Palestine Polytechnic University

College of Engineering

Civil Engineering

Graduation Project

“Comparative study of ordinary resisting frame system and shear wall system”

Project Team

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This project submitted to the College of Engineering in partial fulfillment of requirements of the Bachelor degree of Civil Engineering

Hebron - Palestine

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The undersigned hereby certify that they have read, examined, and recommended to the Department of Civil Engineering and Architecture in the College of Engineering at Palestine Polytechnic University the approval of a project entitled:

“Comparative study of ordinary resisting frame system and shear wall system”

submitted by

Fayez Darwish & **Mahmoud Awawdeh**

for partial fulfillment of the requirements for the bachelor’s degree.

Dr. Belal AL Massri. (Supervisor) :

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“Comparative study of ordinary resisting frame system and shear wall system”

ABSTRACT

The goal behind this research is to highlight on structural systems using shear walls; this approach is widely used in many countries, due to its desired features,. Shear walls are often considered essential in the design of building to resist seismic action. The research includes designing a multi story building according to a specific plan using frame system and other one for the same building using shear walls, then comparing between the two designs to determine the main differences in between the two design.

The research focused on the following points:

1. maximum displacement of the two systems.
2. Quantity of concrete for the construction.
3. Reinforcement steel for the construction.
4. Quantity of formwork, and the required duration to implement each system.

الملخص

الهدف من هذا البحث هو تسليط الضوء على الأنظمة الإنشائية باستخدام جدران القص؛ يستخدم هذا الأسلوب على نطاق واسع في العديد من البلدان، نظرًا لميزاته العديدة غالبًا ما تُعتبر جدران القص ذات أهمية في تصميم المبنى لمقاومة الحركة الزلزالية.

تضمن البحث تصميم مبنى متعدد الطوابق وفق خطة محددة باستخدام "الفريمات" وتصميم آخر لنفس المبنى باستخدام جدران القص، ثم المقارنة بين التصميمين لتحديد الفروق الرئيسية بين التصميمين. ركز البحث على النقاط التالية:

1. **الحد الأقصى من ازاحة المبنى في الحالتين.**
2. **كمية الخرسانة اللازمة للإنشاء.**
3. **كمية حديد التسليح اللازم للبناء.**
4. **كمية الاعمال الخشبية والمدة المطلوبة لإنجازها.**

كما ويقع المبنى المقترح في مدينة الخليل -شارع السلام - بجانب كازية الأنصار، وهو عبارة عن فندق 3 نجوم تحت اسم فندق رزق

ويتكون المبنى من ستة طبقات تقدر مساحتها ب 6800 م^2 , وتم توزيع العناصر الانشائية بما لا يتعارض مع التصميم المعماري، وتم حساب الاحمال المؤثرة على المبنى بالاعتماد على الكود الأردني للأحمال و الكود الأمريكي ASCE-16 وتم الاعتماد على IBC-2009 , UBC-97 لتقدير الاحمال الزلزالية ومن ثم تصميم ما يقاومها , وتم استخدام برامج تصميمة لإخراج مخططات مبدئية قابلة للتنفيذ.

Table of Contents

LIST OF ABBREVIATIONS

As	Area Of Non-Prestressed Tension Reinforcement.
As′	Area Of Non-Prestressed Compression Reinforcement.
Ag	Gross Area Of Section.
Av	Area Of Shear Reinforcement Within A Distance (S).
At	Area Of One Leg Of A Closed Stirrup Resisting Tension Within A (S).
b	Width Of Compression Face Of Member.
bw	Web Width, Or Diameter Of Circular Section.
d	Distance From Extreme Compression Fiber To Centroid Of Tension Reinforcement.
Ec	Modulus Of Elasticity Of Concrete.
fy	Specified Yield Strength Of Non-Prestressed Reinforcement.
h	Overall Thickness Of Member.
I	Moment Of Inertia Of Section Resisting Externally Applied Factored Loads.
ln	Length Of Clear Span , Measured Face-To-Face Of Supports In Slabs Without Beams And Face To Face Of Beam Or Other Supports In Other Cases.
M	Bending Moment.
Mu	Factored Moment At Section.
Mn	Nominal Moment.
S	Spacing Of Shear Or In Direction Parallel To Longitudinal Reinforcement.
Vc	Nominal Shear Strength Provided By Concrete.
Vn	Nominal Shear Stress.
Vs	Nominal Shear Strength Provided By Shear Reinforcement.
ρ	Ratio Of Steel Area.
εc	Compression Strain Of Concrete=0.003mm /Mm
Fsd,r	Total Additional Tension Force Above The Support.
Ved,0	Shear Force At Critical Section.
Vu	Factored Shear Force At Section.
Wu	Factored Load Per Unit Length.
Φ	Strength Reduction Factor.

CHAPTER 1

INTRODUCTION

1. ARCHITECTURAL DESCRIPTION
2. STRUCTURAL DESCRIPTION
3. WORK PROCEDURE

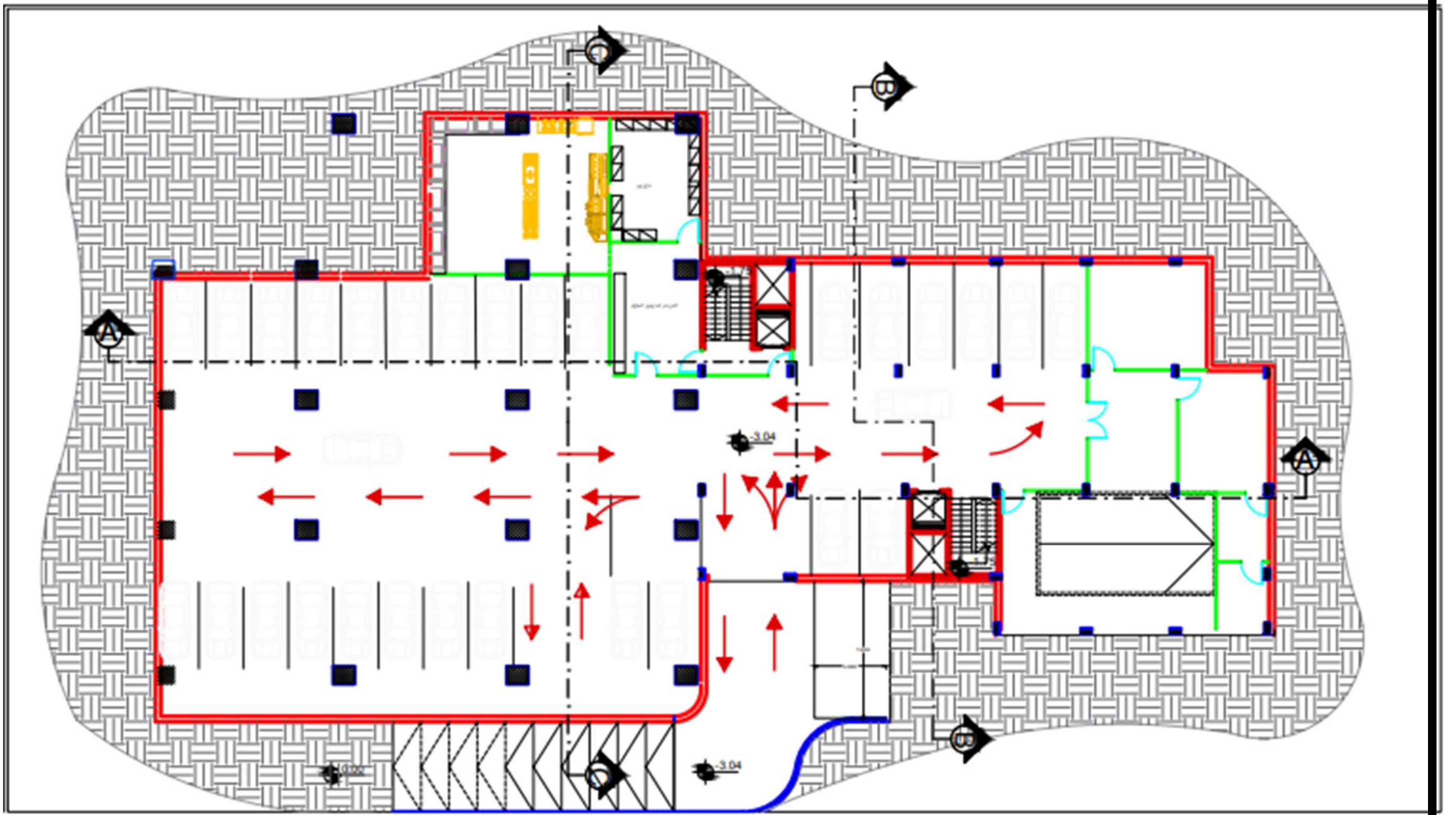
1.1 ARCHITECTURAL DESCRIPTION

1.1.1 General Identification of the project

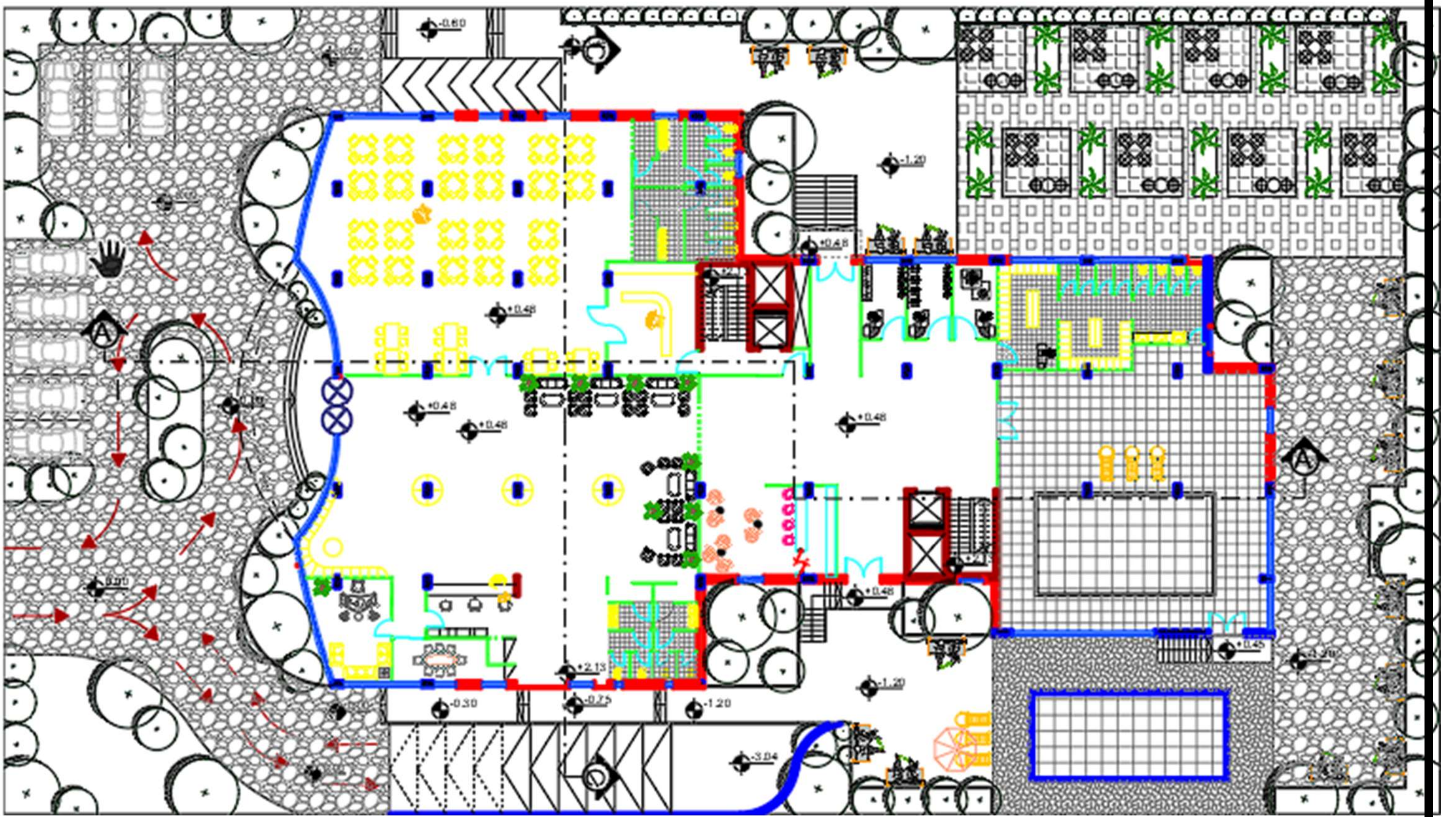
This hotel is located in Hebron city on plot of 2500m2 with a building area around 6800 m2
Which is has basement floor that contain the parking, storage, restaurant, water tanks, water pumps And the ground floor has receptions and hall for waiting and swimming pool, first floor has another hall for eating and rooms. The second, third, fourth floors for visitors' rooms and toilets.

1.1.2 Floors Description

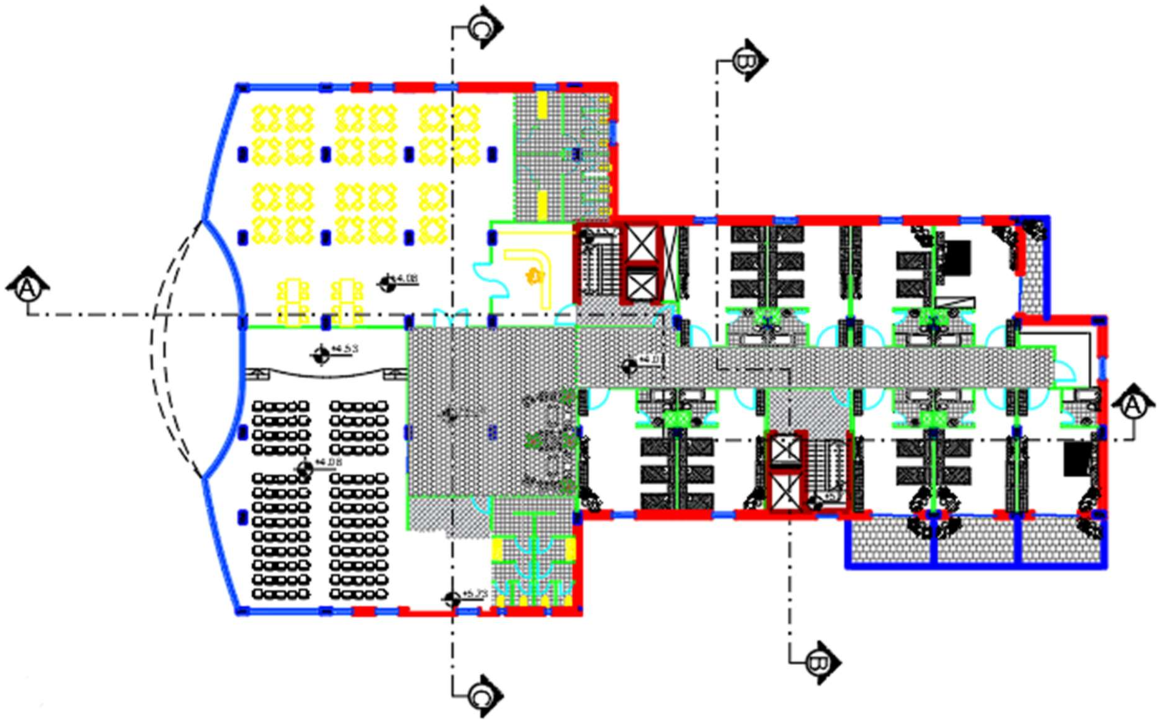
BASEMENT FLOOR :



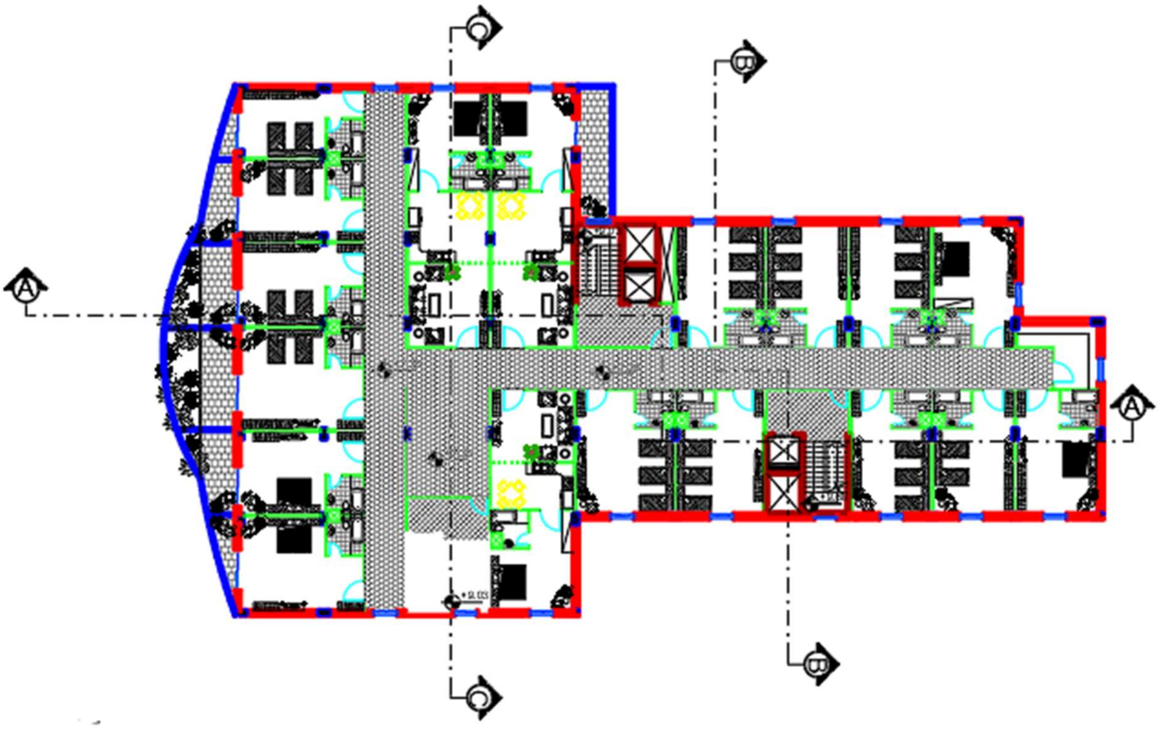
GROUND FLOOR:



FIRST FLOOR:

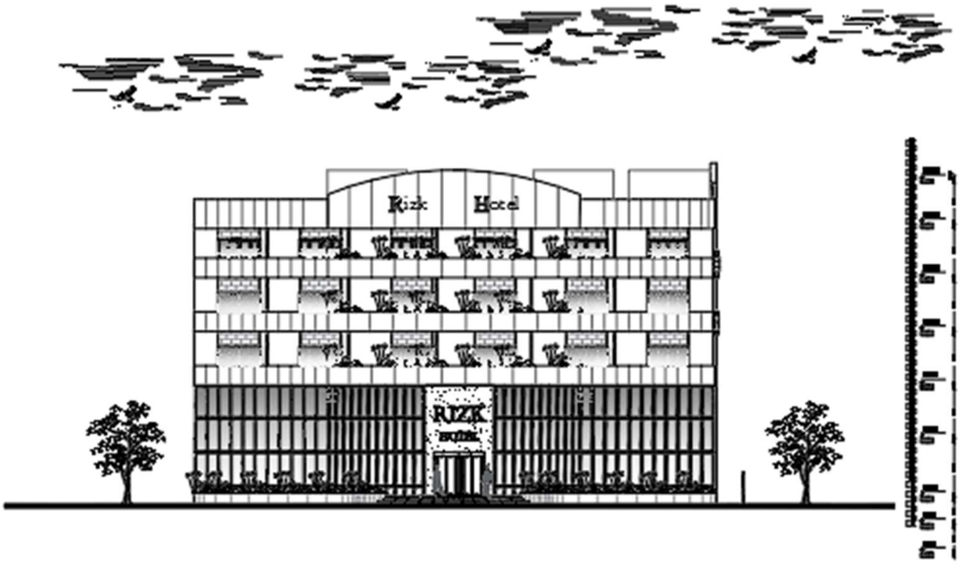


SECOND, THIRD AND FOURTH FLOOR:



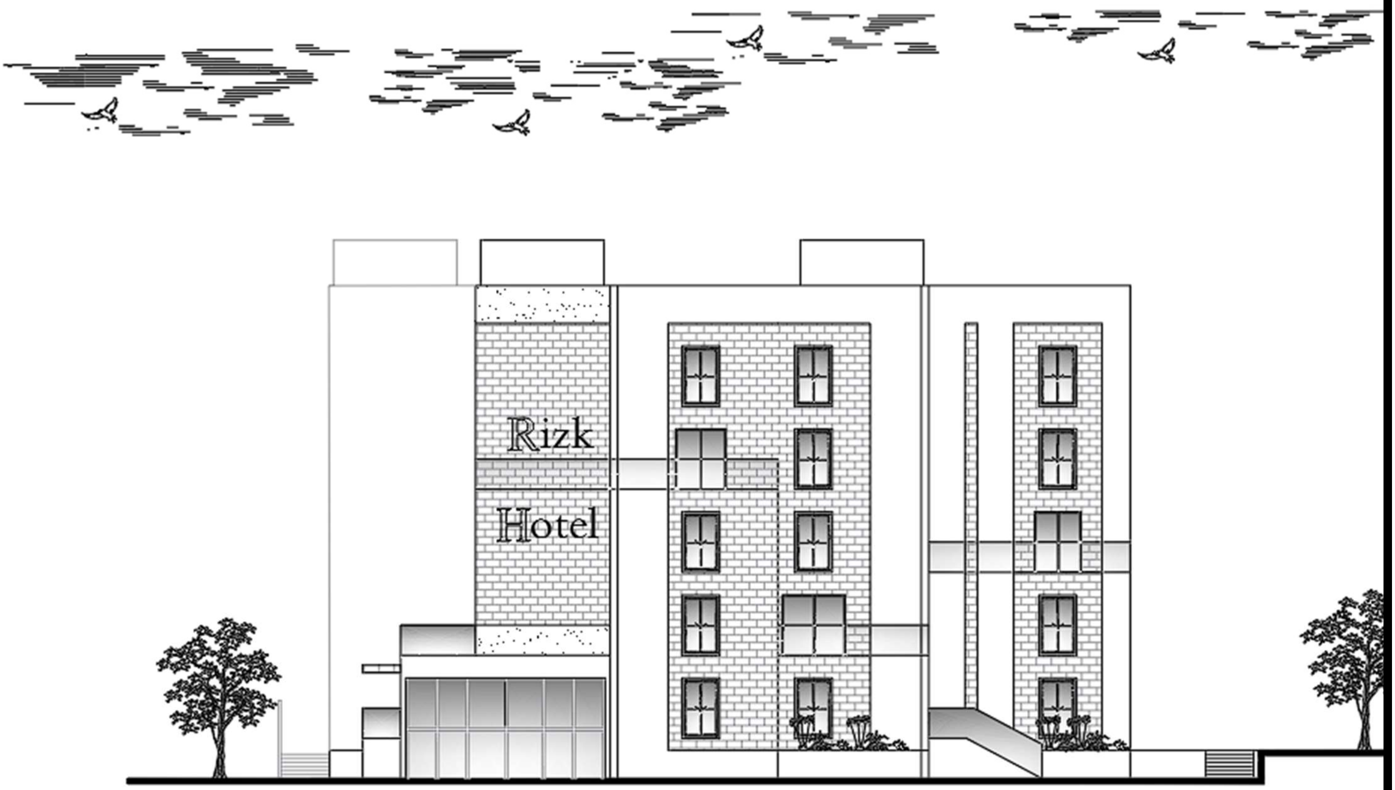
1.1.3 Elevations Description

East elevation:

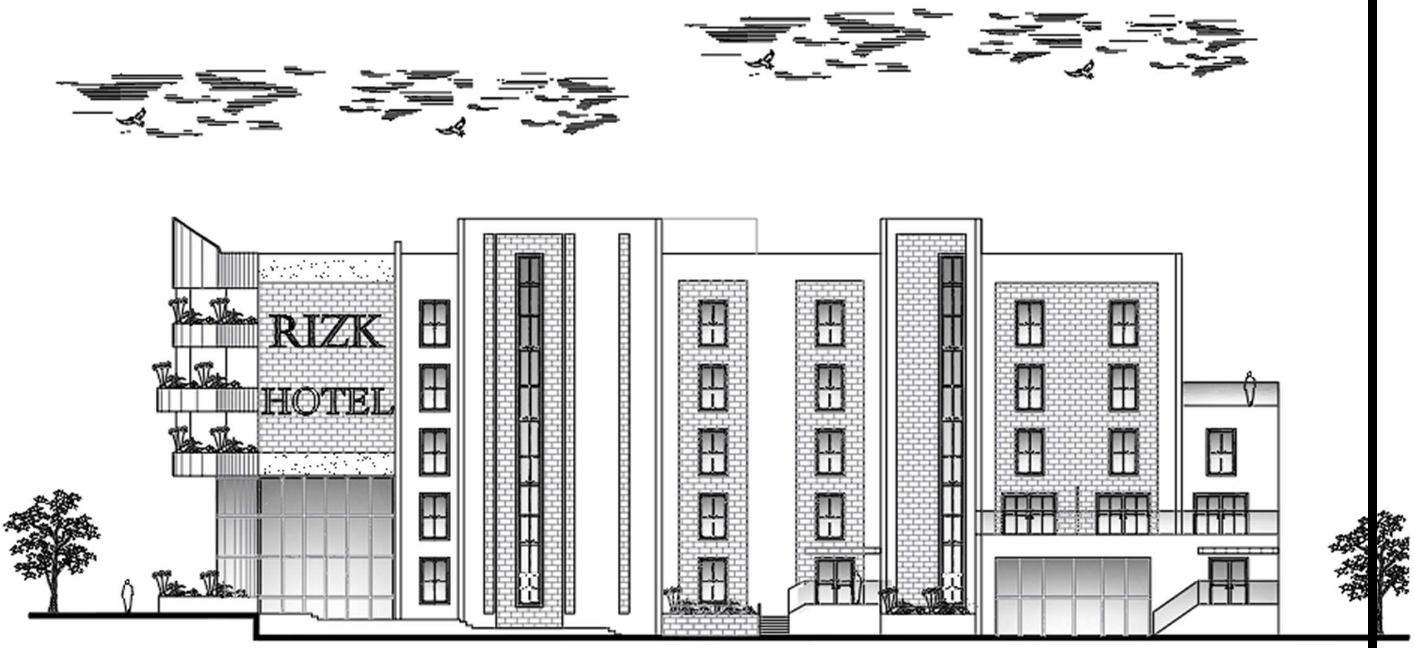


NORTH elevation:

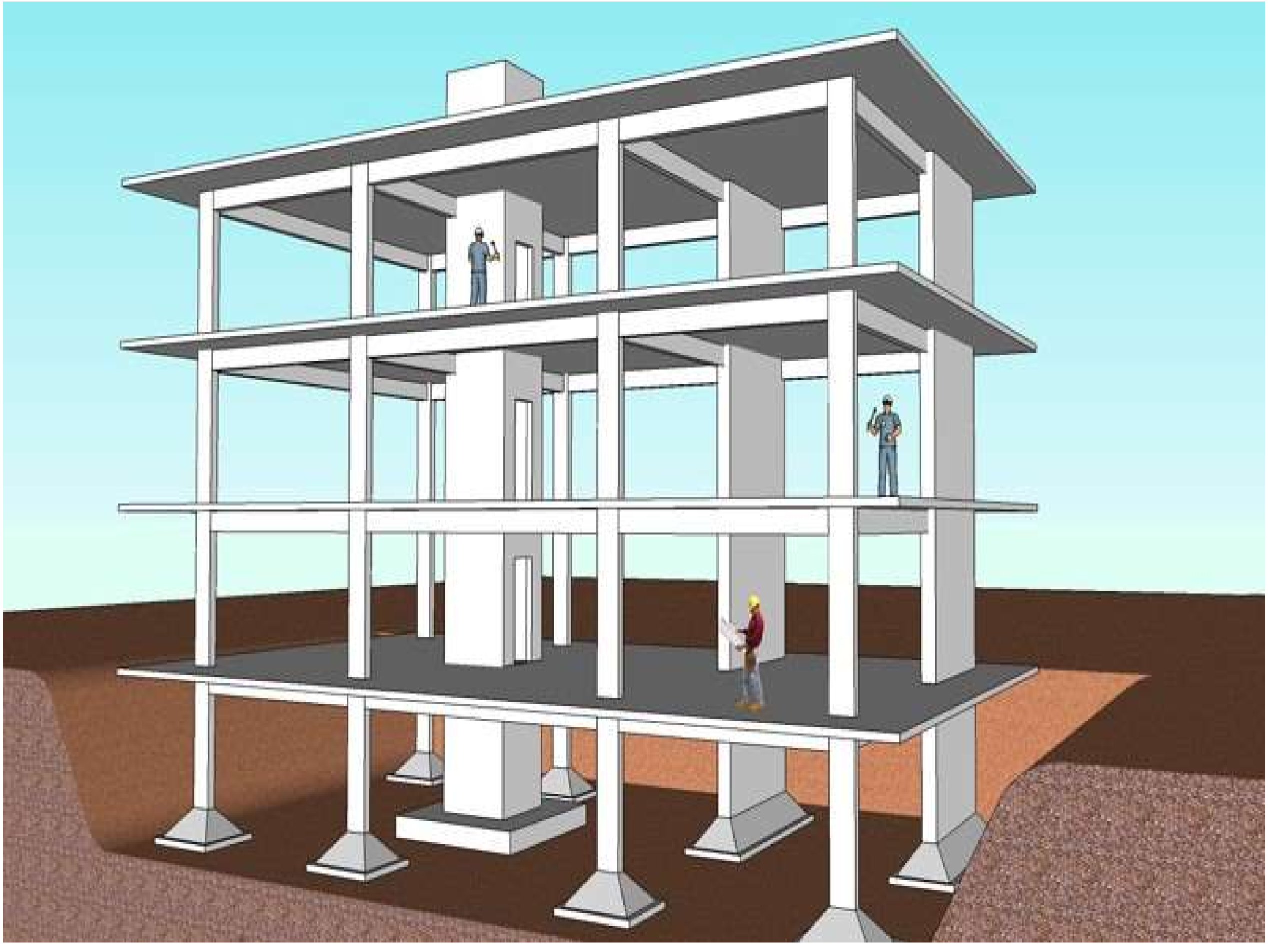
EAST elevation :



South elevation:



1.2 STRUCTURAL DESCRIPTION



1.2.1 Introduction

Frames:

Introduction of Moment Frame:

The **Moment Frames** are the fundamentals structure which consists of two-dimensional series of interconnected members which are joined with each other.

It is very important that the structure should have a resistance to the lateral moment. Moment frames are one of the best frames which have a strong resistance towards the lateral moment.

The designing of the Building to resist the **seismic loads** is a little bit expensive as compared to other designs. **Moment frame** is one of the best options which is widely used in the construction industry to design the structure in earthquake-prone areas.

What Is Moment Frame?

The **Moment Frame** is basically an assembly that consists of **beams** and **columns**. The Beams and columns of the assembly are rigidly connected with each other.

In the **Moment frames**, there is a resistance to the lateral forces which is provided by the rigid frame action with the development of the Bending moment and Shear force in the members and joints.

Moment frames mainly consist of a series of beams and columns where the attachments are formed with the help of welding and bolting. This type of connection are known as **Moment connections.**

Moment frames are structurally designed to carry the **horizontal** as well as **vertical loads** in the same plane. The **Moment frames** are the special type of frames that are rigidly connected with each other of its components so that it will able to resist the overturning forces and Lateral forces acting on it because of the **Bending moment** and **shear strength**.

The overall strength of the **moment frames** while design it to carry the seismic loads mainly depends upon the stiffness and strength of the members.

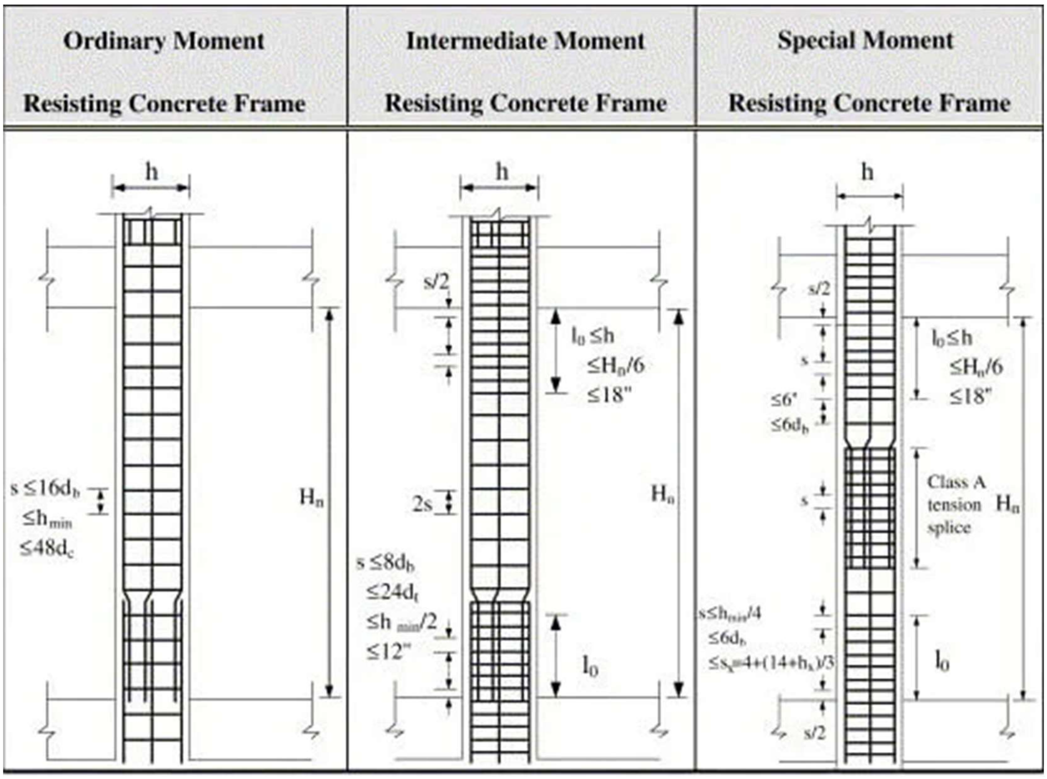
Moment frames are transferred through the connections and it relies mainly on the rigid connections which will help to transfer the lateral loads to the underneath strata of the foundation.

The **analysis of the Moment frames** is a little bit complex as compared to other types of frames and the construction cost required for moment frames also expensive.

The one of the advantages of moment frames that they have more capability of deformation and very little stiffness as compared to other types of Frames.

Moment frames are used to resist the earthquake loads and the construction of the moment frames are not infilled with the materials such as concrete or another masonry which major changes the behavior of the moment frames.

Types of Moment Frames:



There are various types of moment Frames which are as follows.

1. Ordinary Moment Frame (OMF)

This Types of Portal frame system provides very little resistance towards the lateral movement and so it is only used for zero or low seismic areas.

2. Intermediate Moment Frame(IMF)

The intermediate moment frames are designed in such a way that it will release the Limited inelastic deformations in the Moment Frames. The inelastic deformation in the moment frames are developed due to the lateral forces.

3. Special Moment Frame (SMF)

Special moment frames connections are one of the strongest and stable connections that are specially designed to which stand the inelastic deformation in the members when it undergoes the lateral forces.

What Do Moment Resisting Frames Do?

In **moment resisting frames**, the joints or connections between columns and beams **are** designed to be rigid. This causes the columns and beams to bend during an earthquake, so these structural members **are** designed to be strong in bending.

What Is a Moment Frame Structure?

A **moment frame** is a special type of **frame** that uses rigid connections between each of its constituent members. This configuration is able to resist lateral and overturning forces because of the bending **moment** and shear strength that is inherent in its members and the connecting joints.

What Is Concrete Moment Frame?

Reinforced **concrete** special **moment frames** are made up of beams, columns, and beam-column joints. The **frames** are proportioned and detailed to resist flexural, axial, and shearing actions that result as a building sway through multiple displacement cycles during earthquake ground shaking.

Shear walls:

They are structural load-bearing elements that resist vertical and horizontal forces located on them and are mainly used to resist horizontal loads such as wind and earthquake forces.

These walls are armed with two layers of steel to increase their efficiency to resist the horizontal forces. The two directions taking into consideration that the distance between the center of resistance formed by the shear walls in each direction and the center of gravity of the building is minimal. And that these walls are sufficient to prevent or reduce the generation of torque waves and their effects on the walls of the building resisting horizontal forces.

1.2.2 **Loads Acting on the Building**

1.2.2.1 dead loads

Dead loads consist of the weight of all materials of construction incorporated into the building .

1.2.2.2 live load

Live loads are those loads produced by the use and occupancy of the building

1.2.2.3 wind load

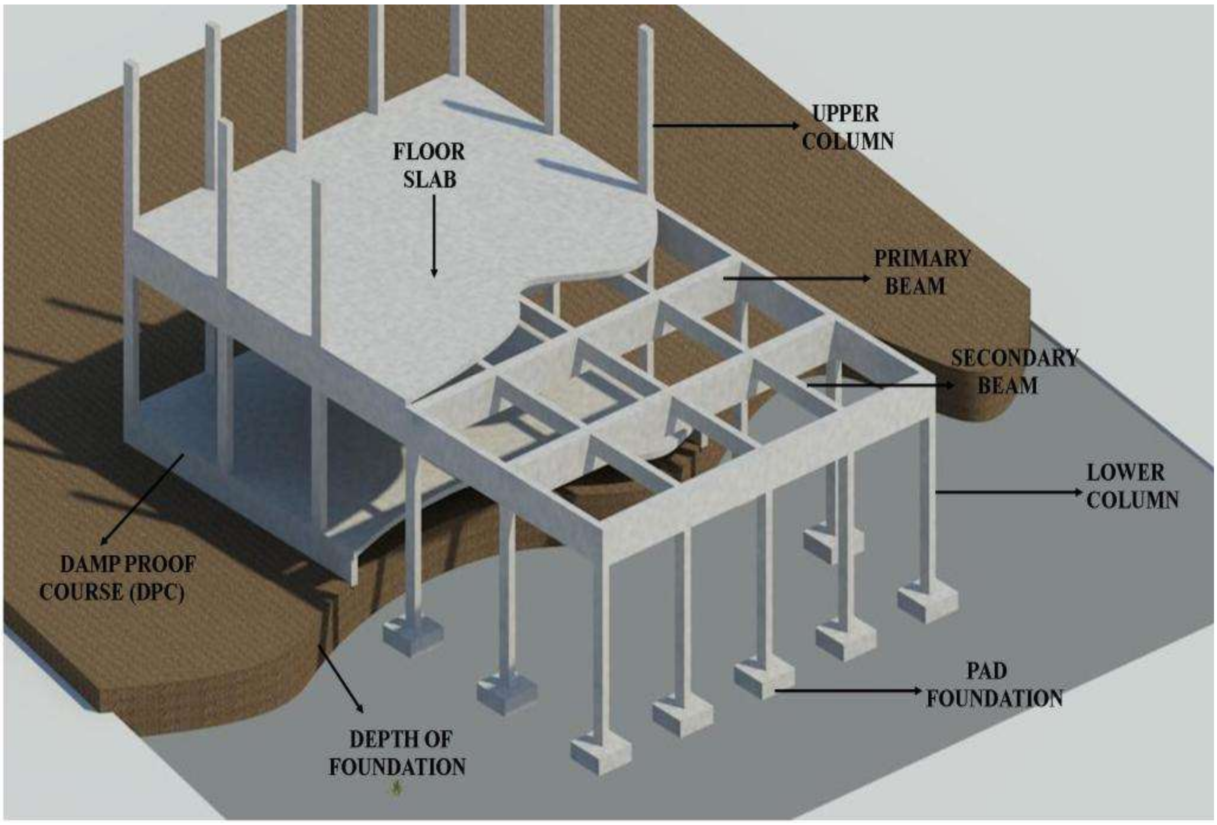
They are horizontal forces that affect the building and their effect appears in tall buildings.

1.2.2.4 seismic load

horizontal and vertical forces that generate torque, and can be resisted by using shear walls designed with thicknesses and sufficient reinforcement to ensure the safety of the building.

1.2.3 **Structural Elements of the Building**

All buildings usually consist of a set of structural elements that work together to maintain the continuity of the building and its suitability for human use, The most important of these slabs, beams, columns, and load-bearing walls, etc. are being defined.



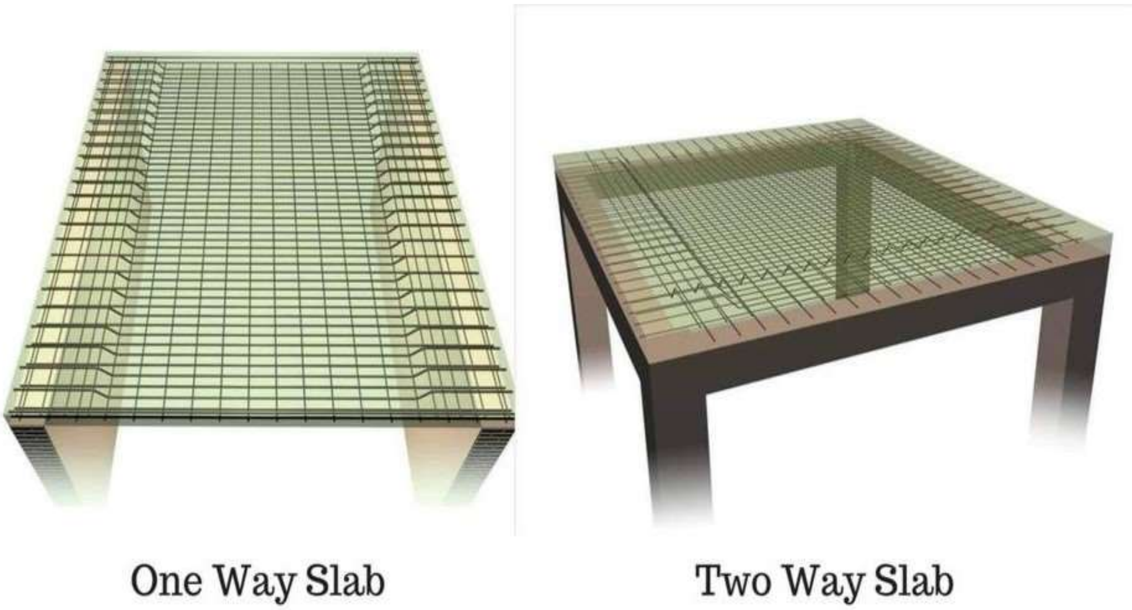
1.2.3.1 Slabs

Structural elements are capable of delivering vertical forces due to the loads affecting the building's load-bearing structural elements such as beams, columns, and walls, without distortions.

There are many different Structural systems of reinforced concrete slabs, including the following:

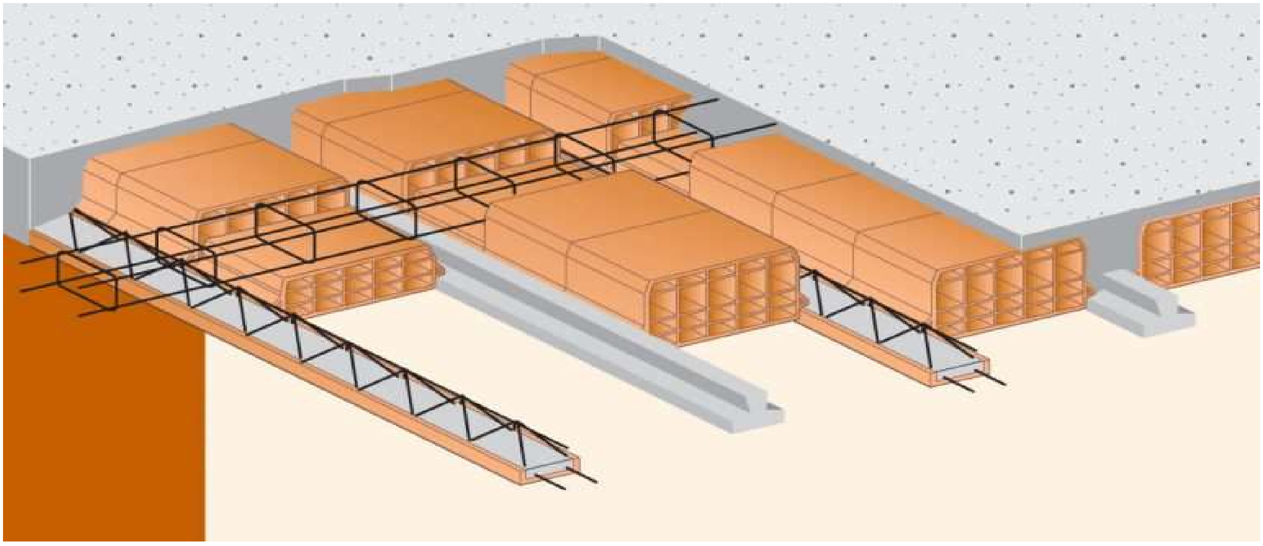
1.2.3.1.1 Solid slab (one or two way)

Solid Slabs are fully customizable concrete slabs of varying width, length, and thickness. They can be used in a variety of applications such as bridges, piers, and building floors. It is known that solid slabs should be supported by drop beams.



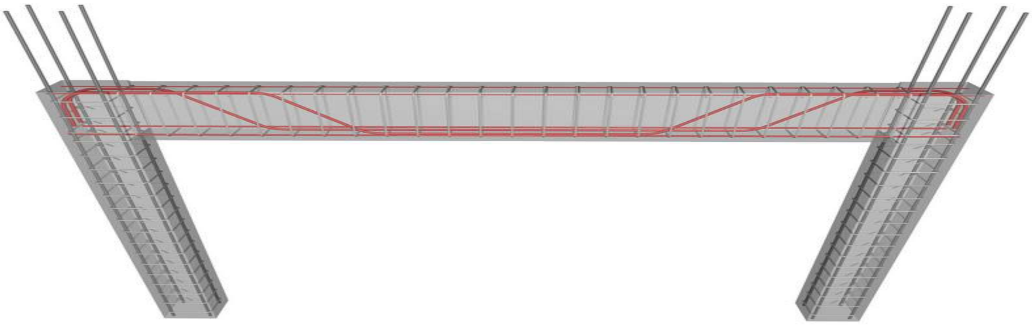
1.2.3.1.2 Ribbed slab (one or two way)

It's the most common system used in Palestine. They are made up of wide band beams running between columns with narrow ribs spanning the orthogonal direction. Normally the ribs and the beams are the same depth. A thin topping slab completes the system. It can be designed to carry loads either in one direction only, or in two directions.



1.2.3.2 Beams

They are basic structural elements in transferring loads from slabs to the columns, and they are of two types, hidden inside the slab and Dropped Beams that emerge from the slab from the bottom.



1.2.3.3 Columns

Columns are the main member in transporting loads from slabs and beams to foundations, and as such, they are a necessary structural component for conveying loads and building stability. Therefore, they must be designed to be able to carry and distribute the loads on them.

1.3 Work Procedure

To achieve the objectives of the project following steps were followed :

1. Architectural study in which the site, building plans, and elevations were been studied.
2. Structural planning of the building, in which the location of columns, beams, and shear walls was determined to fit with architectural design.
3. Structural study in which all structural members were identified and different loads were been estimated.
4. Starting analysis and design for elements according to the ACI Code.
5. Preparation of Structural drawings of all existing elements in the building.

1.3.1 programs used

There are several computer programs used in this project:

1. Microsoft Office: text writing and project output.
2. AUTOCAD 2019: for detailed drawings of structural elements.
3. ATIR18: Structural design and analysis of structural elements.
4. Etabs17:design of structural elements.
5. Safe16: design of slabs & footings .

CHAPTER 2

Previous Studies & Differences

There is a lot of studies that have talked about this subject, but in general cases or virtual system.

And there we have some of them and their results and the difference between these studies and our study.

1-Seismic Performance Comparison of Mid-Rise Moment Resisting Frame And Shear Wall System. By (**Mojtaba Harati** , **Hassan Moghaddam**)[5].

Study summary:

In this study, seismic performance of the concrete mid-rise buildings, with and without shear walls have been evaluated. It seems that in the buildings with shear walls, the earthquake deformation demands are decreased dramatically. These decreases include both of the plastic hinge rotations and inter-story drift ratios at once. In some few cases, damages in beams of the aforementioned structural systems have been witnessed

2-Performance of frames with shear walls for seismic loads in low And Medium Rise Buildings. By (Hirebet Sharada Bai , S V Venkatesh) [6] .

Study summary:

In this study, Different LLRS consisting of external and internal shear wall were studied in the form of parametric study in this paper. Of all the LLRS considered internal shear wall seems to be a more suitable LLRS. However, LLRS consisting of external shear wall is a good proposition in the event of retrofitting medium rise-building for seismic load, when initially designed only for gravity loads.

3-Design and Cost Comparison Between Frame & Shear Walls Structural Systems for Multi Story Buildings. By (**Tarek Edrees Saaed**) [7].

Study summary:

In this study, it seems that shear walls will be done in shorter time than frame system , and the clear space will be decreased in frame system because of drop beams and the amount of concrete will be decreased in shear wall system on the other Hand the amount of reinforce bars increase .

Methodology:

In these studies, they used virtual systems with symmetry dimensions which help with calculations and study the case in general, and lots of factor have been assumed and this make these studies far away from the real cases.

Our study methodology:

Our study stands on real case, that is applicable on ground with architectural design and we used all the factors for seismic based on the place that our building will be constructed at it. In order to compare the two structural systems; moment resistant frame system, and dual system shear walls system . Two typical models were analyzed in CSI ETABS 2016 software, and tested for earthquake loads according to the ACSI.

Study steps:

Step 1 - We started with frame system, we distributed the frames according to the architectural plans, then we find out the dimension of the frame according to the code.

Step 2 – we modeled frame system then we calculated the loads that acting on the system.

Step 3 – we find the coefficient that causing the earthquake force.

Step 4 – we have the area of steal for each frame, then we draw each frame with it reinforce.

Step 5 – we do the same thing for the shear wall system.

Step 6 – we took information for our study like story drift, amount of concrete and amount of steal.

CHAPTER 3

STRUCTURAL ANALYSIS AND DESIGN (By Calculations)

3.1 Introduction

3.2 Factored load

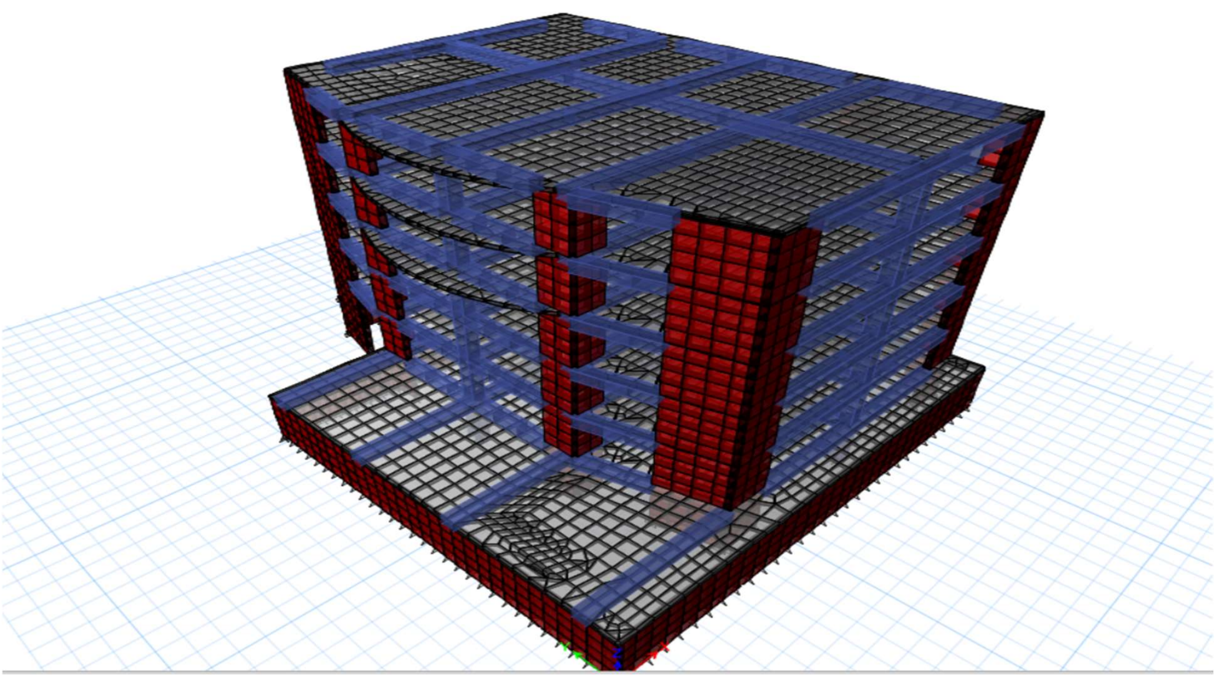
3.3 Determination of slab thickness

3.4 Design of topping

5.5 Design of one-way ribbed slab

5.6 Design of Beam B1

5.7 Design of Two-way ribbed slab



3.1 Introduction:

Concrete is the only major building material that can be delivered to the job site in a plastic state. This unique quality makes concrete desirable as a building material because it can be molded to virtually any form or shape.

A bond forms between the steel and the concrete, and stresses can be transferred between both components. The design strength provided by a member flexure, and load, and shear is taken as the nominal strength calculated in accordance with the requirements and assumptions of ACI-code.

NOTE:

*Concrete B300, {fc'= 24 MPa for rectangular and L section}.

*The specified yield strength of the reinforcement {fy = 420MPa}.

3.2 Factored load

The structure may be exposed to different loads such as dead and live loads. The value of the load depends on the structure type and the intended use. The factored loads on which the structural analysis and design is based for our project members, is determined as follows:

$q_u = 1.2DL + 1.6L$

ACI – 318 - 14 (9.2.1)

3.3Manual calculations:

3.3.1 Determination of slab thickness

Determination of Thickness for One Way Ribbed Slab:

According to ACI-Code-318-08, the minimum thickness of no prestressed beams or one-way slabs unless deflections are computed as follow:

The maximum span length for one end continuous (for ribs):

h_{min} for one-end continuous = L/18.5

=590 /18.5 = **31.8 cm**

The maximum span length for both end continuous (for ribs):

h_{min} for both-end continuous = L/21

= 660/21 = **31.4 cm**

Select Slab thickness **h= 35cm** with **block 27 cm** & **Topping 8cm.**

Load calculations:

One-way ribbed slab:

For the one-way ribbed slabs, the total dead load to be used in the analysis and design is calculated as in the following table:

Table (5 – 1) Calculation of the total dead load for one-way rib slab.

Parts of Rib	Density	Calculation
RC. Rib	25	0.27*0.14*25= 0.945 KN/m
Top Slab	25	0.08*0.54*25 = 1.08 KN/m.
Plaster	22	0.03*0.54*22 = 0.356 KN/m.
Block	12	0.4*0.27*12= 1.296 KN/m
Sand Fill	17	0.07*0.54*17= 0.643 KN/m
Tile	23	0.03*0.54*23 = 0.373KN/m

Mortar	**22**	**0.03*0.54*22 =0.356 KN/m.**
partition	**-**	**2.3*0.54 =1.242 KN/m**

Nominal Total Dead load = **6.3 KN/m** of rib

Nominal Total live load =5*0.54=**2.7 KN/m** of rib

3.3.2 Design of topping

The calculation of the total dead load for the topping is shown below:

Table (5 – 2) Calculation of the total dead load on topping

No.	Material	Calculation
1	**Tile**	**0.03*23*1= 0.69 KN/m**
2	**mortar**	**0.03*22*1= 0.66 KN/m**
3	**Coarse sand**	**0.07*17*1= 1.19 KN/m**
4	**topping**	**0.08*25*1 = 2.0 KN/m**
5	**Interior partitions**	**2.3 *1 =2.3 KN/m**
Sum		**6.84 KN/m**

W_u = 1.2 DL + 1.6 LL

= 1.2 * 6.3+ 1.6 * 5 = 15.56 KN/m^2.   (Total Factored Load)

3.3.3 Design of one-way Ribbed slab

Material: -

concrete B300 Fc' = 24 N/mm^2

Reinforcement Steel fy = 420 N/mm^2

Section: -

b =14cm b_f=54cm

h =35cm T_f=8 cm

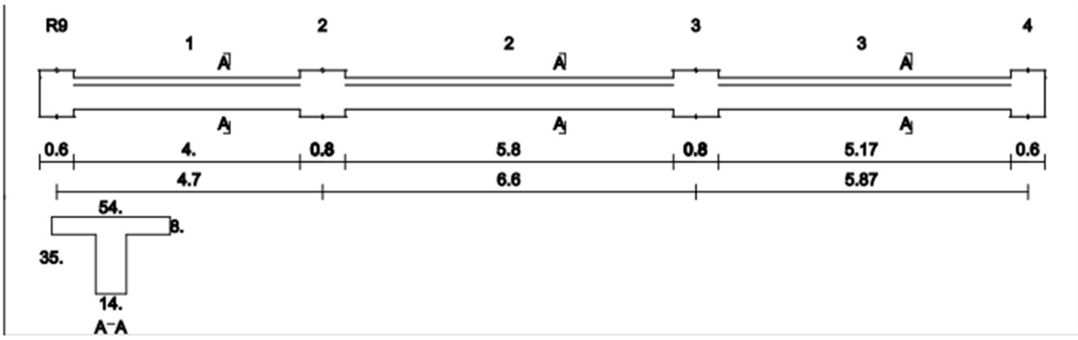


Figure (5-1): Rib geometry.

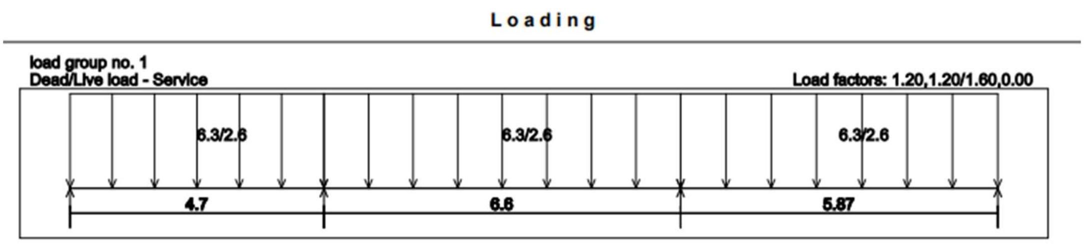


Figure (5-2): loading of rib (9)

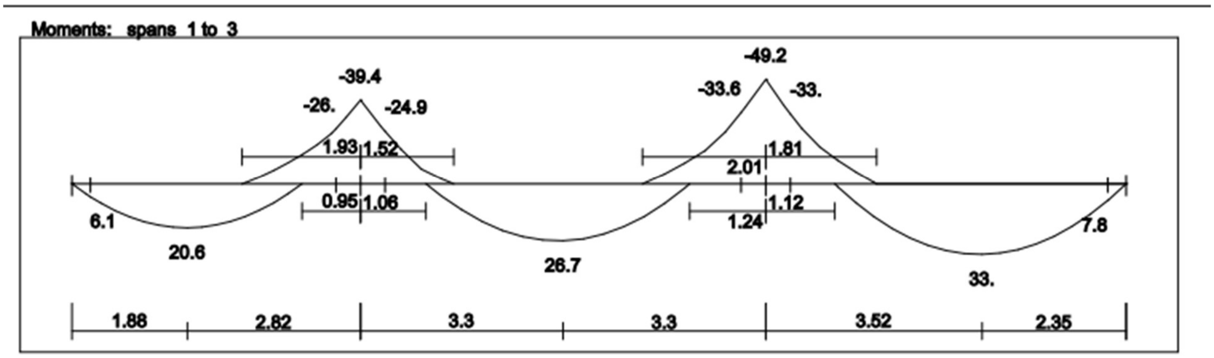


Figure (5-3): Moment Envelop of rib (9)

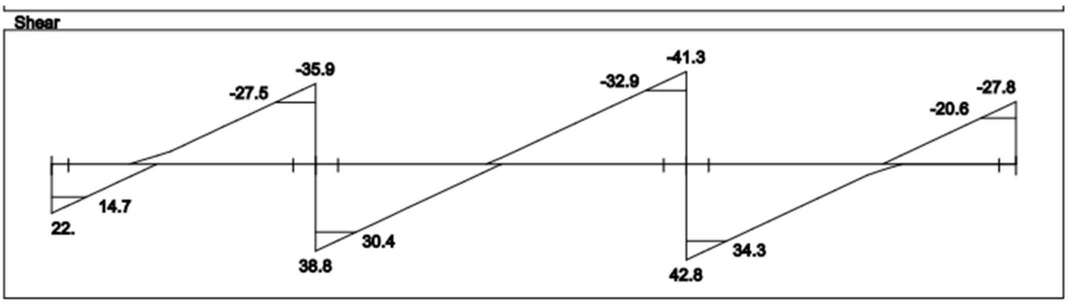


Figure (5-4): Shear Envelop of rib (9)

Design of flexure: -

5.5.1.1 Design of Positive moment of rib (RIB 9):

d = depth - cover – diameter of stirrups – (diameter of bar/ 2)

$= 350 - 20 - 10 - \frac{12}{2} = 314$ mm.

$\rightarrow M_{u\ max} = 49.2$ KN.m

be ≤ Distance center to center between ribs = 540 mm……….. Controlled.

≤ Span/4 = 6600/4 = 1650 mm.

≤ $(16 * t_f) + b_w = (16 * 80) + 140 = 1420$ mm.

→b_E= 540 mm.

$$\rightarrow M_{nf} = 0.85\, f_c' * b_E * t_f * \left(d - \frac{t_f}{2}\right)$$

$$= 0.85 * 24 * 0.54 * 0.08 * \left(0.314 - \frac{0.08}{2}\right) * 10^3 = 241.47\ KN.m$$

ϕM_{nf} = 0.9 * 241.47 = 217.32 KN.m

→ϕM_{nf} =217.32> $M_{u\ max}$= 49.2 KN.m.

∴ **DESIGN AS RECTANGULAR SECTION**.

1) Maximum positive moment $Mu^{(+)}$ **= 33 KN.m**

M_n = Mu /ϕ =33/ 0.9 = 36.66 KN.m

$$m = \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*24} = 20.58$$

$$R_n = \frac{M_n}{b*d^2} = \frac{36.66*10^6}{540*\,(314)^2} = 0.688 \text{ MPa}$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}\right) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2*0.688*20.58}{420}}\right) = 0.00165$$

→$A_s = \rho$ * b *d = 0.00165 * 540 * 314= 280.51 mm^2.

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \;\geq\; \frac{1.4}{f_y} * b_w * d \quad \text{…………(ACI-10.5.1)}$$

$$= \frac{\sqrt{24}}{4*420} * 140 * 314 \;\geq\; \frac{1.4}{420} * 140 * 314$$

= 128.2 mm^2<146.53 mm^2 ………… Larger value is control.

→As_{min} = 146.53 mm^2<As_{req} = 280.51 mm^2.

∴ As = 280.51 mm^2.

2 Φ14 =307.88 mm^2>As_{req} = 280.51 mm^2 . OK.

∴ **Use 2 Φ14**

→ **Check for strain:-($\varepsilon_s \geq 0.005$)**

Tension = Compression

$A_s * fy = 0.85 * f_c' * b * a$

307.88* 420 = 0.85 * 24 * 140 * a

a = 45.28 mm.

f_c' = 24 MPa< 28 MPa→ $\beta_1 = 0.85$

$c = \frac{a}{\beta_1} = \frac{45.28}{0.85}$ = 53.27 mm.

$\varepsilon_s = \frac{d-c}{c} * 0.003$

$= \frac{313 \quad .27}{53.27} * 0.003 = 0.0146 > 0.005$ ∴ ϕ =0.9 … OK!

Maximum negative moment $Mu^{(-)}$ = 49.2 KN.m

M_n = Mu /ϕ =49.2/ 0.9 =54.66 KN.m

$m = \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*24} = 20.58$

$R_n = \frac{M_n}{b*d^2} = \frac{54.66*10^6}{540*\,(314)^2}$ = 1.02 MPa

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}) = \frac{1}{20.58}\left(1 - \sqrt{1 - \frac{2*1.02*20.58}{420}}\right) = 0.002509$

→$A_s = \rho * b * d = 0.002509 * 140 * 314 = 110.302\ mm^2$.

$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \ \geq \ \frac{1.4}{f_y} * b_w * d$ ………..(ACI-10.5.1)

$= \frac{\sqrt{24}}{4*420} * 140 * 314 \ \geq \ \frac{1.4}{420} * 140 * 314$

$= 128.2\ mm^2 < 146.53\ mm^2$ …………. Larger value is control.

→$As_{min} = 146.53\ mm^2 \geq As_{req} = 55.35\ mm^2$.

∴ $As = 146.53\ mm^2$.

2 Φ*12* =226 mm^2>$As_{req} = 146.53\ mm^2$. OK.

∴ Use 2 Φ12

4.6.2 Design of shear of rib (RIB 1):

1) Vu = 32.9 KN.

$V_c = \frac{\sqrt{f_c'}}{6} * b_w * d$

$= 1.1 * \frac{\sqrt{24}}{6} * 0.14 * 0.313 * 10^3 = 39.36$ KN.

$\phi V_c = 0.75 * 39.36 = 29.52$ KN.

→Check for Cases: -

1- <u>Case 1:</u> $V_u \leq \frac{\phi V_c}{2}$.

$32.9 \leq \frac{29.52}{2} = 14.76$

∴ Case (1) is NOT satisfied

2- <u>Case 2:</u> $\frac{\phi V_c}{2} < V_u \leq \phi V_c$

$14.76 \leq 32.9 \leq 29.52$

∴ Case (2) is NOT satisfied → shear reinforcement is required.

Vs = $\frac{V_u}{\phi} - Vc = 4.77$

Vs max = $\frac{2}{3} * \sqrt{f_c'} * d * b_w = \frac{2}{3} * \sqrt{24} * 140 * 313 * 10^{-3} = 143.11$

Vs′ = $\frac{Vs\ max}{2} = 71.56$

Vs min= $\frac{1}{16} * \sqrt{f_c'} * b_w * d = 13.42$

Vs min= $\frac{1}{3} * b_w * d = 14.61$ ....**Control.**

<u>Try 2Φ8: -</u>

$\frac{100.5*420*313}{S} = 14.61*10^3 \rightarrow S = 904.3$ mm.

$S \leq \frac{d}{2} = \frac{313}{2} = 156.5$ mm. … **Control**

≤ 600 mm.

∴ Use 2Φ8 @ 15 Cm

3.3.4 Design of Beam 2

Material: -

concrete B300 Fc' = 24 N/mm^2

Reinforcement Steel fy = 420 N/mm^2

Section: -

B = 80 cm

h =50 cm "choose h=50 , for deflection requirments L/240"

According to ACI-Code-318-08, the minimum thickness of no prestressed beams or one way slabs unless deflections are computed as follow:

h_{min} for one end cont. = L/18.5

=600/18.5 = 32.4 cm.

→Select Total depth of beam **h=50cm. (35cm slab and 15 cm drop)**

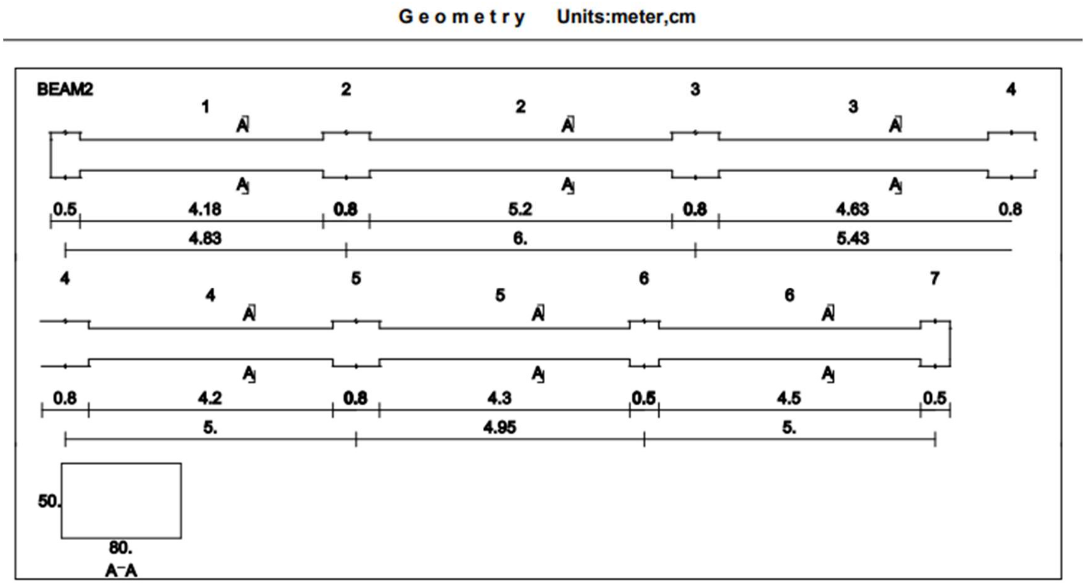


Figure (5-5): Beam Geometry.

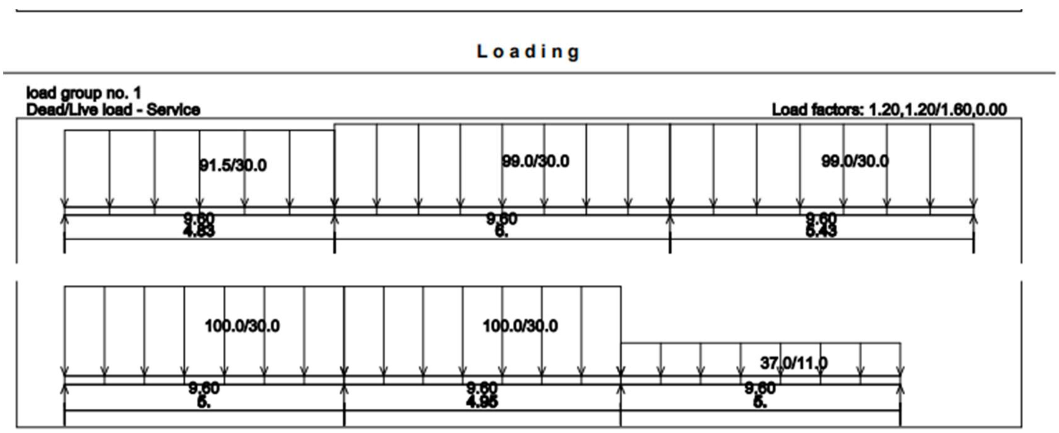


Figure (5-6): Load of Beam (B 2)

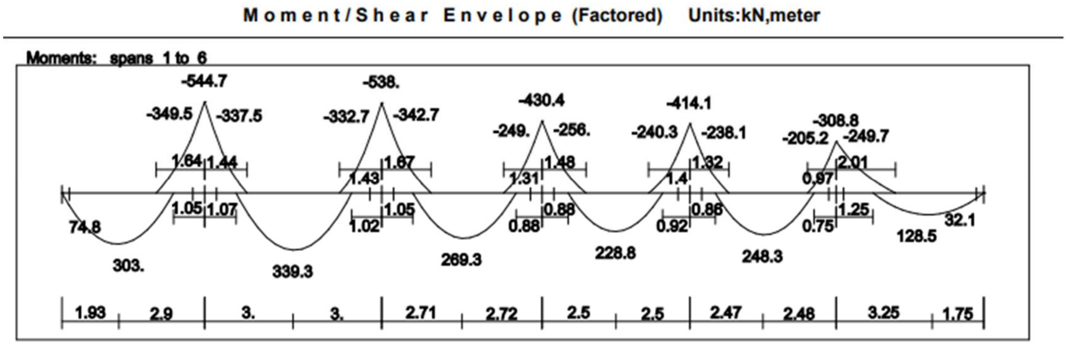


Figure (5-7): Moment Envelop for Beam (B 2)

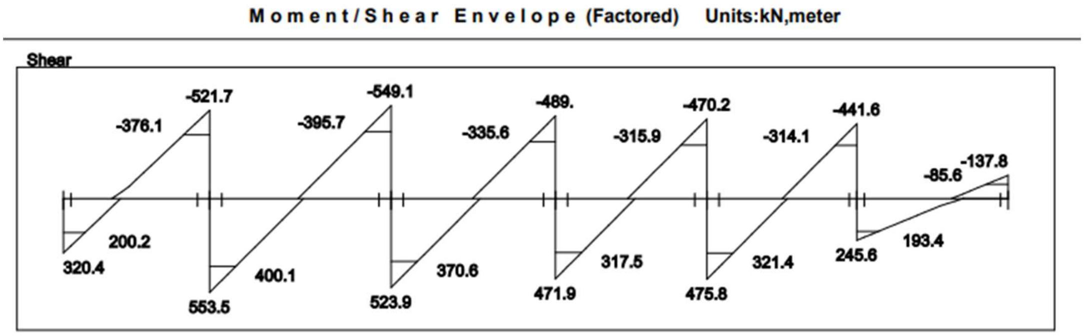


Figure (5-8): Shear Envelop for Beam (B 2)

5.7.1 Design of flexure: -

5.7.1.1 Design of Positive moment: -

→Mu_{max} =544.7 KN.m

b_w = 80 Cm.   h =50 Cm.

d = depth - cover – diameter of stirrups – (diameter of bar/ 2)

$=500 - 40 - 10 - \frac{18}{2} = 441mm$

$C_{max} = \frac{3}{7} * d = \frac{3}{7} * 441 = 189$ mm.

$f_c' = 24$ MPa < 28 MPa→$\beta_1 = 0.85$

$a_{max} = \beta_1 * C_{max} = 0.85 * 189 = 160.65$mm.

*Note:

$M_{n max} = 0.85 * f_c' * b * a * (d - \frac{a}{2})$

$= 0.85 * 24 * 0.8 * 0.161 * (0.441 - 0.161/2) * 10^3$

=947.22 KN.m

$\epsilon_s = 0.004$

$\phi = 0.65 + \frac{250}{3} * (0.004 - 0.002) = 0.82$

→ϕMn_{max} = 0.82 * 947.22=776.72 KN.m

→ Mu = 544.7 KN.m< ϕMn_{max} 776.72 KN.m

∴Singly reinforced concrete section.

1) Maximum positive moment $Mu^{(+)}$ = 339.3 KN.m

Mn = Mu /ϕ= 339.3/ 0.9 =377 KN.m .

→m=20.58

$R_n = \frac{M_n}{b*d^2} = \frac{377*10^6}{800*(441)^2} = 2.423$ MPa

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$

$\frac{1}{20.6}\left(1-\sqrt{1-\frac{2*2.42*20.58}{420}}\right)$=0.006159

$A_s = \rho * b * d$ =0.006159*800*441=2173.16 mm^2

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b * d \;\; \geq \; \frac{1.4}{f_y} * b * d$$

$$\frac{\sqrt{24}}{4 * 420} * 800 * 441 \;\; \geq \; \frac{1.4}{420} * 800 * 441$$

=1028.7mm^2<1176mm^2 …. Larger value is CONTROL

As= 1947.46 mm^2

Use Φ20…. As=314.15mm^2

#of bars= (2173.16/314.15) =8 bars

∴ **Use 8 Φ20…As=2513.2** > 2173.16mm^2

→ **Check for strain:-($\varepsilon_s \geq 0.005$)**

Tension = Compression

A_s * fy = 0.85 * f_c' * b * a

2513.2* 420 = 0.85 * 24 * 800 * a

a = 64.6 mm.

f_c' = 24 MPa< 28 MPa→ β_1 = 0.85

$c = \frac{a}{\beta_1} = \frac{64.67}{0.85}$ = 76.09 mm.

$\varepsilon_s = \frac{d-c}{c}$ * 0.003

$= \frac{441-7\;\;.09}{61.64}$ * 0.003 = 0.0177 > 0.005 ∴ ϕ =0.9 … OK!

5.7.2 Design of shear: -

1) Vu = 400 KN .

ϕVc = ϕ * $\frac{\sqrt{f_c'}}{6}$ * b * d

$= 0.75 * \frac{\sqrt{24}}{6} * 800 * 441 * 10^{-3} = 216.044$KN.

\→ **Check For Cases:-**

1- <u>Case1 :</u>

$V_u \leq \frac{\phi V_c}{2}$.

$400 \leq \frac{216.04}{2} = 108.02$

∴ Case (1) is NOT satisfied

2- <u>Case 2 :</u>

$\frac{\phi V_c}{2} < V_u \leq \phi V_c$

108.08 <400≤216.04

∴ Case (2) is NOT satisfied

3- <u>Case 3 :</u> $\phi V_c < V_u \leq \phi V_c + \phi Vs_{min}$

$\phi\ Vs_{min} \geq \frac{\phi}{16}\sqrt{f_c'} * b_w * d = \frac{0.75}{16}\sqrt{24} * 0.8 * 0.441 * 10^3 = 81.01$KN.

$\geq \frac{\phi}{3} * b_w * d = \frac{0.75}{3} * 0.8 * 0.441 * 10^3 = 88.2$KN …. CONTROL.

$\therefore \phi Vs_{min} = 88.2$ KN.

$\phi V_c + \phi Vs_{min} = 216.04 + 88.2 = 304.24$KN.

$\phi V_c < V_u \leq \phi V_c + \phi Vs_{min}$

216.04<400≤ 304.24 NOT OK

∴ Case (3) is NOT satisfied → $\left(\frac{Av}{S}\right) = \frac{Vs}{(fy_t * d)}$.

4- <u>Case 4 :</u> Φ (V_c + $V_{s\,min}$) < V_u ≤ Φ(V_c + V_s')

$V_s = \left(\frac{Vu}{\phi} - V_c\right)$

$V_c = \frac{216.04}{0.75} = 288.05KN$

$V_s = \left(\frac{400}{0.75} - 288.05\right) = 245.283$ KN.

$V_{s\,min} = 117.6$ KN.

$V_s' = \frac{1}{3} * \sqrt{f_c'} * b_w * d = 576.119$ KN.

$\Phi (V_c + V_{s\,min}) < V_u \leq \Phi(V_c + V_s')$

= 0.75(288.05 + 117.6) < 400 ≤ 0.75 (288.05 + 576.119)

304.23 < 400 ≤ 648.12

∴ **Case (4) is satisfied**.

Try 4Φ8 =4 * 50.26= 201.061 mm^2.

$\frac{201.061}{S} = \frac{245.283 * 10^{-3}}{(420 * 441)} \longrightarrow$ s = 151.82 mm …. CONTROL

$s \leq \frac{d}{2} = \frac{441}{2} = 220.5$ mm

≤ 600 mm.

∴ **Use Φ8 @ 10 Cm 4L.**

3.3.5 Design of Column (C22)

Calculation of Loads act on Column (C22)

Loads acting on columns are obtained from support reaction when analyzing the system on etabs

Dead Load = (Service Dead reaction from etabs)

=2225 KN

Live Load = (Service Live reaction from etabs)

=592 KN

Loads acting on column (C8) are as follows:

Factored loads (Pu) = 3617.2

Calculation of Required Dimension of Column (C8)

Total load Pu =3617.2 KN

Pn = 3617.2 /(0.65) = 5564.92 KN

$\rho g = 2.0\ \%$

$\mathrm{Pn} = 0.8 * \mathrm{Ag}\{0.85 * \mathrm{fc}' + \rho\mathrm{g}(\mathrm{fy} - 0.85\mathrm{fc}')\}$

$5564.92 * 10^{-3} = 0.8 * \mathrm{Ag}[0.85 * 24 + 0.02 * (420 - 0.85 * 24)]$

$\mathrm{Ag} = 2451\ cm^2$

∴**Select 60*60cm with Ag = 3600 cm².**

- **Check Slenderness Effect :**

For braced system if $\lambda \le 34 - 12\frac{M1}{M2} \le 40$, then column is classified as short column and slenderness effect shall not be considered.

$$\lambda = \frac{Klu}{r}$$

Where :

Lu: Actual unsupported (unbraced) length = 3.65 m

K: effective length factor (K= 1 for braced frame).

R: radius of gyration → for rectangular section = $\sqrt{\frac{I}{A}}$ 0.3 h

System about X

$\rightarrow \lambda = \frac{1 * 3.65}{0.3 * 0.6} = 20.28$

$\lambda \leq 34 - 12(1) = \mathbf{22} \leq 40$

$\lambda = 20.28 < 22$ ∴ Short about X .

System about Y

$\rightarrow \lambda = \frac{1 * 3.65}{0.3 * 0.6} = 20.28$

$\lambda \leq 34 - 12(1) = \mathbf{22} \leq 40$

∴ Column is Short , So Slenderness effect will not be considered.

Calculation of Required Reinforcement Ratio

Since Column is short and slenderness effect will not be considered, then Design Strength of column can be calculated using the following equation :

$\phi Pn = 0.65 * 0.8 * Ag\{0.85 * fc' + \rho g(fy - 0.85fc')\}$

Where , Pu =3617.2 KN

$3617.2 * 10^3 = 0.65 * 0.8 * 600 * 600\{0.85 * 24 + \rho g(420 - 0.85 * 24)\}$

$\Rightarrow \rho_g = 0.02034 > \rho_{min} = 0.01 \ \& < \rho_{max} = 0.08$

As req = 0. 02034 *600 *600 =7325.6 mm^2

Use Φ 20 >> $\# of\ bar = \frac{7325.6}{314.15} = 24\ bar$

∴ **Use 24 Ø 20 with As = 7539.822 mm² >As $_{req}$ = 7325.6 mm²**

- **Check spacing between the bars :**

$S = \frac{600- \ \ *40-2*10-7*20}{6} = 60\ mm$

$S = 60\ mm \ \ \geq 40mm$

$\geq 1.5db = 37.5\ mm$

Determination of Stirrups Spacing

According to ACI :

$\text{Spacing} \leq 16 \times d_b(\text{Longitudinal. bar. diameter}) = 16 \times 2 = 32\ \text{cm.}$

$\text{Spacing} \leq 48 \times d_t(\text{tie. bar. diameter}) = 48 \times 1.0 = 48\text{cm.}$

$\text{Spacing} \leq \text{Least. dim e nsion} = 60\ \text{cm}$

∴ **Select Ø 10/20cm**

Column (C22) Section is shown in figure(4-11) where bars arrangement and stirrups detailing appear :

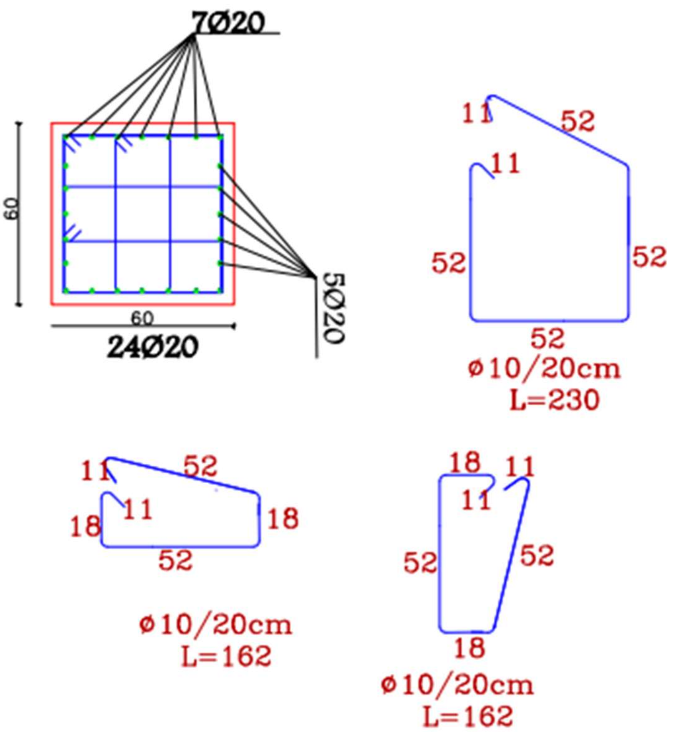


Figure (5- 9): C22 Reinforcement Details

3.3.6 Design of Isolated Footing

Loads that act on footing are :

- PD =7494.5 kN , PL = 1730 kN → Pu = 1.2 * 7494.5 + 1.6*1730= 11761 kN

The following parameters are used in design:

- γ_{concrete} = 25 kN/m^3
- γ_{soil} = 18 kN/m^3
- σ_{allow} = 450 kN/m^2
- clear cover = 7.5 cm

Determination of footing dimension (a)

Footing dimension can be determined by designing the soil against bearing pressure.

→ Assume h = 100 cm

→ $\sigma_{b(allow)net}$ = 450 – 25*1 = 425 kN/m²

→ $\sigma_{\text{bu(allow . net)}}$ = 1.4* 425 = 595 kN/m²

→ $\sigma_{bu} = Pu/_{Areq} \leq \sigma_{\text{bu(allow . net)}}$

∴ $11761/a^2$ = 602 → **a=4.3m** → **Select a=4.5m**

→ $Bearing\ Pressure\ \sigma bu = Pu/_A = \frac{11761}{4.5*4.5}$ **= 580.1 kN/m^2** ≤ 602 kN/m² **…. (SAFE)**

Determination of footing depth (h)

To determine depth of footing both of one and two way shear must be designed.

Design of one way shear

→ $d = h - cover - \emptyset = 1000 - 75 - 18 = 907\ mm$

→ Vu at distance d from the face of column :

Vu = $FRB = \sigma bu \times 0.493 \times b$

$= 580.7 \times 0.943 \times 4.5 = 2464.2\ kN$

→ $\emptyset * Vc = 0.75 * \frac{1}{6} * \sqrt{fc'} * b * d$

$= 0.75 * \frac{1}{6} * \sqrt{24} * 4500 * 907$ =2500 kN >Vu

∴ **h = 100 cm is correct** ✓

Design of Punching (two-way shear)

→ $d = 907\ mm$
→ $b_{\circ} = 2\times 1706 + 2\times 1706 = 6824$ mm
→ Bc = 1
→ αs = 40 (interior column)

Ø×Vc is the smallest of :

1. $Vc = \left(2 + \frac{4}{Bc}\right) \times \frac{\sqrt{fc'}}{12} \times b_o \times d$

$= \left(2 + \frac{4}{1}\right) \times \frac{\sqrt{24}}{12} \times 6824 * 907$

$= 15160.8$

2. $Vc = \left(\frac{\alpha_s \times d}{b_o} + 2\right) \times \frac{\sqrt{fc'}}{12} \times b_o \times d$

$= \left(\frac{40 \times 907}{6824} + 2\right) \times \frac{\sqrt{24}}{12} \times 6824 \times 907$

$= 14385$ kN

3. $Vc = 4 \times \frac{\sqrt{fc'}}{12} \times b_o \times d$

$= 4 \times \frac{\sqrt{24}}{12} \times 6824 \times 907 =$ **10107 kN.** ∢ **cont.**

→ Ø×Vc = 0.75 × 10107 = **7580.25** kN > Vu = **5159.7** kN

∴ **h = 100 cm is correct** ✓

Design of Reinforcement

Mu = 580.7 *1.85*4.5*(1.85/2) = 4466.3 kN.m

→ $m = \frac{Fy}{0.85 * Fc'} = \frac{420}{0.85*24} = 20.6$

→ Mn = = 4962.62 kN.m

→ Rn = $\frac{Mn}{b*d^2}$ = $\frac{4962.62 *10^6}{4500*907^2}$ = 1.34 MPa

→ $\rho = \frac{1}{m} * (1- \sqrt{1 - \frac{2*Rn*m}{Fy}}) = 0.0033$

→ Asreq = $\rho * b * d = 0.0033 * 4500 * 907 =$ 13466.9 mm^2

→ As (min) = 0.0018*b*h = 0.0018*4500 * 1000 = 8100 mm^2

→ Asreq > As (min)

∴ **Select for both directions: 44Ø20@15cm with As = 13816 mm^2 > Asreq … (ok)**

3.4Programs calculations:

Mapped acceleration parameters.
The mapped maximum considered earthquake spectral response acceleration at short periods (S_S) shall be 0.281 g and at 1-second period (S_1) shall be 0.073 g. The mapped long-period transition period (T_L) shall be 6 seconds.

Site class definitions.
Based on the site soil properties, the site shall be classified as either Site Class A, B, C, D, E or F in accordance with Table 1613.5.2. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the commissioner or geotechnical data determines that Site Class E or F soil is present at the site.

TABLE 1613.5.2
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, SEE SECTION 1613.5.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m², 1 pound per square foot = 0.0479 kPa. N/A = Not applicable

Site coefficients and risk-targeted maximum considered earthquake (MCE R) spectral response acceleration parameters. The MCER spectral response acceleration parameters for short periods, S_{MS} , and at 1-second period, S_{M1}, adjusted for site class effects shall be determined by Equations 16-47 and 16-48, respectively:
$S_{MS} = F_a S_s$ **(Equation 16-47)**
$S_{M1} = F_v S_l$ **(Equation 16-48)**
where:

F_a = Site coefficient defined in Table 1613.5.3(1).

F_v = Site coefficient defined in Table 1613.5.3(2).

S_s = The mapped MCERspectral accelerations for short periods as determined in Section 1613.5.1.

S_l = The mapped MCER spectral accelerations for a 1-second period as determined in Section 1613.5.1.

Table 1613.5.3(1)
Values of Site Coefficient F_a as a Function of Site Class and MappedSpectral Response Acceleration at Short Periods (SS)[a]

Site Class	F_a
A	0.80
B	1.00
C	1.20
D	1.57
E	2.37
F	Note a

a. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values, except that for structures with periods of vibration equal or less than 0.5 second, values of F_a for liquefiable soils are permitted to be taken equal to the values for the site class determined without regard to liquefaction in Section 1613.5.5.

Table 1613.5.3(2)

Values of Site Coefficient F_y as a Function of Site Class and Mapped Spectral Response Acceleration at 1-Second Period (S_1)[a]

Site Class	F_y
A	0.80
B	1.00
C	1.70
D	2.40
E	3.50
F	Note a

a. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values, except that for structures with periods of vibration equal or less than 0.5 second, values of F_v for liquefiable soils are permitted to be taken equal to the values for the site class determined without regard to liquefaction in Section 1613.5.5.

16Design spectral response acceleration parameters.

Five-percent damped design spectral response acceleration at short periods, S_{DS} , and at 1-second period, S_{D1} , shall be determined from Equations 16-49 and 16-50, respectively:

$$S_{DS} = \frac{2}{3} S_{MS} \quad \textbf{(Equation 16-49)}$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad \textbf{(Equation 16-50)}$$

where:

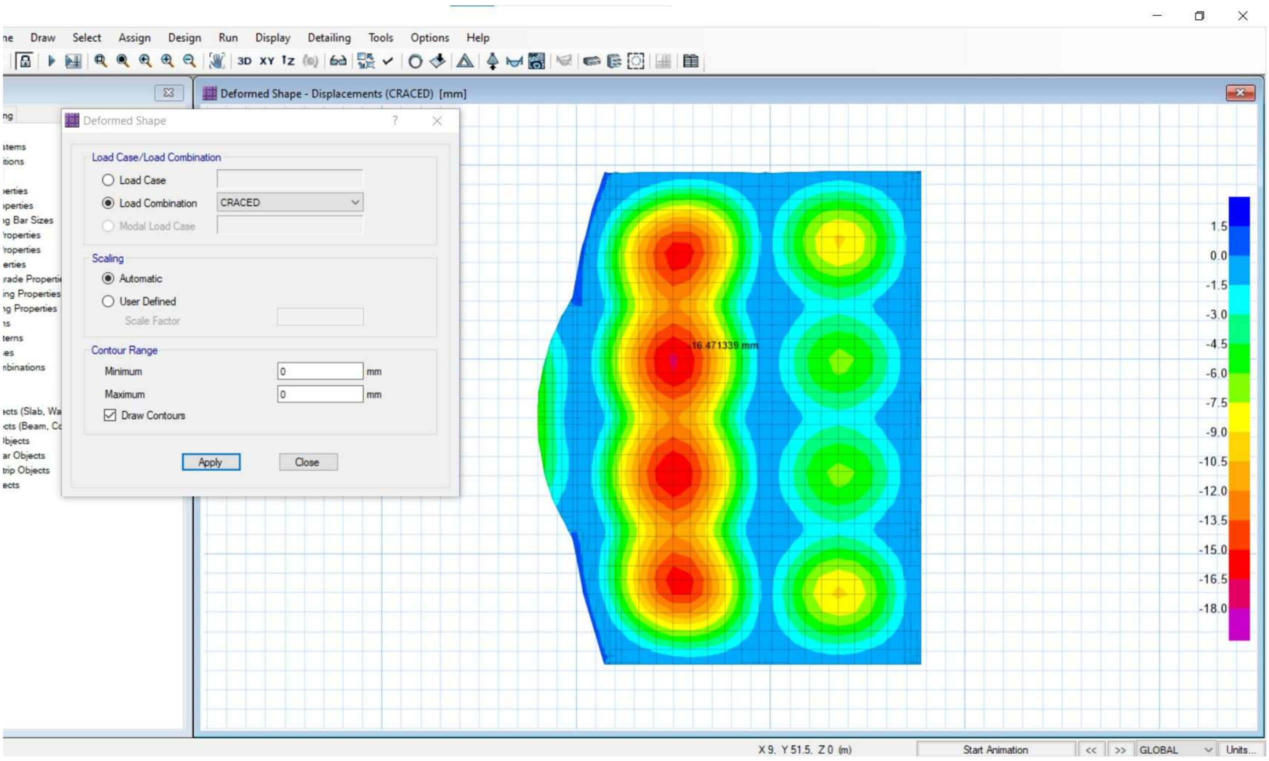
S_{MS} = The MCE_R spectral response accelerations for short period as determined in Section 1613.5.3.

S_{M1} = The MCE_R spectral response accelerations for 1 second period as determined in Section 1613.5.3.

3.4.1 Solid slab design:

Check Long term deflection 4th floor slab :

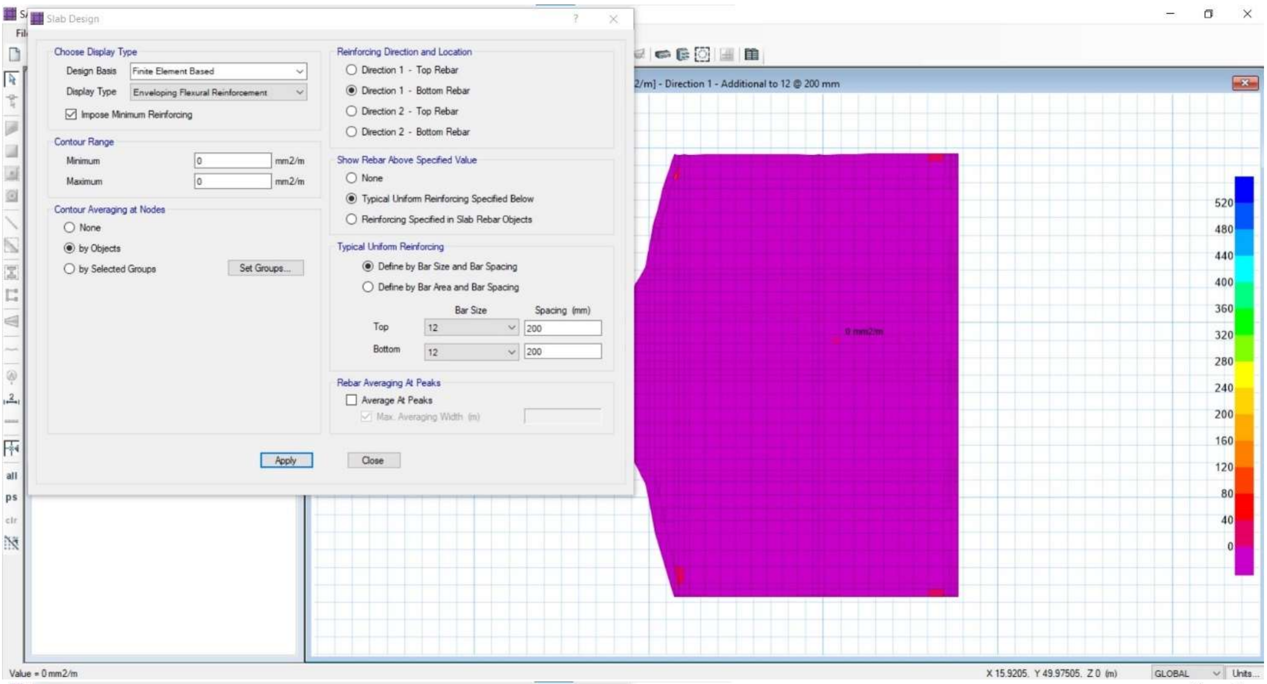
USING SAFE PROGRAM :

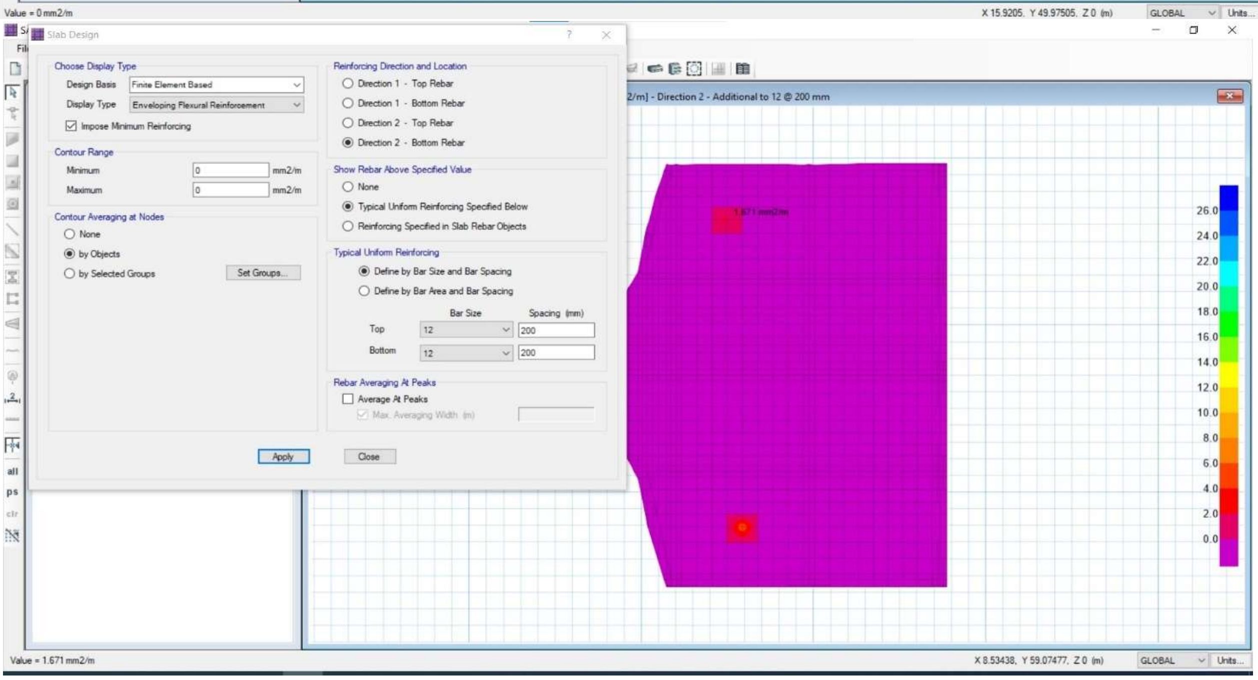


max deflection = $\frac{L}{240} = \frac{7220}{240}$ =30.08mm

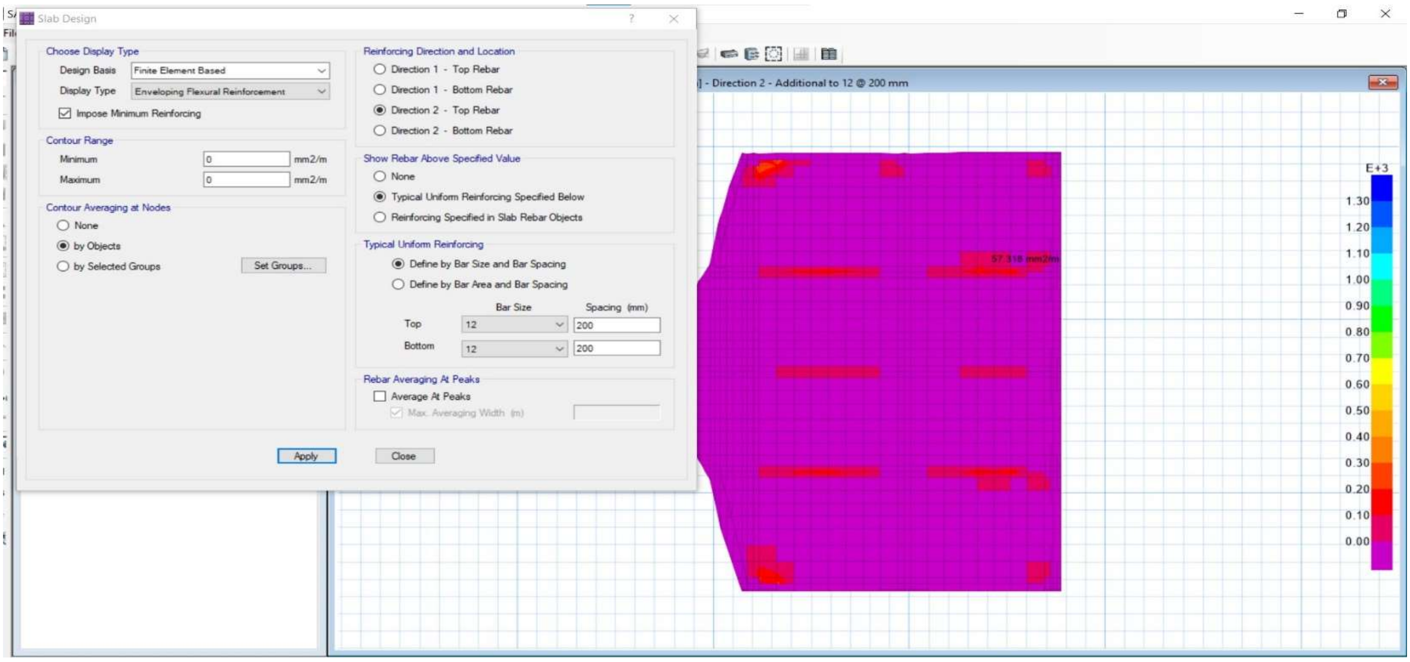
30.08 mm> 23.8 mm , the solid slab thickness are enough

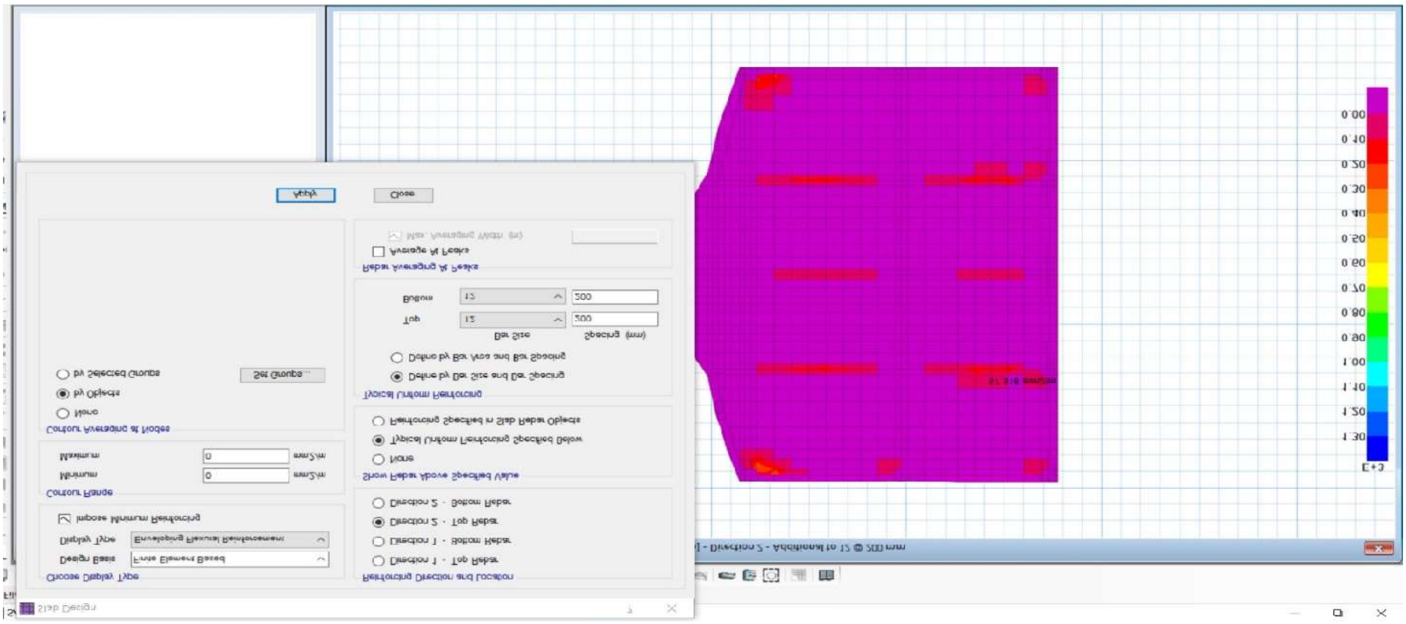
- Flexure analysis: for bottom bars in basement slab
- Basic Mesh bottom reinforcement (ø12@20cm) with AS=565.2 mm^2
- and in short direction chose (ø12@20cm) with AS=565.2 mm^2





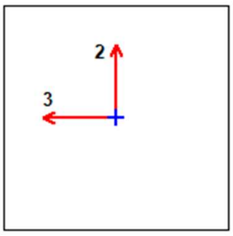
- Flexure analysis for top bars in basement slab
- top reinforcement (ø16@20cm) which equal area / cm 201/0.2 = 1004.8 mm2/m
- and in short direction chose (ø16@20cm)





ETABS Concrete Frame Design

ACI 318-14 Beam Section Design



Beam Element Details (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story2	B9	44	ConcBm	DCon10	3313.1	6483.3	1	Sway Ordinary

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
1000	1000	1000	0	60	60

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
23270	24	1	420	420

Design Code Parameters

φ_T	φ_{CTied}	$\varphi_{CSpiral}$	φ_{Vns}	φ_{Vs}	φ_{Vjoint}
0.9	0.65	0.75	0.75	0.6	0.85

Design Moment and Flexural Reinforcement for Moment, M_{u3}

	Design Moment kN-m	Design P_u kN	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	-1612.8921	-45.9728	4849	61	3086	4849
Bottom (-2 Axis)	0	-45.9728	0	61	81	81

Shear Force and Reinforcement for Shear, V_{u2}

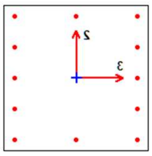
Shear V_{u2} kN	Shear φV_c kN	Shear φV_s kN	Shear V_p kN	Rebar A_v /S mm²/m
611.9756	573.5671	38.4085	632.0721	131.69

Torsion Force and Torsion Reinforcement for Torsion, T_u

T_u kN-m	φT_{th} kN-m	φT_{cr} kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
157.8382	75.187	300.7481	7055.9	3644.4	360.5	3398

ETABS Concrete Frame Design

ACI 318-14 Column Section Design



Column Element Details (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story2	C5	8	ConcCol	DCon13	0	4000	0.4	Sway Ordinary

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
800	800	60	27.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
23270	24	1	420	420

Design Code Parameters

φ_T	φ_{CTied}	$\varphi_{CSpiral}$	φ_{Vns}	φ_{Vs}	φ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
7577.4713	1232.6602	-297.34	297.34	297.34	14819	2.32

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Effective Length mm
Major Bend(M3)	0.738725	1	1	1	3000
Minor Bend(M2)	0.412099	1	1	1	3000

Shear Design for V_{u2} , V_{u3}

	Shear V_u kN	Shear φV_c kN	Shear φV_s kN	Shear φV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	31.4199	671.3781	0	0	0
Minor, V_{u3}	603.9016	671.3781	153.064	0	666.67

ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
Story2	P6	29930	32650.3	2300	400	0.605

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
23270	24	1	420	420

Design Code Parameters

Φ_T	Φ_C	Φ_V	Φ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 mm	Left Y_1 mm	Right X_2 mm	Right Y_2 mm	Length mm	Thickness mm
Top	Leg 1	29930	31500.3	29930	33800.3	2300	400
Bottom	Leg 1	29930	31500.3	29930	33800.3	2300	400

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	2300	0.0025	0.0034	DWal26	1835.0074	88.5064	-151.8257	920000
Bottom	3688	0.004	0.0034	DWal23	791.367	-93.4682	1942.2836	920000

Shear Design

Station Location	ID	Rebar mm²/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	ΦV_c kN	ΦV_n kN
Top	Leg 1	1000	DWal11	1844.3865	-445.6709	653.5698	566.245	1137.1309
Bottom	Leg 1	1000	DWal11	1963.9865	2173.1372	655.8343	683.654	1254.5399

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa	C Depth mm	C Limit mm
Top–Left	Leg 1	378.2	DWal7	3430.0746	-354.4051	4.73	4.8	608.2	511.1
Top–Right	Leg 1	Not Stressed	DWal7	0	0	0	0		
Bottom–Left	Leg 1	343.6	DWal12	3191.4425	-1263.8245	7.05	4.8	573.6	511.1
Botttom–Right	Leg 1	395.6	DWal12	3549.6746	518.4871	5.33	4.8	625.6	511.1

Thickness of all slabs $t = 28cm$

Loads for all slabs Dead load (uniform) $DL = 4.66kN/m^2$

Live load (uniform) $LL = 2.5\ kN/m^2$

Materials concrete B300 $f_c = 25\ Mpa$

reinforcement $f_y = 420\ Mpa$

Self-weight for all elements is auto-calculated.

The following table shows cross section for elements in both models according to the floor:

	STORY	FIRST MODEL	SECOND MODEL
SHEAR WALLS THICKNESS [cm]	1,2,3,4,5,6	40	-
COLUMNS CROSS SECTION [cm x cm]	1,2,3,4,5,6	80X80	80X80
BEAMS CROSS SECTION [cm x cm]	All stories	80X80	100X100

Table 1: Cross section for elements in both models

ASCE 7-16 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQX according to ASCE 7-16, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [ASCE Table 12.8-2]	$C_t = 0.02$ft
Coefficient, x [ASCE Table 12.8-2]	$x = 0.75$
Structure Height Above Base, h_n	$h_n = 78.74$ ft
Long-Period Transition Period, T_L [ASCE 11.4.5]	$T_L = 4$ sec

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	$R = 5$
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]	$\Omega_0 = 2.5$
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	$C_d = 4.5$
Importance Factor, I [ASCE Table 1.5-2]	$I = 1.25$

Ss and S1 Source = 0.75

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.2]	$S_s = 0.56$g
Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.2]	$S_1 = 0.28$g
Site Class [ASCE Table 20.3-1] = A - Hard Rock	
Site Coefficient, F_a [ASCE Table 11.4-1]	$F_a = 0.8$
Site Coefficient, F_v [ASCE Table 11.4-2]	$F_v = 0.8$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.4, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 0.448$g
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.4, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.224$g
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.5, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.298667$g
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.5, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.149333$g

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2]	$C_S = \frac{S_{DS}}{(\frac{R}{I})}$
[ASCE 12.8.1.1, Eq. 12.8-3]	$C_{S,max} = \frac{S_{D1}}{T(\frac{R}{I})}$

[ASCE 12.8.1.1, Eq. 12.8-5]

$$C_{S,min} = \max(0.044 S_{DS} \mathbf{I}, 0.01) = 0.016427$$

[ASCE 12.8.1.1, Eq. 12.8-6]

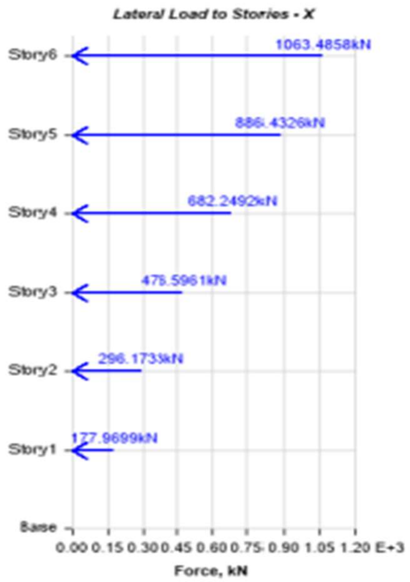
$$C_{S,min} = 0.5 \frac{S_1}{\left(\frac{R}{I}\right)} \text{ for } S_1 = 0.6g$$

$$C_{S,min} \leq C_s \leq C_{S,max}$$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
X	0.847	0.0441	81245.196	3582.9068

Applied Story Forces



Direction = y

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
Y	0.644	0.057945	81245.196	4707.791

Applied Story Forces

ASCE 7-16 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQX according to ASCE 7-16, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [ASCE Table 12.8-2]	$C_t = 0.03\text{ft}$
Coefficient, x [ASCE Table 12.8-2]	$x = 0.75$
Structure Height Above Base, h_n	$h_n = 78.74$ ft
Long-Period Transition Period, T_L [ASCE 11.4.5]	$T_L = 4$ sec

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	$R = 3$
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]	$\Omega_0 = 3$
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	$C_d = 2.5$
Importance Factor, I [ASCE Table 1.5-2]	$I = 1.25$

Ss and S1 Source = 0.75

Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.2]	$S_s = 0.56g$

Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.2]		$S_1 = 0.28g$
Site Class [ASCE Table 20.3-1] = A - Hard Rock		
Site Coefficient, F_a [ASCE Table 11.4-1]		$F_a = 0.8$
Site Coefficient, F_v [ASCE Table 11.4-2]		$F_v = 0.8$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.4, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 0.448g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.4, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.224g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.5, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.298667g$
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.5, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.149333g$

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2] $$C_S = \frac{S_{DS}}{\left(\frac{R}{T}\right)}$$

[ASCE 12.8.1.1, Eq. 12.8-3] $$C_{S,max} = \frac{S_{D1}}{T\left(\frac{R}{T}\right)}$$

[ASCE 12.8.1.1, Eq. 12.8-5] $$C_{S,min} = \max(0.044 S_{DS} I, 0.01) = 0.016427$$

[ASCE 12.8.1.1, Eq. 12.8-6] $$C_{S,min} = 0.5\frac{S_1}{\left(\frac{R}{T}\right)} \text{for} S_1 = 0.6g$$

$$C_{S,min} \le C_s \le C_{S,max}$$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
X	1.168	0.053267	87633.3943	4668.0031

Applied Story Forces

Direction = y

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
Y	0.909	0.068484	87633.3943	6001.4862

Applied Story Forces

Lateral Load to Stories - Y
Story6
Story5
Story4
Story3
Story2
Story1
Base
1802.5563kN
1498.0855kN
1145.0634kN
794.0285kN
487.2731kN
0.00 0.25 0.50 0.75 1.00 1.25 1.50 1.75 2.00 E+3
Force, kN

3.5 Results & comparisons:

ذ

3.5.1 story displacement:

Frame system:

story Displacement EQX

story	elevation	x (mm)	y (mm)
Story6	24	33.18316998	5.716837566
Story5	20	31.04076252	5.679846404
Story4	16	27.43879413	5.619910255
Story3	12	22.69606591	5.535902304
Story2	8	17.22182793	5.381534233
Story1	4	10.583512	4.560667827
Base	0	0	0

story Displacement EQY

story	elevation	x (mm)	y (mm)
Story6	24	2.782	25.128
Story5	20	2.789	23.057
Story4	16	2.816	19.517
Story3	12	2.853	14.815
Story2	8	2.883	9.356
Story1	4	3.905	4.355
Base	0	0	0

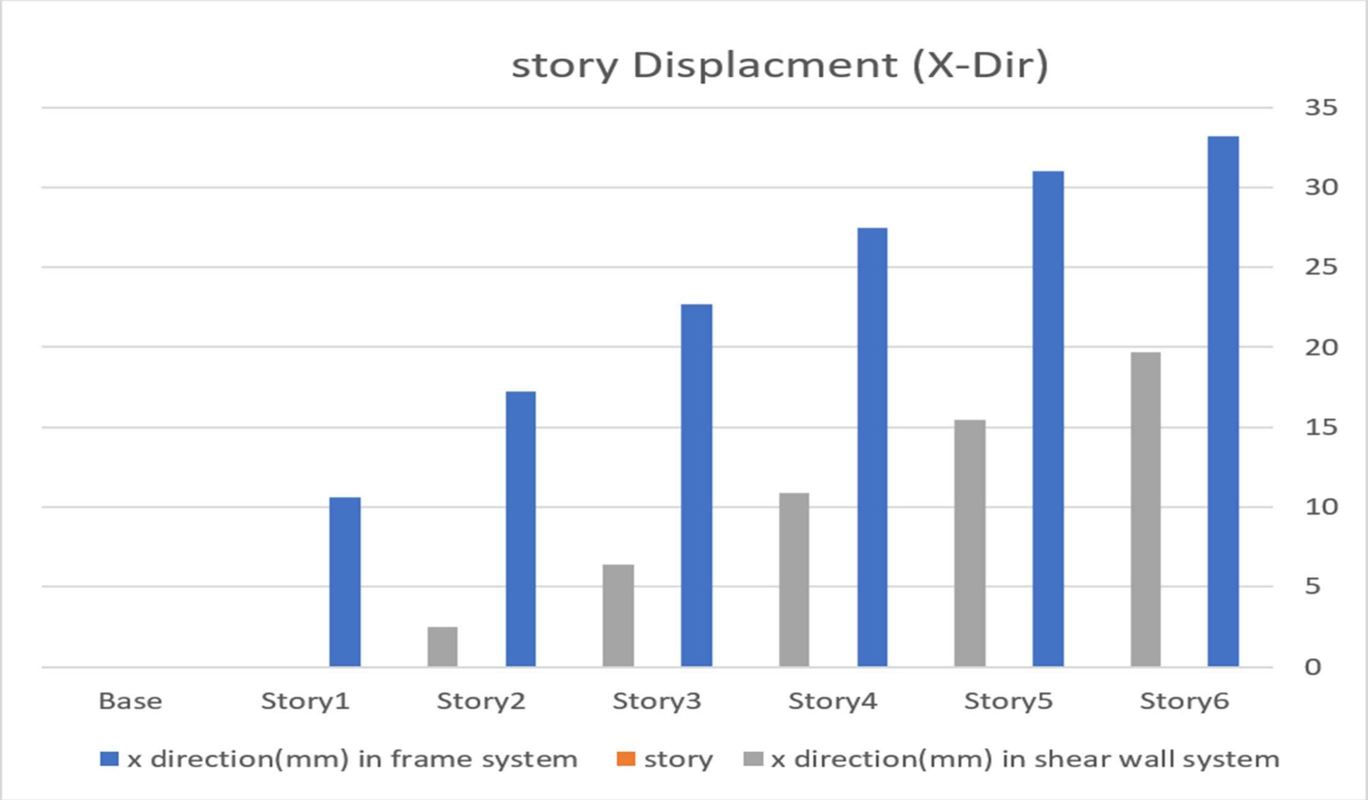
Shear wall system:

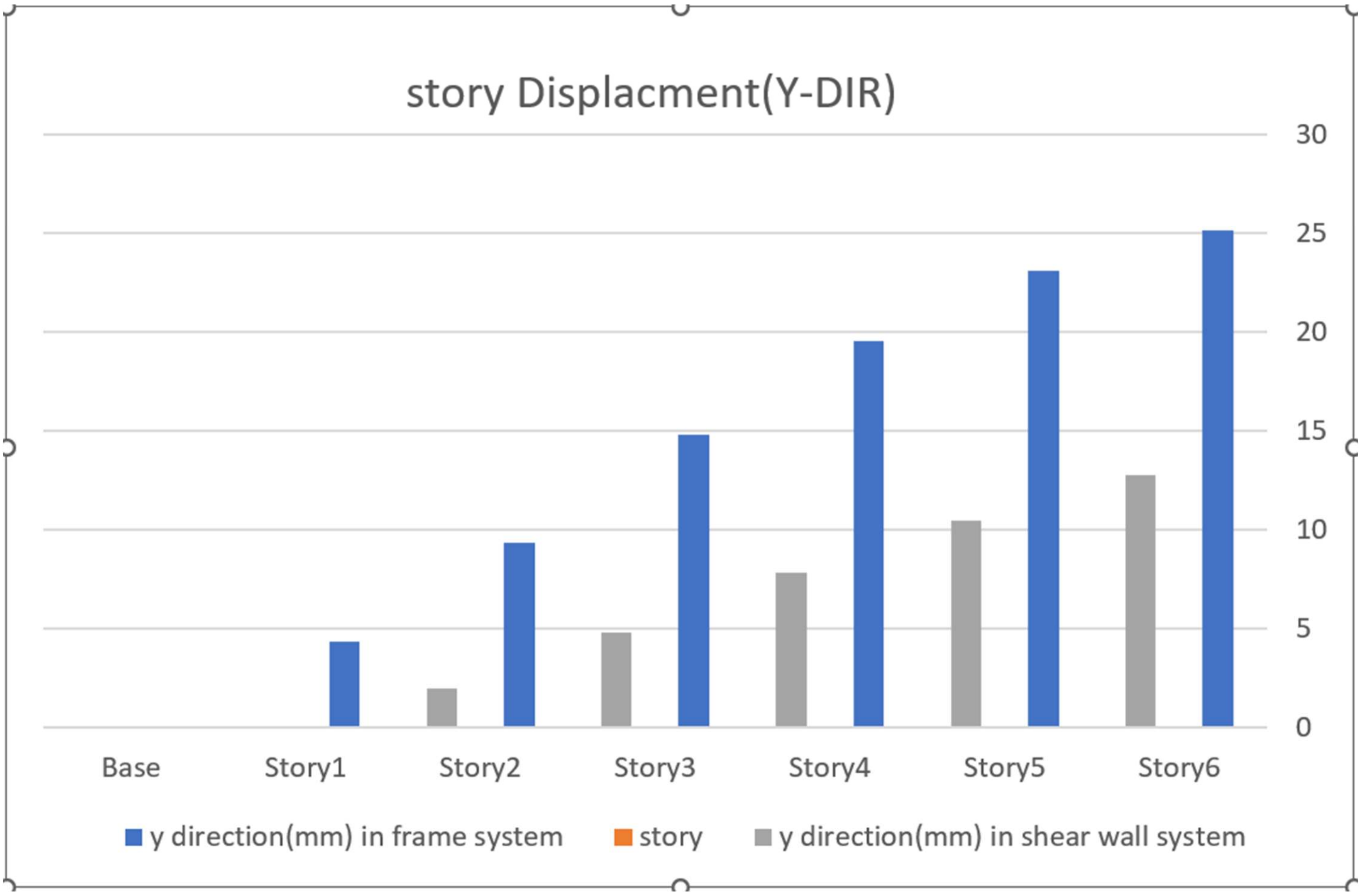
story Displacement EQX

story	elevation	x (mm)	y (mm)
Story6	24	19.705	0.754
Story5	20	15.444	0.656
Story4	16	10.909	0.546
Story3	12	6.406	0.367
Story2	8	2.48	0.207
Story1	4	0	0
Base	0	0	0

story Displacement EQX

story	elevation	x (mm)	y (mm)
Story6	24	19.705	0.754
Story5	20	15.444	0.656
Story4	16	10.909	0.546
Story3	12	6.406	0.367
Story2	8	2.48	0.207
Story1	4	0	0
Base	0	0	0

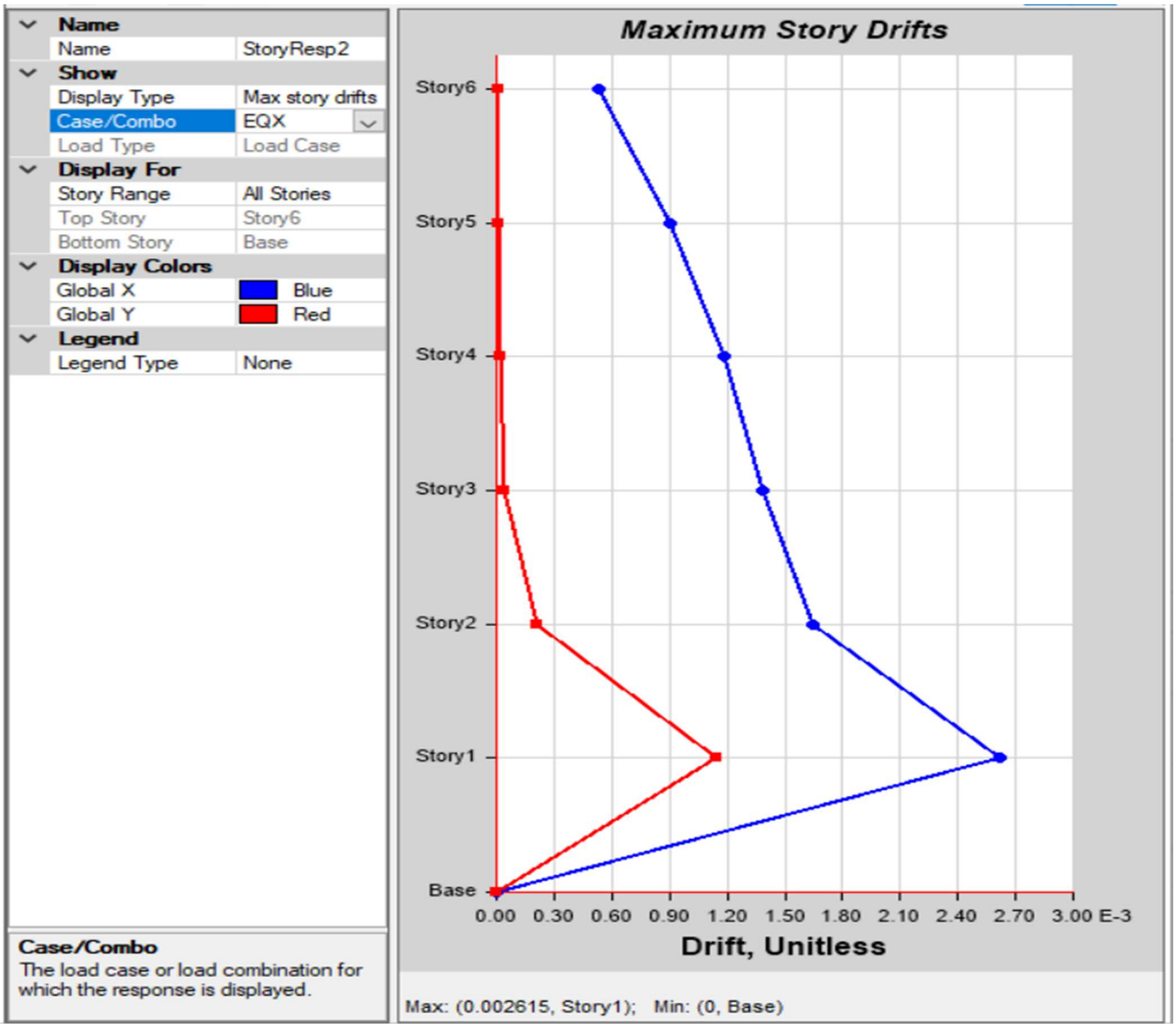




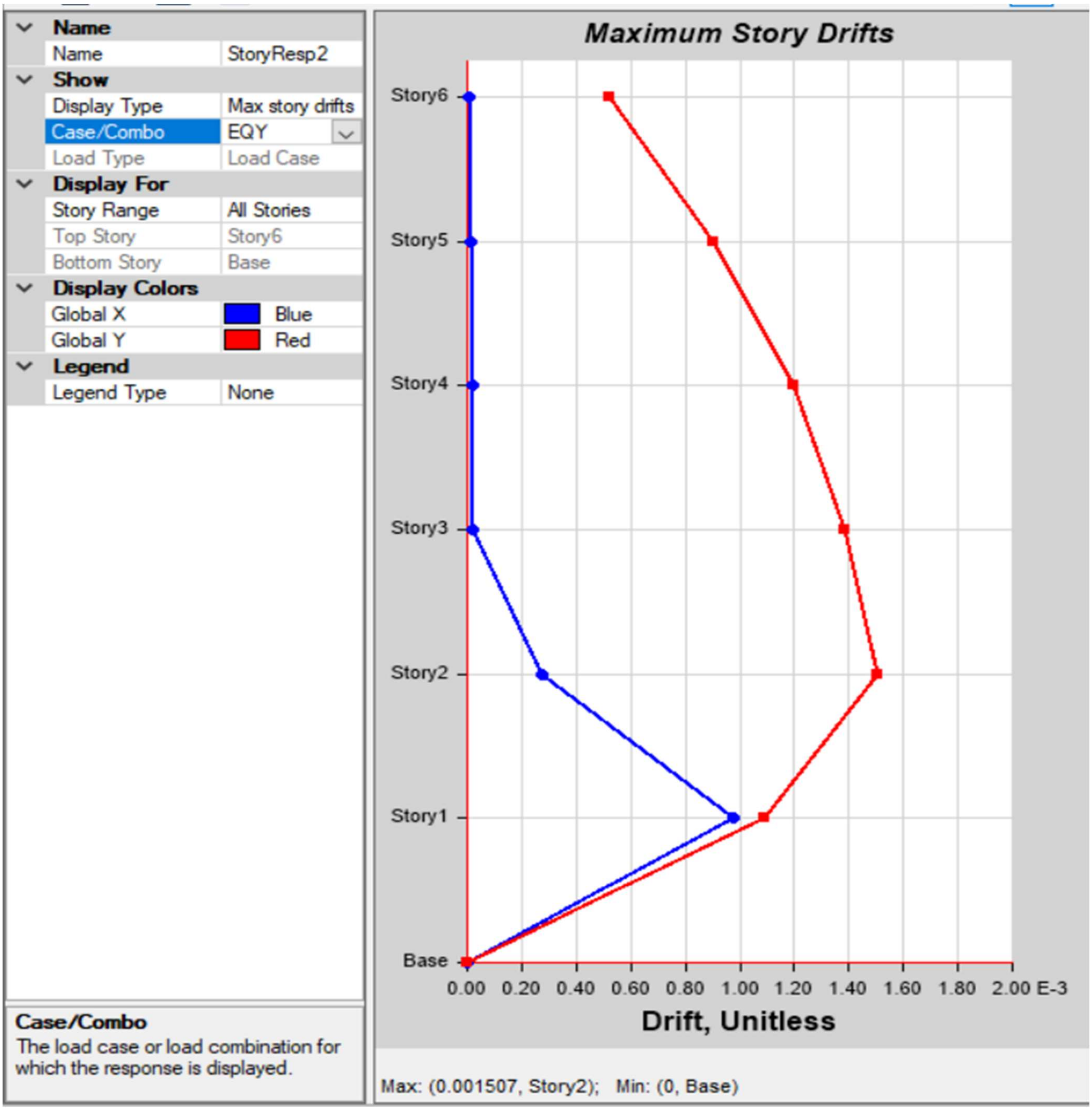
We noticed here that the shear wall system has less story displacement in all stories and that’s a point for the shear wall system.

3.5.2 story drift:

Frame system:

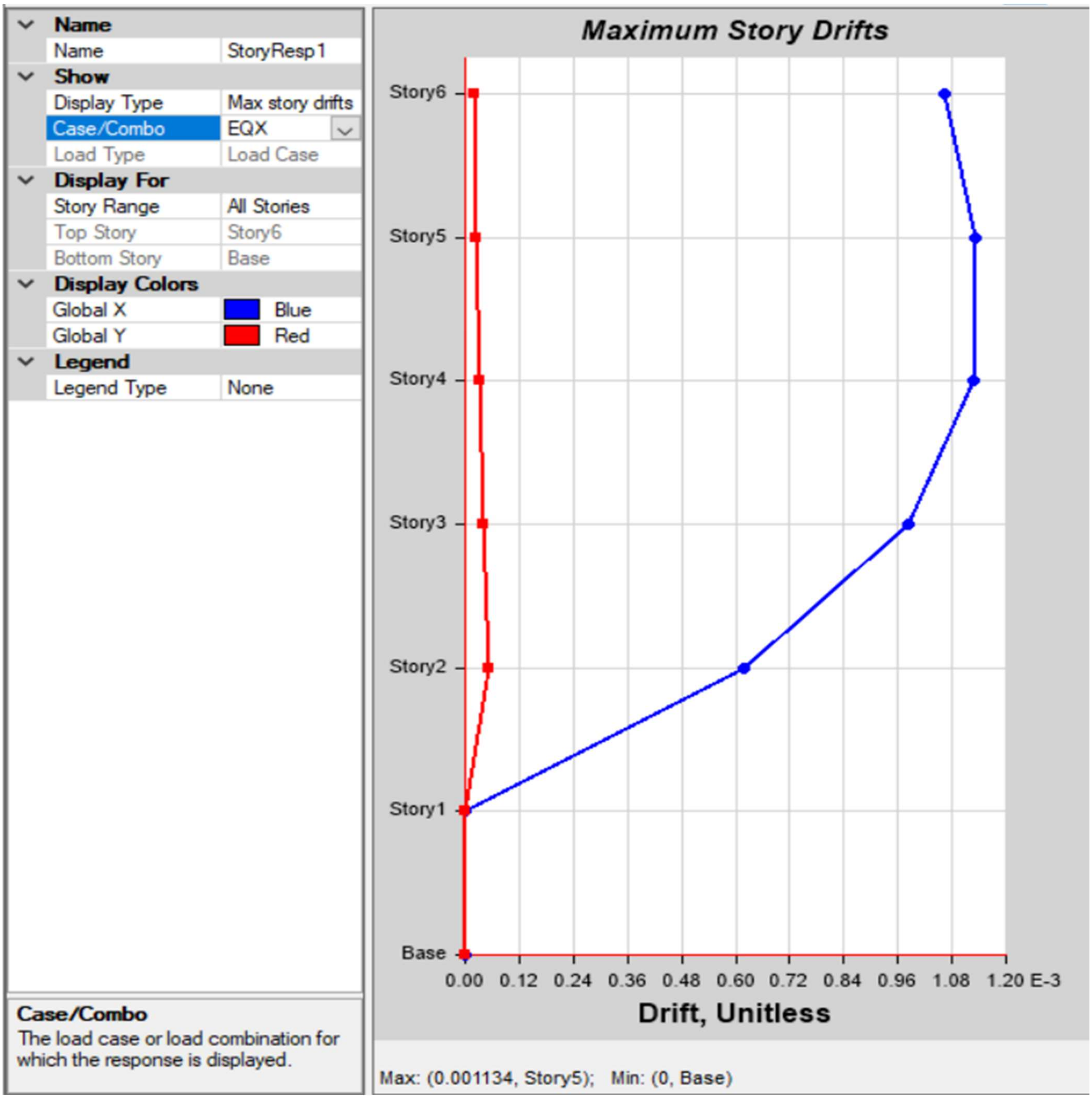


Frame system story drift @ x

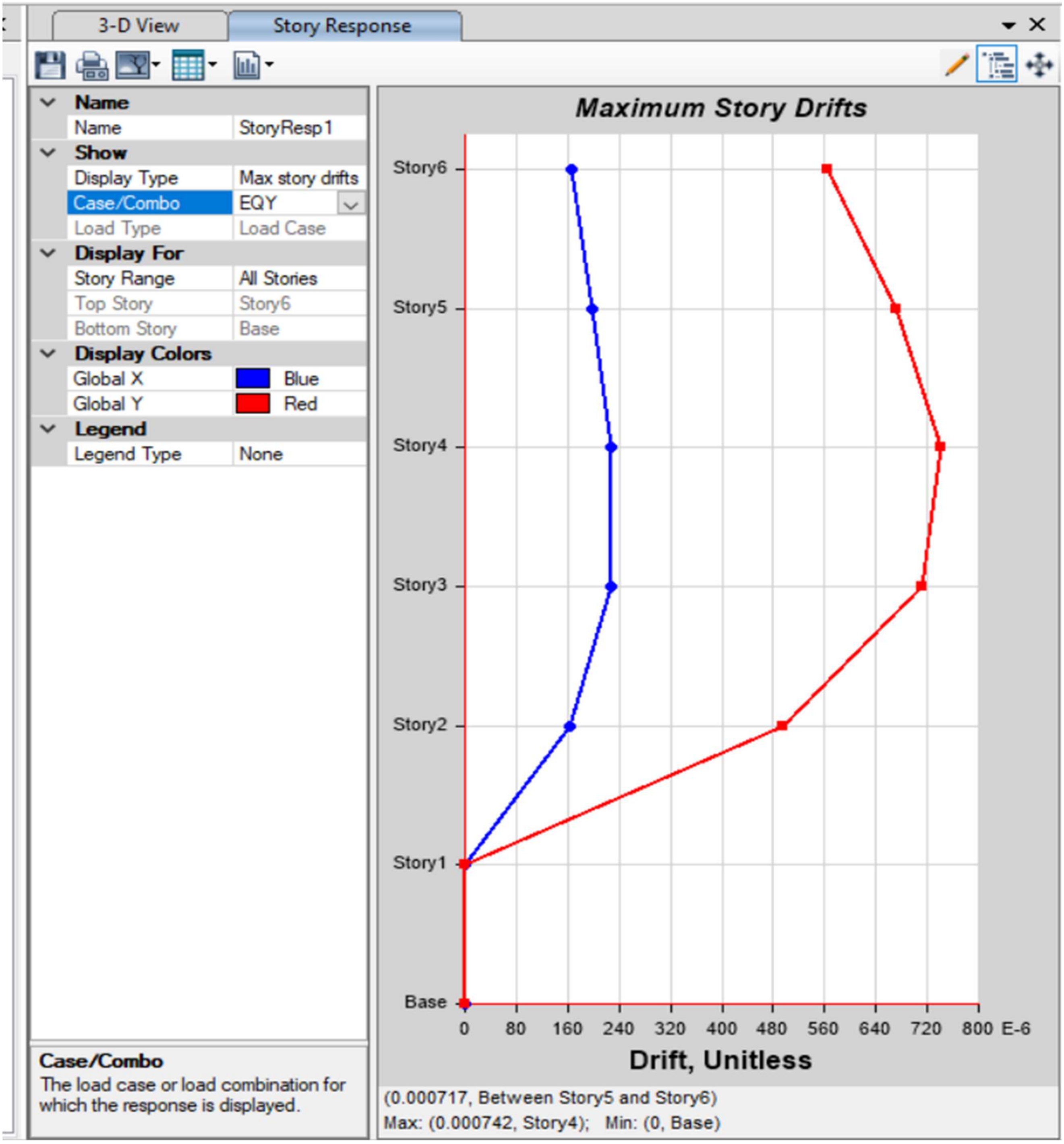


Frame system story drift @ y

Shear wall system:



Shear wall system story drift @ x



Shear wall system story drift @ y

We noticed that the shear wall system has the less story drift, and that's another point for the shear wall system.

3.5.3 Amount of concrete:

*Slabs:

Frame system slabs:

Story	Sectional area(m^2)	Height (m)	Amount(m^3)
Basement	912	0.28	255.3
1+2	695	0.28	389.2
3+4+5	715	0.28	600.6
		sum	1245

Shear wall system slabs:

Story	Sectional area(m^2)	Height (m)	Amount(m^3)
Basement	912	0.28	255.3
1+2	695	0.28	389.2
3+4+5	715	0.28	600.6
		sum	1245

*Beams:

Frame system beams:

Story	Sectional area(m^2)	length (m)	Amount(m^3)
Basement	1	221.5	221.5
1+2	1	176.9	353.7
3+4+5	1	176.9	530.6
		sum	1106

Shear wall system beams:

Story	Sectional area(m^2)	length (m)	Amount(m^3)
Basement	0.86	143.37	123.3
1+2	0.86	98.74	169.8
3+4+5	0.86	98.74	254.7
		sum	548

*Columns:

Frame system columns:

Story	Sectional area(m^2)	No°	Height (m)	Amount(m^3)
Basement	0.64	19	3	36.5
1+2	0.64	15	3	57.6
3+4+5	0.64	15	3	86.4
			sum	182

Shear wall system columns:

Story	Sectional area(m^2)	No°	Height (m)	Amount(m^3)
Basement	0.64	7	3.2	14.3
1+2	0.64	7	3.2	28.7
3+4+5	0.64	7	3.2	43.1
			sum	86

Shear wall system walls :

Story	Sectional area(m^2)	Height (m)	Amount(m^3)
Basement	13.6	3.2	43.52
1+2	13.6	3.2	87.04
3+4+5	13.6	3.2	130.56
		sum	261.12

*The total amount of concrete:

System	Amount of concrete(m³)
Frames	2533
Shear walls	2140

Which give us a difference between the two systems = 393 (m^3) of concrete

And that's another point for the shear wall system.

3.5.4 Amount of steel:

*Slabs:

System	Amount of steel (ton)
Frames	134
Shear walls	127

*Beams:

System	Amount of steel (ton)
Frames	58
Shear walls	34.8

*Columns:

System	Amount of steel (ton)
Frames	22
Shear walls	10.2

*Shear walls:

System	Amount of steel (ton)
Frames	-
Shear walls	8.9

*The total amount of concrete:

System	Amount of steel (ton)
Frames	214
Shear walls	181

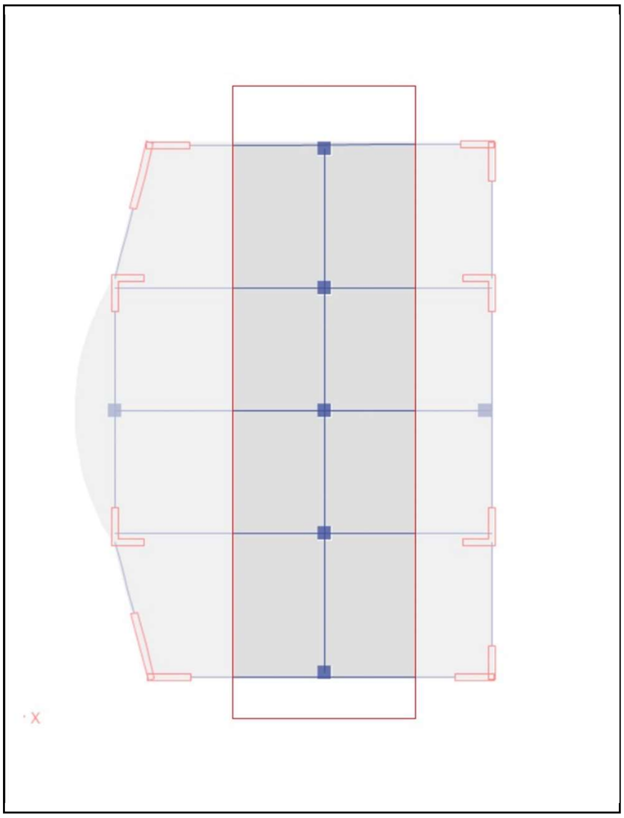
we can see also the difference in the amount of steal = 33 ton

which gives the shear wall system another point.

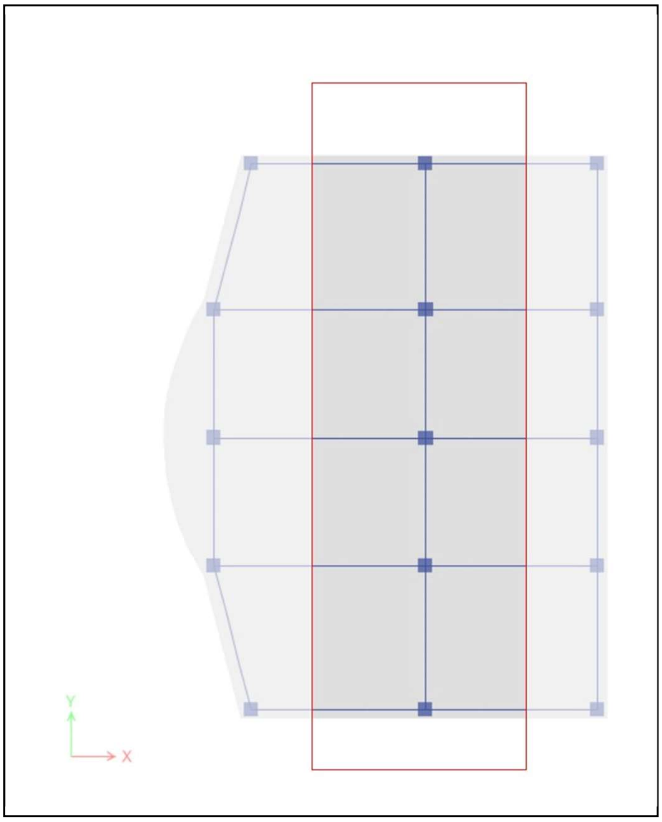
3.5.5 Time to do:

By asking a lot of builders and structural engineers, they gather that the shear wall system works done in shorter time than the frame system, and cost for the builder is different in the two system favor the shear wall system.

3.5.6 explanation about the difference :

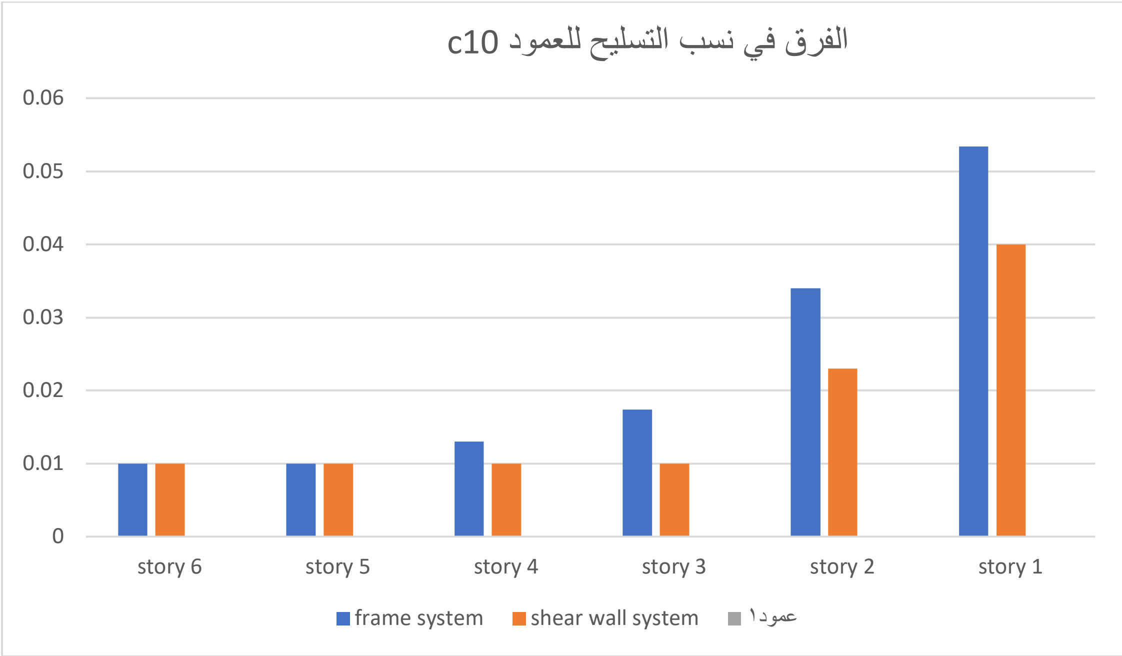


Shear wall system



Frame system

In this case we have these group of columns with the same diminution but in to different systems, and the next chart show the difference in reinforcement ratio.


الفرق في نسب التسليح للعمود c10
0.06
0.05
0.04
0.03
0.02
0.01
0
story 6
story 5
story 4
story 3
story 2
story 1
frame system
shear wall system
عمود١

CHAPTER 4

Conclusion & recommendation.

4.1 conclusion.

4.2 recommendation.

4.1 conclusion:

As apparent in the previous diagrams, the amount of steel in the moment-resisting frame system is about 1.6 more than that in the shear wall system for beams and 1.1 for slabs and 1.3 for columns steel ratio ; and the amount of concrete is about 35% more than that of the shear wall system for the building.

Also, the story displacement in x direction and y direction showed less values in shear walls earthquakes system than in MRF

Considering shorter installation time for shear walls over the MRF system

Thus, when considering an

earthquake resisting system in a building in any of the above- mentioned zones at any heights it is more advisable to choose the shear wall system above any other system if cost-efficiency is required by the client in addition to optimum safety.

4.2 recommendation:

In the near future we recommend further seismic assessment to be done on large scale taking into account different seismic zones using both linear and nonlinear analysis in order to find the safer and optimum option.

References.

[1]	Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE7-16).
[2]	Building code requirements for structural concrete (ACI-318-14), USA: American Concrete Institute, 2014.
[3]	Uniform Building Code : UBC -97 code .
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