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THE EFFECT OF HEIGHT DIFFERENCE IN SEISMIC REGULATIONS IN THE SEISMIC DESIGN OF REINFORCED CONCRETE TALL BUILDINGS

Abdulkhalek KAIRALLAH

Master’s Thesis

Supervisor

Prof. Dr. Zeki HASGÜR

Istanbul, 2023

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The thesis titled THE EFFECT OF HEIGHT DIFFERENCE IN SEISMIC REGULATIONS IN THE SEISMIC DESIGN OF REINFORCED CONCRETE TALL BUILDINGS prepared by ABDULKHALEK KAIRALLAH and submitted on 00/00/2022 has been **accepted unanimously** for the degree of Master of Science in Civil Engineering.

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Prof. Dr. Zeki HASGÜR

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I hereby declare that this thesis meets all format and submission requirements of a Master's thesis.

Submission date of the thesis to Institute of Graduate Studies: ___/___/___

I hereby declare that all information/data presented in this graduation project has been obtained in full accordance with academic rules and ethical conduct. I also declare all unoriginal materials and conclusions have been cited in the text and all references mentioned in the Reference List have been cited in the text, and vice versa as required by the abovementioned rules and conduct.

Abdulkhalek KAIRALLAH

Signature

ABSTRACT

THE EFFECT OF HEIGHT DIFFERENCE IN SEISMIC REGULATIONS IN THE SEISMIC DESIGN OF REINFORCED CONCRETE TALL BUILDINGS

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This thesis as mentioned in its title, will discuss the different between the high-rise buildings (tall buildings) and the normal height ones. The domain of the comparing will be held among several aspects that affect the design and occupancy of the constructions such as: the response of these two types to the lateral loads, the drafts, the lateral settlements, the amount of loads occurs, the needed reinforced concrete and so on. As a sample, 5 floors building (will be assumed for the normal height prototype while a 20 floors building will be taken as a high rise building example. To have a fair comparison, collect correct data and get true information, the same working plane will be chosen for both cases of study. The huge development of the technologies has contributed to reduce the efforts and help the designers to solve the problems in no times comparing with the past decades. So, the simulation of the samples will be performed on a computer software which is ETABS 18.0.1. I hope it will be a useful and valuable thesis.

Keywords: The Height Effect, Seismic Codes, Tall Buildings, American Concrete Institute, Structural Analysis Computer Program .

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ABBREVIATIONS

ASCE7-10	:	American Society of Civil Engineers Code 7-10
SBC	:	Saudi Building Code
ACI	:	American Concrete Institute
t/m^2	:	Ton Per Square Meter (Aerial Load Unit)
m	:	Linear Meter (Length Unit)
t/m	:	Ton Per Meter (Linear Load Unit)
km/h	:	Kilometer Per Hour (Speed Unit)
mph	:	Mile Per Hour (Speed Unit)
K_{zt}	:	Topographic Factor
K_d	:	Wind Directionality Factor
G	:	Gust Factor
S_1	:	The Mapped Maximum Considered Earthquake Spectral Response Acceleration At A Period Of 1-Sec
S_s	:	The Mapped Maximum Considered Earthquake Spectral Response Acceleration At Short Periods
R^a	:	Response Modification Coefficient
Ω_0^f	:	System Over Strength Factor
C_d^b	:	Deflection Amplification Factor
I	:	The Occupancy Importance Factor
MPa	:	Mega Pascal (Pressure Unit)
Centimeter	:	Mega Pascal (Pressure Unit)
AutoCAD	:	Sketching Computer Program
ETABS	:	Structural Analysis Computer Program
DXF	:	Type Of Computer File (Etabs Can Read)
Γ_1	:	Participation Factor Of The Fundamental Mode Of Vibration Of The Structure In The Direction Of Interest
C_{S1}	:	Seismic Response Coefficient Of The Fundamental Mode Of Vibration Of The Structure In The Direction Of Interest

1. INTRODUCTION

1.1 STRUCTURES IMPROVEMENT OVERVIEW

The development of the construction is parallel with the development of the humanity. While the human being improved, their needs to shelters and storehouses also improved. Began with the mud houses, to the modern landmarks building civil engineering structures had been throw huge mega scale improvement. Last century, the needed to high rise building and sky strippers increased. Before the invention of the computers, engineers used to design manually with a theoretical procedures and it took a large amount of time and efforts. These efforts are increased with increases of the buildings heights and areas. After the fast development of the computer technologies, the analysis and design of these building became much easier. The new software let the designers imagine the behavior of the building during the earthquake. So, the high rise building spreads all over the world.

Other than the land marks should we go vertically or horizontally in the modern cities' construction?

1.2 VERTICALLY OR HORIZONTALLY?

Modern cities due to lack of land downtown, they expand vertically. The investors tends to have attractive building to attract the tourist. Also, the overseas companies reflects there advancement on their buildings and their headquarters. But this orientation as it has advantages it also has disadvantages. Some of these disadvantages is making a huge pressure on the infrastructures that designed mainly on a normal scale structures. Also, it caused the crowd increment and exacerbate the crowded and the pollutions in the modern world. Thus, we don't yet consider that the mistakes in designing or execution in these type of constructions is fatal. Also, in some societies the normal building scale are more common. The crowded are lesser and the infrastructures are Sufficient without problems. You didn't see much more landmarks but you see a normal not crowded societies.

1.3 CIVIL ENGINEER PERSPECTIVE

In high rise building the designer need to make the construction safe against the gravity loads and the lateral loads. The governor loads is the lateral loads. As the building not going more than 100m height, usually the seismic load is the worst case of loading epically in the regions that continuously exposed to earthquakes. Deliver the building safely against lateral force means making the building more stable and increase its rigidity. This means enlarge the concrete sections (if it is a concrete building which is our case of study), pour more concrete add more reinforcement and it needs more expertise to perform the design than normal ones. Also, on site you need trained labors, experienced engineers, skilled foreman and huge instruments such as mobile cranes to complete the work. The large experienced consultancy company and high levelled contractor also will be needed.

On the other hand, the normal height building doesn't require large experience. The governor force that exposed on it is the gravity load. The concrete and reinforcement needed to safely deliver the construction are common and doesn't need to be increased comparing with designing against vertical and lateral force at the same time. Mid-level engineering company can design this type of building correctly. Also, normal labors, Forman and ordinary tools could easily use. Mid-level contractor and consultant companies can safely execute and deliver the project without need much more potential.

So, if we have two alternative one high rise and couple of normal building from structural designing engineering aspect what is the different between these alternatives?

This thesis will discuss this question and give an answer to it.

2. GENERAL INFORMATION

2.1 COMPUTER PROGRAMME USED

The computer programme used in this dissertation is ETABS 2018.

2.2 CODES USED

a. SBC code.

b. ASCE7-10 code.

c. ACI 318-14 code.

2.3 LOCATION CHARECTIRISTICS, TYPE OF BUILDING AND NUMBER OF FLOORS

The building samples is residential building one is 5 floors (normal height) which will be referred as (BL5) construction and the other one is 20 floors (high rise) construction which will be referred as (BL20). The area of the construction is 400 m^2. The height of each floor is 3 m. So, the high-rise building is 60 m and the normal one is 15m height. The building is located in category III as it is a more than 300 people building which mentioned in section (a) as seen in Table 2.1 (SBC301 Table 1.6-1 P1/9). The location is assumed to be in Saudi Arabia – AlMadinah Al Monawarah. The altitude of the location is around 636 m above sea level. The soil at the site is very dense soil and soft rock as seen in Table 2.2 (class C as per SBC301 Table 14.1.1 P14/2).

Table 2.1: Classification of Buildings and Other Structures for Flood, Wind and Earthquake Loads.

Nature of Occupancy	Category
1) Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: a) Agricultural facilities b) Certain temporary facilities c) Minor storage facilities	I
All buildings and other structures except those listed in Categories I, III, and IV	II
1) Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: a) Buildings and other structures where more than 300 people congregate in one area b) Buildings and other structures with day care facilities with capacity greater than 150 c) Buildings and other structures with elementary school or secondary school facilities with capacity greater than 250 d) Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities e) Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities f) Jails and detention facilities g) Power generating stations and other public utility facilities not included in Category IV 2) Buildings and other structures not included in Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of hazardous materials to be dangerous to the public if released. 3) Buildings and other structures containing hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.6.2 that a release of the hazardous material does not pose a threat to the public.	III
1) Buildings and other structures designated as essential facilities including, but not limited to: a) Hospitals and other health care facilities having surgery or emergency treatment facilities b) Fire, rescue, ambulance, and police stations and emergency vehicle garages c) Designated earthquake, hurricane, or other emergency shelters d) Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response e) Power generating stations and other public utility facilities required in an emergency f) Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Category IV structures during an emergency g) Aviation control towers, air traffic control centers, and emergency aircraft hangars h) Water storage facilities and pump structures required to maintain water pressure for fire suppression i) Buildings and other structures having critical national defense functions 2) Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing extremely hazardous materials where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. 3) Buildings and other structures containing extremely hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.6.2 that a release of the extremely hazardous material does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

Table 2.2: Site Classification.

<table>
<tr><th>Site Class</th><th>$\overline{\upsilon}_s$</th><th>$\overline{N}$ or $\overline{N}_{ch}$</th><th>$\overline{s}_u$</th></tr>
<tr><td>A
Hard rock</td><td>> 1500 m/s</td><td>not applicable</td><td>not applicable</td></tr>
<tr><td>B
Rock</td><td>760 to 1500 m/s</td><td>not applicable</td><td>not applicable</td></tr>
<tr><td>C
Very dense soil and soft rock</td><td>370 to 760 m/s</td><td>> 50</td><td>> 100 kPa</td></tr>
<tr><td>D
Stiff soil</td><td>180 to 370 m/s</td><td>15 to 50</td><td>50 to 100 kPa</td></tr>
<tr><td rowspan="2">E
Soil</td><td><180 m/s</td><td>< 15</td><td><50 kPa</td></tr>
<tr><td colspan="3">Any profile with more than 3 m of soil having the following characteristics:
- Plasticity index PI > 20,
- Moisture content w $\geq$ 40%, and
- Undrained shear strength $\overline{s}_u$ < 25 kPa</td></tr>
<tr><td colspan="2">F Soils requiring site-specific evaluation</td><td colspan="2">1. Soils vulnerable to potential failure or collapse
2. Peats and/or highly organic clays
3. Very high plasticity clays
4. Very thick soft/medium clays</td></tr>
</table>

2.4 ASSUMED LOADING PATTERNS FOR THE PROTOTYPE

2.4.1 Dead Load

The dead load comes from the covering material on the flooring system, the load of the partitions of the building and the own weight of the structural elements. The covering material load is assumed to equal 2 kN/m^2 .The interior partition loads is distributed on the whole floor and taken as 2 kN/m^2. The exterior partitions is drawn on the outer beams as linear load equal to 10 kN/m. The own weight of the slab equal to 6.25 kN/m^2 (25 *0.25 which is 0.25 m is the thickness of the slab and 25 kN/m^3 is the density of the reinforced concrete).

2.4.2 Live Load

As it's a residential building the assumed live load as 2 kN/m^2 as shown in Table 3.2 (SBC301 code Table 4.1 P4/6).

Table 2.3: Minimum Uniformly Distributed Live Loads, Lo, and Minimum Concentrated Live Loads.

Occupancy or Use	Uniform kN/m²	Conc. kN
Apartments (see residential)		
Access floor systems Office use Computer use	 2.5 5	 9 9
Armories and drill rooms	7.5	
Assembly areas and theaters • Fixed seats (fastened to floor) • Lobbies • Movable seats • Platforms (assembly) • Stage floors	 3 5 5 5 7.5	
Balconies (exterior) On one- and two-family residences only, and not exceeding 10 m^2	5 3	
Bowling alleys, poolrooms, and similar recreational areas	4	
Catwalks for maintenance access	2	1.5
Corridors First floor Other floors, same as occupancy served except as indicated	 5	
Mosques	5	
Decks (patio and roof) Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	5	
Dwellings (see residential)		
Elevator machine room grating (on area of 2500 mm^2)		1.5
Finish light floor plate construction (on area of 650 mm^2)		1
Fire escapes	5	
Fixed ladders	See Section 4.4	
Garages (passenger vehicles only) Trucks and buses	2 Note (2)	Note (1) Note (2)
Grandstands (see stadium and arena bleachers)		
Gymnasiums, main floors, and balconies	5 Note (4)	
Handrails, guardrails, and grab bars	See Section 4.4	
Hospitals • Operating rooms, laboratories • Private rooms • Wards • Corridors above first floor	 3 2 2 4	 4.5 4.5 4.5 4.5
Hotels (see residential)		
Libraries • Reading rooms • Stack rooms • Corridors above first floor	 3 7.5 Note (3) 4	 4.5 4.5 4.5
Manufacturing • Light • Heavy	 6 12	 9 13.5
Marquees and canopies	4	
Office buildings • File and computer rooms shall be designed for heavier loads based on anticipated occupancy: • Lobbies and first floor corridors • Offices • Corridors above first floor	 5 2.5 4.0	 9 9 9
Penal institutions Cell blocks Corridors	 2 5	

2.4.3 WIND LOAD

The wind speed in AlMadinah Al Monawarah is 155 km/h (97mph) as shown in the Figure 2.1 (SBC301 Figure 6.4-1 P6/10). K_d is taken 0.85 as described in Figure 2.2 (SBC301 code Table 6.4-1 P6/7). Importance factor equal to 1.15 (SBC301 code Table 6.5-1 P6/9). Exposure category taken as type B (SBC301 code P6/7). K_{zt} almost equal 1 ($K_{zt} = (1 + K_1K_2K_3)^2$ SBC301 P6/8). G is equal to 0.85 (SBC301 P7/4). All of these factors are applied in the ETABS on the computer and the wind load will be added and analyzed depend on the diaphragms.

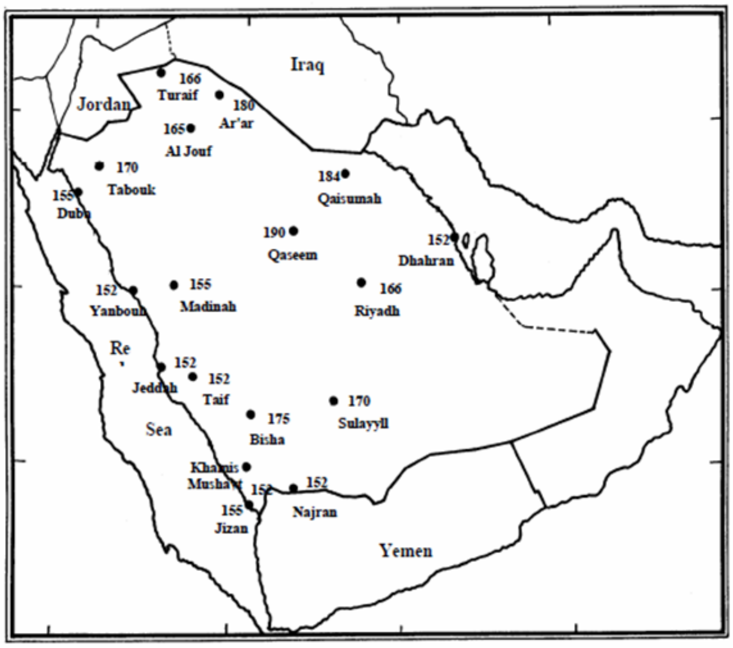

Figure 2.1: Basic 3-Second Gust Wind Speed in Km/H for Selected Cities of Saudi Arabia. Adopted from Saudi Aramco Data Saes A-112.

Table 2.4: Wind Directionality Factor, Kd.

Structure Type	Directionality Factor K_d*
Buildings Main Wind Force Resisting System Components and Cladding	 0.85 0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures Square Hexagonal Round	 0.90 0.95 0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers Triangular, square, rectangular All other cross sections	 0.85 0.95

* Directionality Factor K_d has been calibrated with combinations of loads specified in Chapter 2. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

Table 2.5: Importance Factor, I.

Category[1]	Regions with V = 136-160 km/h	Regions with V > 160 km/h
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

2.4.4 Seismic Load

As the location assumed to be in AlMadinah Al Monawarah, it's located on region 3 as seen in Figure 2.2 (SBC301 Figure 9.4.1 (a) P9/16). S_1 = 25.4 (% of g) as appeared in Figure 2.3 (SBC301 Figure 9.4.1 (e) P9/20). S_s = 7.3 (% of g) as appeared in Figure 2.4 (SBC301 Figure

9.4.1 (m) P9/28). $R^a = 5.5$, $\Omega_0{}^f = 2.5$, $C_{db} = 6$ as mentioned in Table 2.6 (SBC301 Table 10.2 P10/4) because it is Ordinary reinforced concrete shear walls construction. Values of approximate period parameters C_t and x = 0.044 (0.016 ft) and 0.9 as seen in Table 2.7 (SBC301 Table 10.9.3.2 P10/16) as it is a Moment resisting frame systems of reinforced concrete in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frame from deflecting when subjected to seismic forces. I = 1.25 as the occupancy category is III as investigated previously (SBC301 Table 9.5 P9/14). The long period (transition period) is assumed to be equal to 8 this amount is practical and taken depend on constructions that designed in similar areas.

SEISMIC DESIGN CRITERIA

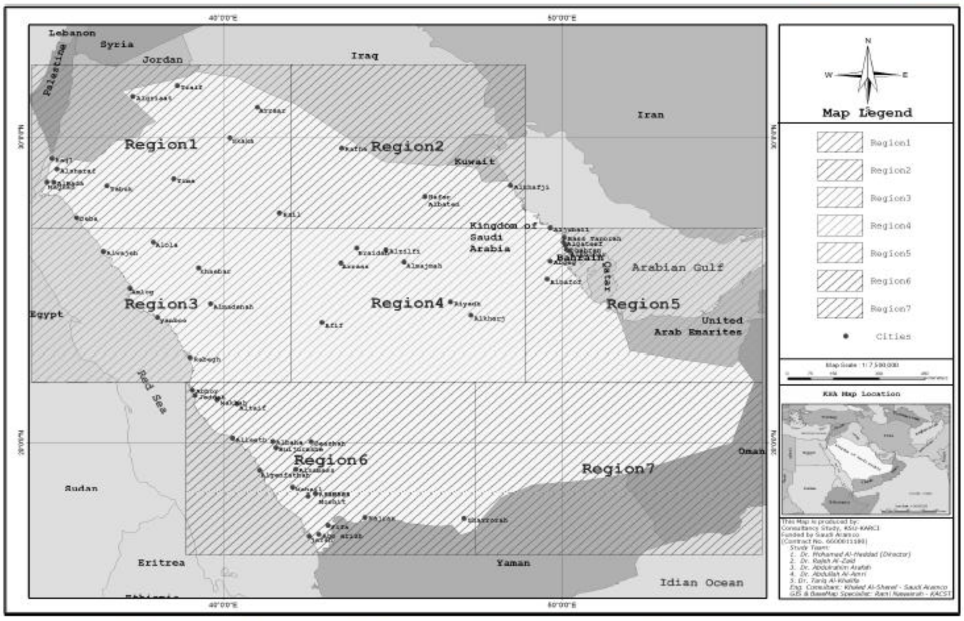

Figure 9.4.1(a): Regions for Determination of the Maximum Considered Earthquake Ground Motion in the Kingdom of Saudi Arabia.

SBC 301 2007 9/16

Figure 2.2: Regions for Determination of the Maximum Considered Earthquake Ground Motion in the Kingdom of Saudi Arabia.

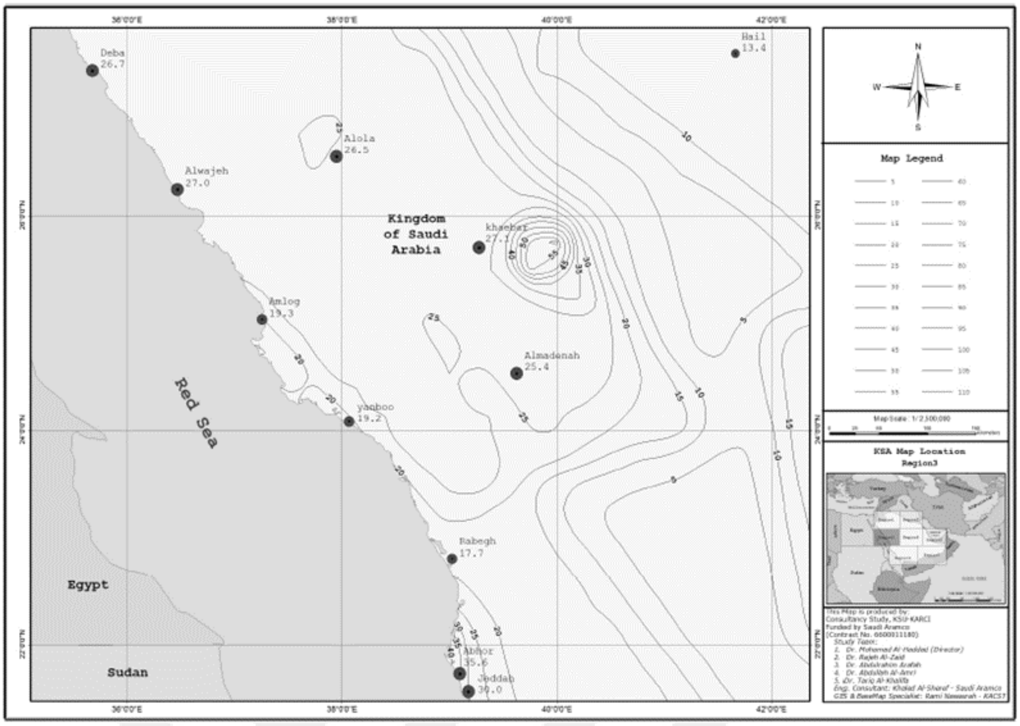

Figure 2.3: Maximum Considered Earthquake Ground Motion for the Kingdom of 0.2 Sec spectral Response Acceleration (Ss in %g) (5 Percent of Critical Damping), Site Class B(Region 3).

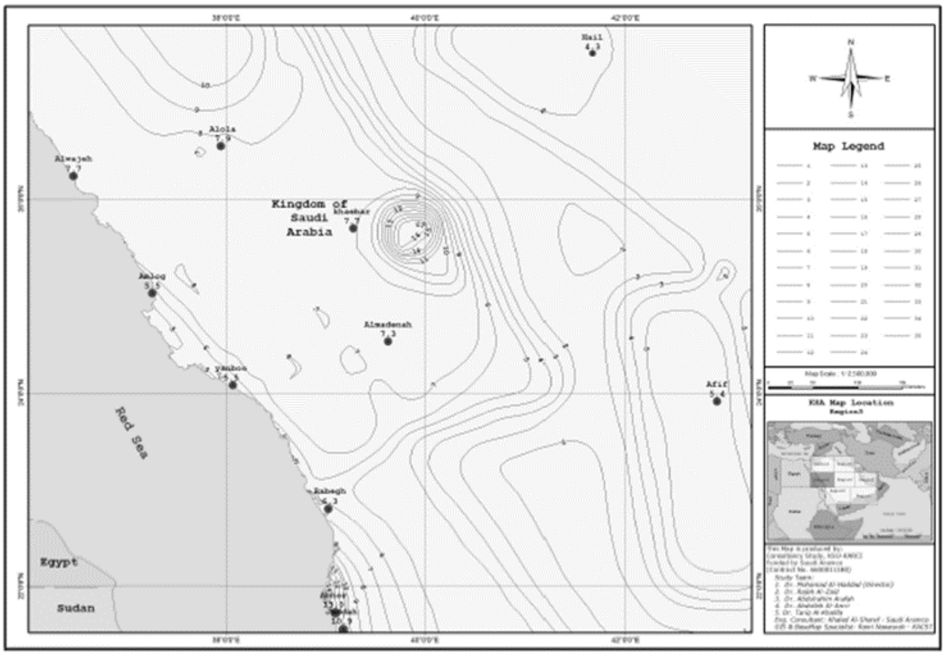

Figure 2.4: Maximum Considered Earthquake Ground Motion for the Kingdom of 1 SEC spectral Response Acceleration (S1 in %g) (5 Percent of Critical Damping), Site Class B. (Region 3).

Table 2.6: Design Coefficients and Factors For Basic Seismic Force-Resisting Systems.

Basic Seismic Force-Resisting System	Response Modification Coefficient, R[a]	System Over-strength Factor, Ω_0 [f]	Deflection Amplification Factor, C_d[b]	Structural System Limitations and Building Height (m) Limitations[c]		
				Seismic Design Category		
				A&B	C	D[d]
Dual Systems with Special Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces						
Steel eccentrically braced frames, moment resisting connections, at columns away from links	7	2.5	4	NL	NL	NL
Steel eccentrically braced frames, non-moment resisting connections, at columns away from links	6	2.5	4	NL	NL	NL
Special steel concentrically braced frames	7	2.5	6.5	NL	NL	NL
Special reinforced concrete shear walls	6.5	2.5	6.5	NL	NL	NL
Ordinary reinforced concrete shear walls	5.5	2.5	6	NL	NL	NP
Composite eccentrically braced frames	6.5	2.5	4	NL	NL	NL
Composite concentrically braced frames	5	2.5	5	NL	NL	NL
Composite steel plate shear walls	6.5	2.5	6.5	NL	NL	NL
Special composite reinforced concrete shear walls with steel elements	6.5	2.5	6.5	NL	NL	NL
Ordinary composite reinforced concrete shear walls with steel elements	5.5	2.5	6	NL	NL	NP
Special reinforced masonry shear walls	5.5	3	6.5	NL	NL	NL
Intermediate reinforced masonry shear walls	4.5	2.5	5	NL	NL	NL
Ordinary steel concentrically braced frames	5	2.5	5	NL	NL	NL
Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces						
Special steel concentrically braced frames[e]	4	2.5	4.5	NL	NL	10
Special reinforced concrete shear walls	4.5	2.5	5	NL	NL	50
Ordinary reinforced masonry shear walls	2.5	3	2.5	NL	50	NP
Intermediate reinforced masonry shear walls	4	3	4.5	NL	NL	NP
Composite concentrically braced frames	4	2.5	4.5	NL	NL	50
Ordinary composite braced frames	3.5	2.5	3	NL	NL	NP
Ordinary composite reinforced concrete shear walls with steel elements	4	3	4.5	NL	NL	NP
Ordinary steel concentrically braced frames	4	2.5	4.5	NL	NL	50
Ordinary reinforced concrete shear walls	4.5	2.5	4.5	NL	NL	NP

Table 2.7: Values of Approximate Period Parameters Ct AND x.

Structure Type	C_t	x
Moment resisting frame systems of steel in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces	0.068	0.8
Moment resisting frame systems of reinforced concrete in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frame from deflecting when subjected to seismic forces	0.044	0.9
Eccentrically braced steel frames	0.07	0.75
All other structural systems	0.055	0.75

Table 2.8: Values of Scismic Parameter For Madinah In Saudi Arabia.

S_1	S_s	R^a	Ω_0^f	C_{db}	C_t	x	I	Long Period (sec) (1st assumption)
25.4	7.3	5.5	2.5	6	0.044	0.9	1.25	8

2.5 PROPERTIES OF THE STRUCTURAL ELEMENT

2.5.1 Type of the Skeleton Used

The type of slab used in the structures is flat slab with perimeter beams. The system that resists the lateral forces is the shear wall system. The compressive strength of the concrete is 35 MPa. (This compressive strength is changed during the checking of the high-rise building)

2.5.2 Properties of The Slabs

The slabs of the two buildings and its’ floors are typical. The thickness of the slab is 250 mm. The interior columns has drop panels. The thickness of the drop panel is 400 mm.

2.5.3 Properties of The Beams

The beams of the project have the same section. The section is 300 mm width and 600 mm height.

2.5.4 Properties and Location of The Columns

The columns' section is assumed in the beginning of the analysis part as 600 mm height and 300 mm width. This section remains the same in the lower building and adjusted in the high-rise building as needed. The columns are around 5 m apart (center to center) in the two directions.

Table 2.8: First Assumption of The Properties of The Members.

First Assumption	Dimensions (m)	Percent of reinforcement	The compressive strength of the concrete (MPa)
Slabs	0.25 (thickness)	0.0025	35
Drop Panels	0.4 (thickness)	0.0025	35
Beams	0.3x0.6	0.0033	35
Columns	0.3x0.6	0.01	35
Shear Wall (Core)	0.2 (thickness)	0.01	35

2.5.5 Properties and Location of The Shear Walls

The wall is designed to be as core wall in the centre of the building. The thickness of the wall is 200 mm. also, this assumption is changed in the high-rise building due to huge lateral forces.

2.6 LAYOUT OF THE REPEATED FLOOR

As mentioned before, the floor is 400 m^2, the width is 20m and the length is 20m (square shape). The distance between the columns is about 5m in the both directions. The core wall is located in the centre of the construction as it appears in the Figure 2.5.

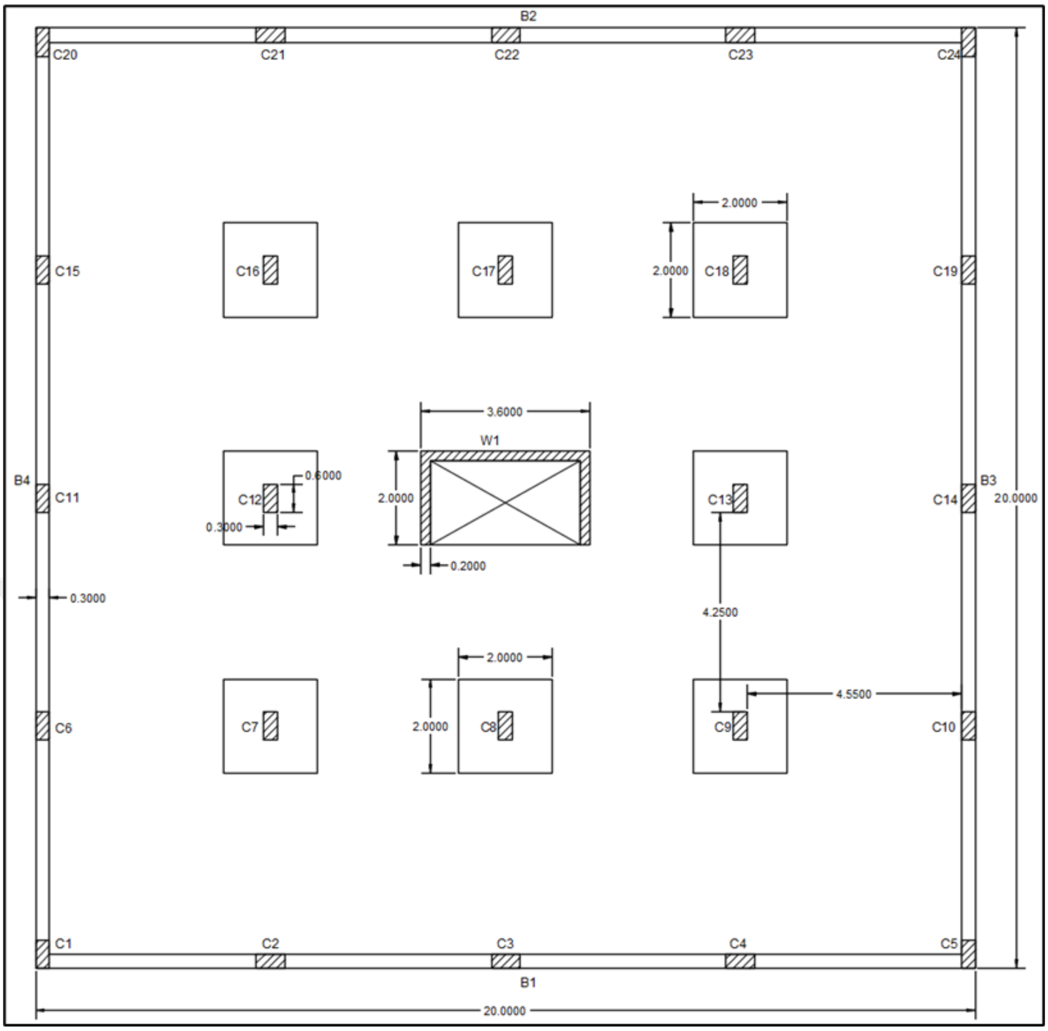

Figure 2.5: Layout of The Repeated Floor.

3. METHODOLOGY

3.1 OVERVIEW

The results in this thesis is conducted mainly form the analysis of the computer program (ETABS). The procedure is generally assumed a plan and sections of the structural parts and then altered after the first analysis depends on the analysis outputs to assure the structure safety epically the high-rise one. Firstly, the two structures are modelled depend on the characteristics mentioned previously. Then, the loads are assigned. After that, the analysis is taken place. Then, the high-rise structure altered and enforced to resist the forces. Next, the result obtained. Finally, the comparison is taken place and the conclusion is conducted.

3.2 DETAILED PROCEDURE FOR THE NORMAL-HEIGHT BUILDING

3.2.1 Exporting The Plan

The plan sketched on AutoCAD, made it ready and transferred to DXF file. After that, the file ready to be exported to ETABS.

3.2.2 Adjusting Etabs File

ETABS program opened and adjusted the Code to ACI 318-14 metric and the units to be metric SI as in Figure 3.1. In the next board the XY grids should left blank. In the story tab, the story height is changed to 3m. In the story data pad, number of story altered to 5 for the normal height. Finally, the 1st story changed to be master story and the rest of story was made similar to the master story as appeared in Figure 3.2.

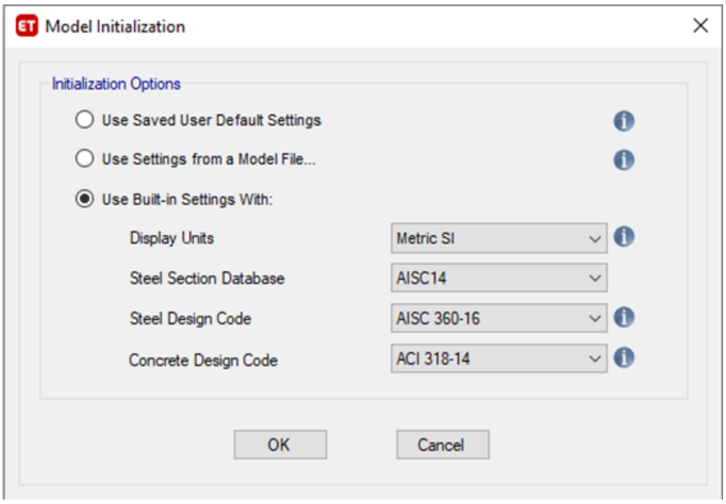

Figure 3.1: Etabs Model Initialization.

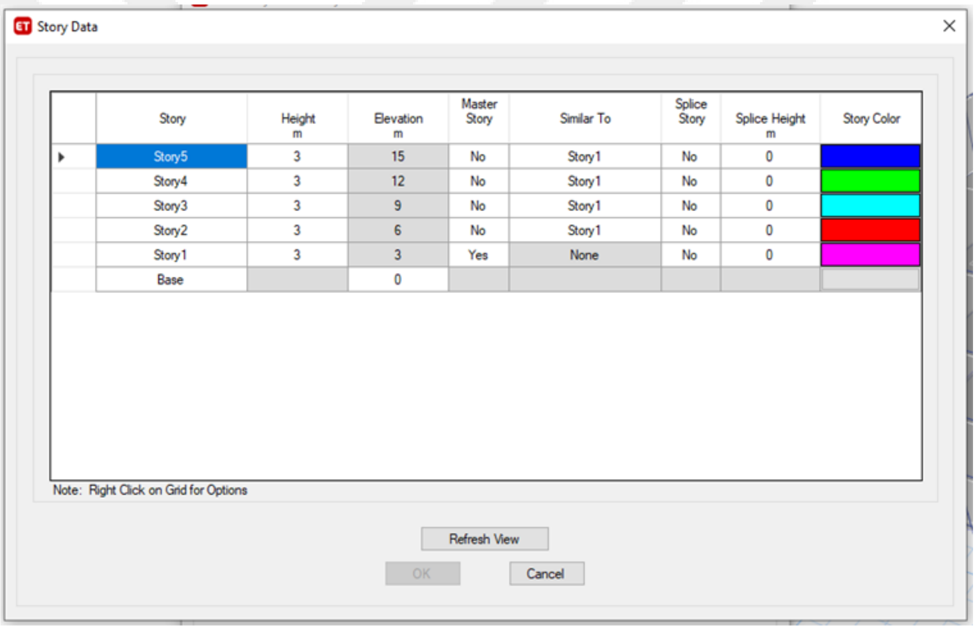

Figure 3.2: Etabs Story Data.

3.2.3 Importing Dxf File and Modeling

After that, the plan which is drown and saved as dxf file is imported. Then, every section of every type of element is drown as mentioned in part II. Thereafter, since every type of elements is plotted on different layer, the layer of column is chosen and the other layers is hidden. It's hidden from the architecture plan option panel in display panel in the program. In the next step, similar story mode is selected because if the element modeled on unique story all its similar story will also modeled on. Then, the option "draw column" chosen and the section stated in part 2.6 is selected (the source of architectural plan is selected to be modeled). So, all the columns of the project in this step is modeled as seen in Figure 3.3. This step is repeated for the rest of the construction's element (walls, beams, slabs and drop panels). All the construction's part is modeled after this process as Figure 3.4 shows. After that, the moment of inertia of the sections is modified to get the real moment of inertia of the sections after analysis (sections after cracking). As per ACI318-14 code, the column's and wall's moment of inertia should multiplied by 0.7, the beam's moment of inertia should factored by 0.35 and finally the slab's moment of inertia should decreased to 0.25% as shown in Figure 3.5 (ACI318-14 Table 6.6.3.1.1(a) p72). The next step is changing the restrains of the columns and walls in the base from pin support to fixed support. This step is essential because the program assume the support as pin when the vertical parts are molded. But in the real works the reinforcement of the columns is extended, bended and combined with the reinforcement of the foundation. So, it must be fixed. This assumption affects the lateral displacement of the structure.

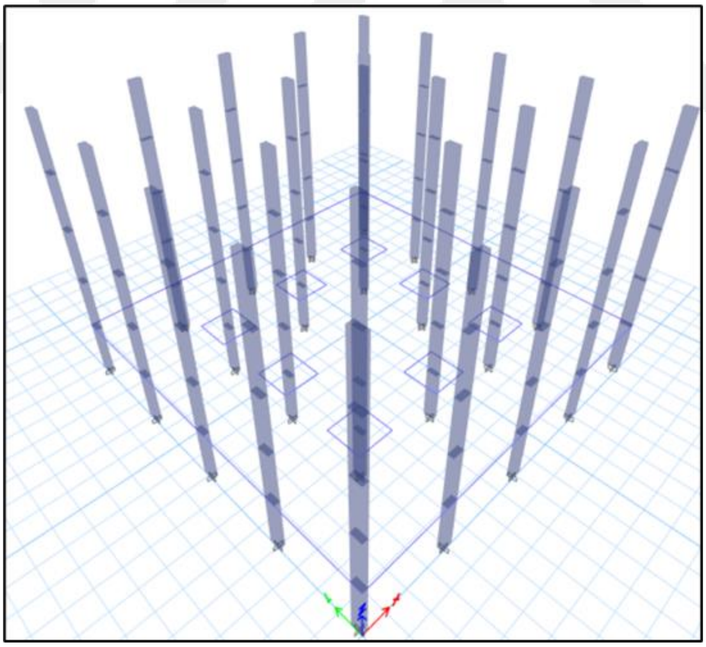

Figure 3.3: Normal-Height Construction's Columns.

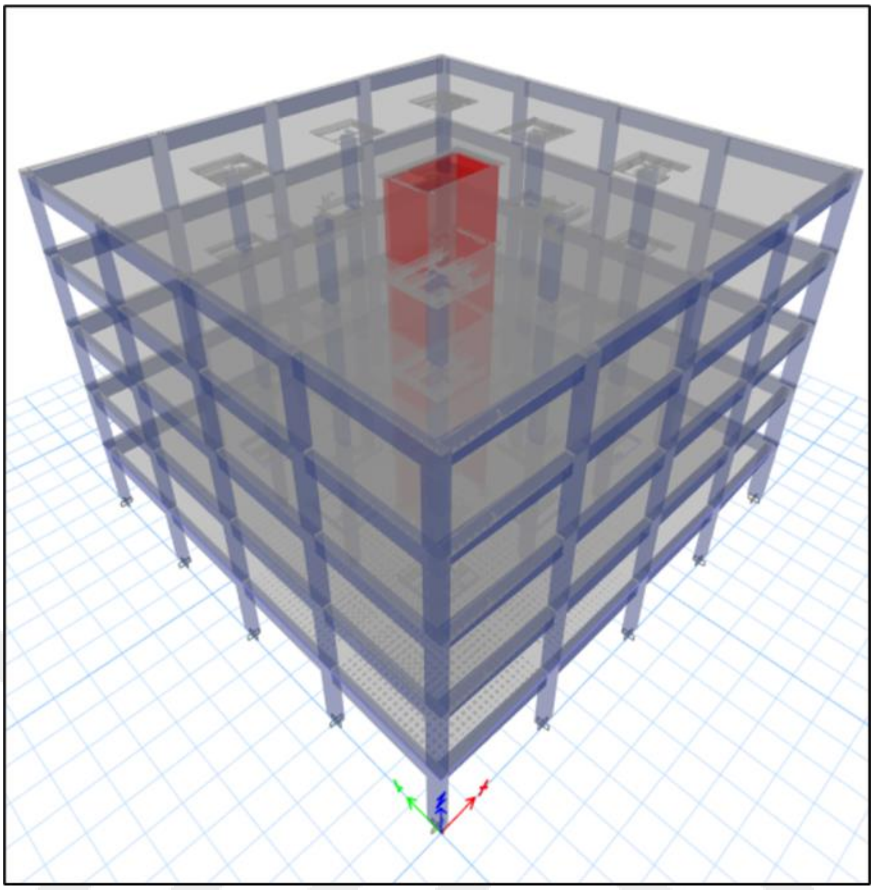

Figure 3.4: Normal-Height Construction's Model.

Table 6.6.3.1.1(a)—Moment of inertia and cross-sectional area permitted for elastic analysis at factored load level

Member and condition		Moment of Inertia	Cross-sectional area
Columns		$0.70I_g$	$1.0A_g$
Walls	Uncracked	$0.70I_g$	
	Cracked	$0.35I_g$	
Beams		$0.35I_g$	
Flat plates and flat slabs		$0.25I_g$	

Figure 3.5: Moment of Inertia and Cross Sectional Area Permitted For Elastic Analysis at Factored Load Level.

3.2.4 Load Patterns Definitions

As mentioned in part 2.5, the dead, live, wind and seismic load is defined from the load pattern panel. Firstly, the dead load should defined and the self-weight multiplier should be 1. The step is done to include the self-weight of the structural element in the analysis. Secondly, the live load is added. Then, the wind load is defined and the code used is chosen to be ASCE7-10 as mentioned before. The parameters of wind load is configured as stated in section 2.5.3 as shown in Figure 3.6. The exposure height panel should be altered to be from the base to the story5. Exposure from extents of diaphragm should be selected. Finally, the seismic load is defined similarly to the wind load. The deference here that the seismic load should defined in the X, Y, positive and negative directions. It defined and the code chosen is also ASCE7-10. The diaphragm ecc.is chosen to be 0.05 and the seismic parameters is token as mentioned in section 2.5.4 as shown in Figure 3.7. The story range should also be from base to the story5.

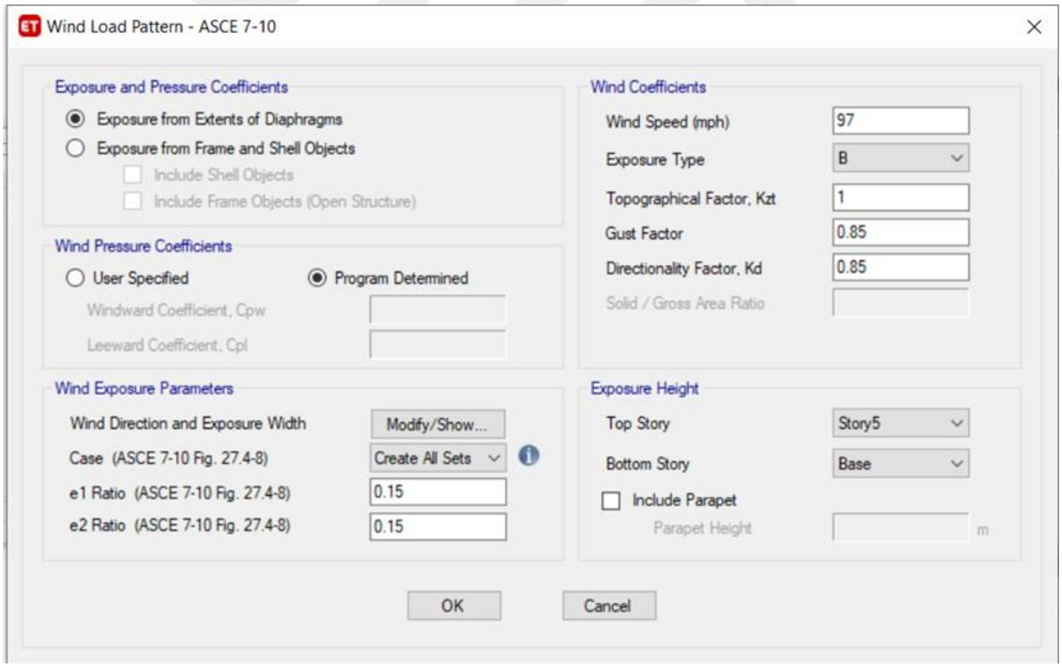

Figure 3.6: Wind Load Characteristic Factors.

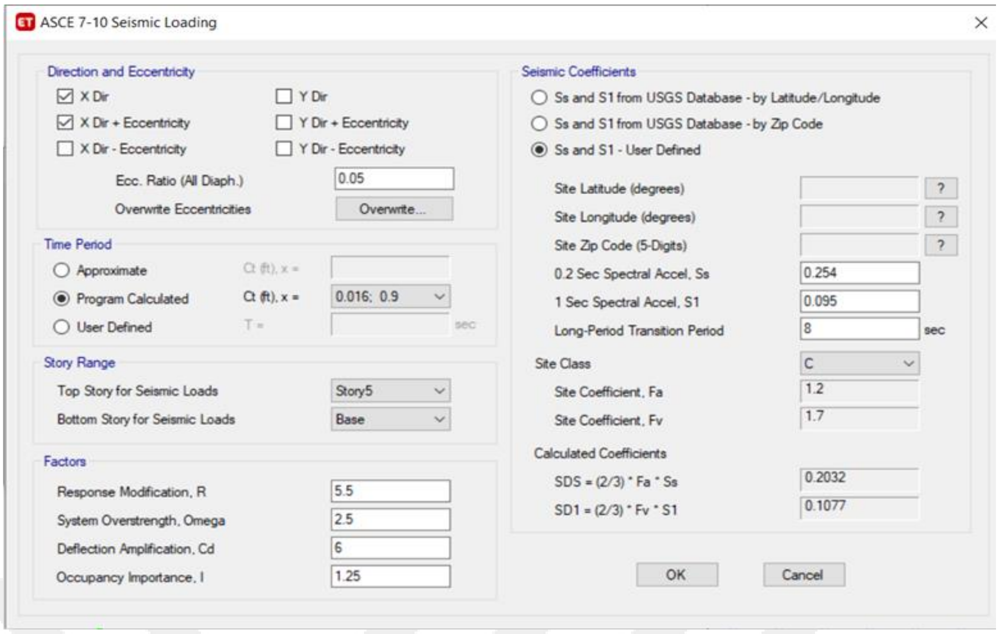

Figure 3.7: Seismic Load Coefficients.

3.2.5 Diaphragm Definition and Assignment

Every story of the structure should be defined discreetly. This step is important to assign the wind and seismic load to the floors. It defined from define diaphragm panel. Five different diaphragms should be defined. Then, every singular story should be attached to its diaphragm.

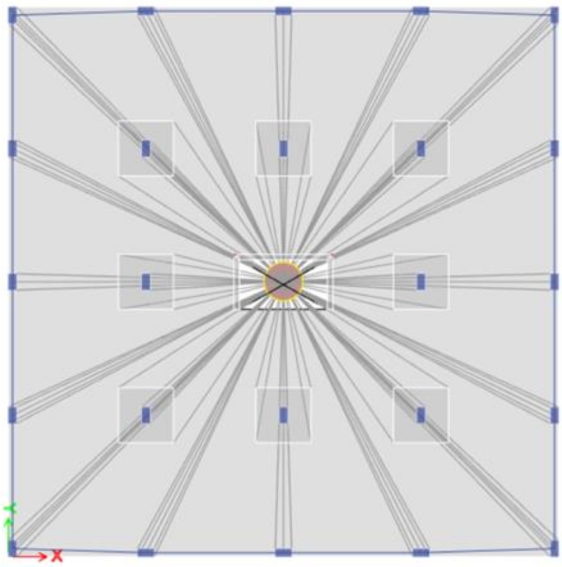

Figure 3.8: Diaphragm (Center of Mass).

3.2.6 Load Assignment

Started with dead load, the floors is loaded with 0.4 t/m^2 and the beams loaded with 1t/m load as detailed in section 2.5. Also, live load are attached to the floors with amount of 0.2 t/m^2. The wind and the seismic load are attached automatically after the diaphragms link as shown in Figure 3.9.

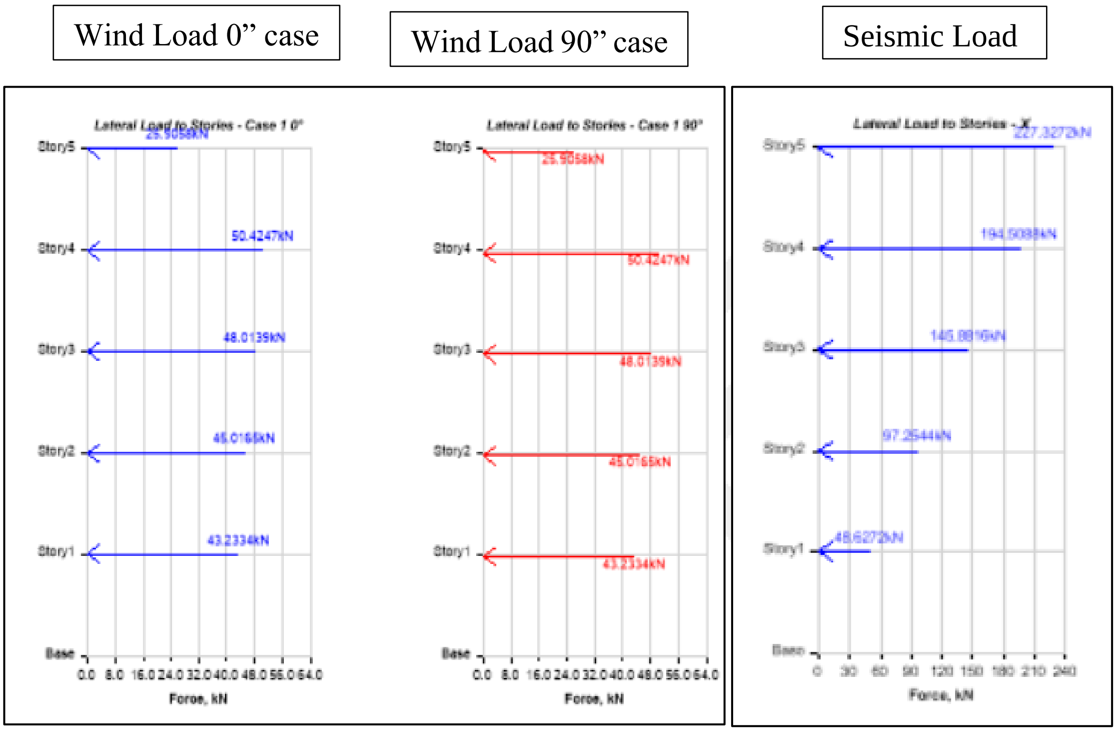

Figure 3.9: Wind and Seismic Load (BL5).

3.2.7 Finalizing the Model and Preparing to Run the Analysis

The last step is to let the program make an automatic mesh to the walls and slabs. It mainly token as 0.5m x 0.5m. After that, review should be taken all around the model to check if any blunder taken place. By ending this step the model is ready for the analysis.

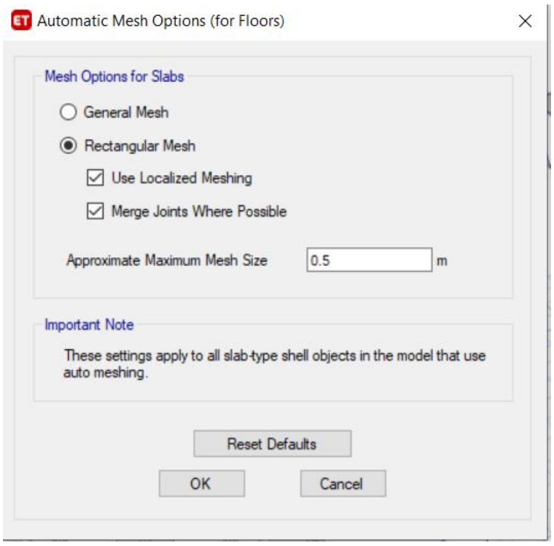

Figure 3.10: Automatic Mesh Adding.

3.3 HIGH-RISE BUILDING MODELING

3.3.1 First Analysis Iteration and Modelling

After the analysis of normal height construction, the file is copied. Then, it is opened in order to make it 20 floors building. Edit panel is opened and "replicate" button is chosen. It repeated the floor and made the building 20 floors as seen in Figure 3.9 as mentioned before. All the loads, beams, columns, walls, slab characteristics is assigned same as the lower floors. Addition steps should be done before analysis: The seismic and wind load should be assigned to all 20 floors not just the lower floors (the loads assigned to only the lower 5 floors because the file is a copy from the 5 floors). It is altered from the load pattern tabs. (All of the 4 type of seismic and the wind load is altered as seen in Figure 3.10. Also, the diaphragms of new floors should defined and assigned to the floors. Now, the model is ready to run.

3.3.2 Second Analysis Iteration

3.3.2.1 Vertical elements properties change

The same properties is taken also to the high rise building to compare with the normal construction. Now, the properties altered to safely carry the loads act on it. The new plan of columns and walls is shown in Figure 3.11. Four walls around the perimeter of the building with section of 30cm x 180cm is added (W2, W3, W4 and W5) instead of side columns (C3, C11, C14 and C22) to maintain the construction against the lateral load. Also, the central core thickness (W1) is increased to 30 cm. The section of corner columns (C1, C5, C20 and C24) became 60cm x 40cm. Exterior side columns (C2, C4, C6, C10, C15, C19, C21 and C23) turn into 70cm x 40cm. Interior columns (C7, C8, C9, C12, C13, C16, C17 and C18) enhanced to 80cm x 45cm. The compressive strength is also increased to attain 40 MPa.

Table 3.1: Members Changing Between Assumptions.

First Assumption						Second Assumption			
Location	Member ID	Type of member	Dimensions (m)	The compressive strength of the concrete (MPa)		Member ID	Type of member	Dimensions (m)	The compressive strength of the concrete (MPa)
Core	W1	Wall	0.2 (thickness)	35		W1	Wall	0.3 (thickness)	40
Perimeter Columns to wall	C3,C11,C14 and C22	Column	0.3x0.6	35	→	W2,W3,W4 and W5	Wall	0.3x1.8	40
Corner Columns	C1, C5, C20 and C24	Column	0.3x0.6	35		C1, C5, C20 and C24	Column	0.4x0.6	40
Perimeter Columns	C2, C4, C6, C10, C15, C19, C21 and C23	Column	0.3x0.6	35		C2, C4, C6, C10, C15, C19, C21 and C23	Column	0.4x0.7	40
Interior Columns	C7, C8, C9, C12, C13, C16, C17 and C18	Column	0.3x0.6	35		C7, C8, C9, C12, C13, C16, C17 and C18	Column	0.45x0.8	40

3.3.2.2 Dynamic Analysis

As the building height goes beyond the 50m, the dynamic analysis is very useful and gives more economic procedure which reduce the unnecessary expenses and non-preferable large concrete sections. Earth quake acceleration (a_{max}) =0.2032g at period 0.106 and 0.5299 as shown in the Figure 3.12 (a_{max} determination). The soil type is very dense soil and soft rock (class C as per SBC301 Table 14.1.1 P14/2) as mentioned before and the type of foundation is Piles (deep foundation).

The dynamic analysis of the 20 floors building is done throw the following procedure:

a. Define the response of Spectrum of the project depend on the C_v and C_a.

b. Determination the number of modes used. It usually taken as 3*no. of floors (60 in our case).

c. Define the load cases of the response Spectrum in the X direction and name it E1.

d. Alter the scale factor to make the base shear in dynamic analysis typical to static analysis and by adding this expression (g*I/R).

e. Do the same procedure in Y direction as done in X direction.

f. Include the vertical mass from the “mass source data” panel.

g. Calibrate the base shear in dynamic analysis with the one in static analysis in both X and Y direction to be the same value. (V_x and V_y in both type of analysis).

h. Define the main and secondary combos in both directions to extract the displacements and design the shear wall depend on it. The main comb is (1.2D + L + E) and the secondary combo is (0.9D + E). The model is ready now to obtain the results.

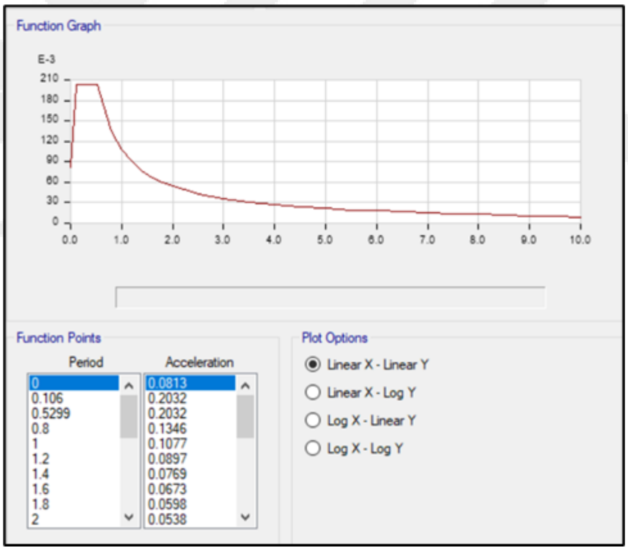

Figure 3.11: Response of Spectrum.

*Note: The response of spectrum is computed from the periods that is derived from the modes analysis given in Table 4.3.

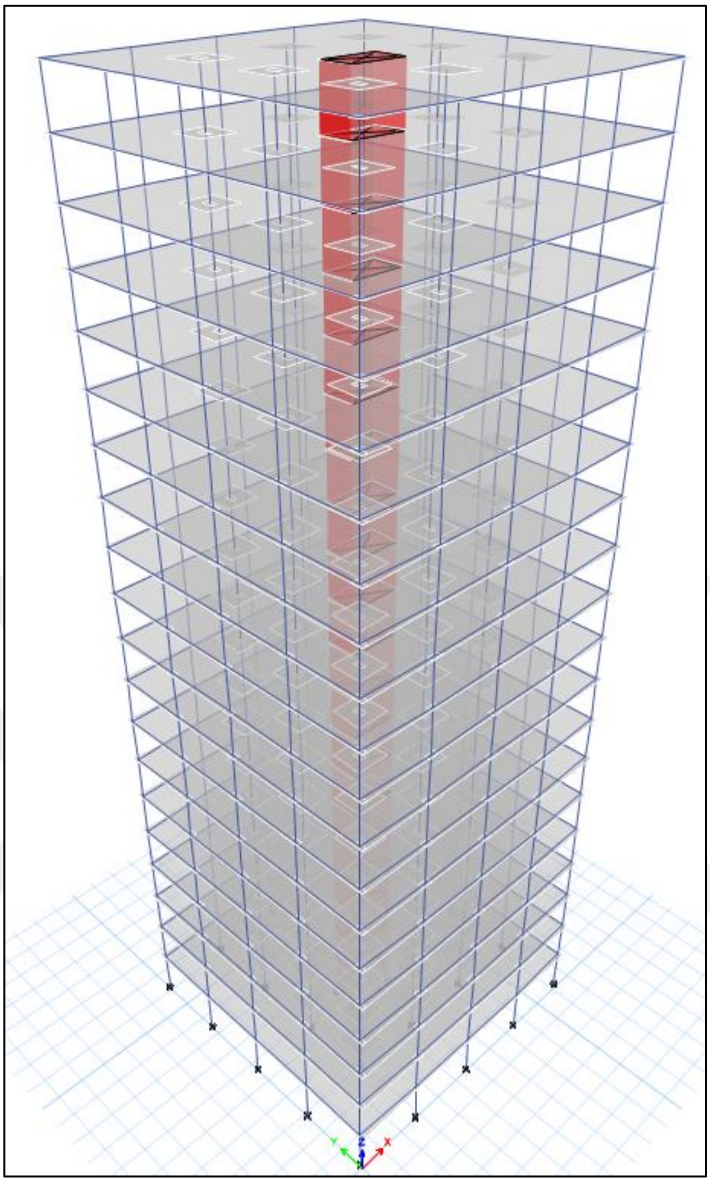

Figure 3.12: High Rise Building Model (BL20).

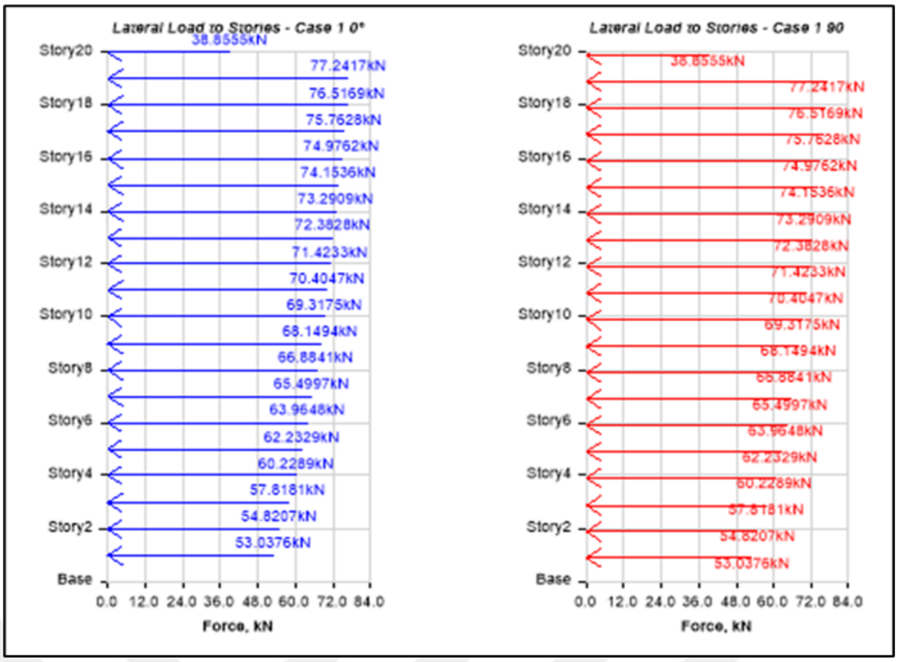

Figure 3.13: Wind Load Acted on High Rise Building Model (BL20).

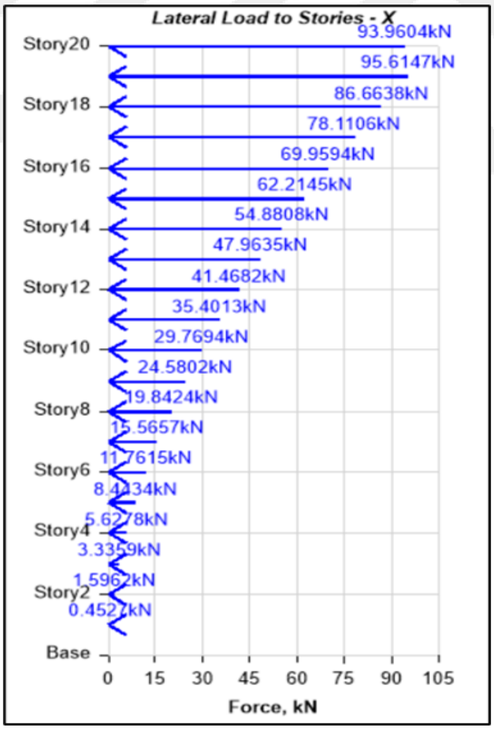

Figure 3.14: Seismic Load Acted on High Rise Building Model (BL20).

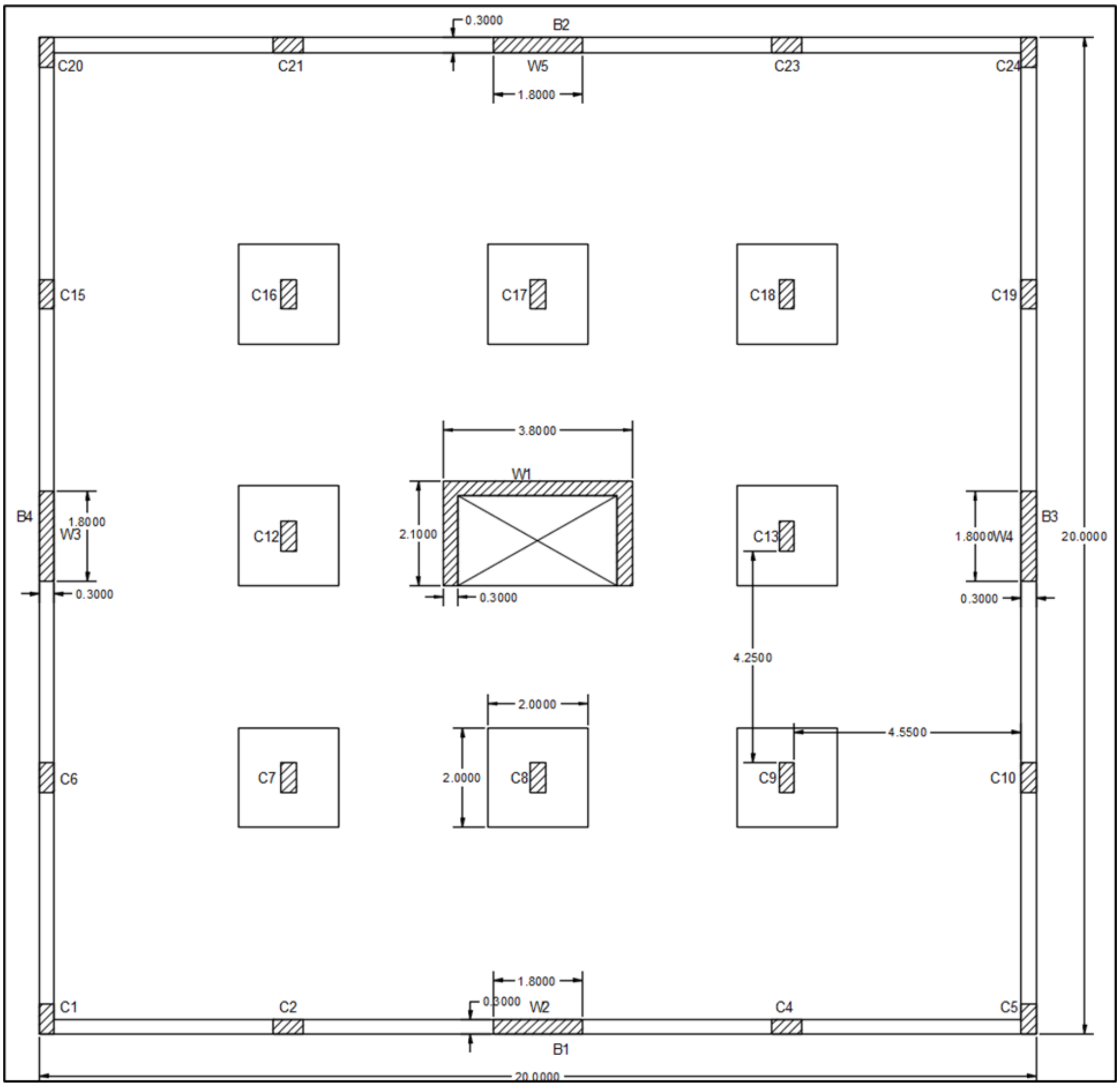

Figure 3.15: Altered Plan for High Rise Building (BL20).

4. RESULTS

4.1 GENERAL CONSTRUCTION CHECKS

4.1.1 Punching Shear Checking for the Slab

The slab thickness is 250 mm. The drop panel thickness is 400 mm. Its compressive strength equal to 35 Mpa. The outer columns have perimeter beams, so, it don't need punching checking because the beams stirrups safely stands the punching shear. The slab on the interior columns will be checked. Maximum load that its punching will checking is at is at C16. (1.2D+1.6L) combination is giving the maximum unfavourable load which equal to the difference of axial load of V_p = N2 – N1. Two samples will be checked:

a. Interior Column:

V_p = N2 – N1 = 7223.6 – 6818.73 = 404.87 kN. The punching shear checking has three equation (as per ACI318- Table 22.6.5.2—Calculation of vc for two-way shear –pg 364). The least value is chosen as the following:

i. $V_c = 4\lambda\sqrt{fc'}\,*b_o*d$ = 1776 KN.

ii. $V_c = (2+\frac{4}{\beta})\lambda\sqrt{fc'}\,*b_o*d$ = 1830 KN.

iii. $V_c = (2+\frac{\acute{\alpha}s\ d}{bo})\ \lambda\sqrt{fc'}\,*b_o*d$ = 2910 KN.

λ = 1 (for normal Wight concrete), fc' = 35 MPa, d = 370 mm (400 – cover), b_o = 3280 mm, $\acute{\alpha}s$ = 40 (for internal columns as per (ACI 318 - 22.6.5.3 – pg365), β = 600/300 = 2.

The smallest value equal to 1776 KN which is quite bigger than 403.2 KN.

($V_c > V_u$) >>> It's safe against punching shear.

*Note: The eccentricity calculations is neglected because the case of study columns are internal columns and the spans between columns are almost equals (the differences between moments goes to zero). (Unbalanced Moment Mu2 = -3.0514 kN.m).

b. Corner Column:

Avg. Eff. Slab Thickness = 0.217 m

Eff. Punching Perimeter = 1.1175 m

Cover = 0.033 m

Conc. Comp. Strength = 35000 kN/m2

Reinforcement Ratio = 0.0025

Section Inertia I22 = 0.014106 m4

Section Inertia I33 = 0.003933 m4

Section Inertia I23 = 0 m4

Gamma_v2 = 0.46748

Gamma_v3 = 0.4

Moment Mu2 = 23.0426 kN-m

Moment Mu3= -11.7403 kN-m

Shear Force = 117.281 KN

Unbalanced Moment Mu2 = 10.7719 kN-m

Unbalanced Moment Mu3= -4.6961 kN-m

Max Design Shear Stress = 744.53 kN/m2

Conc. Shear Stress Capacity = 1473.72 kN/m2

Punching Shear Ratio = 0.51 OK

4.1.2 Decreasing the Dimensions of the Columns with Respect to the Remaining Floors

The loads of corner columns is decrease every floor around 158.6 KN, the exterior side columns loss about 252.6 KN while the interior columns load down 403.2 KN every floor. So, the as we have 20 floors, it is more economic to shrink the columns with respect to the height. The most efficiency decreasing in this situation is decreasing the columns every 3 floors. The smallest standard dimension is 300x600 mm. The following Table (Table 4.1) will summarize this decreasing.

4.1.3 Displacements Checking

The maximum lateral displacement of the (BL5) is equal to 3.3 mm. while the maximum lateral displacement of the (BL20) is equal to 24.5 mm. According to ASCE7-10 code (17.5.6 pg. 172) the maximum story drift of the structure above the isolation system shall not exceed $0.015h_{sx}$. The maximum allowable for (BL5) = 0.015*5*3 = 225mm. 3.3<225 **ok.** The maximum allowable for (BL20) = 0.015*20*3 = 900mm. 24.5<900 **ok.**

4.1.4 Ductility Calculation

Ductility Coefficient μ, which is the ratio of the maximum displacement (u_{max}) of the nonlinear system to the yielding displacement (u_y). This ratio according to ASCE7-10 (18.6.3 pg 192) should be equal or greater than 1. u_{max} = 24.5 mm. u_y (D_y) = $(g/4\pi^2)(\Omega_0 C_d/R)T_1^2 C_{s1}\,\Gamma_1$ (ASCE7-10 (18.6-10) pg 192). g = 9.81, π = 3.14, Ω_0 = 2.5, C_d = 6, R = 5.5, T_1 = 2.277, C_{S1} = (R/C_d) $(S_{DS}/\ \Omega_0 B_{1D})$ (ASCE7-10 (18.4-4) pg 185) B_{1D} = 1.0 (ASCE7-10 Table 18.6-1 pg 190). So, C_{S1} = 0.0745. $\Gamma_1 = W_i /\sum W$ = w/20w = 0.05. u_y = 0.013 = 13 mm.

$u_{max}/\ u_y$ = 24.5/13 = 1.89 > 1 ok.

Table 4.1: Columns Shrinkage.

Columns Floors	Interior columns dimensions (mm)	Exterior side columns dimensions (mm)	Corner columns dimensions (mm)
1-3	800*450	700*400	600*400
4-6	800*400	700*350	600*350
7-9	750*400	700*300	600*300
10-12	700*400	650*300	600*300
13-15	650*400	600*300	600*300
16-18	600*400	600*300	600*300
19-20	600*350	600*300	600*300

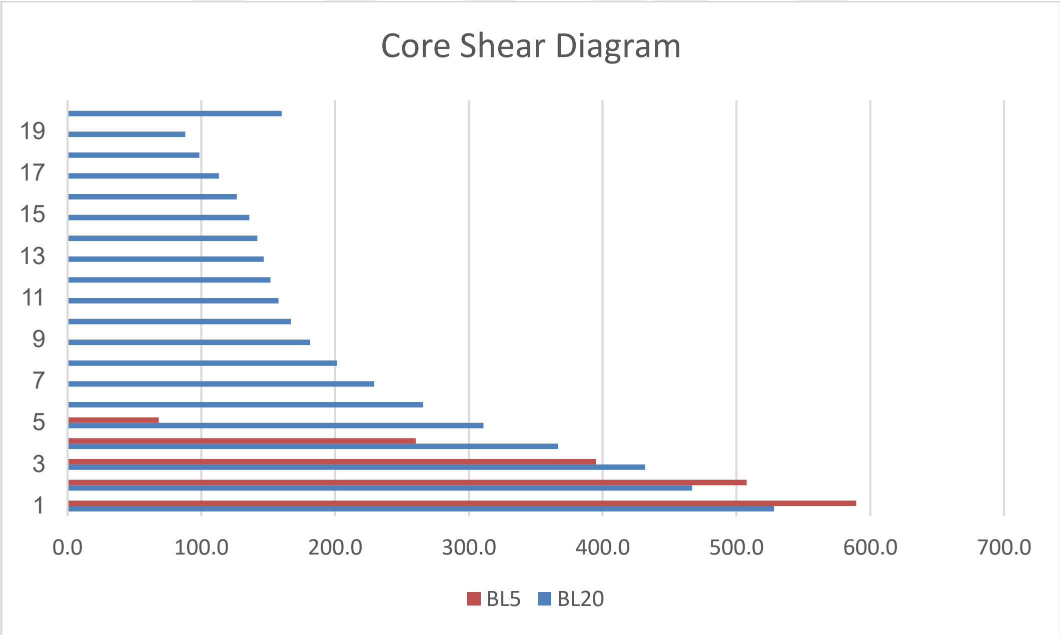

Figure 4.1: Core Shear Diagram.

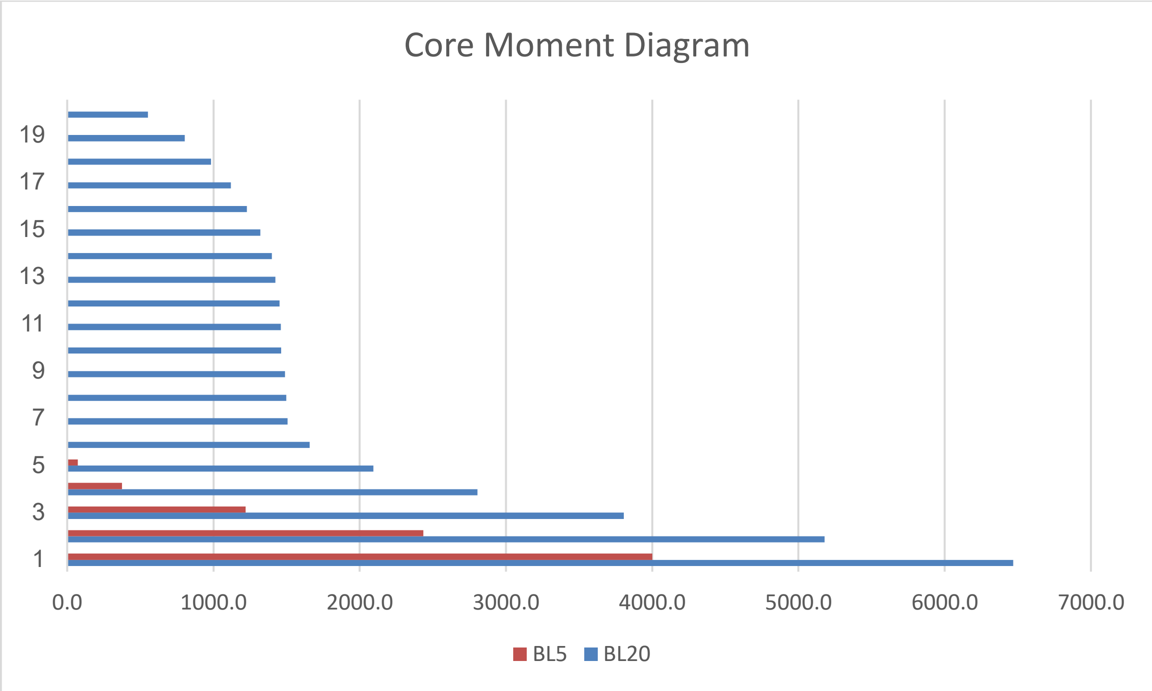

Figure 4.2: Core Moment Diagram.

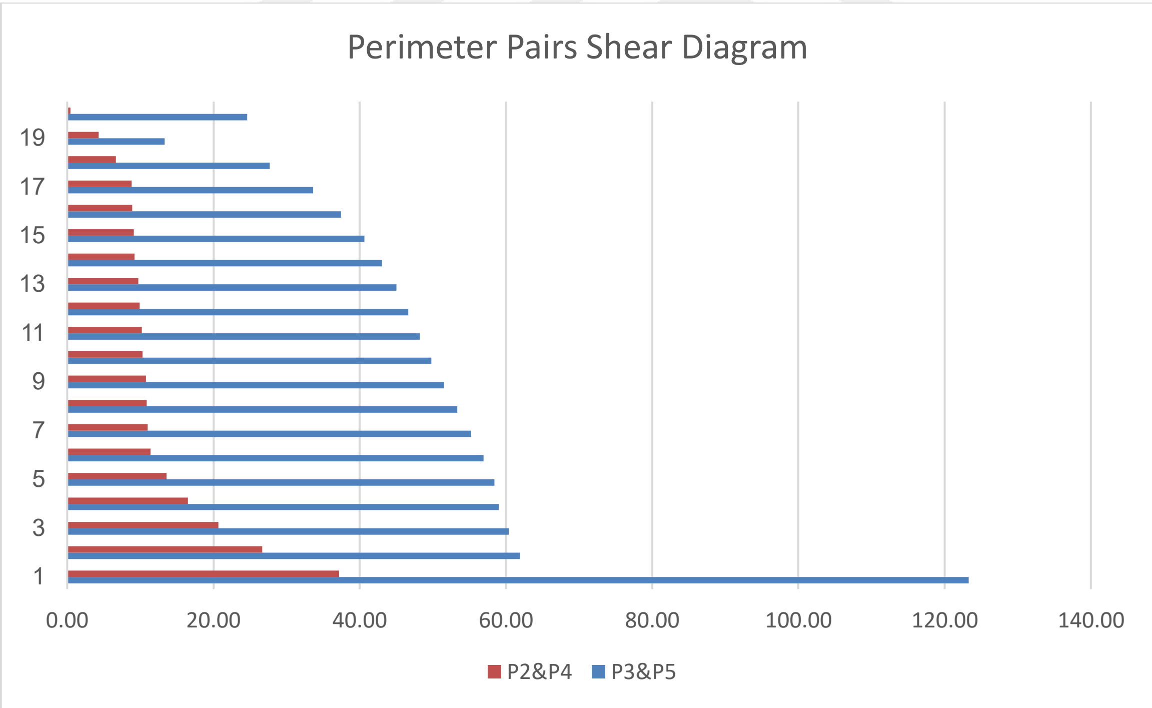

Figure 4.3: Perimeter Pairs Shear Diagram.

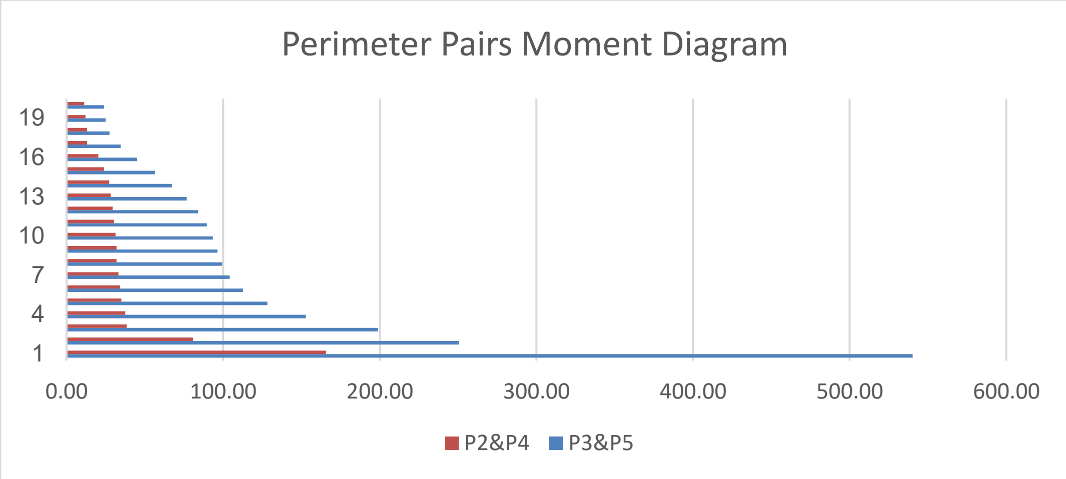

Figure 4.4: Perimeter Pairs Moment Diagram.

4.2 COMPARISON TABLE

The final results of the 3 types of models are summarized in the coming Table:

Table 4.2: Result Summary.

Category/No. of floors	BL5	BL20	BL20 (Edited)
Base Shear forces (KN)	589.6	527.9	527.9
Type of analysis	Static	Static	Dynamic
Wind load at 5 floor level (KN)	19.8	63.5	63.5
Max. Wind load (KN)	38.5	78.8	78.8
Seismic load at 5 floor level (KN)	232	6.25	6.25
Max. Seismic load (KN)	232	93.5	93.5
X Max. Story Drift (m)	0.0033	0.0537	0.0233
Y Max. Story Drift (m)	0.0034	0.0544	0.0245
Z Max. Displacement (m)	0.0045	0.0401	0.0213
Max. moment (KN.m)	4005	10764.9	6467.7
Max. Shear Force (KN)	590.4	863.7	527.9
No. of modes	12	40	40
Period at mode 1 (sec)	0.503	2.277	2.277
Type of foundation (suggested)	Single shallow	Piles deep	Piles deep

4.3 MODAL PARTICIPATING MASS RATIOS

Table 4.3: Modal Participating Mass Ratios for 20 Floors Building.

Mode	Period (sec)	UX	UY	UZ	RX	RY	RZ
1	2.277	0.000	0.758	0.000	0.224	0.000	0.000
2	2.136	0.706	0.000	0.000	0.000	0.246	0.024
3	1.834	0.020	0.000	0.000	0.000	0.010	0.759
4	0.706	0.000	0.109	0.000	0.398	0.000	0.000
5	0.628	0.086	0.000	0.000	0.000	0.238	0.028
6	0.576	0.041	0.000	0.000	0.000	0.107	0.068
7	0.374	0.000	0.042	0.000	0.057	0.000	0.000
8	0.342	0.008	0.000	0.000	0.000	0.012	0.031
9	0.289	0.045	0.000	0.000	0.000	0.066	0.007
10	0.236	0.000	0.025	0.000	0.075	0.000	0.000
11	0.225	0.003	0.000	0.000	0.000	0.008	0.019
12	0.175	0.026	0.000	0.000	0.000	0.082	0.003
13	0.166	0.000	0.000	0.780	0.000	0.000	0.000
14	0.163	0.000	0.015	0.003	0.012	0.000	0.000
15	0.160	0.002	0.000	0.000	0.000	0.003	0.012
16	0.152	0.000	0.001	0.001	0.081	0.000	0.000
17	0.151	0.000	0.000	0.000	0.000	0.045	0.000
18	0.137	0.000	0.000	0.000	0.000	0.000	0.000
19	0.125	0.000	0.000	0.002	0.000	0.000	0.000
20	0.120	0.000	0.000	0.002	0.000	0.000	0.000

Table 4.3: Modal Participating Mass Ratios for 20 Floors Building “Table Continued”.

21	0.120	0.000	0.000	0.000	0.000	0.000	0.010
22	0.118	0.018	0.000	0.000	0.000	0.044	0.001
23	0.117	0.000	0.011	0.000	0.027	0.000	0.000
24	0.115	0.000	0.001	0.000	0.000	0.000	0.000
25	0.111	0.000	0.000	0.000	0.000	0.002	0.000
26	0.104	0.000	0.000	0.000	0.004	0.000	0.000
27	0.104	0.000	0.000	0.000	0.000	0.005	0.000
28	0.100	0.000	0.000	0.000	0.000	0.000	0.000
29	0.093	0.000	0.000	0.000	0.000	0.000	0.000
30	0.092	0.001	0.000	0.000	0.000	0.002	0.007
31	0.091	0.000	0.000	0.011	0.000	0.000	0.000
32	0.089	0.000	0.008	0.000	0.021	0.000	0.000
33	0.088	0.003	0.000	0.000	0.000	0.006	0.001
34	0.087	0.008	0.000	0.000	0.000	0.020	0.000
35	0.087	0.000	0.000	0.017	0.000	0.000	0.000
36	0.086	0.000	0.000	0.000	0.001	0.000	0.000
37	0.083	0.000	0.000	0.000	0.000	0.000	0.000
38	0.077	0.000	0.000	0.000	0.000	0.000	0.000
39	0.076	0.000	0.000	0.000	0.000	0.000	0.000
40	0.076	0.000	0.000	0.061	0.000	0.000	0.000

Table 4.4: Modal Participating Mass Ratios for 5 Floors Building.

Mode	Period (sec)	UX	UY	UZ	RX	RY	RZ
1	0.503	0.000	0.771	0.000	0.088	0.000	0.000
2	0.503	0.093	0.000	0.000	0.000	0.009	0.749
3	0.409	0.667	0.000	0.000	0.000	0.084	0.099
4	0.164	0.009	0.000	0.000	0.000	0.010	0.090
5	0.143	0.000	0.138	0.000	0.185	0.000	0.000
6	0.115	0.162	0.000	0.000	0.000	0.190	0.010
7	0.096	0.004	0.000	0.000	0.000	0.004	0.032
8	0.079	0.000	0.000	0.594	0.000	0.000	0.000
9	0.076	0.000	0.000	0.000	0.000	0.369	0.000
10	0.074	0.000	0.000	0.000	0.381	0.000	0.000
11	0.071	0.000	0.000	0.020	0.002	0.000	0.000
12	0.069	0.000	0.000	0.000	0.000	0.000	0.015

5. DISCUSSION AND CONCLUSIONS

Obviously from the result summary, the analysis data has a significant deferent between the two types of samples. Beginning with the vertical element, the amount of reinforced concrete added is very large. This amount increased not just for the axial force, shear and moment. But also for the huge quantity of lateral force acted on the construction. As seen in the base shear forces, the base shear of the two buildings almost the same. At the first impression it kind a wired and non logical. But after reviewing of the calculation sheet, the base shear is a function of the seismic design coefficient (C_s) and the weight of the structure. The seismic design coefficient (C_s) has a maximum limit that is a function of the period determination (T). This term is linked to the number of story of the building. Its formula is T =0.1N (N=number of stories) (ASCE7-10, 12.8-8, P91). This formula means that the building that goes after 10 floors the base shear will be limited by the maximum value of the seismic design coefficient (C_s). Even though, the torsion and the moment has a great effect on the higher building due to the height. This is because as it is known, the moment is a function of length and force. Even the buildings has almost the same forces the height is enlarge the moment to its maximum levels. Also, the distributing of this shear force is various due to the height deference. One distributes its load on 15m and the other one distributes it on 60m. On the other hand, the wind load isn’t multiplied by 4. It approximately just doubled. That because it is not have a linear proportion with the height. Wind load has a many parameters that affect its value. But also, its effect on the high rise building is very big. The type of study for the two construction is also various. The static analysis adequately match the 5 floors building analysis needs. But for the high rise building the dynamic analysis method is a must to have a logical, correct, not over calculated analysis. The displacement in X, Y and Z direction is also alter and has a huge differences between the two types. The high rise building has a large value of displacements before adjusting. Even after the adjusting, the distinction between the two types is still huge. For the 20 floor structure, even though the displacement is maintained to become between the code limits the designers keep it on low value. This quantity is important to remain in it lower amount in order to not damages the non-structural elements according to the codes. These elements is usually very sensitive and have little resistance against the lateral loads. Also, as this consideration should be kept in mind during the designing process, the economic aspects should be taken as a priority. The cost of the construction is mainly the strong determination agent to choose wither the structure will executed or not. So, the analysis and designing phase should take the non-structural elements safety and the economic value as a limits in its analysis. Also, the type of foundation is varied between these two types. The 15m structure will use single shallow footing safely and adequately. But the 60m construction will need piles deep foundation. This type of foundation cost very much sources and need trained labors and experienced contractors to deliver the construction correctly. This differences becoming severe and large with the growth of the lateral loads factors. Sites that locate in worse seismic areas will face a more differences between these two types. So, if there are an adequate sources, experienced executers and allowed regulation, the high rise building is a good fit. If it is not and the site located in a severe seismic location the normal height building is much better.

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