.167	
B4	10	643.707	35010.9	2863.35	155736.02	15842	10.22064	3.197	3.2m X 3.2m

APPENDIX 5

Appendix 5.1 Shear Wall Calculation (RCC Model)

floor	LL used for the floor (KN/m ²)	Load per Floor (KN)	Cumulative load (KN)	Load per Floor (Kips)	Cumulative load (Kips)	AREA OF SHEAR WALL (in ²)	Area of Shear Wall (m ²)	Thickness of the Shear Wall (m)
56	5	6171.17	6171.17	1387.335	1387.33456	630.6066166	0.4068422	0.007961686
55	5	6171.17	12342.34	1387.335	2774.66911	1261.213233	0.8136843	0.015923372
54	5	6171.17	18513.51	1387.335	4162.00367	1891.81985	1.2205265	0.023885059
53	5	6171.17	24684.68	1387.335	5549.33823	2522.426466	1.6273687	0.031846745
52	7.5	7185.19	31869.87	1615.295	7164.6336	3256.651639	2.1010614	0.041116661
51	7.5	7185.19	39055.06	1615.295	8779.92898	3990.876811	2.5747541	0.050386577
50	7.5	7185.19	46240.25	1615.295	10395.2244	4725.101983	3.0484468	0.059656493
49	7.5	7185.19	53425.44	1615.295	12010.5197	5459.327155	3.5221395	0.068926409
48	10	8199.19	61624.63	1843.252	13853.7714	6297.168839	4.0626814	0.079504529
47	10	8199.19	69823.82	1843.252	15697.0232	7135.010523	4.6032234	0.090082649
46	10	8199.19	78023.01	1843.252	17540.2749	7972.852207	5.1437653	0.10066077
45	10	8199.19	86222.2	1843.252	19383.5266	8810.693891	5.6843073	0.11123889
44	10	8199.19	94421.39	1843.252	21226.7783	9648.535575	6.2248492	0.12181701
43	10	8199.19	102620.58	1843.252	23070.03	10486.37726	6.7653912	0.13239513
42	10	8696.58	111317.16	1955.069	25025.0994	11375.04519	7.3387242	0.143614954
41	10	8696.58	120013.74	1955.069	26980.1689	12263.71313	7.9120572	0.154834778
40	10	8199.19	128212.93	1843.252	28823.4206	13101.55481	8.4525991	0.165412898
39	10	8199.19	136412.12	1843.252	30666.6723	13939.39649	8.993141	0.175991018
38	10	8199.19	144611.31	1843.252	32509.924	14777.23818	9.533683	0.186569139
37	10	8199.19	152810.5	1843.252	34353.1757	15615.07986	10.074225	0.197147259
36	10	8199.19	161009.69	1843.252	36196.4274	16452.92155	10.614767	0.207725379
35	10	8199.19	169208.88	1843.252	38039.6791	17290.76323	11.155309	0.218303499
34	10	8199.19	177408.07	1843.252	39882.9308	18128.60491	11.695851	0.228881619
33	10	8199.19	185607.26	1843.252	41726.1825	18966.4466	12.236393	0.239459739
32	5	6171.17	191778.43	1387.335	43113.5171	19597.05321	12.643235	0.247421426
31	5	6171.17	197949.6	1387.335	44500.8516	20227.65983	13.050077	0.255383112
30	5	6171.17	204120.77	1387.335	45888.1862	20858.26645	13.456919	0.263344798
29	5	6171.17	210291.94	1387.335	47275.5207	21488.87306	13.863761	0.271306484
28	5	6171.17	216463.11	1387.335	48662.8553	22119.47968	14.270604	0.27926817
27	5	6171.17	222634.28	1387.335	50050.1899	22750.0863	14.677446	0.287229857
26	5	6171.17	228805.45	1387.335	51437.5244	23380.69291	15.084288	0.295191543
25	5	6171.17	234976.62	1387.335	52824.859	24011.29953	15.49113	0.303153229
24	5	6171.17	241147.79	1387.335	54212.1935	24641.90615	15.897972	0.311114915

23	5	6171.17	247319	1387.335	55599.53	25272.51	16.30481	0.319076601
22	5	6171.17	253490.1	1387.335	56986.86	25903.12	16.71166	0.327038287
21	5	6171.17	259661.3	1387.335	58374.2	26533.73	17.1185	0.334999973
20	5	6171.17	265832.5	1387.335	59761.53	27164.33	17.52534	0.34296166
19	5	6171.17	272003.6	1387.335	61148.87	27794.94	17.93218	0.350923346
18	5	6171.17	278174.8	1387.335	62536.2	28425.55	18.33903	0.358885032
17	5	6171.17	284346	1387.335	63923.54	29056.15	18.74587	0.366846718
16	10	8199.19	292545.2	1843.252	65766.79	29893.99	19.28641	0.377424838
15	10	8199.19	300744.4	1843.252	67610.04	30731.84	19.82695	0.388002959
14	10	8199.19	308943.6	1843.252	69453.29	31569.68	20.36749	0.398581079
13	10	8199.19	317142.7	1843.252	71296.54	32407.52	20.90804	0.409159199
12	10	8199.19	325341.9	1843.252	73139.79	33245.36	21.44858	0.419737319
11	10	8199.19	333541.1	1843.252	74983.05	34083.2	21.98912	0.430315439
10	10	8199.19	341740.3	1843.252	76826.3	34921.04	22.52966	0.440893559
9	10	8199.19	349939.5	1843.252	78669.55	35758.89	23.0702	0.45147168
8	10	8199.19	358138.7	1843.252	80512.8	36596.73	23.61074	0.4620498
7	10	8199.19	366337.9	1843.252	82356.05	37434.57	24.15129	0.47262792
6	10	8199.19	374537.1	1843.252	84199.3	38272.41	24.69183	0.48320604
5	10	8199.19	382736.3	1843.252	86042.56	39110.25	25.23237	0.49378416
4	10	8199.19	390935.5	1843.252	87885.81	39948.09	25.77291	0.50436228
3	10	8199.19	399134.6	1843.252	89729.06	40785.94	26.31345	0.514940401
2	10	8199.19	407333.8	1843.252	91572.31	41623.78	26.854	0.525518521
1	10	8199.19	415533	1843.252	93415.56	42461.62	27.39454	0.536096641
B1	10	8199.19	423732.2	1843.252	95258.81	43299.46	27.93508	0.546674761
B2	10	8199.19	431931.4	1843.252	97102.07	44137.3	28.47562	0.557252881
B3	10	8199.19	440130.6	1843.252	98945.32	44975.14	29.01616	0.567831001
B4	10	8199.19	448329.8	1843.252	100788.6	45812.99	29.55671	0.578409122

APPENDIX 6

Appendix 6.1 Column Calculation (CLT Model)

FLOOR	LL used for the floor (KN/m2)	LOAD (Kips) per FLOOR	LOAD (Kips) Total	LOAD (KN) per FLOOR	Cumulative load (KN) Total	AREA OF COLUMNS (in^2)	Area of columns	Total Area	Column dimensions B*D (m ²)
56	5	387.78	387.78	1724.93	1724.93	165.7179	0.106915	0.326978	
55	5	387.78	775.56	1724.93	3449.86	331.4359	0.213829	0.462417	
54	5	387.78	1163.34	1724.93	5174.79	497.1538	0.320744	0.566342	
53	5	387.78	1551.12	1724.93	6899.72	662.8718	0.427658	0.653956	0.7 m X 0.7 m
52	7.5	471.73	2022.85	2098.36	8998.08	864.4658	0.557719	0.746806	
51	7.5	471.73	2494.58	2098.36	11096.44	1066.06	0.687779	0.829325	
50	7.5	471.73	2966.31	2098.36	13194.80	1267.654	0.81784	0.904345	
49	7.5	471.73	3438.04	2098.36	15293.16	1469.248	0.9479	0.973602	1 m X 1 m
48	10	555.68	3993.72	2471.79	17764.94	1706.718	1.101106	1.049336	
47	10	555.68	4549.4	2471.79	20236.73	1944.188	1.254312	1.119961	
46	10	555.68	5105.08	2471.79	22708.52	2181.658	1.407519	1.186389	
45	10	555.68	5660.76	2471.79	25180.31	2419.128	1.560725	1.24929	
44	10	555.68	6216.44	2471.79	27652.09	2656.598	1.713931	1.309172	
43	10	555.68	6772.12	2471.79	30123.88	2894.068	1.867137	1.366432	
42	10	596.86	7368.98	2654.96	32778.84	3149.137	2.031697	1.425376	
41	10	596.86	7965.84	2654.96	35433.81	3404.205	2.196257	1.481977	1.5m X 1.5m
40	10	555.68	8521.52	2471.79	37905.59	3641.675	2.349463	1.532796	
39	10	555.68	9077.2	2471.79	40377.38	3879.145	2.502669	1.581983	
38	10	555.68	9632.88	2471.79	42849.17	4116.615	2.655876	1.629686	
37	10	555.68	10188.6	2471.79	45320.96	4354.085	2.809082	1.676032	
36	10	555.68	10744.2	2471.79	47792.74	4591.556	2.962288	1.72113	
35	10	555.68	11299.9	2471.79	50264.53	4829.026	3.115494	1.765076	
34	10	555.68	11855.6	2471.79	52736.32	5066.496	3.2687	1.807955	
33	10	555.68	12411.3	2471.79	55208.10	5303.966	3.421907	1.84984	1.8m X 1.8m
32	5	387.78	12799.1	1724.93	56933.03	5469.684	3.528821	1.878516	
31	5	387.78	13186.8	1724.93	58657.96	5635.402	3.635736	1.906761	
30	5	387.78	13574.6	1724.93	60382.90	5801.12	3.74265	1.934593	
29	5	387.78	13962.4	1724.93	62107.83	5966.838	3.849565	1.962031	
28	5	387.78	14350.2	1724.93	63832.76	6132.556	3.95648	1.98909	
27	5	387.78	14738	1724.93	65557.69	6298.274	4.063394	2.015786	
26	5	387.78	15125.7	1724.93	67282.62	6463.991	4.170309	2.042133	
25	5	387.78	15513.5	1724.93	69007.55	6629.709	4.277223	2.068145	2.1m X 2.1m
24	5	387.78	15901.3	1724.93	70732.48	6795.427	4.384138	2.093833	
23	5	387.78	16289.1	1724.93	72457.41	6961.145	4.491053	2.11921	
22	5	387.78	16676.9	1724.93	74182.34	7126.863	4.597967	2.144287	

21	5	387.78	17064.6	1724.93	75907.27	7292.58	4.704882	2.169074	
20	5	387.78	17452.4	1724.93	77632.20	7458.3	4.811796	2.193581	
19	5	387.78	17840.2	1724.93	79357.13	7624.02	4.918711	2.217817	
18	5	387.78	18228	1724.93	81082.06	7789.74	5.025625	2.241791	
17	5	387.78	18615.8	1724.93	82806.99	7955.45	5.13254	2.265511	2.3m X 2.3m
16	10	555.68	19171.4	2471.79	85278.78	8192.92	5.285746	2.299075	
15	10	555.68	19727.1	2471.79	87750.57	8430.39	5.438952	2.332156	
14	10	555.68	20282.8	2471.79	90222.35	8667.86	5.592159	2.364775	
13	10	555.68	20838.5	2471.79	92694.14	8905.33	5.745365	2.396949	
12	10	555.68	21394.2	2471.79	95165.93	9142.8	5.898571	2.428697	
11	10	555.68	21949.8	2471.79	97637.71	9380.27	6.051777	2.460036	
10	10	555.68	22505.5	2471.79	100109.50	9617.74	6.204983	2.49098	
9	10	555.68	23061.2	2471.79	102581.29	9855.21	6.35819	2.521545	2.5m X 2.5m
8	10	555.68	23616.9	2471.79	105053.08	10092.7	6.511396	2.551744	
7	10	555.68	24172.6	2471.79	107524.86	10330.2	6.664602	2.581589	
6	10	555.68	24728.2	2471.79	109996.65	10567.6	6.817808	2.611093	
5	10	555.68	25283.9	2471.79	112468.44	10805.1	6.971014	2.640268	
4	10	555.68	25839.6	2471.79	114940.22	11042.6	7.124221	2.669124	
3	10	555.68	26395.3	2471.79	117412.01	11280	7.277427	2.697671	
2	10	555.68	26951	2471.79	119883.80	11517.5	7.430633	2.725919	
1	10	555.68	27506.6	2471.79	122355.58	11755	7.583839	2.753877	2.7m X 2.7m
B1	10	555.68	28062.3	2471.79	124827.37	11992.4	7.737045	2.781555	
B2	10	555.68	28618	2471.79	127299.16	12229.9	7.890252	2.808959	
B3	10	555.68	29173.7	2471.79	129770.94	12467.4	8.043458	2.836099	
B4	10	555.68	29729.4	2471.79	132242.73	12704.9	8.196664	2.862982	2.9m X 2.9m

APPENDIX 7

Appendix 7.1 Shear Wall Calculation (CLT Model)

floor	LL used for the floor (KN/m2)	Load per Floor (KN)	Cumulative load (KN)	Load per Floor (Kips)	Cumulative load (Kips)	AREA OF SHEAR WALL (in^2)	Area of Shear Wall (m^2)	Thickness of the Shear Wall (m)
56	5	5107.97	6171.17	1148.318	1148.31763	497.1072	0.32071368	0.006276197
55	5	5107.97	11279.14	1148.318	2296.63526	994.2144	0.64142736	0.012552395
54	5	5107.97	16387.11	1148.318	3444.95288	1491.322	0.96214104	0.018828592
53	5	5107.97	21495.08	1148.318	4593.27051	1988.429	1.28285472	0.025104789
52	7.5	6121.99	27617.07	1376.278	5969.54896	2584.22	1.66723559	0.03262692
51	7.5	6121.99	33739.06	1376.278	7345.82741	3180.012	2.05161646	0.04014905
50	7.5	6121.99	39861.05	1376.278	8722.10586	3775.803	2.43599732	0.04767118
49	7.5	6121.99	45983.04	1376.278	10098.3843	4371.595	2.82037819	0.055193311
48	7.5	6121.99	52105.03	1376.278	11474.6628	4967.386	3.20475906	0.062715441
47	7.5	6121.99	58227.02	1376.278	12850.9412	5563.178	3.58913993	0.070237572
46	7.5	6121.99	64349.01	1376.278	14227.2197	6158.97	3.97352079	0.077759702
45	7.5	6121.99	70471	1376.278	15603.4981	6754.761	4.35790166	0.085281833
44	10	7136.01	77607.01	1604.239	17207.7374	7449.237	4.80594972	0.094049897
43	10	7136.01	84743.02	1604.239	18811.9767	8143.713	5.25399777	0.10281796
42	10	7136.01	91879.03	1604.239	20416.2159	8838.189	5.70204583	0.111586024
41	10	7136.01	99015.04	1604.239	22020.4552	9532.665	6.15009389	0.120354088
40	10	7136.01	106151.1	1604.239	23624.6945	10227.14	6.59814194	0.129122151
39	10	7136.01	113287.1	1604.239	25228.9337	10921.62	7.04619	0.137890215
38	10	7633.38	120920.4	1716.053	26944.9863	11664.5	7.52546638	0.147269401
37	10	7633.38	128553.8	1716.053	28661.0388	12407.38	8.00474276	0.156648586
36	10	7136.01	135689.8	1604.239	30265.2781	13101.85	8.45279082	0.16541665
35	10	7136.01	142825.8	1604.239	31869.5173	13796.33	8.90083888	0.174184714
34	10	7136.01	149961.9	1604.239	33473.7566	14490.8	9.34888693	0.182952778
33	10	7136.01	157097.9	1604.239	35077.9959	15185.28	9.79693499	0.191720841
32	10	7136.01	164233.9	1604.239	36682.2352	15879.76	10.244983	0.200488905
31	10	7136.01	171369.9	1604.239	38286.4744	16574.23	10.6930311	0.209256969
30	10	7136.01	178505.9	1604.239	39890.7137	17268.71	11.1410792	0.218025032
29	10	7136.01	185641.9	1604.239	41494.953	17963.18	11.5891272	0.226793096
28	5	5107.97	190749.9	1148.318	42643.2706	18460.29	11.9098409	0.233069293
27	5	5107.97	195857.8	1148.318	43791.5882	18957.4	12.2305546	0.239345491
26	5	5107.97	200965.8	1148.318	44939.9059	19454.5	12.5512683	0.245621688
25	5	5107.97	206073.8	1148.318	46088.2235	19951.61	12.8719819	0.251897885
24	5	5107.97	211181.8	1148.318	47236.5411	20448.72	13.1926956	0.258174082
23	5	5107.97	216289.7	1148.318	48384.8587	20945.83	13.5134093	0.26445028

22	5	5107.97	221397.7	1148.318	49533.1764	21442.9335	13.834123	0.270726477
21	5	5107.97	226505.7	1148.318	50681.494	21940.0407	14.1548366	0.277002674
20	5	5107.97	231613.6	1148.318	51829.8116	22437.1479	14.4755503	0.283278871
19	5	5107.97	236721.6	1148.318	52978.1292	22934.2551	14.796264	0.289555069
18	5	5107.97	241829.6	1148.318	54126.4469	23431.3623	15.1169777	0.295831266
17	5	5107.97	246937.5	1148.318	55274.7645	23928.4695	15.4376914	0.302107463
16	5	5107.97	252045.5	1148.318	56423.0821	24425.5767	15.758405	0.30838366
15	5	5107.97	257153.5	1148.318	57571.3998	24922.6839	16.0791187	0.314659858
14	5	5107.97	262261.5	1148.318	58719.7174	25419.7911	16.3998324	0.320936055
13	5	5107.97	267369.4	1148.318	59868.035	25916.8983	16.7205461	0.327212252
12	10	7136.01	274505.4	1604.239	61472.2743	26611.3741	17.1685941	0.335980316
11	10	7136.01	281641.4	1604.239	63076.5136	27305.85	17.6166422	0.34474838
10	10	7136.01	288777.5	1604.239	64680.7528	28000.3259	18.0646903	0.353516443
9	10	7136.01	295913.5	1604.239	66284.9921	28694.8018	18.5127383	0.362284507
8	10	7136.01	303049.5	1604.239	67889.2314	29389.2776	18.9607864	0.371052571
7	10	7136.01	310185.5	1604.239	69493.4706	30083.7535	19.4088344	0.379820635
6	10	7136.01	317321.5	1604.239	71097.7099	30778.2294	19.8568825	0.388588698
5	10	7136.01	324457.5	1604.239	72701.9492	31472.7053	20.3049305	0.397356762
4	10	7136.01	331593.5	1604.239	74306.1885	32167.1812	20.7529786	0.406124826
3	10	7136.01	338729.5	1604.239	75910.4277	32861.657	21.2010266	0.414892889
2	10	7136.01	345865.5	1604.239	77514.667	33556.1329	21.6490747	0.423660953
1	10	7136.01	353001.5	1604.239	79118.9063	34250.6088	22.0971228	0.432429017
B1	10	7136.01	360137.6	1604.239	80723.1455	34945.0847	22.5451708	0.441197081
B2	10	7136.01	367273.6	1604.239	82327.3848	35639.5605	22.9932189	0.449965144
B3	10	7136.01	374409.6	1604.239	83931.6241	36334.0364	23.4412669	0.458733208

APPENDIX 8

Appendix 8.1 Foundation Design (RCC Model)

Coloumn	Location	X (m)	Y (m)	Load (KN)	Load× X	Load ×Y	X'	Y'
C1	A1	1.6	1.6	104246.5	166794.4	166794.4	62.05649	32.61575
C2	A2	7.15	1.6	110157.4	787625.41	176251.84		
C3	A3	12.1	1.6	112217.9	1357836.59	179548.64		
C4	A4	17.25	1.6	112773	1945334.25	180436.8		
C5	A6	22.75	1.6	114468	2604147	183148.8		
C6	A8	28.25	1.6	112167.3	3168726.225	179467.68		
C7	A9	33.4	1.6	109502.7	3657390.18	175204.32		
C8	A10	38.35	1.6	103455.9	3967533.765	165529.44		
C9	A11	41.6	1.6	97639	4061782.4	156222.4		
C10	A12	46.6	1.6	7946.9	370325.54	12715.04		
C11	A13	51.6	1.6	5041.8	260156.88	8066.88		
C12	A14	56.6	1.6	4125.1	233480.66	6600.16		
C13	A15	61.6	1.6	4343.3	267547.28	6949.28		
C14	A16	66.6	1.6	3961.9	263862.54	6339.04		
C15	A17	71.6	1.6	5042	361007.2	8067.2		
C16	A18	76.6	1.6	8277.1	634025.86	13243.36		
C17	A19	81.6	1.6	104199.3	8502662.88	166718.88		
C18	A20	87.15	1.6	11243.9	979905.885	17990.24		
C19	A21	92.1	1.6	114212.5	10518971.25	182740		
C20	A22	97.25	1.6	114093.7	11095612.33	182549.92		
C21	A24	102.75	1.6	114160.9	11730032.48	182657.44		
C22	A26	108.25	1.6	110217.6	11931055.2	176348.16		
C23	A27	113.4	1.6	106841.9	12115871.46	170947.04		
C24	A28	118.35	1.6	100294.8	11869889.58	160471.68		
C25	A29	121.6	1.6	98053.5	11923305.6	156885.6		
C26	B1	1.6	7.575	109410.5	175056.8	828784.538		
C27	B4	17.25	7.575	133359.9	2300458.275	1010201.24		
C28	B8	28.25	7.575	132980.4	3756696.3	1007326.53		
C29	B11	41.6	7.575	108464.8	4512135.68	821620.86		
C30	B14	56.6	7.575	11294	639240.4	85552.05		
C31	B16	66.6	7.575	11294.1	752187.06	85552.8075		
C32	B19	81.6	7.575	114591.7	9350682.72	868032.128		
C33	B22	97.25	7.575	135233.5	13151457.88	1024393.76		
C34	B26	108.25	7.575	130817.1	14160951.08	990939.533		
C35	B29	121.6	7.575	103780.7	12619733.12	786138.803		
C36	C1	1.6	13.55	110301.3	176482.08	1494582.62		
C37	C2	7.15	13.55	122135	873265.25	1654929.25		
C38	C3	12.1	13.55	132219.7	1599858.37	1791576.94		
C39	C9	33.4	13.55	129242.7	4316706.18	1751238.59		
C40	C10	38.35	13.55	113131	4338573.85	1532925.05		

C40	C10	38.35	13.55	113131	4338573.85	1532925.05
C41	C11	41.6	13.55	105063.1	4370624.96	1423605.005
C42	C12	46.6	13.55	9785.8	456018.28	132597.59
C43	C13	51.6	13.55	9495	489942	128657.25
C44	C17	71.6	13.55	9628.9	689429.24	130471.595
C45	C18	76.6	13.55	10121.1	775276.26	137140.905
C46	C19	81.6	13.55	111456.4	9094842.24	1510234.22
C47	C20	87.15	13.55	125435.1	10931668.97	1699645.605
C48	C21	92.1	13.55	135862.6	12512945.46	1840938.23
C49	C27	113.4	13.55	125525	14234535	1700863.75
C50	C28	118.35	13.55	109258.8	12930778.98	1480456.74
C51	C29	121.6	13.55	104407.6	12695964.16	1414722.98
C52	D1	1.6	19.05	109555.8	175289.28	2087037.99
C53	D2	7.15	19.05	119319.4	853133.71	2273034.57
C54	D4	17.25	19.05	152940.4	2638221.9	2913514.62
C55	D8	28.25	19.05	149171.5	4214094.875	2841717.075
C56	D10	38.35	19.05	112044.9	4296921.915	2134455.345
C57	D11	41.6	19.05	103839.2	4319710.72	1978136.76
C58	D12	46.6	19.05	8388.6	390908.76	159802.83
C59	D13	51.6	19.05	7929.1	409141.56	151049.355
C60	D15	61.6	19.05	13388.3	824719.28	255047.115
C61	D17	71.6	19.05	8090	579244	154114.5
C62	D18	76.6	19.05	8702.6	666619.16	165784.53
C63	D19	81.6	19.05	110288.6	8999549.76	2100997.83
C64	D20	87.15	19.05	123567.8	10768933.77	2353966.59
C65	D22	97.25	19.05	152450.2	14825781.95	2904176.31
C66	D26	108.25	19.05	151266.6	16374609.45	2881628.73
C67	D28	118.35	19.05	107439.5	12715464.83	2046722.475
C68	D29	121.6	19.05	103384.4	12571543.04	1969472.82
C69	E4	17.25	23.15	98199.6	1693943.1	2273320.74
C70	E8	28.25	23.15	67328.2	1902021.65	1558647.83
C71	E22	97.25	23.15	95255.7	9263616.825	2205169.455
C72	E26	108.25	23.15	71189.5	7706263.375	1648036.925
C73	F1	1.6	24.55	109204.9	174727.84	2680980.295
C74	F2	7.15	24.55	121523.8	868895.17	2983409.29
C75	F10	38.35	24.55	113544.6	4354435.41	2787519.93
C76	F11	41.6	24.55	104993	4367708.8	2577578.15
C77	F12	46.6	24.55	10112.2	471228.52	248254.51
C78	F14	56.6	24.55	9475.7	536324.62	232628.435
C79	F16	66.6	24.55	9464.8	630355.68	232360.84
C80	F18	76.6	24.55	10466.4	801726.24	256950.12

C81	F19	81.6	24.55	110355.6	9005016.96	2709229.98
C82	F20	87.15	24.55	125611	10946998.65	3083750.05
C83	F28	118.35	24.55	108588.2	12851413.47	2665840.31
C84	F29	121.6	24.55	104488.3	12705777.28	2565187.765
C85	G1	1.6	30.05	107980.3	172768.48	3244808.015
C86	G2	7.15	30.05	120478.5	861421.275	3620378.925
C87	G10	38.35	30.05	112757.6	4324253.96	3388365.88
C88	G11	41.6	30.05	105346.4	4382410.24	3165659.32
C89	G13	51.6	30.05	10023.2	517197.12	301197.16
C90	G15	61.6	30.05	9391.2	578497.92	282205.56
C91	G17	71.6	30.05	9691.4	693904.24	291226.57
C92	G19	81.6	30.05	111723.1	9116604.96	3357279.155
C93	G20	87.15	30.05	124865.4	10882019.61	3752205.27
C94	G28	118.35	30.05	107777.6	12755478.96	3238716.88
C95	G29	121.6	30.05	102133.4	12419421.44	3069108.67
C96	H1	1.6	35.55	105751.5	169202.4	3759465.825
C97	H2	7.15	35.55	117772	842069.8	4186794.6
C98	H10	38.35	35.55	11000.6	421873.01	391071.33
C99	H11	41.6	35.55	100622.1	4185879.36	3577115.655
C100	H12	46.6	35.55	10592.1	493591.86	376549.155
C101	H14	56.6	35.55	9300.3	526396.98	330625.665
C102	H16	66.6	35.55	9279.8	618034.68	329896.89
C103	H18	76.6	35.55	10920.3	836494.98	388216.665
C104	H19	81.6	35.55	107004.5	8731567.2	3804009.975
C105	H20	87.15	35.55	122055.6	10637145.54	4339076.58
C106	H28	118.35	35.55	105384.3	12472231.91	3746411.865
C107	H29	121.6	35.55	99685	12121696	3543801.75
C108	I1	1.6	41.05	102479.2	163966.72	4206771.16
C109	I2	7.15	41.05	112431.8	803887.37	4615325.39
C110	I10	38.35	41.05	104832.1	4020311.035	4303357.705
C111	I11	41.6	41.05	96438.8	4011854.08	3958812.74
C112	I12	46.6	41.05	9354	435896.4	383981.7
C113	I14	56.6	41.05	9060.4	512818.64	371929.42
C114	I16	66.6	41.05	9060	603396	371913
C115	I18	76.6	41.05	9667.6	740538.16	396854.98
C116	I19	81.6	41.05	103226.1	8423249.76	4237431.405
C117	I20	87.15	41.05	116431.8	10147031.37	4779525.39
C118	I28	118.35	41.05	100830.5	11933289.68	4139092.025
C119	I29	121.6	41.05	96115.4	11687632.64	3945537.17

C120	J4	17.25	42.45	5587.4	96382.65	237185.13
C121	J8	28.25	42.45	81797.6	2310782.2	3472308.12
C122	J22	97.25	42.45	51719.3	5029701.925	2195484.285
C123	J26	108.25	42.45	83795.6	9070873.7	3557123.22
C124	K1	1.6	46.55	98359.6	157375.36	4578639.38
C125	K2	7.15	46.55	106201.5	759340.725	4943679.825
C126	K3	12.1	46.55	109507.9	1325045.59	5097592.745
C127	K6	22.75	46.55	119011.6	2707513.9	5539989.98
C128	K9	33.4	46.55	106724.6	3564601.64	4968030.13
C129	K10	38.35	46.55	98202.1	3766050.535	4571307.755
C130	K11	41.6	46.55	92042.9	3828984.64	4284596.995
C131	K12	46.6	46.55	8519.7	397018.02	396592.035
C132	K13	51.6	46.55	7149.3	368903.88	332799.915
C133	K15	61.6	46.55	7716.8	475354.88	359217.04
C134	K17	71.6	46.55	7242.6	518570.16	337143.03
C135	K18	76.6	46.55	8874.6	679794.36	413112.63
C136	K19	81.6	46.55	99160.3	8091480.48	4615911.965
C137	K20	87.15	46.55	108484	9454380.6	5049930.2
C138	K21	92.1	46.55	111378.3	10257941.43	5184659.865
C139	K24	102.75	46.55	117428.3	12065757.83	5466287.365
C140	K27	113.4	46.55	103911.8	11783598.12	4837094.29
C141	K28	118.35	46.55	95211.6	11268292.86	4432099.98
C142	K29	121.6	46.55	91892	11174067.2	4277572.6
C143	L1	1.6	51.2	95482.9	152772.64	4888724.48
C144	L3	12.1	51.2	103687.5	1254618.75	5308800
C145	L4	17.25	51.2	104163.7	1796823.825	5333181.44
C146	L6	22.75	51.2	104814.2	2384523.05	5366487.04
C147	L8	28.25	51.2	103210.3	2915690.975	5284367.36
C148	L9	33.4	51.2	100019	3340634.6	5120972.8
C149	L11	41.6	51.2	91588.4	3810077.44	4689326.08
C150	L13	51.6	51.2	6058.2	312603.12	310179.84
C151	L14	56.6	51.2	5562	314809.2	284774.4
C152	L15	61.6	51.2	5244.2	323042.72	268503.04
C153	L16	66.6	51.2	5582.9	371821.14	285844.48
C154	L17	71.6	51.2	6133.2	439137.12	314019.84
C155	L19	81.6	51.2	98737.8	8057004.48	5055375.36
C156	L21	92.1	51.2	104500.5	9624496.05	5350425.6
C157	L22	97.25	51.2	104851.8	10196837.55	5368412.16
C158	L24	102.75	51.2	104159.6	10702398.9	5332971.52
C159	L26	108.25	51.2	101578.5	10995872.63	5200819.2
C160	L27	113.4	51.2	98411.6	11159875.44	5038673.92

C161	L29	121.6	51.2	89073.4	10831325.44	4560558.08
C162	M1	1.6	56.45	90749.6	145199.36	5122814.92
C163	M2	7.15	56.45	95936.2	685943.83	5415598.49
C164	M3	12.1	56.45	95426.6	1154661.86	5386831.57
C165	M4	17.25	56.45	96074.4	1657283.4	5423399.88
C166	M6	22.75	56.45	96622.8	2198168.7	5454357.06
C167	M8	28.25	56.45	95344.2	2693473.65	5382180.09
C168	M9	33.4	56.45	92866.2	3101731.08	5242296.99
C169	M10	38.35	56.45	88894.4	3409100.24	5018088.88
C170	M11	41.6	56.45	83799.3	3486050.88	4730470.485
C171	M12	46.6	56.45	7720.4	359770.64	435816.58
C172	M13	51.6	56.45	4896.9	252680.04	276430.005
C173	M14	56.6	56.45	4538.2	256862.12	256181.39
C174	M15	61.6	56.45	4566.8	281314.88	257795.86
C175	M16	66.6	56.45	4546.6	302803.56	256655.57
C176	M17	71.6	56.45	5002.6	358186.16	282396.77
C177	M18	76.6	56.45	8134.4	623095.04	459186.88
C178	M19	81.6	56.45	91203.8	7442230.08	5148454.51
C179	M20	87.15	56.45	97479	8495294.85	5502689.55
C180	M21	92.1	56.45	96977.4	8931618.54	5474374.23
C181	M22	97.25	56.45	97047.3	9437849.925	5478320.085
C182	M24	102.75	56.45	96330.7	9897979.425	5437868.015
C183	M26	108.25	56.45	93874.6	10161925.45	5299221.17
C184	M27	113.4	56.45	90867.2	10304340.48	5129453.44
C185	M28	118.35	56.45	86581.5	10246920.53	4887525.675
C186	M29	121.6	56.45	84095.1	10225964.16	4747168.395
C187	N1	1.6	61.6	84689	135502.4	5216842.4
C188	N2	7.15	61.6	87004.5	622082.175	5359477.2
C189	N3	12.1	61.6	87224.6	1055417.66	5373035.36
C190	N4	17.25	61.6	87769.9	1514030.775	5406625.84
C191	N6	22.75	61.6	88087.4	2003988.35	5426183.84
C192	N8	28.25	61.6	87137.6	2461637.2	5367676.16
C193	N9	33.4	61.6	85041.9	2840399.46	5238581.04
C194	N10	38.35	61.6	81459.3	3123964.155	5017892.88
C195	N11	41.6	61.6	76989.2	3202750.72	4742534.72
C196	N12	46.6	61.6	5473.7	255074.42	337179.92
C197	N13	51.6	61.6	3130.1	161513.16	192814.16
C198	N14	56.6	61.6	2965.8	167864.28	182693.28
C199	N15	61.6	61.6	2983.3	183771.28	183771.28
C200	N16	66.6	61.6	2971.4	197895.24	183038.24
C201	N17	71.6	61.6	3170.4	227000.64	195296.64

C202	N18	76.6	61.6	5860.5	448914.3	361006.8
C203	N19	81.6	61.6	84041.1	6857753.76	5176931.76
C204	N20	87.15	61.6	88621.7	7723381.155	5459096.72
C205	N21	92.1	61.6	88985.3	8195546.13	5481494.48
C206	N22	97.25	61.6	88925.9	8648043.775	5477835.44
C207	N24	102.75	61.6	88003.2	9042328.8	5420997.12
C208	N26	108.25	61.6	85844.8	9292699.6	5288039.68
C209	N27	113.4	61.6	83169	9431364.6	5123210.4
C210	N28	118.35	61.6	79495.5	9408292.425	4896922.8
C211	N29	121.6	61.6	78470	9541952	4833752
Σ				16175882.4	1003818564	527588548.1

Location	Strip 1 loads "A19-N19" (KN)	Factored Load of strip 1
A19	104199.3	82863.67015
B19	114591.7	91128.14415
C19	111456.4	88634.82159
D19	110288.6	87706.13787
F19	110355.6	87759.41909
G19	111723.1	88846.91266
H19	107004.5	85094.48329
I19	103226.1	82089.74054
K19	99160.3	78856.44521
L19	98737.8	78520.45542
M19	91203.8	72529.10144
N19	84041.1	66833.02085
Σ	1245988.3	

Location	Strip 2 loads "M1-M29" (KN)	Factored Load of strip 2
M1	90749.6	83379.49829
M2	95936.2	88144.87583
M3	95426.6	87676.66228
M4	96074.4	88271.85211
M6	96622.8	88775.71457
M8	95344.2	87600.95428
M9	92866.2	85324.20158
M10	88894.4	81674.96576
M11	83799.3	76993.65717
M12	7720.4	7093.398523
M13	4896.9	4499.205122
M14	4538.2	4169.63644
M15	4566.8	4195.913732
M16	4546.6	4177.354246
M17	5002.6	4596.320845
M18	8134.4	7473.776092
M19	91203.8	83796.81107
M20	97479	89562.37949
M21	96977.4	89101.51623
M22	97047.3	89165.7394
M24	96330.7	88507.33706
M26	93874.6	86250.70579
M27	90867.2	83487.54757
M28	86581.5	79549.90469
M29	84095.1	77265.43419
Σ	1709576.2	

APPENDIX 9

Appendix 9.1 Foundation Design (CLT Model)

Coloumn	Location	X (m)	Y (m)	Load (KN)	Load × X	Load × Y	X'	Y'
C1	A1	1.45	1.45	91027.2	131989.44	131989.44	61.90586	32.33495
C2	A2	7	1.45	95800.8	670605.6	138911.16		
C3	A3	11.95	1.45	97323.8	1163019.41	141119.51		
C4	A4	17.1	1.45	97464.9	1666649.79	141324.105		
C5	A6	22.6	1.45	98179.4	2218854.44	142360.13		
C6	A8	28.1	1.45	96806.4	2720259.84	140369.28		
C7	A9	33.25	1.45	94660.1	3147448.325	137257.145		
C8	A10	38.2	1.45	89598.6	3422666.52	129917.97		
C9	A11	41.45	1.45	84832.5	3516307.125	123007.125		
C10	A12	46.45	1.45	6728.3	312529.535	9756.035		
C11	A13	51.45	1.45	4591.1	236212.095	6657.095		
C12	A14	56.45	1.45	3680.6	207769.87	5336.87		
C13	A15	61.45	1.45	3846	236336.7	5576.7		
C14	A16	66.45	1.45	3560	236562	5162		
C15	A17	71.45	1.45	4600.3	328691.435	6670.435		
C16	A18	76.45	1.45	7006.1	535616.345	10158.845		
C17	A19	81.45	1.45	90800	7395660	131660		
C18	A20	87	1.45	97385.2	8472512.4	141208.54		
C19	A21	91.95	1.45	98840.1	9088347.195	143318.145		
C20	A22	97.1	1.45	98553.3	9569525.43	142902.285		
C21	A24	102.6	1.45	97918.2	10046407.32	141981.39		
C22	A26	108.1	1.45	95180.7	10289033.67	138012.015		
C23	A27	113.25	1.45	92559.5	10482363.38	134211.275		
C24	A28	118.2	1.45	87241.2	10311909.84	126499.74		
C25	A29	121.45	1.45	85300.5	10359745.73	123685.725		
C26	B1	1.45	7.425	95166.8	137991.86	706613.49		
C27	B4	17.1	7.425	114943	1965525.3	853451.775		
C28	B8	28.1	7.425	114552.2	3218916.82	850550.085		
C29	B11	41.45	7.425	93813.7	3888577.865	696566.7225		
C30	B14	56.45	7.425	10867.7	613481.665	80692.6725		
C31	B16	66.45	7.425	10848.4	720876.18	80549.37		
C32	B19	81.45	7.425	99394	8095641.3	738000.45		
C33	B22	97.1	7.425	116578.1	11319733.51	865592.3925		
C34	B26	108.1	7.425	112694.5	12182275.45	836756.6625		
C35	B29	121.45	7.425	89914.5	10920116.03	667615.1625		
C36	C1	1.45	13.4	95531.9	138521.255	1280127.46		
C37	C2	7	13.4	104995.1	734965.7	1406934.34		
C38	C3	11.95	13.4	112761.8	1347503.51	1511008.12		
C39	C9	33.25	13.4	110090.8	3660519.1	1475216.72		
C40	C10	38.2	13.4	97006.2	3705636.84	1299883.08		

C41	C11	41.45	13.4	90635.3	3756833.185	1214513.02
C42	C12	46.45	13.4	8413.6	390811.72	112742.24
C43	C13	51.45	13.4	9072.2	466764.69	121567.48
C44	C17	71.45	13.4	9163.1	654703.495	122785.54
C45	C18	76.45	13.4	8700.4	665145.58	116585.36
C46	C19	81.45	13.4	96588.3	7867117.035	1294283.22
C47	C20	87	13.4	107615.1	9362513.7	1442042.34
C48	C21	91.95	13.4	115916.7	10658540.57	1553283.78
C49	C27	113.25	13.4	106983	12115824.75	1433572.2
C50	C28	118.2	13.4	93938.3	11103507.06	1258773.22
C51	C29	121.45	13.4	89962.6	10925957.77	1205498.84
C52	D1	1.45	18.9	94632.8	137217.56	1788559.92
C53	D2	7	18.9	102453.9	717177.3	1936378.71
C54	D4	17.1	18.9	131420.2	2247285.42	2483841.78
C55	D8	28.1	18.9	127991.4	3596558.34	2419037.46
C56	D10	38.2	18.9	9607.5	367006.5	181581.75
C57	D11	41.45	18.9	89289.1	3701033.195	1687563.99
C58	D12	46.45	18.9	7084.4	329070.38	133895.16
C59	D13	51.45	18.9	7681.2	395197.74	145174.68
C60	D15	61.45	18.9	13011.1	799532.095	245909.79
C61	D17	71.45	18.9	7771.5	555273.675	146881.35
C62	D18	76.45	18.9	7347.9	561746.955	138875.31
C63	D19	81.45	18.9	95310.8	7763064.66	1801374.12
C64	D20	87	18.9	105960.3	9218546.1	2002649.67
C65	D22	97.1	18.9	130868.6	12707341.06	2473416.54
C66	D26	108.1	18.9	129926.9	14045097.89	2455618.41
C67	D28	118.2	18.9	92227.5	10901290.5	1743099.75
C68	D29	121.45	18.9	88769	10780995.05	1677734.1
C69	E4	17.1	23	86603	1480911.3	1991869
C70	E8	28.1	23	59446.2	1670438.22	1367262.6
C71	E22	97.1	23	84171.3	8173033.23	1935939.9
C72	E26	108.1	23	62663.6	6773935.16	1441262.8
C73	F1	1.45	24.4	94020.8	136330.16	2294107.52
C74	F2	7	24.4	103474.7	724322.9	2524782.68
C75	F10	38.2	24.4	96815.5	3698352.1	2362298.2
C76	F11	41.45	24.4	89761.6	3720618.32	2190183.04
C77	F12	46.45	24.4	8790.9	408337.305	214497.96
C78	F14	56.45	24.4	9125.4	515128.83	222659.76
C79	F16	66.45	24.4	9112.1	605499.045	222335.24
C80	F18	76.45	24.4	9099.4	695649.13	222025.36
C81	F19	81.45	24.4	95179	7752329.55	2322367.6

C82	F20	87	24.4	107068.8	9314985.6	2612478.72
C83	F28	118.2	24.4	92600.7	10945402.74	2259457.08
C84	F29	121.45	24.4	88938.6	10801592.97	2170101.84
C85	G1	1.45	29.9	92772.2	134519.69	2773888.78
C86	G2	7	29.9	102416.6	716916.2	3062256.34
C87	G10	38.2	29.9	95840.4	3661103.28	2865627.96
C88	G11	41.45	29.9	89973.6	3729405.72	2690210.64
C89	G13	51.45	29.9	9411.6	484226.82	281406.84
C90	G15	61.45	29.9	9016.7	554076.215	269599.33
C91	G17	71.45	29.9	9273.3	662577.285	277271.67
C92	G19	81.45	29.9	95991.1	7818475.095	2870133.89
C93	G20	87	29.9	106315.8	9249474.6	3178842.42
C94	G28	118.2	29.9	91503.6	10815725.52	2735957.64
C95	G29	121.45	29.9	87078.4	10575671.68	2603644.16
C96	H1	1.45	35.4	90766.4	131611.28	3213130.56
C97	H2	7	35.4	100024	700168	3540849.6
C98	H10	38.2	35.4	93452.2	3569874.04	3308207.88
C99	H11	41.45	35.4	86046.9	3566644.005	3046060.26
C100	H12	46.45	35.4	9293.3	431673.785	328982.82
C101	H14	56.45	35.4	9055.1	511160.395	320550.54
C102	H16	66.45	35.4	9037.7	600555.165	319934.58
C103	H18	76.45	35.4	9592.8	733369.56	339585.12
C104	H19	81.45	35.4	92038.9	7496568.405	3258177.06
C105	H20	87	35.4	103798.7	9030486.9	3674473.98
C106	H28	118.2	35.4	89435.3	10571252.46	3166009.62
C107	H29	121.45	35.4	84958.1	10318161.25	3007516.74
C108	I1	1.45	40.9	87984.4	127577.38	3598561.96
C109	I2	7	40.9	95579.7	669057.9	3909209.73
C110	I10	38.2	40.9	88969.1	3398619.62	3638836.19
C111	I11	41.45	40.9	82390	3415065.5	3369751
C112	I12	46.45	40.9	8147.5	378451.375	333232.75
C113	I14	56.45	40.9	8824.7	498154.315	360930.23
C114	I16	66.45	40.9	8818.2	585969.39	360664.38
C115	I18	76.45	40.9	8418.9	643624.905	344333.01
C116	I19	81.45	40.9	88701.5	7224737.175	3627891.35
C117	I20	87	40.9	98972.1	8610572.7	4047958.89
C118	I28	118.2	40.9	85581.8	10115768.76	3500295.62
C119	I29	121.45	40.9	81958	9953799.1	3352082.2
C120	J4	17.1	42.3	49054.8	838837.08	2075018.04
C121	J8	28.1	42.3	72203.3	2028912.73	3054199.59

C122	J22	97.1	42.3	45642	4431838.2	1930656.6
C123	J26	108.1	42.3	73825.7	7980558.17	3122827.11
C124	K1	1.45	46.55	84548.6	122595.47	3935737.33
C125	K2	7	46.55	90830.5	635813.5	4228159.775
C126	K3	11.95	46.55	93690.2	1119597.89	4361278.81
C127	K6	22.6	46.55	101412	2291911.2	4720728.6
C128	K9	33.25	46.55	90985.5	3025267.875	4235375.025
C129	K10	38.2	46.55	83591.8	3193206.76	3891198.29
C130	K11	41.45	46.55	78756	3264436.2	3666091.8
C131	K12	46.45	46.55	7318.3	339935.035	340666.865
C132	K13	51.45	46.55	6716.4	345558.78	312648.42
C133	K15	61.45	46.55	7323.4	450022.93	340904.27
C134	K17	71.45	46.55	6773.9	483995.155	315325.045
C135	K18	76.45	46.55	7627.3	583107.085	355050.815
C136	K19	81.45	46.55	85278	6945893.1	3969690.9
C137	K20	87	46.55	92531.9	8050275.3	4307359.945
C138	K21	91.95	46.55	95174.6	8751304.47	4430377.63
C139	K24	102.6	46.55	100076.6	10267859.16	4658565.73
C140	K27	113.25	46.55	88752.9	10051265.93	4131447.495
C141	K28	118.2	46.55	81298.1	9609435.42	3784426.555
C142	K29	121.45	46.55	78563.6	9541549.22	3657135.58
C143	L1	1.45	51.05	82127.8	119085.31	4192624.19
C144	L3	11.95	51.05	88659.9	1059485.805	4526087.895
C145	L4	17.1	51.05	88939.6	1520867.16	4540366.58
C146	L6	22.6	51.05	89356.8	2019463.68	4561664.64
C147	L8	28.1	51.05	87946.2	2471288.22	4489653.51
C148	L9	33.25	51.05	85155.3	2831413.725	4347178.065
C149	L11	41.45	51.05	78238.7	3242994.115	3994085.635
C150	L13	51.45	51.05	5601.2	288181.74	285941.26
C151	L14	56.45	51.05	5234	295459.3	267195.7
C152	L15	61.45	51.05	4929.8	302936.21	251666.29
C153	L16	66.45	51.05	5244.6	348503.67	267736.83
C154	L17	71.45	51.05	5656	404121.2	288738.8
C155	L19	81.45	51.05	84694.9	6898399.605	4323674.645
C156	L21	91.95	51.05	89217.5	8203549.125	4554553.375
C157	L22	97.1	51.05	89646.7	8704694.57	4576464.035
C158	L24	102.6	51.05	88841.4	9115127.64	4535353.47
C159	L26	108.1	51.05	86646.4	9366475.84	4423298.72
C160	L27	113.25	51.05	83994.4	9512365.8	4287914.12
C161	L29	121.45	51.05	76256.1	9261303.345	3892873.905
C162	M1	1.45	56.3	78080	113216	4395904

C163	M2	7	56.3	82261.9	575833.3	4631344.97
C164	M3	11.95	56.3	81648.9	975704.355	4596833.07
C165	M4	17.1	56.3	82064.6	1403304.66	4620236.98
C166	M6	22.6	56.3	82422.6	1862750.76	4640392.38
C167	M8	28.1	56.3	81248.8	2283091.28	4574307.44
C168	M9	33.25	56.3	79059.5	2628728.375	4451049.85
C169	M10	38.2	56.3	75672	2890670.4	4260333.6
C170	M11	41.45	56.3	71694	2971716.3	4036372.2
C171	M12	46.45	56.3	6612.8	307164.56	372300.64
C172	M13	51.45	56.3	4606.4	236999.28	259340.32
C173	M14	56.45	56.3	4231.7	238879.465	238244.71
C174	M15	61.45	56.3	4268	262268.6	240288.4
C175	M16	66.45	56.3	4232.6	281256.27	238295.38
C176	M17	71.45	56.3	4666.5	333421.425	262723.95
C177	M18	76.45	56.3	6946.7	531075.215	391099.21
C178	M19	81.45	56.3	78417.4	6387097.23	4414899.62
C179	M20	87	56.3	83228.2	7240853.4	4685747.66
C180	M21	91.95	56.3	82793.7	7612880.715	4661285.31
C181	M22	97.1	56.3	82879.1	8047560.61	4666093.33
C182	M24	102.6	56.3	82203.5	8434079.1	4628057.05
C183	M26	108.1	56.3	80092.1	8657956.01	4509185.23
C184	M27	113.25	56.3	77583.4	8786320.05	4367945.42
C185	M28	118.2	56.3	74076	8755783.2	4170478.8
C186	M29	121.45	56.3	72004.4	8744934.38	4053847.72
C187	N1	1.45	61.45	72873	105665.85	4478045.85
C188	N2	7	61.45	74615.4	522307.8	4585116.33
C189	N3	11.95	61.45	74665.6	892253.92	4588201.12
C190	N4	17.1	61.45	75023.1	1282895.01	4610169.495
C191	N6	22.6	61.45	75185.5	1699192.3	4620148.975
C192	N8	28.1	61.45	74279.7	2087259.57	4564487.565
C193	N9	33.25	61.45	72370.6	2406322.45	4447173.37
C194	N10	38.2	61.45	69228.9	2644543.98	4254115.905
C195	N11	41.45	61.45	65655.8	2721432.91	4034548.91
C196	N12	46.45	61.45	4509.5	209466.275	277108.775
C197	N13	51.45	61.45	2843.4	146292.93	174726.93
C198	N14	56.45	61.45	2717.1	153380.295	166965.795
C199	N15	61.45	61.45	2743.1	168563.495	168563.495
C200	N16	66.45	61.45	2716.5	180511.425	166928.925
C201	N17	71.45	61.45	2869.5	205025.775	176330.775
C202	N18	76.45	61.45	4816	368183.2	295943.2
C203	N19	81.45	61.45	72070.8	5870166.66	4428750.66

C204	N20	87	61.45	75629.4	6579757.8	4647426.63
C205	N21	91.95	61.45	75981.3	6986480.535	4669050.885
C206	N22	97.1	61.45	75961.7	7375881.07	4667846.465
C207	N24	102.6	61.45	75137.1	7709066.46	4617174.795
C208	N26	108.1	61.45	73277.4	7921286.94	4502896.23
C209	N27	113.25	61.45	71006.8	8041520.1	4363367.86
C210	N28	118.2	61.45	67965.4	8033510.28	4176473.83
C211	N29	121.45	61.45	67158.5	8156399.825	4126889.825
Σ				14031955	868660226.3	453722548

Location	Strip 1 loads "A19-N19" (KN)	Factored Load of strip 1
A19	90800	72441.07536
B19	99394	79297.44762
C19	96588.3	77059.03435
D19	95310.8	76039.8331
F19	95179	75934.68185
G19	95991.1	76582.5827
H19	92038.9	73429.48118
I19	88701.5	70766.87275
K19	85278	68035.57296
L19	84694.9	67570.37041
M19	78417.4	62562.12316
N19	72070.8	57498.74729
Σ	1074464.7	

Location	Strip 2 loads "M1-M29" (KN)	Factored Load of strip 2
M1	78080	72059.64941
M2	82261.9	75919.10443
M3	81648.9	75353.36973
M4	82064.6	75737.01722
M6	82422.6	76067.41367
M8	81248.8	74984.1194
M9	79059.5	72963.62516
M10	75672	69837.31801
M11	71694	66166.0413
M12	6612.8	6102.920717
M13	4606.4	4251.223989
M14	4231.7	3905.415195
M15	4268	3938.916287
M16	4232.6	3906.2458
M17	4666.5	4306.689984
M18	6946.7	6411.075391
M19	78417.4	72371.03421
M20	83228.2	76810.89796
M21	82793.7	76409.90004
M22	82879.1	76488.71528
M24	82203.5	75865.20735
M26	80092.1	73916.60664
M27	77583.4	71601.3397
M28	74076	68364.37743
M29	72004.4	66452.50794
Σ	1462994.8	

APPENDIX 10

Appendix 10.1 Pile Calculation

Length of pile : 3m												
Pile Diameter (m)	L (m)	B (m)	Spacing : 10D (m)	No. of Piles (X-axis)	Cosidered No. of piles (X-axis)	No. of Piles (Y-axis)	Cosidered No. of piles (Y-axis)	Tolat No. of piles	Qs (KPa)	Qs (KN)	Qi,tot al (KPa)	Qall,total
0.3	123.2	63.2	3	41.06667	41	21.06667	21	861	169.56	11.98	36622	37117.42
0.4	123.2	63.2	4	30.8	30	15.8	15	450	226.08	28.4	25558	26053.63
0.5	123.2	63.2	5	24.64	24	12.64	12	288	282.6	55.46	20471	20966.83
0.6	123.2	63.2	6	20.53333	20	10.53333	10	200	339.12	95.84	17080	17575.63
Length of pile : 4m												
Pile Diameter (m)	L (m)	B (m)	Spacing : 10D (m)	No. of Piles (X-axis)	Cosidered No. of piles (X-axis)	No. of Piles (Y-axis)	Cosidered No. of piles (Y-axis)	Tolat No. of piles	Qs (KPa)	Qs (KN)	Qi,tot al (KPa)	Qall,total
0.3	123.2	63.2	3	41.06667	41	21.06667	21	861	226.08	15.97	48788	49283.35
0.4	123.2	63.2	4	30.8	30	15.8	15	450	301.44	37.86	34036	34531.63
0.5	123.2	63.2	5	24.64	24	12.64	12	288	376.8	73.95	27254	27749.23
0.6	123.2	63.2	6	20.53333	20	10.53333	10	200	452.16	127.8	22732	23227.63
Length of pile : 5m												
Pile Diameter (m)	L (m)	B (m)	Spacing : 10D (m)	No. of Piles (X-axis)	Cosidered No. of piles (X-axis)	No. of Piles (Y-axis)	Cosidered No. of piles (Y-axis)	Tolat No. of piles	Qs (KPa)	Qs (KN)	Qi,tot al (KPa)	Qall,total
0.3	123.2	63.2	3	41.06667	41	21.06667	21	861	282.6	19.97	60954	61449.28
0.4	123.2	63.2	4	30.8	30	15.8	15	450	376.8	47.33	42514	43009.63
0.5	123.2	63.2	5	24.64	24	12.64	12	288	471	92.43	34036	34531.63
0.6	123.2	63.2	6	20.53333	20	10.53333	10	200	565.2	159.7	28384	28879.63

Length of pile : 6m												
Pile Diameter (m)	L (m)	B (m)	Spacing : 10D (m)	No. of Piles (X-axis)	Cosidered No. of piles (X-axis)	No. of Piles (Y-axis)	Cosidered No. of piles (Y-axis)	Tolat No. of piles	Qs (KPa)	Qs (KN)	Qi,total (KPa)	Qall,total
0.3	123.2	63.2	3	41.0667	41	21.067	21	861	339.12	23.959	73120	73615.21
0.4	123.2	63.2	4	30.8	30	15.8	15	450	452.16	56.791	50992	51487.63
0.5	123.2	63.2	5	24.64	24	12.64	12	288	565.2	110.92	40818	41314.03
0.6	123.2	63.2	6	20.5333	20	10.533	10	200	678.24	191.67	34036	34531.63

APPENDIX 11

Appendix 11.1 Beam Reinforcement (RCC Model)

1250 mm X 1000 mm														
Loads on floor	Span (mm)	area req. TOP (mm ²)	Bar Dia. Used TOP (mm)	area of the bar (mm ²)	Number of bars	provide As TOP (mm ²)	Spacing C/C (mm)	area req. BOTTOM (mm ²)	Bar Dia. Used BOTTOM (mm)	area of the bar (mm ²)	Number of bars	provide As BOTTOM (mm ²)	Spacing C/C (mm)	Shear (KN)
DL+ LL=10 KN/m ²	5500	14308	32	803.84	18	14469.12	111	6994	24	452.16	16	7234.56	128	2742.2
	9900	11993	32	803.84	16	12861.44	126.857	5952	22	379.94	16	6079.04	128.286	1518.7
	10650	0	18	254.34	10	2543.4	225.5	9704	28	615.44	16	9847.04	127.429	1392.4
	15650	9845	28	615.44	16	9847.04	127.429	5952	22	379.94	16	6079.04	128.286	1185
DL+ LL=7.5 KN/m ²	5500	5952	22	379.94	16	6079.04	128.286	3265	20	314	12	3768	180	1051.2
	8750	0	18	254.34	10	2543.4	225.5	5870	22	379.94	16	6079.04	128.286	563.75
	11950	7178	25	490.625	16	7850	127.857	4755	22	379.94	14	5319.16	149.667	1125.5
	13350	4352	22	379.94	14	5319.16	149.667	2184	18	254.34	10	2543.4	225.5	458.74
DL+ LL=5 KN/m ²	5500	16515	36	1017.36	18	18312.48	110.5	8096	28	615.44	14	8616.16	148.667	3106.6
	10100	372	18	254.34	10	2543.4	225.5	18762	38	1133.54	18	20403.7	110.25	1780.2
	11950	10671	28	615.44	18	11077.92	111.5	5952	22	379.94	16	6079.04	128.286	1285.6
	15650	9823	28	615.44	16	9847.04	127.429	5952	22	379.94	16	6079.04	128.286	1250.8
1400 mm X 1200 mm														
DL+ LL=10 KN/m ²	10100	18495	36	1017.36	22	22381.92	108.4	11095	28	615.44	20	12308.8	121.333	1739.5
	15000	10337	28	615.44	18	11077.92	136.5	5113	20	314	18	5652	137.5	799.94
	20000	8022	28	615.44	14	8616.16	182	4824	20	314	18	5652	137.5	1052.2
	21550	12823	32	803.84	16	12861.44	155.429	8022	28	615.44	14	8616.16	182	1596.5
DL+ LL=7.5 KN/m ²	10100	13882	32	803.84	18	14469.12	136	8022	28	615.44	14	8616.16	182	1552.5
	10100	966	18	254.34	10	2543.4	275.5	7124	26	530.66	14	7429.24	182.333	615.04
	21300	9731	28	615.44	16	9847.04	156	6461	26	530.66	14	7429.24	182.333	1415.2
	21550	8760	28	615.44	16	9847.04	156	5813	20	314	20	6280	122.222	1231.4
DL+ LL=5 KN/m ²	10100	0	18	254.34	10	2543.4	275.5	23810	40	1256	20	25120	120	566.35
	10100	15820	32	803.84	20	16076.8	120.889	8022	24	452.16	18	8138.88	137	2002.1
	21300	21460	38	1133.54	20	22670.8	120.222	10535	28	615.44	18	11077.9	136.5	2843.7
	21550	8022	28	615.44	16	9847.04	156	5118	24	452.16	12	5425.92	219.2	623.25

APPENDIX 12

Appendix 12.1 Beam Reinforcement (CLT Model)

1250 mm X 1000 mm														
Loads on floor	Span (mm)	area req. TOP (mm ²)	Bar Dia. Used TOP (mm)	area of the bar (mm ²)	Number of bars	provide As TOP (mm ²)	Spacing C/C (mm)	area req. BOTTOM (mm ²)	Bar Dia. Used BOTTOM (mm)	area of the bar (mm ²)	Number of bars	provide As BOTTOM (mm ²)	Spacing C/C (mm)	Shear (KN)
DL+ LL=10 KN/m ²	5500	14855	36	1017.36	16	16277.8	126.29	7416	26	530.7	14	7429.24	149	2780.06
	9900	12013	32	803.84	16	12861.4	126.86	5952	22	379.9	16	6079.04	128.2857	1527.23
	10650	0	18	254.34	10	2543.4	225.5	10258	28	615.4	18	11077.92	111.5	1449
	15650	10342	28	615.44	18	11077.9	111.5	5952	22	379.9	16	6079.04	128.2857	1234.77
DL+ LL=7.5 KN/m ²	5500	5952	22	379.94	16	6079.04	128.29	3057	20	314	12	3768	180	933.16
	8750	0	18	254.34	10	2543.4	225.5	4818	20	314	16	5024	128.5714	450.46
	11950	6140	24	452.16	14	6330.24	149.33	4074	20	314	14	4396	150	997.97
	13350	3704	20	314	12	3768	180	1861	16	201	10	2009.6	226	373.76
DL+ LL=5 KN/m ²	5500	16065	36	1017.36	16	16277.8	126.29	7832	26	530.7	16	8490.56	127.7143	2975.36
	10100	244	14	153.86	10	1538.6	226.5	19385	40	1256	16	20096	125.7143	1872.24
	11950	10364	28	615.44	18	11077.9	111.5	5952	22	379.9	16	6079.04	128.2857	1235.89
	15650	9657	28	615.44	16	9847.04	127.43	5952	22	379.9	16	6079.04	128.2857	1189.33
1350 mm X 1100 mm														
DL+ LL=10 KN/m ²	10100	2944	18	254.34	12	3052.08	200.4	4971	20	314	16	5024	142.8571	229.8
	15000	2401	18	254.34	10	2543.4	250.5	10772	28	615.4	18	11077.92	124	641.34
	20000	6982	26	530.66	14	7429.24	165.67	3492	20	314	16	5024	142.8571	777.93
	21550	11059	32	803.84	16	12861.4	141.14	7078	26	530.7	14	7429.24	165.6667	1382.9
DL+ LL=7.5 KN/m ²	10100	9503	28	615.44	16	9847.04	141.71	6288	26	530.7	12	6367.92	198.8	1044.27
	10100	862	14	153.86	10	1538.6	251.5	5889	24	452.2	14	6330.24	166	452.49
	21300	7078	26	530.66	14	7429.24	165.67	4505	22	379.9	12	4559.28	199.6	987.7
	21550	7363	26	530.66	14	7429.24	165.67	4885	22	379.9	14	5319.16	166.3333	1049.86
DL+ LL=5 KN/m ²	10100	0	18	254.34	10	2543.4	250.5	11701	30	706.5	18	12717	123.75	473.44
	10100	19136	36	1017.36	20	20347.2	109.33	9213	24	452.2	20	9043.2	110.6667	1993.4
	21300	19111	36	1017.36	20	20347.2	109.33	9321	26	530.7	18	9551.88	124.25	2480.88
	21550	7078	24	452.16	16	7234.56	142.29	3910	20	314	14	4396	166.6667	442.91

APPENDIX 13

Appendix 13.1 Column Reinforcement (RCC Model)

Cross section	Width (mm)	Length (mm)	Gross area	area of steel required (mm ²)	number of bar in width	number of bar in length	total number of bars	area of one bar required	Bar Dia. Used (mm)	area of the bar (mm ²)	area of steel provide	Spacing C/C (mm)	ρ	Shear (KN)
3.2X3.2	3200	3200	10240000	102400	31	29	120	853.333	36	1017.4	122083	102.133	0.0119	470.4
3.1X3.1	3100	3100	9610000	96100	30	28	116	828.448	36	1017.4	118014	102.207	0.0123	432.7
2.8X2.8	2800	2800	7840000	78400	27	25	104	753.846	32	803.84	83599.4	102.615	0.0107	1093.6
2.6X2.6	2600	2600	6760000	67600	25	23	96	704.167	30	706.5	67824	102.917	0.01	1341.5
2.4X2.4	2400	2400	5760000	57600	23	21	88	654.545	30	706.5	62172	103.182	0.0108	1426.9
2.1X2.1	2100	2100	4410000	44100	20	18	76	580.263	28	615.44	46773.4	103.789	0.0106	1363.6
1.8X1.8	1800	1800	3240000	32400	17	15	64	506.25	28	615.44	39388.2	104.5	0.0122	1221.6
1.3X1.3	1300	1300	1690000	16900	12	10	44	384.091	25	490.63	21587.5	106.818	0.0128	820.4
1.2X1.2	1200	1200	1440000	14400	11	9	40	360	22	379.94	15197.6	107.8	0.0106	55.5
0.7X0.7	700	700	490000	4900	6	4	20	245	18	254.34	5086.8	116.4	0.0104	143.2

APPENDIX 14

Appendix 14.1 Column Reinforcement (CLT Model)

Cross section	Width (mm)	Length (mm)	Gross area	area of steel required (mm ²)	number of bar in width	number of bar in length	total number of bars	area of one bar required	Bar Dia. Used (mm)	area of the bar (mm ²)	area of steel provide	Spacing C/C (mm)	ρ	Shear (KN)
2.9X2.9	2900	2900	8410000	84100	28	26	108	778.7037	32	803.84	86814.7	102.519	0.0103	362
2.7X2.7	2700	2700	7290000	72900	26	24	100	729	32	803.84	80384	102.72	0.011	509
2.5X2.5	2500	2500	6250000	62500	24	22	92	679.3478	30	706.5	64998	103.043	0.0104	1011.5
2.3X2.3	2300	2300	5290000	52900	22	20	84	629.7619	30	706.5	59346	103.333	0.0112	1238.5
2.1X2.1	2100	2100	4410000	44100	20	18	76	580.2632	28	615.44	46773.4	103.789	0.0106	1301.6
1.8X1.8	1800	1800	3240000	32400	17	15	64	506.25	28	615.44	39388.2	104.5	0.0122	1203.5
1.5X1.5	1500	1500	2250000	22500	15	13	56	401.7857	25	490.63	27475	98.2143	0.0122	1004.7
1.2X1.2	1200	1200	1440000	14400	9	7	32	450	25	490.63	15700	134.375	0.0109	673
1X1	1000	1000	1000000	10000	7	5	24	416.6667	25	490.63	11775	145.833	0.0118	32.6
0.7X0.7	700	700	490000	4900	6	4	20	245	18	254.34	5086.8	116.4	0.0104	131.3

APPENDIX 15

Appendix 15.1 Cost Estimation (RCC Model)

Concrete Columns					
Floors	Number of floors	Area of column (m ²)	No. of columns in single floor	Height of columns (m)	Volumn of column (m ³)
1-4	4	10.24	148	4	24248.32
1-4 (middle)	4	1.44	57	4	1313.28
5-12	8	9.61	148	4	45512.96
13-20	8	7.84	148	4	37130.24
21-28	8	6.76	148	4	32015.36
29-36	8	5.76	148	4	27279.36
37-44	8	4.41	148	4	20885.76
45-52	8	3.24	148	4	15344.64
53-56	4	1.69	148	4	4001.92
57-60	4	0.49	148	5	1450.4
				Total	140220.2
Shear wall					
Sides	Thickness	Length (m)	Height (m)	Number of stories	Volumn of Shear wall (m ³)
Side 1	0.6	11	4.066667	60	1610.4
Side 2	0.6	11	4.066667	60	1610.4
Side 3	0.6	14.9	4.066667	60	2181.36
Side 4	0.6	14.9	4.066667	60	2181.36
				Total	7583.52
				Total x2	15167.04
Slab					
Number of floors	Slab Thickness (m)	Area of Floor (m ²)	Shear wall area (m ²)	Area of slab (m ²)	Volumn of Slab (m ³)
8	0.18	7200	163.9	7036.1	10131.98
52	0.18	4800	163.9	4636.1	43393.9
				Total	53525.88

Beam						
Number of floors	beam cross section	Beam height (m)	Beam Width (m)	Total Length (m)	Volumn of Beams (m ³)	
4	1.25m X 1m	1.25	1	2275	2843.75	
56	1.25m X 1m	1.25	1	1525.9	1907.375	
4	1.4m X 1.2m	1.4	1.2	398.2	668.976	
56	1.4m X 1.2m	1.4	1.2	247.3	415.464	
				Total	5835.565	
Foundation						
Area (m ²)		Depth (m)	Volume (m ³)			
7718.24		3.4	26242.02			
Piles						
Number of piles	Diameter (m)	Area (m ²)	Length (m)	Volumn of piles (m ³)		
200	0.6	0.282743	3	169.646		
				Total Volumn (m ³)	241160.3	
				Reinforc ement Weight (Tons)	33762.44	Using a ratio of 140 kg/m3

APPENDIX 16

Appendix 16.1 Cost Estimation (CLT Model)

Concrete Columns						
Floors	Number of floors	Area of column (m ²)	No. of columns in single floor	Height of columns (m)	Volumn of column (m ³)	
1-4	4	8.41	148	4	19914.88	
1-4 (middle)	4	1	57	4	912	
5-12	8	7.29	148	4	34525.44	
13-20	8	6.25	148	4	29600	
21-28	8	5.29	148	4	25053.44	
29-36	8	4.41	148	4	20885.76	
37-44	8	3.24	148	4	15344.64	
45-52	8	2.25	148	4	10656	
53-56	4	1.44	148	4	3409.92	
57-60	4	0.49	148	5	1450.4	
				Total	110005.8	
Shear wall						
Sides	Thickness	Length (m)	Height (m)	Number of stories	Volumn of Shear wall (m ³)	
Side 1	0.5	11	4.066667	60	1342	
Side 2	0.5	11	4.066667	60	1342	
Side 3	0.5	14.9	4.066667	60	1817.8	
Side 4	0.5	14.9	4.066667	60	1817.8	
				Total	6319.6	
				Total x2	12639.2	
Slab						
Number of floors	Slab Thickness (m)	Area of Floor (m ²)	Shear wall area (m ²)	Area of slab (m ²)	Volumn of Slab (m ³)	
4	0.18	7200	163.9	7036.1	5065.992	RCC
52	0.36	4800	163.9	4636.1	86787.79	CLT
4	0.36	7200	163.9	7036.1	10131.98	CLT
				RCC Total	5065.992	
				CLT Total	96919.78	

Foundation						
Area (m ²)	Depth (m)	Volumn (m ³)				
7672.61	3.2	24552.35				
Piles						
Number of piles	Diameter (m)	Area (m ²)	Length (m)	Volumn of piles (m ³)		
200	0.6	0.282743	3	169.646		
				Concrete Total Volumn (m ³)	158142.6	
				Reinforc ement Weight (Tons)	22139.97	Using a ratio of 140 kg/m3
				CLT Total Volumn (m ³)	96919.78	

APPENDIX 17

Appendix 18

Appendix 19

Appendix 20