T.C. İSTANBUL KÜLTÜR UNIVERSITY
INSTITUTE OF GRADUATE STUDIES

EARTHQUAKE PERFORMANCE AND COST COMPARISON OF CORE WALL AND TUNNEL FORMWORK RC HIGH RISE BUILDINGS

MASTER OF SCIENCE THESIS

MOHAMMAD IBRAHIM AMINI

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DEPARTMENT: CIVIL ENGINEERING
PROGRAMME: STRUCTURAL ENGINEERING

Supervisor: Assist. Prof. Dr. MUSTAFA CÖMERT

APRIL 2022

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PREAMBLE

I would like to express my sincere gratitude to **Assist. Prof. Dr. Mustafa Cömert** for his supervision, patience, support, and persuasion throughout this research. I wish to express my appreciation to all faculty lecturers of Istanbul Kültür University. My thanks also extend to numerous individuals who have provided encouragement, advice, and assistance in the completion of this thesis.

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ABBEREVATIONS

ASCE: American Society of Civil Engineering

ACI: American Concrete Institute

PEER: Pacific Earthquake Engineering Centre

LATBSDC: Los Angeles Tall Buildings Structural Design Council

TBI: Tall Building Initiative

MCE: Maximum Considered Earthquake

DBE: Design Based Earthquake

SLE: Service Level Earthquake

LS: Life safety

IO: Immediate Occupancy

CP: Collapse Prevention

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KISA ÖZET

YÜKSEK KATLI BETONARME BİNALARIN ÇEKİRDEK PERDE DUVAR VE TÜNEL KALIP YAPILARININ DEPREM PERFORMANSI VE MALİYET KARŞILAŞTIRMASI

Mohammad Ibrahim Amini

Bu çalışma, 30 katlı yüksek binaların çekirdek perde duvar ve tünel kalıp yapılarının deprem performansı ve maliyet açısından karşılaştırılmasını incelemektedir. Çekirdek duvarlı binalarda zemin ve çatı sistemleri, yerçekimi yüklerini taşımak ve bu yükleri taşıyıcı kirişlere, kolonlara ve temellere aktarmak üzere tasarlanmıştır. Ayrıca, deprem kaynaklı yüklerin iç çekirdek ve kolonlar gibi yanal yüke dayanıklı sistemlere diyafram hareketi ile dağıtılmasında anahtar role sahiptirler. Tünel kalıp yapılarında ise düz döşemeler, yerçekimi yüklerini taşıyacak ve bunları doğrudan perde duvarlara aktaracak ve son olarak da temellere aktaracak şekilde tasarlanmıştır. Bu tür yapı sistemleri, depreme bağlı yüklere, herhangi bir kiriş ve kolon olmaksızın sadece düz döşeme ve perde duvarlar yardımıyla dayanabilen sistemlerdir. Çekirdek perde bina ve tünel kalıp bina sistemleri, geçmiş depremlerde çok iyi neticeler veren yanal yüke dayanıklı sistemler olarak çok yaygın olarak kullanılmaktadır.

Bu araştırmada, (G+29) katlı betonarme konut yüksek katlı binaların modellemesi ETABS yazılımında gerçekleştirilmiş ve veriler analiz amacıyla PERFORM-3D analiz yazılımına aktarılmıştır. (ASCE 7-16) ve (ACI-318-19) kodlarına ve (PEER TBI,2017) ve (LATBSDC,2020) yönergelerine göre doğrusal olmayan dinamik (Zaman geçmişi) analizi yapılmıştır. Tasarım taban kesmesi, yapım süreleri, farklı mod şekilleri, maksimum yer değiştirmeler, öteleme, ötelenme oranı, moment eğriliği ilişkisi ve elde edilen binaların maliyetleri açısından sonuçlar elde edilmiştir.

Anahtar kelimeler: Çekirdek duvar inşası, tünel kalıp inşası, yüksek binalar, perde duvar, binaların deprem performansı, bina maliyetleri.

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ABSTRACT

EARTHQUAKE PERFORMANCE AND COST COMPARISON OF CORE WALL AND TUNNEL FORMWORK RC HIGH RISE BUILDINGS

Mohammad Ibrahim Amini

This study investigates the earthquake performance and cost comparison of core wall and tunnel formwork of 30 story high-rise buildings. Floor and roof systems in core wall buildings are designed to carry gravity loads and transfer these loads to the supporting beams, columns, and to footings. Furthermore, they play a key role in distributing earthquake induced loads to the lateral load resisting systems such as inner cores and columns by diaphragm action. And in tunnel formwork buildings the flat slabs are designed to carry gravity loads and directly transfer them to the shear walls and finally transferred them to the footings. This kind of buildings systems resisting earthquake-induced loads with the help of flat slab and shear walls only, without any beams and columns. Core wall building and tunnel formwork building systems are very commonly used as lateral load resisting systems which have shown very good results in past earthquakes.

In the present research, the modelling of (G+29) story RC residential high-rise buildings was performed in ETABS software then the data have been transferred to PERFORM-3D analysis software for analysis purposes. Nonlinear dynamic (Time-history) analysis has been performed, according to (ASCE 7-16) and (ACI-318-19) codes and according to (PEER TBI,2017), and (LATBSDC,2020) guidelines.

The results in terms of design base shear, building periods, different mode shapes maximum displacements, drift, drift ratio, moment curvature relationship, and costs of the buildings obtained.

Key words: Core wall building, Tunnel formwork building, High rise buildings, Shear wall, Earthquake performance of buildings, Building costs.

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CHAPTER 1

INTRODUCTION

This chapter provides an introduction to the study by first discussing the background and context, followed by the scope of the study, motivation for the research, the research aims and objectives, codes governing, and finally, the structure or organization of the thesis is presented.

1.1. Background

High-rise buildings are becoming more common in the world, the design of high rise buildings need many aspects to take into consideration, such as economy, sustainability, wind and earthquake performances, to make these structures economical, safe and convenient therefore multi-story reinforced concrete (RC) core wall and tunnel formwork buildings structural systems have been increasingly employed for construction industry in many countries, which are commonly used as the main lateral force-resisting systems for high rise buildings and are effective for resisting lateral loads imposed by wind or earthquakes, they provide substantial strength and stiffness as well as the deformation capacity needed to meet the demands of strong earthquake ground motions, during strong ground shaking the core wall building system is intended to dissipate energy by yielding of coupling beams, followed by flexural yielding at the wall base, and the tunnel form building system dissipate the exerted energy by shear walls in all sides.

For this purpose, two distinct residential buildings as the first building is the core wall building system which containing the moment-resisting frame with a core wall, and the second building is the tunnel formwork building system, as a rectangular in shape; Both prototypes of buildings have been designed and modelled with the help of ETABS software and the analyses have been performed in PERFORM-3D analysis software.

The design procedures for both types of buildings are typically conducted using performance-based design procedures recommended by Los Angeles Tall Buildings Structural Design Council (LATBSDC, 2020), Pacific Earthquake Engineering Research Canter of Tall Buildings Initiative (PEER TBI, 2017), American concrete institute (ACI-318-2019), and American Society of Civil Engineering (ASCE 7-16) guidelines. The buildings are assumed to be residential located in Skopje city Macedonia. The outcomes incorporate the base shear, building period, axial forces, shear forces, and bending moments of the structural members.

1.1.1. Core wall building

Concrete core wall buildings have been the most popular seismic force-resisting system in the world for many decades, and recently have become more popular for high-rise buildings. When the shear walls in high-rise buildings located around the perimeter of elevator or stairway shafts which are connected together to form a rectangular tube called a core. Core shear wall can be with a single-core or with a dual-core. Reinforced concrete core shear wall is the most imperative structural component of high-rise buildings to tolerate horizontal and vertical loads. Shear walls are used for stiffening the entire structure, to overcome the effects of seismic activities. The structural member specification is determined according to the minimum requirements of the ASCE code which is validated based on the preliminary design of buildings.

Cores have different configurations; For instance, T, L, I ،C ،and 2C. They are very similar to rectangular shear walls in flexural characteristics but the variant between the core and shear wall is in their torsion resistance. In primeval earthquakes, buildings with an adequate amount of core walls were saved from failure during the strong ground shaking.

In this study the core wall building contains moment-resisting frames and solid core walls as rectangular plan shape of the building. The core walls are located in the centre of the building. The cores or shear walls are supposed to have the same thickness for all thirty floors. these buildings are designed using nonlinear dynamic (Time-History) analysis at the MCE hazard level combined with various prescriptive design procedures. The analysis outputs have been investigated to select the appropriate type and location of the core shear walls for the building. The columns are located along the building perimeter; Finally, specified conclusions have been presented to obtain the optimum behaviour for the core wall building under the effects of earthquake loadings.

1.1.2. Tunnel formwork building

Tunnel formwork buildings technology was invented over 50 years ago; It is a steel formwork in which structural walls and slabs of the building are cast in one operation, steel formwork is used during the placing of the concrete to form the floor and the wall at the same time also called monolithic structures. All the vertical members are made of shear walls and the floor system is a flat plate where walls and slabs having almost the same thickness and hot air blowers for accelerated curing of the concrete is used. A half Tunnel is nothing but a half room sized L- shaped structural steel fabricated form which is used to cast the RC walls and floor slabs of a building as a monolithic structure in a continuous pour where two half Tunnel makes one room-sized full Tunnel. In many countries the tunnel formwork buildings are the primary housing due to their industrialized modular construction technique.

This construction technique is provided with low-cost, high-quality work and saves more construction time. The main components of this construction system are walls and flat plate slabs and do not have beams and columns as their structural component. The walls of these buildings have two functions; First, to resisting the lateral loads (seismic or wind), and the second, to carry the gravity or vertical loads to the ground because all the vertical load-carrying members are consisting of shear walls and floor slab systems.

This advanced construction technique (Tunnel formwork building system) is one of the best industrial scales which is very common in many countries including America, Canada, China, Turkey, Germany, Italy and other European countries, in total more than 60 countries use this construction technique. This system in addition of increasing the speed and quality of work, improves the structural performance and seismic behaviour of buildings as well.

In this study the tunnel formwork building system which containing solid walls and flat slabs. The floor plan of the building is similar as the core wall building. All shear walls have the same thickness for the entire building.
The buildings models are created in ETABS software and analysed in PERFORM-3D software by using shell elements for shear walls and slabs, all the shell elements are assumed as thin elements, and columns and beams are assumed as frame elements.

1.2. Scope of the study

This study covers and spotlights the earthquake performance and costs comparison of core wall and tunnel formwork high-rise structures as designed by dead load, live load, wind load, and earthquake loads. The analyses involve only frames of structures, without considering the effect on its infill. The nonlinear dynamic (Time History) analysis has been conducted on both types of buildings considering 11 different earthquake records. After creating models in the ETABS software as a pre-design procedure, the evaluation of the buildings response due to earthquake loading simulation and analysis has been carried out in PERFORM-3D analysis software. The buildings occupied the same land area of (600 meter^2) with similar each story heights of (3.3 meter) with a total height of (99 meters). The buildings are located in a high seismic zone (ZB) of Skopje city in Macedonia. The building models have been checked against the lateral loads after the analysis. If any structure member failed then the material and reinforcement changes or the cross-section increment would be recommended. Similarly, the outcomes, for instance the maximum displacement, reaction, base shear, building period, drift, drift ratio and moment curvature relationships has been evaluated.

As the world population is increasing day by day and the land space limitation is the more common concern in major cities, led the way to construct high-rise buildings. To limit earthquake ground shaking, manage lateral forces encountered in high-rise buildings, and to provide reliable and earthquake-resistant high-rise buildings, then the core wall and tunnel formwork buildings systems are intended to be relatively flexible relative to many other types of buildings.

This research provides the wide comparative study for core wall and tunnel formwork buildings systems as there is very limited researches related to the core wall and tunnel formwork buildings and no study addressed their comparative learning. Specially not enough studies exist about the tunnel formwork buildings systems. Structural performance in terms of strength, stiffness, and ductility are well thought out in this study; which helps for adopting these two types of buildings systems to perform well during strong earthquakes in earthquake-prone areas. These types of construction systems have shown better results from previous earthquakes; While these developments are undoubtedly beneficial to withstand against lateral forces and mitigating lateral loads which acting on structures. Hence, the design standards must be updated promptly with the intention of safety preservation when new materials and methods are inevitably used in design and construction.

1.3. Motivation for the research

Investigating the earthquake performance of buildings is one of the most significant aspects for design and structural engineers and researchers to take into consideration. Also, the building cost determination and comparison is the second perspective of every person who is willing to have a building. This type of research not only helps manufacturers, construction industries, and Engineers, but it is the aspiration for everyone to get to know about earthquake performance and building costs as far as the earthquake-resistant and cost-effective buildings are concerned.

1.4. Aim and objectives of the study

The aim and objective of this research are to investigate the earthquake performance and cost comparison of core wall and tunnel formwork buildings systems.

The first objective is to design both types of buildings in ETABS software according to the American Society of Civil Engineering (ASCE-7-16) Minimum Design Loads for Buildings and Other Structures, and American Concrete Institute (ACI-318-19) codes assumptions. Then analyse the buildings in (PERFORM-3D) analysis software according to Los Angeles Tall Buildings Structural Design Council (LATBSDC, 2020) guidelines and as per Pacific Earthquake Engineering Research centre of Tall Buildings Initiative (PEER TBI, 2017) Guidelines for Performance-Based Seismic Design of Tall Buildings.

Every step of analysis and design is calculated twice and separately for each building. The main evaluating objectives are as follows:

1. To model the core wall and tunnel formwork (G+29) story buildings in ETABS, version 19 software.
2. To analyse the core wall and tunnel formwork buildings with the same specification in PERFORM-3D, version 7 software.

3. To evaluate the earthquake performance of the core wall and tunnel formwork buildings.
4. To check the lateral displacement, building periods, stiffness, story drift, drift ratio and other specifications of buildings and comparing their results.
5. To calculation the material quantities and costs of core wall and tunnel formwork building systems respectively
6. Detailed cost comparisons between core wall and tunnel formwork building systems.
7. To determine whether the core wall or tunnel form building is more cost-effective.
8. To compare whether the core wall or tunnel formwork building taking less construction time with low cost.
9. To compare the obtained results in existing standards and with other references.
10. To Perform a computational analysis and to determine the reinforced requirements in all structural members.
11. To check the performance of every structural element of the building with codified evidence.

1.5. Governing codes

This thesis study is governed based on the following codes

- American Society of Civil Engineering (ASCE-7-16) Minimum Design Loads for Buildings and Other Structures.
- American Concrete Institute (ACI-318-19)
- Los Angeles Tall Buildings Structural Design Council (LATBSDC, 2020) guidelines
- Tall Buildings Initiative (PEER TBI, 2017)
- American Society of Civil Engineering (ASCE-41-17)

1.6. Organization of the thesis

This thesis is organized into seven individual chapters. A summary of the organization of the thesis is provided below.

Chapter 1. Introduction

In chapter one the context and explanation of the study, introduction to the core wall and tunnel formwork building systems has been introduced, motivation of the research, aims and objectives of the study, the governing codes, and lastly, the overall organization and structure outline of the study has been described.

Chapter 2. Literature review

In this chapter the existing literature regarding high rise core wall and tunnel formwork buildings systems will be reviewed to identify the most powerful approaches and strategies to improve the earthquake performance of buildings with most cost-effective method.

Chapter 3. Methodology

This chapter explains the methodology of the research. The basic concept of the analyses, ground motions selection, modified process for the design and analysis of core wall and tunnel formwork high-rise buildings, and descriptive idealization of the core wall and tunnel formwork buildings is discussed followed by reliable and valid evidence.

Chapter 4. Design of the buildings

This chapter illustrates the designing procedures of core wall and tunnel formwork buildings in ETABS software. And finally provides the quantities and cost estimation and comparison between the core wall and tunnel formwork buildings systems.

Chapter 5. Analysis of the buildings

This chapter demonstrates the circumstances of earthquake performance and the buildings analysis procedures

Chapter 6. Results and Discussion: This chapter provides results of the core wall and tunnel formwork building systems after designing and analysis procedure; And discusses the results about earthquake performance of buildings and their costs.

Chapter 7. Conclusion: This chapter provides a summary conclusion of the study and further recommendations of work.

CHAPTER 2

LITERATURE REVIEW

2.1. General overview

This literature review investigates some studies made by researchers on seismic performance of core wall and tunnel formwork buildings systems and their costs. It has always been a human aspiration to create high-rise buildings. The development of metro cities in urban areas is increasing the demand for high rise buildings. But as the structure extends vertically, it becomes sensitive to seismic loads; And it is necessary to consider the earthquake-resistant parameters of the buildings to ensure that the strength of the structure can resist seismic forces during the earthquake.

We cannot predict the occurrence of an earthquake. It can occur at any place and at any time. Scientific research and work are carried out to develop new methods and construction techniques to make structures resistant to seismic forces. While there have been many types of research for shear wall and tunnel formwork buildings but the literature has not adequately addressed the problem of the earthquake-resistant case for core shear walls and tunnel formwork structure systems of high-rise buildings.

This thesis begins with a review of the design of high-rise buildings and the conceptual design. Then the study is carried out to investigate the comparison of structural performance and cost comparison between tunnel formwork and core wall structure systems of 30 stories reinforced concrete buildings. The design of both types of structures is performed in ETABS software. And nonlinear dynamic (Time history) analysis is carried out with the help of PERFORM-3D software. For the analysis, 11 different earthquakes consisting of near-fault and far-field events were specified and scaled under the Service Level Earthquake (SLE) of the return period of 72 years (50% probability of earthquake exceedance in 50 years) for limited damage and the Designed Based earthquake(DBE) level of the return period of 500 years(10% probability of earthquake exceedance in 50 years) for Life Safety, and Maximum Considered Earthquake (MCE) level of the return period of 2500 years (2% probability of earthquake exceedance in 50 years) for collapse prevention.

After reviewing the literature at the end of the work, conclusions and an outline of the proposed work will be made and will address the earthquake resistance and cost-effectiveness for both types of buildings systems.

2.2. Theoretical framework

Sivan et al. (2021), [1] Studied the dynamic response of high-rise buildings with the diverse location of shear walls. In this study, they modelled and analysed thirteen-story RCC framed buildings of various locations of shear walls using linear static, linear dynamic, and pushover methods of analysis with the help of standard package ETABS. Considering the IS 875 (part-3) code. The study concluded that providing shear walls is found to be effective in enhancing the overall seismic performance characteristics of the buildings.

Yadav and Jamle (2020), [2] This paper discussed finding the optimum building case in two stages to resist the earthquake forces and reduce the building cost. The first stage is a (G+20) earthquake-resistant building with a single-core shear wall and the second stage is a (G+20) earthquake-resistant building with a dual-core shear wall. All with openings and without openings, using STAAD-pro software with different phases as per Indian Standard 1893: 2016 code. After the analysis, the result obtained showed that the building with 25% opening area in a single-core type shear wall and 50% opening area in a dual-core type shear wall performed well with reduced cost of the buildings and structures resistance against the earthquake forces consideration.

Vedant and Mishra (2020), [3] Studied the importance of different types of shear walls used for high rise buildings to resist horizontal forces caused by wind and earthquake loads. And to improve the structural performance of high-rise buildings. From this study it was observed that the lateral deflection in any one story of a multi-story or a high-rise building must not be more than the total building height divided by 480. This study concluded that reinforced cement concrete shear wall system can be the best option for high-rise buildings because of its cost-effectiveness and easier in construction.

Thakre et al. (2020) [4], This study investigated the cost reduction in high-rise buildings due to shear wall area reduction. Here, five buildings were taken into consideration located at Seismic Zone-III with the help of STAAD-Pro software using dynamic response spectrum analysis. The shear walls not only to be designed to withstand against gravity or vertical loads but will be designed for the lateral loads of winds and earthquakes as well. That is why too much area reduction of the shear wall will reduce the seismic resistance of the whole building. This study has shown that if the opening in shear wall excessed beyond the 20 % limit, then the stiffness of the building will be less and the building's components may fail.

Yadav and Jamle (2020) [5] Studied the shear wall with openings in high rise buildings. The analysis has been performed on a single-core and a dual-core shear wall building. And a comparative analytical investigation has been done on different residential buildings with very-large medium and very-small shear wall openings. The study concluded that structures with 80% coverage shear wall area performed best.

Patidar and Jamle (2020), [6] This paper studied the stability increment of the high buildings which are located in a high seismic zone with core shear wall of different concert grades. The building was modelled with different sizes of shear walls and different grades of concrete. The study concluded that to enhance the stability of the high-rise buildings it is required to increase the thickness of the shear wall members with a higher grade of concrete.

Vahid et al. (2019),[7] This study paper discussed the seismic behaviour of tunnel form buildings with the horizontal irregularity of different story tunnel form buildings which are located in high seismic zones. The buildings were designed by ETABS software according to ASCE-07 (2016). And nonlinear analysis is completed in PERFORM-3D software, concerning the models studied in this paper indicated that the tunnel form structural system is proficient to show satisfactory seismic performance despite the presence of horizontal geometric irregularity.

Hong-Gun et al. (2019), [8] This study presented enhancement of the structural performance, constructability, and cost-saving techniques for the core wall structure system by using steel plate columns at the corners of the core section of the structure. After performing nonlinear section and 3-dimensional structural analyses the results achieved that when compared to traditional reinforced concrete core wall case, the use of the corner steel plate columns provided better structural capacity, and the construction time and cost (approximately 15%) can be reduced even with the use of steel plate columns, meanwhile, the lateral displacement was maintained to be the same for both cases.

N. Ostovar et al. (2019), [9] This study discusses the seismic design concepts of different configuration of cores and shear walls in tall buildings. In this study (G+24) story building modelled and analysed with the distinct shape of the cores with the help of SAP2000 software conducting pushover and time history analyses considering EURO and ASCE codes. The El Centro earthquake record was used for lateral excitation of the building. The results showed that the rectangular cores showed better displacement reduction in the building as compared to other shapes of the core walls.

Patel and Jamle (2019), [10] Analysed 25 storied residential buildings with the help of STAAD-Pro software using the response spectrum method along with SRSS combinations, in zone-V with different structure systems such as bracing, shear wall, and outrigger system to resist sever earthquakes and wind effects. The study concluded that the outrigger system provides better performance than the belt truss system.

Aknahad et al. (2019), [11] This study investigated the performance of high-rise buildings with different types of core wall shapes and to identify the efficient core wall system for high-rise buildings. That which shape of the cores is more effective for reducing the seismic vibration. In this study (25) story structures with (8), different types of core wall systems were taken into consideration using Sap-2000 software and the El Centro earthquake record. The time history analysis was performed according to EUROCODE and ASCE standard, the results obtained show that walls and cores are the very best solution for resisting the lateral and vertical forces, this study especially proves that the structure with irregular core (1) existence is very effective one among other cores for reducing the seismic response of the building

Kumar and Reddy (2019), [12] This paper studied and analysed (G+30) story building by considering different earthquake, dead and live loads, using ETABS software. The dynamic analysis either, response spectrum, and time history have been conducted according to the Indian standard. The main results were the determination of the effects of lateral loads, shear force, axial force, base shear, maximum displacement, and tensile forces, and comparison has been performed on seismic zones 2, 3, 4, and 5 and found that the lateral displacements or drifts are more in zone 5 when compared to the zones 4, 3 & 2 then the strength, serviceability, and stability of the structure checked

Banerjee and Srivastava (2019), [13] This paper studied the stiffness inducement by introducing the shear wall for high rise buildings for resisting the lateral forces the study contain the analysis of 16 story building located in seismic zones III and IV by ETABS software to obtain the optimum position of shear wall in the building, response spectrum and time history analysis has been performed with the various positions of the shear wall, the comparative study included that the location of the shear wall plays a more significant role in enhancing the resistance against the lateral forces so, the structure will be able to handle more seismic loads

Balkaya et al. (2018), [14] This study discussed the torsional rigidity improvement and enhancement of structure performance with experimental studies and finite element analysis on tunnel form buildings that have shear wall dominant structural systems. Which are more generally used because of their fast construction technique and lower cost. After analysing and introducing four different strengthening techniques (steel braces, reinforce concrete infill shear wall, precast concrete shear wall, and reinforced concrete shear wall at the facade), the study concluded that steel bracing is the greatest suitable and practical method to improve torsional rigidity.

Vahid et al. (2018) [15] Conducted a case study of the seismic reliability of tunnel form concrete residential (G+10) story building to different configurations of canters of mass and stiffness is figured out in the plan, for its modelling ETABS software and its nonlinear modelling and analysis the PERFORM-3D software was used and the analyses type was Time History and Pushover analyses according to ACI 318-14, and FEMA356, 2000 and ASCE/SEI 41-13 codes, the founded results showed that the seismic response of the structures affected by the eccentricity of mass and stiffness as well as the severity of the earthquake.

Chaudhary (2017), [16] This study had come to know the cost and completion time comparison of conventional structure system and tunnel formwork structure system. It provides to know which one is cheaper and better with a short completion cycle among the conventional system and monolithic system. The data for the research was collected from a questionnaire survey of a case study from the construction site. The results have been obtained that the use of tunnel formwork reduces cost by 40% and time by 60% when compared to conventional formwork.

Shijala and Vijapur (2017), [17] Studied the seismic loading performance of the buildings with tunnel form structures for different wall thicknesses of different story heights with wall openings and without a wall opening using ETABS-2015 version software the comparison has been done by considering the parameters such as wall thickness, story height, story drift, story displacement, and stiffness of the buildings.

Abdulmajeed and Ahmed (2017), [18] This study examined two **(**28) story reinforced concrete core wall structures one with bracing and another one without bracing and the third one with the same story shear wall structure system. All three different structures were subjected to different load combinations. The analysis was performed using ETABS software by equivalent static analysis and response spectrum dynamic analysis. Finally, it was concluded that the core wall bracing structure is better to be used in the term of displacement and drift. And the core wall without bracing structure is better to be used in the term of shear and moment. And shear wall structure system is more economical in terms of reinforcement.

Shalgar and Aradhye (2017), [19] Conducted a case study of advanced tunnel formwork techniques in the real estate construction industry. The study was consisting of four 14 (2G+12) storied buildings located in Pune, India. Where two below floors were parking level and the formwork system type was conventional formwork, and the other 12 floors formwork system type was tunnel formwork. The main purpose of this study was to introduce and improve the use of the advanced tunnel formwork techniques in the construction sectors and concluded that this innovation despite being extremely economical even much more time-saving.

Fathalizadeh (2017), [20] This study deliberated two common types (Steel and Concrete) of shear walls for resisting lateral loads for high-rise buildings. The study showed that a concrete shear wall consists of reinforcements and concrete. Which can be pre-cast or cast in situ, a steel plate shear wall (SPSW) consists of steel infill plates circumscribed by boundary elements. The study concluded that shear walls are one of the finest priorities for resisting lateral loads, which can be constructed by reinforced concrete or steel plates.

Roberta et al. (2016), [21] This study investigated the seismic performance of high-rise shear wall buildings as a case study of four RC structures each with 44 stories in Skopje based on Tall Building Initiative (PEER-TBI) and Los Angeles Tall Buildings Structural Design Council (LATBSDC) Guidelines, where these buildings are the highest one in Macedonia, and Linear elastic, Pushover and nonlinear dynamic analysis with different earthquake records to analogize near-field and far-field events were performed using ETABS and SAP-2000 software. The result obtained indicates that the buildings show suitable seismic performance.

Husain and Mahmood (2016), [22] Analysed (G+9) story reinforced concrete framed building with (9) different building cases. The purpose of the study is to improve the seismic performance of the building by using three types of shear walls such as side shear walls, middle core shear walls, and double core shear walls, using the finite element method with the help of Sap-2000 software according to International Building Code (IBC 2006), the study concluded that seismic resistance capacity of the shear wall framed buildings is expressively enhanced in the presence of shear walls when comparing to the free-wall arrangement, also concluded that side shear wall is the better option if the building length in the x-direction is less than or equal to 20m if its length is more than 20m in the same direction then use of the double core shear wall is the best option.

Dyavappanavar et al. (2015), [23] This study analysed (G+19) story shear walled RC building located in the seismic zone- IV proportionally changing the shear wall position considering different shapes and locations of the shear walls in buildings. The analysis has been performed by ETABS software using equivalent static, response spectrum, and time history analyses methods. It has been concluded that the shear wall increases the strength and stiffness of the structure and compared data from different models showed that the building with the exterior shear wall at corners given better results for lateral displacement and base shear

Lasanovic et al. (2014), [24] Purpose of this case study paper was to compared fundamental period obtained from current codes with an analytical obtained fundamental period on real reinforced concrete tunnel form building located in Osijek's city, Croatia. The analysis of the building has been performed in Sap-2000 software considering Eurocode-8 and ACT-3-06. The results showed that the data found from codes are the closest to the period obtained by numerical modelling of real tunnel form building.

Değer et al. (2014), [25] This paper investigated the comparison of seismic performance and costs of two similar 42 stories reinforced concrete core wall buildings located in Los Angeles. A core wall building without moment-resisting frame (SMRD) and a similar core wall building with perimeter moment resisting frames (SMRF), using PERFORM-3D software according to International Building Code (IBC), Los Angeles Tall Building Structural Design Council (LATBSDC), Pacific Earthquake Engineering Research Centre (PEER, 2010), and ASCE 41-06. After nonlinear response history and cost analysis it has been found that both structural systems usually attained excellent performance, but the core wall building system with moment resisting frame performed somewhat better and it is repair cost is lower, However, it is total costs were higher than the core wall building without moment-resisting frame.

Yüksel (2014), [26] This study presented the experimental investigation on the inelastic seismic behaviour of the shear walls of tunnel form buildings. As all the vertical members are made up of shear walls also, all floors are flat plates. When strong seismic forces occur due to retarding of the plastic hinges in tunnel formwork buildings, there will be no brittle failure in which lateral and vertical loads are assigned to shear walls, And Strength and ductility requirements are satisfied.

Munir and Warnitchai (2012), [27] Studied the 40 story RC core-wall building as a case study building located in a high seismic zone area, and two levels of earthquake ground motions Design Based Earthquake (DBE) and Maximum Considered Earthquake (MCE) are considered. A Flexural plastic hinge is typically allowed to form at the base of the core wall under such a severe earthquake. The analysis is evaluated by Nonlinear Response History Analysis (NLRHA), using the ETABS software and according to International Building Code, Uniform Building Code (UBC-97), and Los Angles Tall Buildings Structural Design Council's (LSTBSD) procedures. The results showed that the core wall is generally much stiffer than the peripheral columns. And to improve the seismic design of high-rise core-wall buildings then the designing of plastic hinges, dual plastic hinges, and installing the energy-absorbing devices will be a good beneficial option.

2.3: Closing remarks

As confirmation from the literature. It is clear that earthquake is one of the most destructive cataclysms in which the power and duration of seismic ground vibrations expected at any area rely on the greatness of the earthquake that is why more precise attention is required to have earthquake-resistant high-rise buildings. To avoid or minimize damages due to earthquake forces. So, core type shear wall structure systems and innovative techniques like tunnel formwork structure systems are the best options to be followed due to withstanding against the lateral forces and their cost-effectiveness

In the future, frequent use of tunnel formwork and core wall buildings systems are recommended. Because these types of structures can good performances against lateral forces during an earthquake.

CHAPTER 3

METHODOLOGY

3.1. General

The present work is carried out to study the earthquake performance and costs comparison of two reinforced concrete buildings models. The buildings are designed in ETABS software and the analyses are performed in PERFORM-3D analysis software Here, the analysis is carried out on totally different buildings which occupied the same land area. The models considered are:

1) Core wall building
2) Tunnel formwork building

The Nonlinear dynamic (Time History) analysis is adopted for seismic evaluation of the buildings, as per the ASCE-7-16, and ACI-318-19 codes and PEER-TBI and LATBSDC guidelines.

3.1.1. Structural configuration

The following parameters are considered in the performance evaluation.

1. Literature review.
2. Modelling of the buildings in ETABS V.19 software
3. The nonlinear dynamic analysis method conducted in PERFORM-3D V.7 analysis software.
4. Two types of structural systems a core wall building system (building with core walls at centre and columns at the periphery of the building) and a tunnel formwork building system (shear walls and flat slabs) have been modelled and analysed with different applied loads.
5. Required data have been extracted from analysis results and interpretation of data is done based on the result of analyses.
6. Evaluating different structural components for both types of buildings
7. In this study two 30 story buildings of a core wall building the tunnel formwork building with a total height of (99 m). With the same plan size of (30 m x 20 m), was considered. So, that the efficiency of each system will be assessed under specified earthquake loading.

8. Accurate prediction of stiffness, strength, drift capacity, mode shapes, and lateral forces effects results.
9. Eleven (11) different seismic records were designated from PEER Ground Motion Databases of the Pacific seismic centre.
10. Finally, the thesis will be ended with the results and conclusion.

3.2. Research methodology

To check the earthquake performance and cost comparison of core wall and tunnel formwork buildings systems. First, accomplish both types of buildings modelling in ETABS software. Then transferred its data to PERFORM-3D software for the analysis purpose.

The methodology consists of the following built points:

- The data is collected from the year 2012 up to the year 2021 end from different countries, cities, and regions all over the world.
- The study utilized convenient sampling techniques from the analytical study of the (PEER) reviewed journals, articles from Google Scholar, Web of Science, Science Direct, other online sources, different books, and from many practically conducted documentaries in the specific study area.
- Modelling: ETABS software.
- Analysis: PERFORM-3D software.
- Analysis Method: Nonlinear Dynamic (Time History) Analysis.
- Codes used: (ASCE 7-16) and (ACI-318-19)
- Guidelines: (LATBSDC, 2020) and (PEER TBI, 2017)
- P-delta effects are taken in consideration
- The buildings used fibers models
- 2.4% of modal damping was used on the structural period segment
- 0.1% of Rayleigh damping was superimposed to eliminate high-frequency vibration.

3.3.Design Procedures

Following steps of methods are adopted in this project:

Step-1: Selection of building (Core wall building or Tunnel formwork building)

Step-2: Collection of data and study literature

Step-3: Modelling (Pre-design) of the buildings in ETABS V.19 software

Step-4: Assigning of loads and load combinations and other specifications as per the American codes

Step-5: Analysis of buildings in ETABS with DBE level earthquake

Step-6: Export buildings from ETABS to PERFORM-3D software

Step-7: analysis of buildings conducted in PERFORM-3D V.7 analysis software.

Step-8: Use Nonlinear dynamic or Time-History analysis method

Step-9: Analysis the buildings for 11 different earthquake records (MCE level means 2475-years earthquake return period.

Step-10: 2.4% of modal damping was used on the structural period segment

Step-11: 0.1% of Rayleigh damping was superimposed to eliminate high-frequency vibration.

Step-12: Required data extraction from analysis results and interpretation of data based on the result of analyses.

Step-13: Evaluating different structural components for both types of buildings

Step-14: Interpretation of the results

Step-15: Calculating the material quantities for the core wall and tunnel formwork building individually

Step-16: Find the estimate cost for both type of buildings.

3.4. Types of analyses

The structure analysis has different types but here we will deliberate about nonlinear dynamic or Time History analysis method

1. Static analysis method
    - Linear static method (equivalent static analysis)
    - Non-linear static method (pushover analysis)
2. Dynamic analysis method
    - Linear dynamic method (response spectrum analysis)
    - Nonlinear dynamic method (time history analysis)

3.4.1. Nonlinear dynamic (Time History) analysis

Nonlinear dynamic analysis is definitely the most realistic and accurate analysis method for structures analysis. This method is also stated as “Nonlinear time history analysis”, “Nonlinear response history analysis” and according to ASCE-41-17 code it is also called as “NDP (Nonlinear Dynamic Procedure)”.

In this study many different earthquake records like Kocaeli Turkey, Chi-Chi Taiwan, Kobe Japan, Bam Iran, Landers, and others earthquakes was taken in consideration for the analysis purpose with geometric nonlinearities (P-delta) effects and 2.4% damping of the structures. The propagation of the ground motion throughout the structure will generate approximately different data. That is why selecting different set of numerous ground motion records is essential for optimum results. And after all, only a narrow, single pointed estimated response of structure will be adopted with codes checking that a structure will withstand or shall not sustain a significant life-threatening damage at a such level of earthquake intensities.

Thus, seismic code (ASCE 7-16) prescribed using ground motion records that match or exceed than the targeted level, Overall, the maximum of the recorded peak responses or mean of the peak responses of the earthquake recodes is taken as the structures demand.

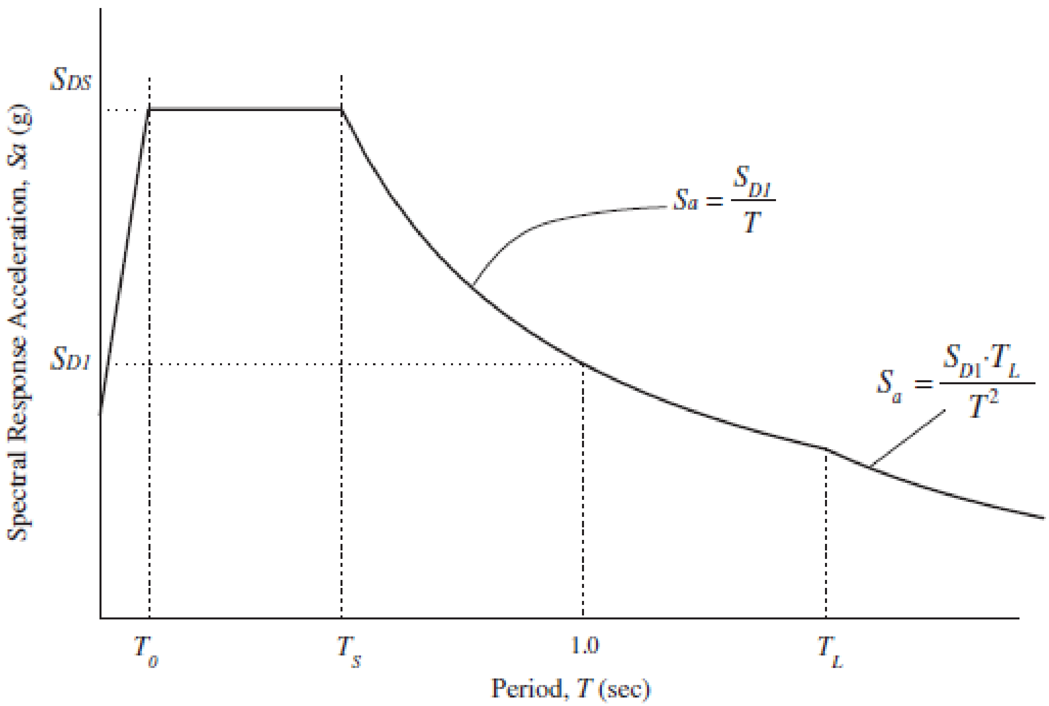


Figure 3.1: Design response spectrum

Nonlinear dynamic analysis is an important technique for structural seismic analysis particularly when the evaluated structural response is nonlinear. To perform such an analysis, a representative earthquake time history analysis is required for a structure being evaluated. Time history analysis is a step-by-step analysis of the dynamic response of a structure to a specified loading that may vary with time. Time history analysis is used to determine the seismic response of a structure under dynamic loading.

Earthquake causes diverse shaking intensities at different locations and the damage induced in buildings at these locations is also different. Therefore, there is essential to construct a structure which is earthquake resistance at a particular level of intensity of shaking a structure.

Determine Site Correction Factors F_1 and F_S

Find

Long period site coefficient (S_1)

Short period site coefficient (S_S)

$S_{D1} = F_1 \times S_1$

$S_{DS} = Fs \times S_S$

Where S_{D1} and S_{DS} are design spectral acceleration

$T_0 = 0.2\ S_{D1} / S_{DS}$

$T_1 = S_{D1} / S_{DS}$

Where T_0 and T_1 are the period of the buildings

Base Shear $V_{te} = M_t \times (S_{ae} / R_a)$

Where

M_t: is the total mass of the building

S_{ae}: is the spectral acceleration

R_a: is the Earthquake Reduction Factor

3.5. Software

For the buildings modelling and analysing the following software has been used

3.5.1. ETABS V.19

ETABS stands for Extended Three-Dimensional Analysis of Building Systems This software can design and analyse any simple, high rise, and most complex types of buildings on a systematic procedure. More than 30 years ago from the original release of the ETABS software it is accepted as a significant and very accurate tool for the designing and analysis of the structures. To reduce the complexity of modelling the preliminary engineering modelling was created in ETABS software, then the models were exported to PERFORM-3D software with material and other given properties.

3.5.2. PERFORM-3D

It is a structural design and analysis software that can perform dynamic and static analysis at the same time specifically intended for performance-based seismic assessment of structures to assess the earthquake performance. The software permits monitoring of inelastic behaviour of structural components with different levels of deformability. After importing the models here from ETABS software the different earthquake records were selected for simulation, the construction in PERFORM-3D all used fibers models. 2.4% of modal damping was used on the structural period segment. And 0.1% of Rayleigh damping was superimposed to eliminate high-frequency vibration. After the analysis, the results come out was indicated that the overall energy dissipation was good, the damping was small. The overall deformation was right, and the seismic performance was also good. Generally, the PERFORM-3D software has a good performance for seismic evaluation behaviour of building.

CHAPTER 4

DESIGN OF THE BUILDINGS

4.1. General

The development of the construction of high-rise and ultra-high-rise buildings reflected the public attention around the world. The design of these high-rise structures in seismically dynamic areas shifts drastically from area to area where thorough execution-based appraisals are needed in a few countries. Recent tendencies in high-rise inhabited construction brought about an assortment in a variety of unusual shapes, configurations, inventive frameworks, and high-performance materials that contest current design practice.

The goal of designing high-rise building models is to certify that the models characterize the features of an apartment building. Nowadays, these structures are diverse in height, shape, and capacities. This makes each building's qualities not quite the same as another such as residential, official, and commercials for model designing principle and better understanding. For the behaviour of high-rise buildings, a sophisticated analysis has been carried out to examine the structural efficiency and structural performance of the different structural systems.

In this study, two different buildings with an equal number of stories, with (G+29) stories having the same floor plan of (30 m) in the x-direction and (20 m) in the y-direction were taken into consideration. The floor height for the core wall and tunnel formwork buildings are assumed as (3.3 meters) for all floors.

The accompanying two particular structure designs are utilized in the study

Design 1. Core wall building

Design 2. Tunnel formwork building

The designing of the structures has been done with the help of ETABS V.19 software and their analysis have been performed in PERFORM-3D software according to the information given underneath.

4.2. Building design

The thesis models are (G+29) story reinforced concrete residential buildings, one is a core wall building and another is a tunnel formwork building system. Each building designing and analysis explanation is described in this study. It is noticed that first, the designing of the buildings is performed in ETABS software, then their analyzation completed in PERFORM-3D analysis software.

4.2.1. Core wall building design

The building is designed with geometrical plan dimension of (30 m) to x-axis, (20 m) to y-axis, and (99 m) to the z-axis. As shown in figure 4.1. (a), (b). The building has the slab thickness of (0.17 m) connected with beams and column elements.

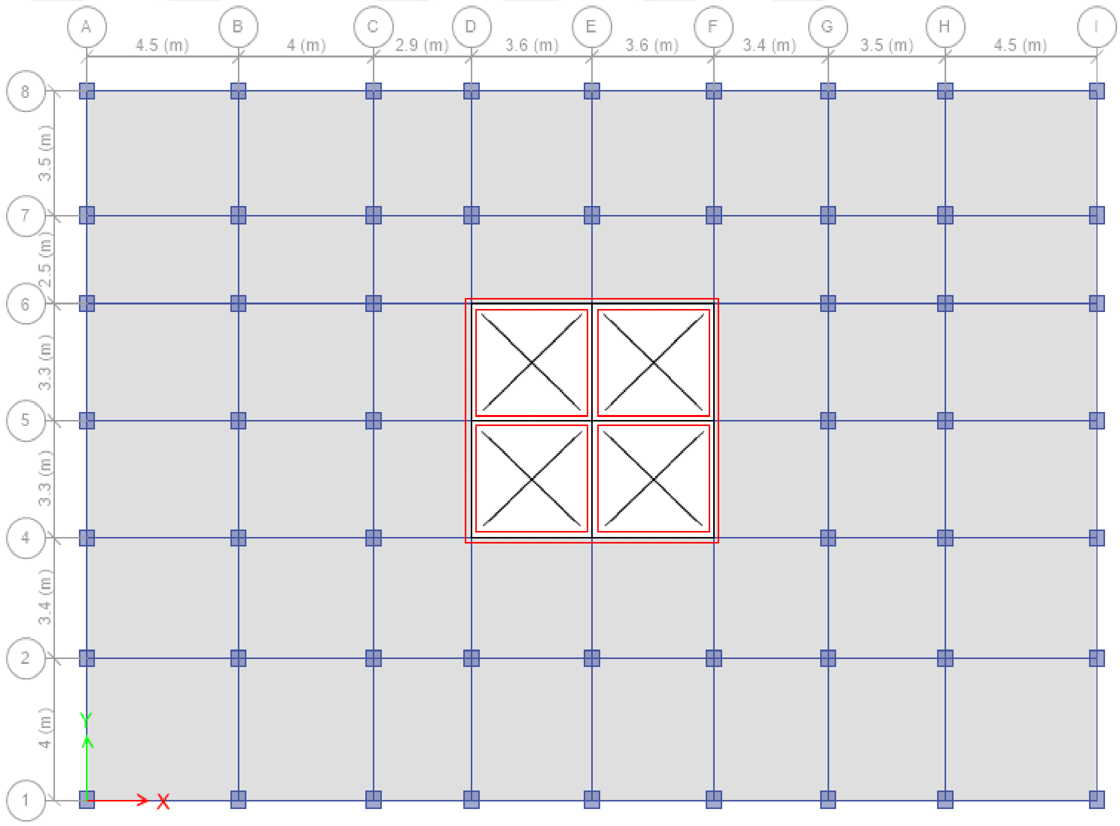


Figure 4.1: (a) Plan view of core wall building

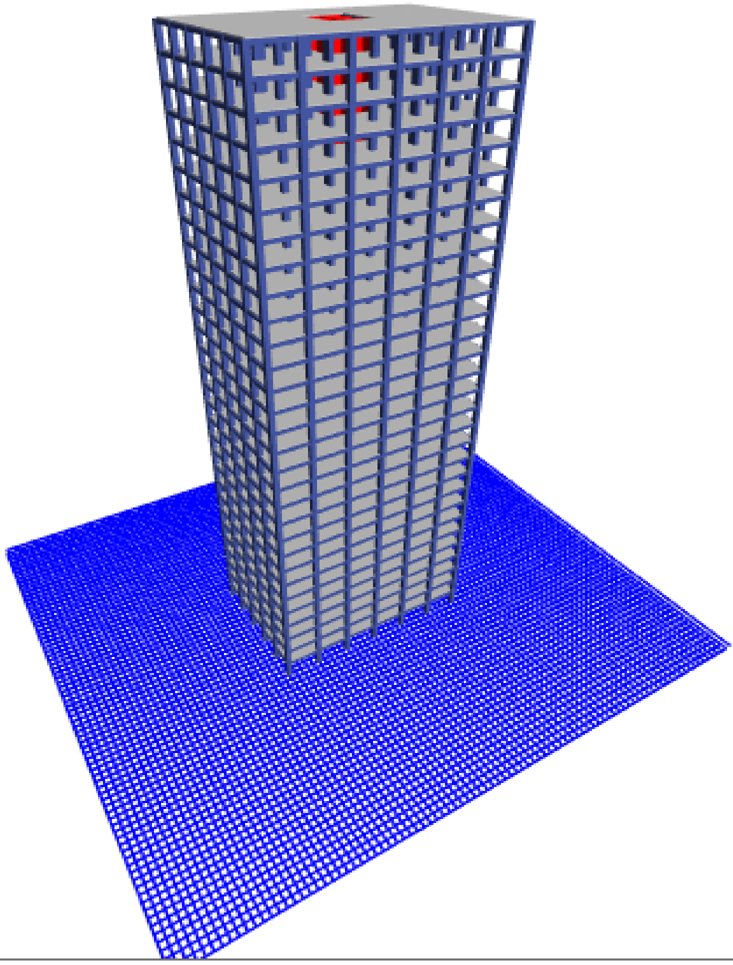


Figure 4.1: (b) Core wall building

Table 4.1. Core wall building specification

Building dimension in X-axis (m)	Building dimension in Y-axis (m)	Building dimension in Z-axis (m)	No. Of stories	Each story Height (m)	Type of structure
30	20	99	30	3.3	Multi-story RC frame structure

Table 4.2. Core wall building elements properties

Section Properties	Dimension (m)	Cross-sectional area (m^2)	Length (m)	Story
Beam	0.50 x 0.50	0.25	Varies	

Column 1	0.55 x 0.55	0.3025	3.3	1-10
Column 2	0.50 x 0.50	0.25	3.3	11-20
Column 3	0.45 x 0.45	0.2025	3.3	21-30

Table 4.3. (a) Core wall building material properties

	Material Properties		Modulus of Elasticity (E)
Design properties for Concrete (Fc`)	Column	4000 Psi	24855 MPa
	Beam	3000 Psi	21525 MPa
	Slab	3000 Psi	21525 MPa

Table 4.3. (b) Core wall building material properties

Design properties for Rebars (Fy)	60 Ksi	Minimum yield strength (Fy)	413.69 MPa
		Minimum tensile strength (Fu)	620.53 MPa
		Expected yield strength (Fy)	455.05 MPa
		Expected tensile strength (Fu)	682.58 MPa

Table 4.4. Core wall building slab and shear wall properties

Define Slab section	Floor Slab thickness	170 mm	As per design
Define Shear Wall	Shear wall thickness	300 mm	Marked as Wall

Table 4.5. Core wall building section properties

Define Frame Section	Clear Cover	40 mm	Design Type
	Beam	Marked as Beam	Reinforcement to be designed
	Column	Marked as Column	Reinforcement to be designed

Table 4.6. Loads for core wall building

Define Mass source	For Dead load	1
	For Live load	0.25

Table 4.7. Code references for core wall building

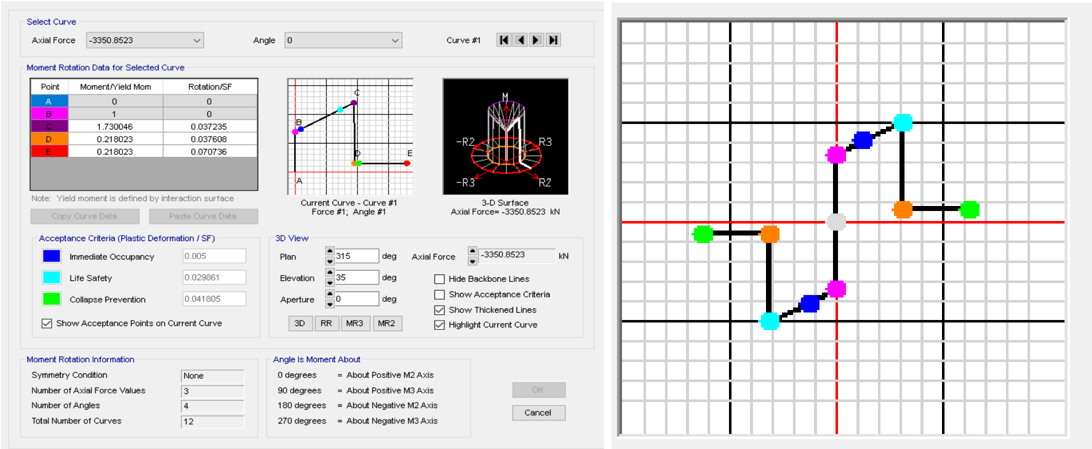
References	RCC Frame design	ACI-318-19
	Slab	
	Shear wall	

Model	Supports	Fixed
Type of occupancy	Residential	

The structure assigned a gravity load composed of a dead load of (1.53 KN/m^2 + self-weight) and 1.44 KN/m^2 live load assigned on each floor. Also, the structure has a mass source combination of (1.0 Dead Load+ 0.25 Live Load).

It is important to choose adequate parameters for beam, column, slab, and other structural members of the building. In order to have balanced range to endure the load coming from the chosen set of records. As well as considering not having over designed structure systems. Therefore, selective design was made beside trial-and-error method for different earthquake records to choose optimum sizes of structure elements and earthquake records for the sake of comparison.

In nonlinear analysis of structures, a plastic / fibre hinge refers to the deformation of a part of beams or columns wherever a plastic bending happens. In those part of the structure that have been stressed to the yield point cannot resist additional stresses, when hinge is applied to a part of structural element, it does have the capacity to resist additional load or stresses and permit free rotation to the element. Therefore, the concept of hinge is very important in order to check the performance of building and considerate the structural failure.



(a) **(b)**

Figure 4.2: (a): Moment-rotation curves (P-M-M) – Column, (b): Rotation hinges – Beam (M3)

4.2.1.1. Beam reinforcement details

Beams are one of the most common structural components. Reinforcements in beam are provided to oppose tensile stresses due to bending and shear in beams. Beams in the core wall building have the (0.5 m x 0.5 m) dimension which is same throughout the building.

Table 4.8. Beam rebars

Area of top reinforcement	1963.43	mm^2
No of 25 mm bars (Top bars)	6	
Area of bottom reinforcement	2945.15	mm^2
No of 25 mm bars (Bottom bars)	4	
Area of steel provided	4909	mm^2
Design shear reinforcement	416.67	mm^2/m
Depth of the beam	500	mm
width of the beam	500	mm
Number of legged stirrups provided	4	
Diameter of stirrups	12	mm
Area of shear reinforcement= 4 x 3.14/4 x 12^2	452.38	mm^2/m
Spacing required	108.57	mm
Spacing provided	100	mm
Provided shear reinforcement of section	452.38	mm^2
	OK	

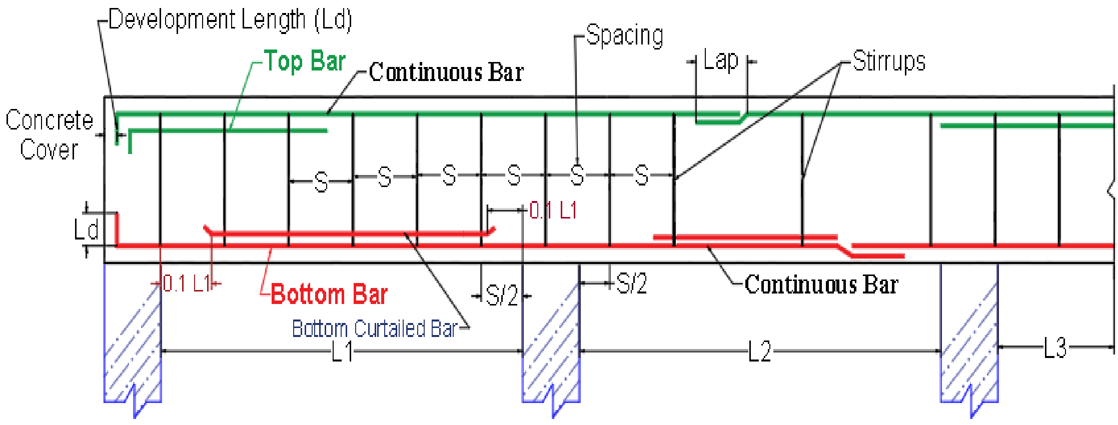


Figure 4.3: Typical Beam in core wall building

The cross section of beam for core wall building is shown in below figure

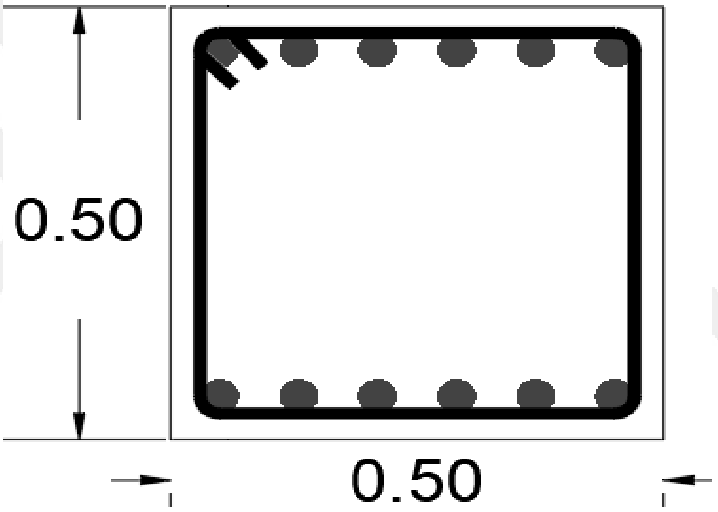


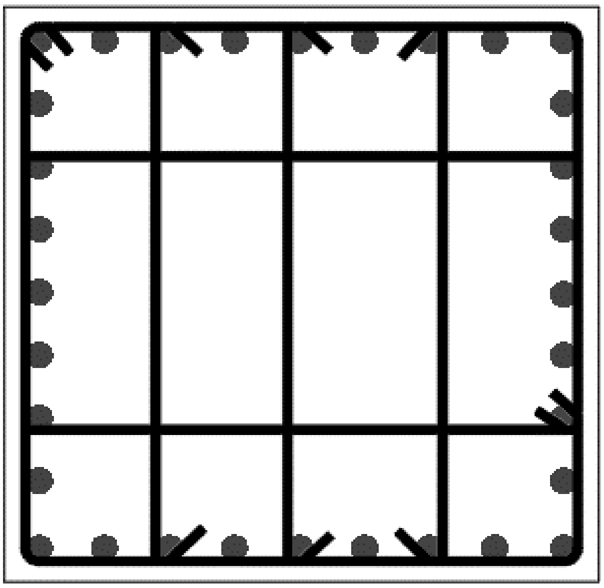
Figure 4.4: Beam section

Table 4.9. Beams shear force (V2) & moment (M3)

Shear force (V2) KN	**Moment (M3) KN-m**
0.23	52.17
2.88	51.59
5.53	50.01
20.32	49.94
23.85	38.90
27.39	26.09
47.21	25.79
50.75	1.30
54.28	-24.96
66.35	-26.54
69.00	-51.92
71.65	-78.29

4.2.1.2. Column reinforcement details

Core wall building contain different size of columns, it has a column size of (0.55 m x 0.55 m) from 1st story up to 10th story and a column size of (0.50 m x 0.50 m) from the 11th story up to 20th story and a column size of (0.45 m x 0.45 m) form 21st story upto 30th story.

Column (0.55 m x 0.55 m)			
B	55	cm	
H	55	cm	
Ac	3025	cm^2	
Reinforcement Required	1%	From ETABS	
As	98.17	cm^2	
As used	122.7	cm^2	
20 #	25		

The columns are designed based on the minimum required reinforcement. And the axial load and moment capacity relationship has been found. The interaction curve data and the graph for column (0.55 x 0.55) are represented below

Table.4.10. Axial force and Moment capacity data

Point	P (KN)	M2 (KN-m)	M3 (KN-m)
1	5679.787	0	0
2	5679.787	265.9438	-265.944
3	5002.613	384.4308	-384.431
4	4166.981	486.5874	-486.587
5	3194.578	574.6519	-574.652
6	1964.803	661.464	-661.464
7	1402.481	745.4608	-745.461
8	501.1922	779.7032	-779.703
9	-805.756	587.5483	-587.548
10	-2518.28	256.0723	-256.072
11	-3655.41	0	0

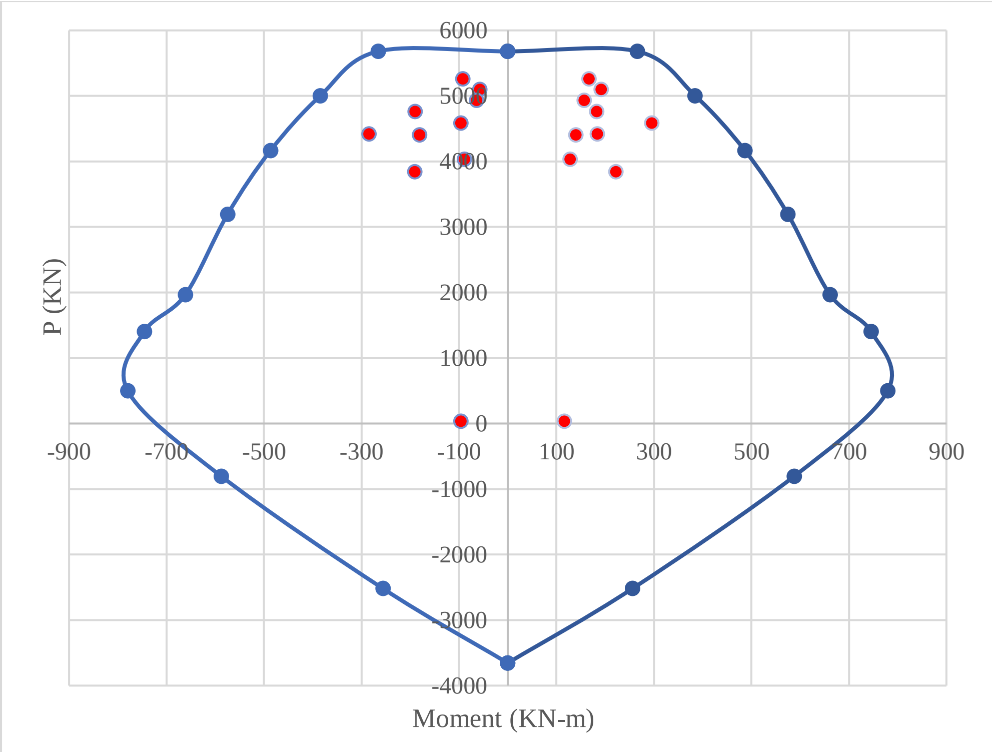
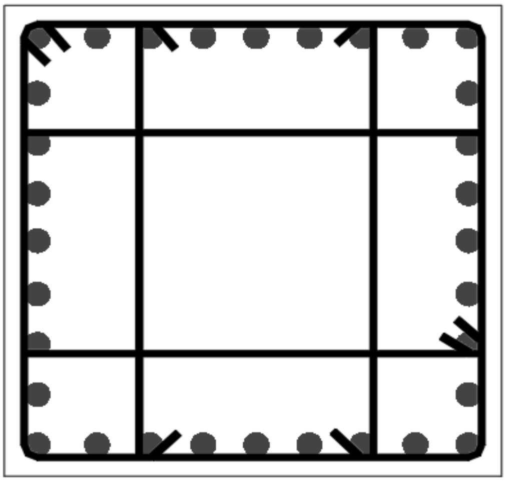


Figure 4.5: Interaction curve of column (C 0.55 x 0.55)

Column (0.50 m x 0.50 m)			
Stirrup's diameter	12	mm	
B	50	cm	
H	50	cm	
Ac	2500	cm^2	
Reinforcement Required	1%	From ETABS	
As	50.26	cm^2	
As used	62.83	cm^2	
	#16	20	

These columns are also designed based on the minimumum reinforcement required. And the axial load and moment capacity relationship has been found. It is data and graph are represented below.

Table 4.11. Axial force and Moment capacity data

Point	P (KN)	M2 (KN-m)	M3 (KN-m)
1	4067.635	0	0
2	4067.635	159.2015	-159.202
3	3677.468	240.6715	-240.672
4	3083.459	305.7656	-305.766
5	2408.856	355.0628	-355.063
6	1616.466	394.0975	-394.098
7	1254.872	435.4391	-435.439
8	718.1873	449.3627	-449.363
9	-131.831	332.9501	-332.95
10	-1189.77	144.3111	-144.311
11	-1871.71	0	0

The moment and axial force curve are a graphical illustration of bending moment (M) and axial force (P) which clarifies the ultimate capacity of the compression member when it is exposed to axial load and moments.

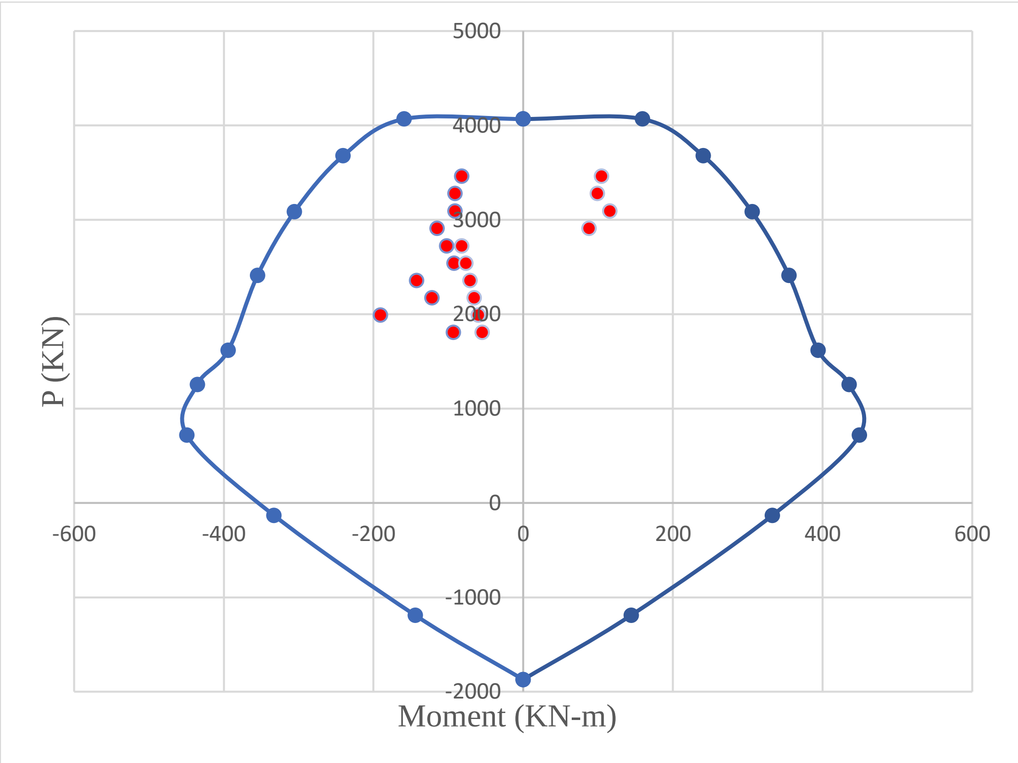


Figure 4.6: Interaction curve of column (0.50 x 0.50)

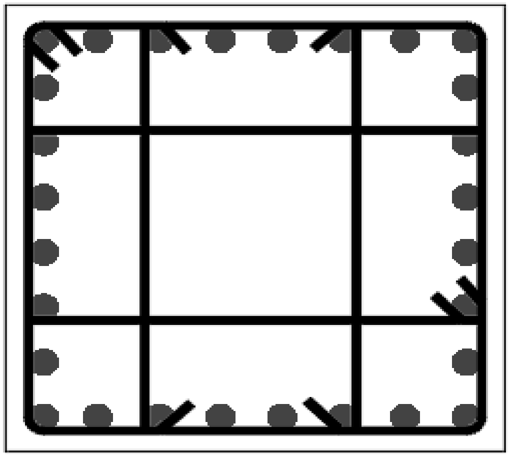
Column (0.45 m x 0.45 m)			
Stirrup's diameter	12	mm	
B	45	cm	
H	45	cm	
Ac	2025	cm^2	
Reinforcement Required	1%	From ETABS	
As	40.71	cm^2	
As used	45.80	cm^2	
	#16	18	

Table 4.12. Axial force and Moment capacity data

Point	P (KN)	M2 (KN-m)	M3 (KN-m)
1	3294.778	0	0
2	3294.778	113.9343	-113.934
3	2974.876	173.3129	-173.313
4	2487.72	220.1617	-220.162
5	1933.241	254.9533	-254.953
6	1275.269	281.3447	-281.345
7	959.312	306.8419	-306.842
8	508.5118	313.5388	-313.539
9	-190.715	227.0547	-227.055
10	-1080.54	85.8736	-85.8736
11	-1516.07	0	0

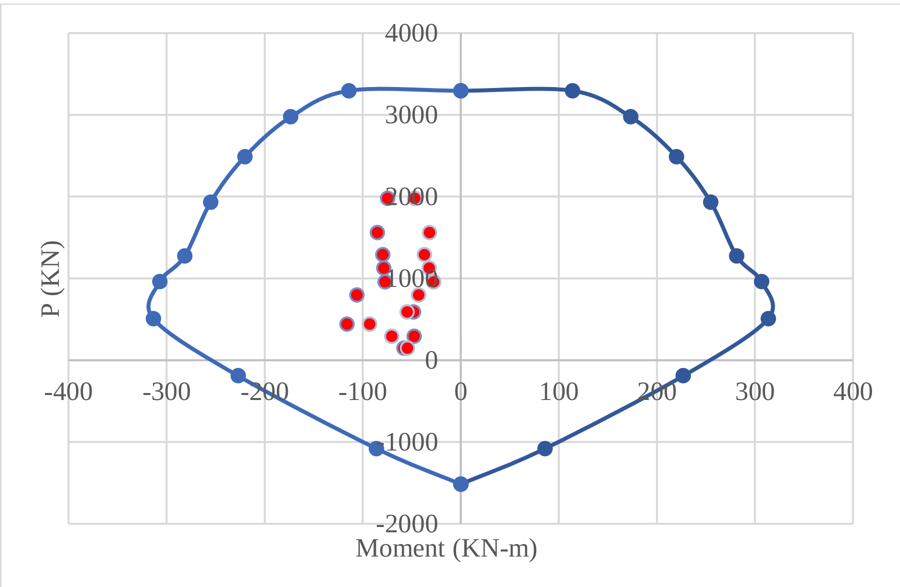


Figure 4.7: Interaction curve of column (0.45 x 0.45)

Shear force of columns in core wall building has been illustrated in below table 4.13.

Table 4.13. Shear force of columns in core wall building

Story	Story Height (m)	Shear force (V2) (KN)	Shear force (V3) (KN)
Story30	99	7.79	-1.47
Story29	95.7	9.25	0.81
Story28	92.4	10.60	-0.46
Story27	89.1	12.09	-1.06
Story26	85.8	13.29	-1.69
Story25	82.5	14.36	-2.23
Story24	79.2	15.28	-2.72
Story23	75.9	16.12	-3.14
Story22	72.6	16.64	-3.53
Story21	69.3	18.27	-3.98
Story20	66	15.57	-3.35
Story19	62.7	16.92	-3.64
Story18	59.4	17.14	-3.86
Story17	56.1	17.45	-4.02
Story16	52.8	17.67	-4.16
Story15	49.5	17.85	-4.26
Story14	46.2	17.98	-4.36
Story13	42.9	18.04	-4.40
Story12	39.6	17.86	-4.45
Story11	36.3	19.06	-4.70
Story10	33	14.90	-3.32
Story9	29.7	15.85	-3.39
Story8	26.4	15.66	-3.42
Story7	23.1	15.62	-3.41
Story6	19.8	15.51	-3.39
Story5	16.5	15.39	-3.36
Story4	13.2	15.24	-3.35
Story3	9.9	15.23	-3.26
Story2	6.6	14.45	-3.53
Story1	3.3	18.50	-2.71

Table 4.14. Demand from load combination & columns shear capacity

Story Height (m)	Demand from load combination (KN)	Shear capacity-V2 (KN)	Demand from load combination (KN)	Shear capacity-V3 (KN)	
99	35.89	138.55	40.32	138.55	
95.7	30.61	138.55	27.83	138.55	Response modification coefficient (R) = 7
92.4	32.32	138.55	29.77	138.55	
89.1	33.47	138.55	29.16	138.55	
85.8	2.46	138.55	46.34	138.55	
82.5	2.2	138.55	47.11	138.55	
79.2	1.94	138.55	47.9	138.55	
75.9	1.71	138.55	48.51	138.55	Overstrength factor (D) = 3
72.6	1.44	138.55	49.69	138.55	
69.3	1.12	138.55	46.76	109.88	
66	1.5	109.88	58.37	109.88	
62.7	1.18	109.88	54.88	109.88	$V_t / R = V_{tn} \times D$
59.4	0.87	109.88	55.78	109.88	
56.1	0.57	137.22	55.78	109.88	
52.8	0.31	137.22	55.75	109.88	
49.5	0.25	109.88	55.53	109.88	
46.2	0.31	109.88	55.14	109.88	
42.9	0.61	109.88	54.45	109.88	
39.6	0.94	107.05	54.13	107.05	
36.3	1.26	100.56	49.99	100.56	
33	1.52	88.63	57.82	88.61	
29.7	1.87	82.58	52.34	82.58	
26.4	2.25	76.56	50.91	76.56	
23.1	2.63	70.59	48.37	70.59	
19.8	3.04	64.68	45.56	64.68	
16.5	3.46	58.82	42.24	58.82	
13.2	3.92	51.45	38.43	51.45	
9.9	4.32	12.76	33.85	46.18	
6.6	4.97	40.91	29.02	40.91	
3.3	6.25	35.63	23.6	35.63	

Table 4.15. Column's capacity ratio in core wall building

Story	Column Section	Location	Capacity ratio
Story30	C 0.45 x 0.45	Top	0.24
Story30	C 0.45 x 0.45	Bottom	0.277
Story29	C 0.45 x 0.45	Top	0.195
Story29	C 0.45 x 0.45	Bottom	0.264
Story28	C 0.45 x 0.45	Top	0.224
Story28	C 0.45 x 0.45	Bottom	0.298
Story27	C 0.45 x 0.45	Top	0.253
Story27	C 0.45 x 0.45	Bottom	0.319
Story26	C 0.45 x 0.45	Top	0.291
Story26	C 0.45 x 0.45	Bottom	0.347
Story25	C 0.45 x 0.45	Top	0.331
Story25	C 0.45 x 0.45	Bottom	0.387
Story24	C 0.45 x 0.45	Top	0.375
Story24	C 0.45 x 0.45	Bottom	0.428
Story23	C 0.45 x 0.45	Top	0.421
Story23	C 0.45 x 0.45	Bottom	0.468
Story22	C 0.45 x 0.45	Top	0.467
Story22	C 0.45 x 0.45	Bottom	0.512
Story21	C 0.45 x 0.45	Top	0.512
Story21	C 0.45 x 0.45	Bottom	0.547
Story20	C 0.50 x 0.50	Top	0.459
Story20	C 0.50 x 0.50	Bottom	0.494
Story19	C 0.50 x 0.50	Top	0.491
Story19	C 0.50 x 0.50	Bottom	0.528
Story18	C 0.50 x 0.50	Top	0.53
Story18	C 0.50 x 0.50	Bottom	0.566
Story17	C 0.50 x 0.50	Top	0.569
Story17	C 0.50 x 0.50	Bottom	0.605
Story16	C 0.50 x 0.50	Top	0.608
Story16	C 0.50 x 0.50	Bottom	0.643
Story15	C 0.50 x 0.50	Top	0.648
Story15	C 0.50 x 0.50	Bottom	0.683
Story14	C 0.50 x 0.50	Top	0.687
Story14	C 0.50 x 0.50	Bottom	0.722
Story13	C 0.50 x 0.50	Top	0.726
Story13	C 0.50 x 0.50	Bottom	0.761
Story12	C 0.50 x 0.50	Top	0.771
Story12	C 0.50 x 0.50	Bottom	0.802
Story11	C 0.50 x 0.50	Top	0.817
Story11	C 0.50 x 0.50	Bottom	0.835
Story10	C 0.55 x 0.55	Top	0.621
Story10	C 0.55 x 0.55	Bottom	0.624
Story9	C 0.55 x 0.55	Top	0.657
Story9	C 0.55 x 0.55	Bottom	0.659
Story8	C 0.55 x 0.55	Top	0.693
Story8	C 0.55 x 0.55	Bottom	0.695
Story7	C 0.55 x 0.55	Top	0.728
Story7	C 0.55 x 0.55	Bottom	0.730

Story6	C 0.55 x 0.55	Top	0.763
Story6	C 0.55 x 0.55	Bottom	0.767
Story5	C 0.55 x 0.55	Top	0.796
Story5	C 0.55 x 0.55	Bottom	0.805
Story4	C 0.55 x 0.55	Top	0.831
Story4	C 0.55 x 0.55	Bottom	0.842
Story3	C 0.55 x 0.55	Top	0.868
Story3	C 0.55 x 0.55	Bottom	0.877
Story2	C 0.55 x 0.55	Top	0.906
Story2	C 0.55 x 0.55	Bottom	0.911
Story1	C 0.55 x 0.55	Top	0.945
Story1	C 0.55 x 0.55	Bottom	0.950

4.2.1.3. Core shear walls reinforcement details

Reinforce concrete core shear walls are the most important structural components of high-rise buildings in order to tolerate horizontal and vertical load; Which provide stability to structures from lateral loads such as wind, seismic loads. These shear walls usually start at foundation level and are continuous throughout the building height. Shear walls are more effectual in resisting lateral loads in high rise buildings.

In Figure 4.8. (a) & (b) the core shear wall is shown with its reinforcement provisions

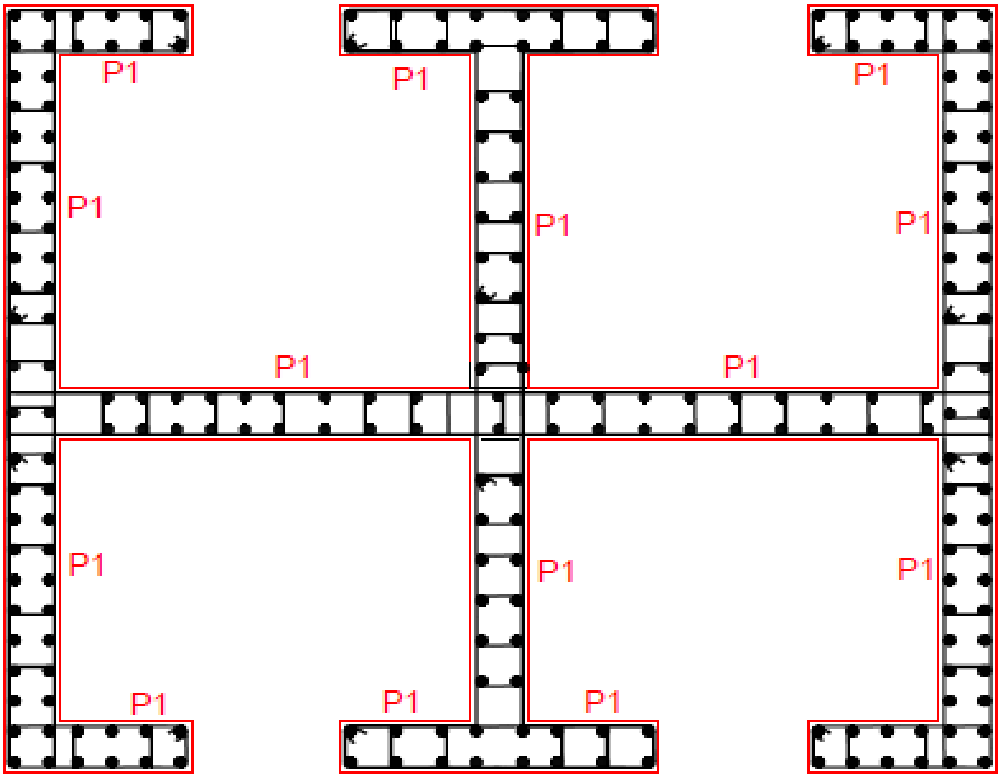


Figure 4.8: (a) Core shear wall

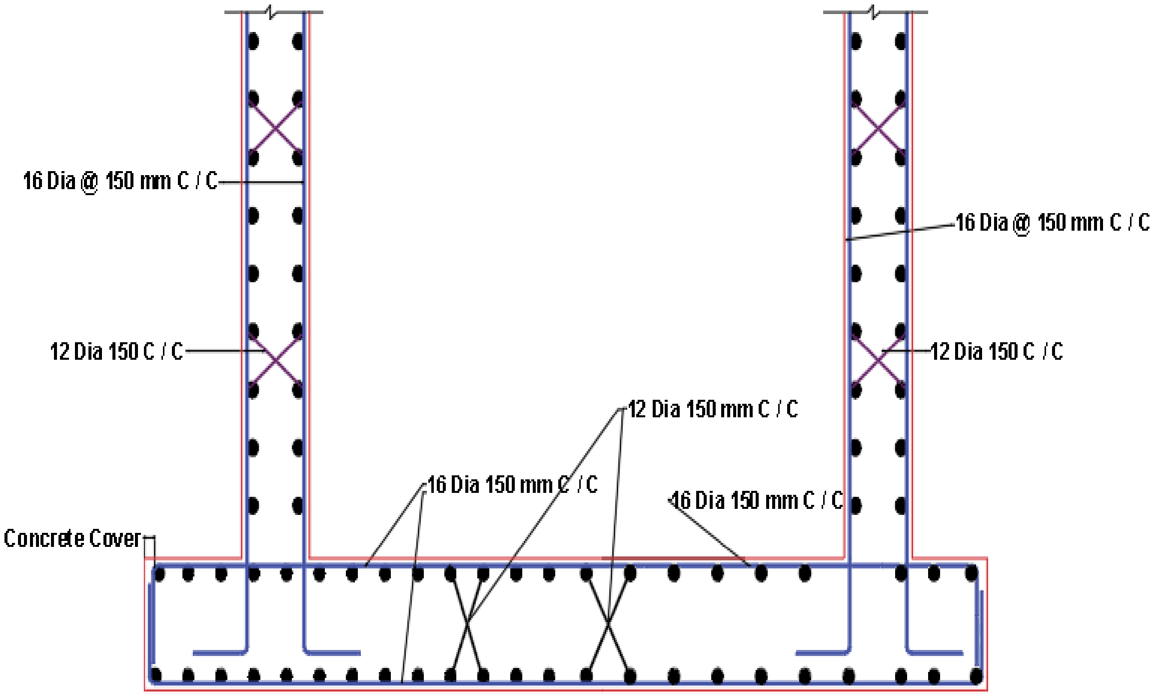


Figure 4.8: (b) Typical Core shear wall

This shear wall section represent the whole core wall as a single pier section of a building. The axial force and moment data and the graph for a core shear walls has been illustrated as below

Table 4.16. Axial force and Moment capacity

Point	P (kN)	M2 (kN-m)	M3 (kN-m)
1	143519.8	-242.036	242.0355
2	135381.7	148535.1	-148535
3	125408.6	174179.3	-174179
4	119981.2	184501.1	-184501
5	110289.6	196974.3	-196974
6	83291.02	209151.9	-209152
7	64541.73	240183.6	-240184
8	61418.4	275654.1	-275654
9	51741.25	263597.2	-263597
10	37572.98	226307.4	-226307
11	-26472.4	355.2574	-355.257

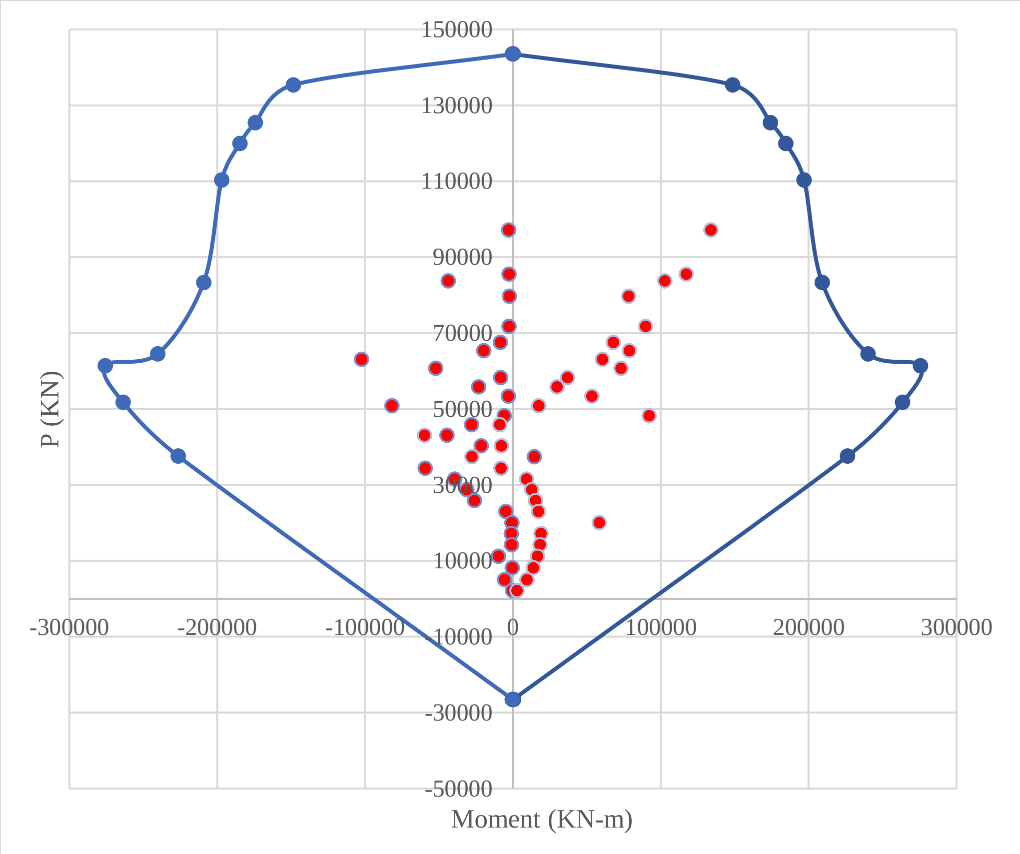


Figure 4.9: Interaction curve of core shear wal

Shear walls must provide the essential lateral strength to resist horizontal earthquake forces. When shear walls are strong enough, they will transfer the horizontal forces to the next element in the load path underneath them. These other components in the load pathway may be other shear walls, floors, foundation walls, slabs, and footings.

In this building system the the shear walls around lift shafts and stairwells as a core wall are used to give the required lateral strength and essential stiffens to the building. The lateral force resisting system for high rise building is occasionally concentrated in relatively few walls disseminated around the floor or within a central core wall to provide adequate lateral strength and stiffness required to limit the lateral deformations to suitable levels. Shear walls give huge strength and stiffness to the structures which significantly diminishes lateral sway of the building and subsequently lessens damage to building and its contents.

Table 4.17. Pier shear and torsional forces in core wall building

Story	Pier	Location	V2 (KN)	V3 (KN)	T (KN-m)
Story30	P1	Top	-8.57	1279.18	-341.60
Story30	P1	Bottom	-5.78	1274.24	-576.48
Story29	P1	Top	-3.71	439.98	-766.98
Story29	P1	Bottom	-3.61	428.57	-1215.17
Story28	P1	Top	-8.02	28.91	-1151.89
Story28	P1	Bottom	-10.57	16.81	-1801.14
Story27	P1	Top	-2.38	-407.09	-1516.19
Story27	P1	Bottom	-2.77	-418.44	-2369.83
Story26	P1	Top	-3.47	-776.76	-1836.95
Story26	P1	Bottom	-3.66	-787.41	-2884.68
Story25	P1	Top	-2.50	-1115.15	-2127.50
Story25	P1	Bottom	-3.36	-1125.32	-3359.82
Story24	P1	Top	-2.83	-1417.27	-2398.76
Story24	P1	Bottom	-3.07	-1427.48	-3796.41
Story23	P1	Top	-2.62	-1690.83	-2648.65
Story23	P1	Bottom	-2.76	-1700.65	-4194.07
Story22	P1	Top	-2.77	-1925.25	-2877.56
Story22	P1	Bottom	-2.47	-1935.10	-4558.09
Story21	P1	Top	-2.69	-2271.99	-3125.21
Story21	P1	Bottom	-1.95	-2281.44	-4936.71
Story20	P1	Top	-3.21	-1884.02	-3079.86
Story20	P1	Bottom	-2.79	-1894.40	-4943.34
Story19	P1	Top	-3.79	-2214.23	-3306.09
Story19	P1	Bottom	-3.53	-2226.49	-5256.87
Story18	P1	Top	-1.44	-2368.78	-3446.60
Story18	P1	Bottom	-3.70	-2380.53	-5498.35
Story17	P1	Top	-2.69	-2528.89	-3581.71
Story17	P1	Bottom	-2.63	-2540.60	-5710.11
Story16	P1	Top	-2.71	-2674.97	-3697.83
Story16	P1	Bottom	-2.79	-2686.54	-5900.28
Story15	P1	Top	-2.64	-2812.94	-3800.17
Story15	P1	Bottom	-2.79	-2824.30	-6067.33
Story14	P1	Top	-2.59	-2944.94	-3889.59
Story14	P1	Bottom	-2.76	-2956.01	-6212.99
Story13	P1	Top	-2.52	-3076.78	-3967.30
Story13	P1	Bottom	-2.67	-3087.43	-6338.85
Story12	P1	Top	-2.42	-3189.96	-4032.95
Story12	P1	Bottom	-2.55	-3200.17	-6444.14
Story11	P1	Top	-2.12	-3458.02	-4137.63

Story11	P1	Bottom	-1.91	-3467.16	-6589.54
Story10	P1	Top	-2.78	-3040.09	-3929.78
Story10	P1	Bottom	-2.91	-3049.08	-6341.14
Story9	P1	Top	-2.47	-3385.38	-4038.92
Story9	P1	Bottom	-2.90	-3394.70	-6461.41
Story8	P1	Top	-2.47	-3549.58	-4071.62
Story8	P1	Bottom	-2.98	-3556.62	-6512.22
Story7	P1	Top	-2.34	-3759.20	-4101.98
Story7	P1	Bottom	-2.89	-3764.19	-6554.22
Story6	P1	Top	-2.15	-3989.94	-4129.26
Story6	P1	Bottom	-2.66	-3992.39	-6588.08
Story5	P1	Top	-1.89	-4252.86	-4158.17
Story5	P1	Bottom	-2.25	-4252.27	-6616.05
Story4	P1	Top	-1.56	-4556.00	-4198.76
Story4	P1	Bottom	-1.65	-4551.76	-6641.74
Story3	P1	Top	-1.10	-4903.18	-4272.39
Story3	P1	Bottom	-0.77	-4894.68	-6668.32
Story2	P1	Top	-0.53	-5341.85	-4469.32
Story2	P1	Bottom	0.1731	-5328.13	-6742.28
Story1	P1	Top	2.2091	-5595.32	-3716.22
Story1	P1	Bottom	5.6772	-5573.98	-5465.50

Table 4.18. Pier D/C ratio in core wall building

Story	Pier Section	Station	Pier	Location	D/C Ratio
Story30	P1	Top	Pier	Top Leg 1	0.017
Story30	P1	Bottom	Pier	Bottom Leg 1	0.037
Story29	P1	Top	Pier	Top Leg 1	0.048
Story29	P1	Bottom	Pier	Bottom Leg 1	0.063
Story28	P1	Top	Pier	Top Leg 1	0.072
Story28	P1	Bottom	Pier	Bottom Leg 1	0.084
Story27	P1	Top	Pier	Top Leg 1	0.093
Story27	P1	Bottom	Pier	Bottom Leg 1	0.103
Story26	P1	Top	Pier	Top Leg 1	0.111
Story26	P1	Bottom	Pier	Bottom Leg 1	0.120
Story25	P1	Top	Pier	Top Leg 1	0.129
Story25	P1	Bottom	Pier	Bottom Leg 1	0.140
Story24	P1	Top	Pier	Top Leg 1	0.146
Story24	P1	Bottom	Pier	Bottom Leg 1	0.161

Story23	P1	Top	Pier	Top Leg 1	0.166
Story23	P1	Bottom	Pier	Bottom Leg 1	0.182
Story22	P1	Top	Pier	Top Leg 1	0.185
Story22	P1	Bottom	Pier	Bottom Leg 1	0.203
Story21	P1	Top	Pier	Top Leg 1	0.204
Story21	P1	Bottom	Pier	Bottom Leg 1	0.223
Story20	P1	Top	Pier	Top Leg 1	0.222
Story20	P1	Bottom	Pier	Bottom Leg 2	0.243
Story19	P1	Top	Pier	Top Leg 1	0.242
Story19	P1	Bottom	Pier	Bottom Leg 1	0.265
Story18	P1	Top	Pier	Top Leg 1	0.261
Story18	P1	Bottom	Pier	Bottom Leg 1	0.287
Story17	P1	Top	Pier	Top Leg 1	0.281
Story17	P1	Bottom	Pier	Bottom Leg 1	0.308
Story16	P1	Top	Pier	Top Leg 1	0.300
Story16	P1	Bottom	Pier	Bottom Leg 1	0.330
Story15	P1	Top	Pier	Top Leg 1	0.320
Story15	P1	Bottom	Pier	Bottom Leg 1	0.353
Story14	P1	Top	Pier	Top Leg 1	0.339
Story14	P1	Bottom	Pier	Bottom Leg 1	0.375
Story13	P1	Top	Pier	Top Leg 1	0.359
Story13	P1	Bottom	Pier	Bottom Leg 1	0.397
Story12	P1	Top	Pier	Top Leg 1	0.379
Story12	P1	Bottom	Pier	Bottom Leg 1	0.418
Story11	P1	Top	Pier	Top Leg 1	0.399
Story11	P1	Bottom	Pier	Bottom Leg 1	0.44
Story10	P1	Top	Pier	Top Leg 1	0.419
Story10	P1	Bottom	Pier	Bottom Leg 1	0.460
Story9	P1	Top	Pier	Top Leg 1	0.438
Story9	P1	Bottom	Pier	Bottom Leg 1	0.482
Story8	P1	Top	Pier	Top Leg 1	0.457
Story8	P1	Bottom	Pier	Bottom Leg 1	0.510
Story7	P1	Top	Pier	Top Leg 1	0.476
Story7	P1	Bottom	Pier	Bottom Leg 1	0.541
Story6	P1	Top	Pier	Top Leg 1	0.495
Story6	P1	Bottom	Pier	Bottom Leg 1	0.578
Story5	P1	Top	Pier	Top Leg 1	0.524
Story5	P1	Bottom	Pier	Bottom Leg 1	0.618
Story4	P1	Top	Pier	Top Leg 1	0.555

Story4	P1	Bottom	Pier	Bottom Leg 1	0.666
Story3	P1	Top	Pier	Top Leg 1	0.591
Story3	P1	Bottom	Pier	Bottom Leg 1	0.721
Story2	P1	Top	Pier	Top Leg 1	0.637
Story2	P1	Bottom	Pier	Bottom Leg 1	0.794
Story1	P1	Top	Pier	Top Leg 1	0.690
Story1	P1	Bottom	Pier	Bottom Leg 1	0.883

The shear capacity of shear wall in core wall building along the height of the building has been illustrated in the following table individually for each pier.

Table 4.19. Core shear capacity along the story height

Story Height (m)	Pier	Demand from load combination (KN)	Shear capacity-V2 (KN)	Shear capacity-V3 (KN)	
99	P1	193.52	666.05	1192.62	
95.7	P1	178.39	666.05	1192.62	Response modification coefficient (R) = 7
92.4	P1	168.3	666.05	1192.62	
89.1	P1	338.23	666.05	1192.62	
85.8	P1	486.17	666.05	1192.62	
82.5	P1	634.98	670.34	1192.62	
79.2	P1	757.43	675.73	1192.62	
75.9	P1	862.24	670.46	1193.74	Overstrength factor (D) = 3
72.6	P1	959.46	669.77	1195.37	
69.3	P1	1114	668.79	1195.80	
66	P1	967.36	666.05	1192.62	
62.7	P1	1126.49	666.05	1192.62	$V_t / R = V_{tn} \times D$
59.4	P1	1185.41	666.05	1196.19	
56.1	P1	1241.76	666.05	1196.74	
52.8	P1	1300.91	666.05	1195.33	
49.5	P1	1356.07	666.05	1196.31	
46.2	P1	1408.02	666.05	1192.62	
42.9	P1	1458.85	666.05	1192.62	

39.6	P1	1499.75	666.05	1192.62	
36.3	P1	1611.91	666.05	1192.62	
33	P1	1433.72	666.05	1192.62	
29.7	P1	1566.8	666.05	1192.62	
26.4	P1	1615.14	666.05	1192.62	
23.1	P1	1679.13	660.26	1192.62	
19.8	P1	1748.01	661.71	1192.62	
16.5	P1	1825.79	661.50	1191.09	
13.2	P1	1961.1	660.04	1189.46	
9.9	P1	2023.39	657.86	1188.04	
6.6	P1	2197.28	654.94	1187.19	
3.3	P1	2313.05	651.60	11940.47	

We know that tensile strain is the fractional increase in length of an object / element ($\varepsilon = \Delta\ell/\ell0$) due to a tensile stress. Compressive strain is the fractional decrease in length of an object or element ($\varepsilon = \Delta\ell/\ell0$) due to a compressive stress. Generally, strain is defined as the amount of deformation in the direction of the applied force divided by the initial length of the element. The compressive strain of concrete and tensile strain of the rebars in shear wall pier section of core wall building are illustrated below.

Table 4.20. Strains in core shear wall

Story	**Compressive Strain** mm/mm	**Tensile Strain** mm/mm
1	0.00059	0.00245
2	0.00055	0.00308
3	0.00052	0.00368
4	0.00048	0.00423
5	0.00045	0.00474
6	0.00042	0.00522
7	0.00040	0.00566
8	0.00037	0.00608
9	0.00035	0.00648
10	0.00032	0.00683
11	0.00030	0.00722
12	0.00028	0.00759
13	0.00026	0.00795
14	0.00023	0.00815

15	0.00021	0.00831
16	0.00019	0.00848
17	0.00018	0.00860
18	0.00017	0.00802
19	0.00018	0.00873
20	0.00015	0.00870
21	0.00015	0.00863
22	0.00014	0.00894
23	0.00014	0.00897
24	0.00013	0.00900
25	0.00012	0.00855
26	0.00010	0.00871
27	0.00009	0.00868
28	0.00007	0.00889
29	0.00005	0.00762
30	0.00002	0.00864

4.2.1.4. Slab reinforcement details

Slab is a structural element, typically made up of reinforced concrete. They help in transferring the loads father to beams. Slabs are classified into one way and two-way slab depending upon the aspect ratio and the supports on the corresponding edges. And other different types

In core wall building the slabs are supported by beams and beams are supported by columns or shear walls. The slab has to carry a distributed permanent load of 1.53 KN/m^2 (excluding slab self weight) and a variable load of 1.44 KN/m^2 as live load
In below figure 4.9 shown the number and size of bars at the top and bottom of the layer A and layer B of the design strips, which designate the top and bottom reinforcements of the slab.

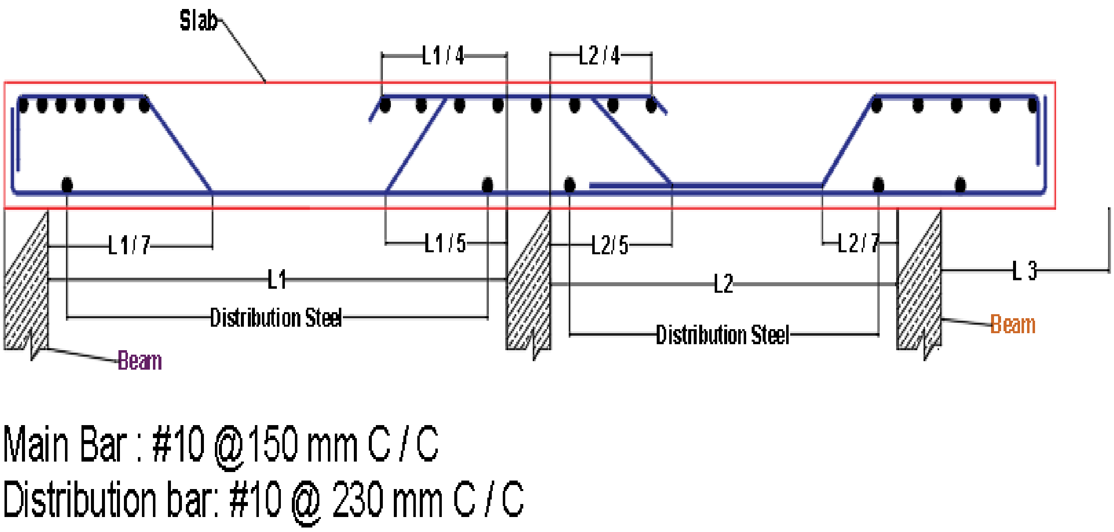


Figure 4.10: Typical Slab in core wall building

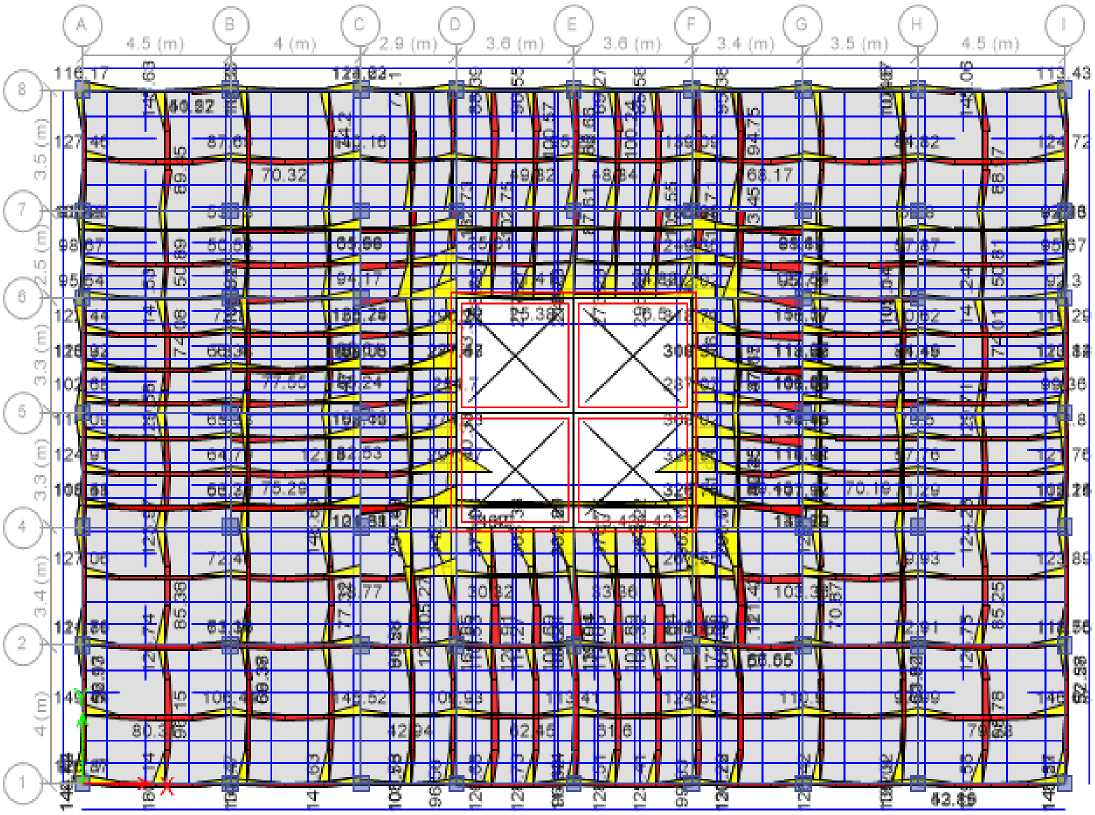


Figure 4.11: Bars in layer A and B of slab

ACI 318-19 recommends the effective stiffness value of 0.25 for bending stiffness for slabs. So, a conservative value of (0.25 Ig) is used for effective stiffness of slabs in linear and nonlinear analyses.

4.2.2. Tunnel formwork building design

The Tunnel formwork building is designed with a geometrical plan dimension of (30 m) to the x-axis, (20 m) to the y-axis, and (99 m) to the z-axis. As shown in figure 4.13. (a), (b). The building has a slab thickness of (0.3 m) and a shear wall of (0.35 m) thick which is the same for the entire building but the reinforcement is varying in upper floors. This kind of building has no beams and column elements.

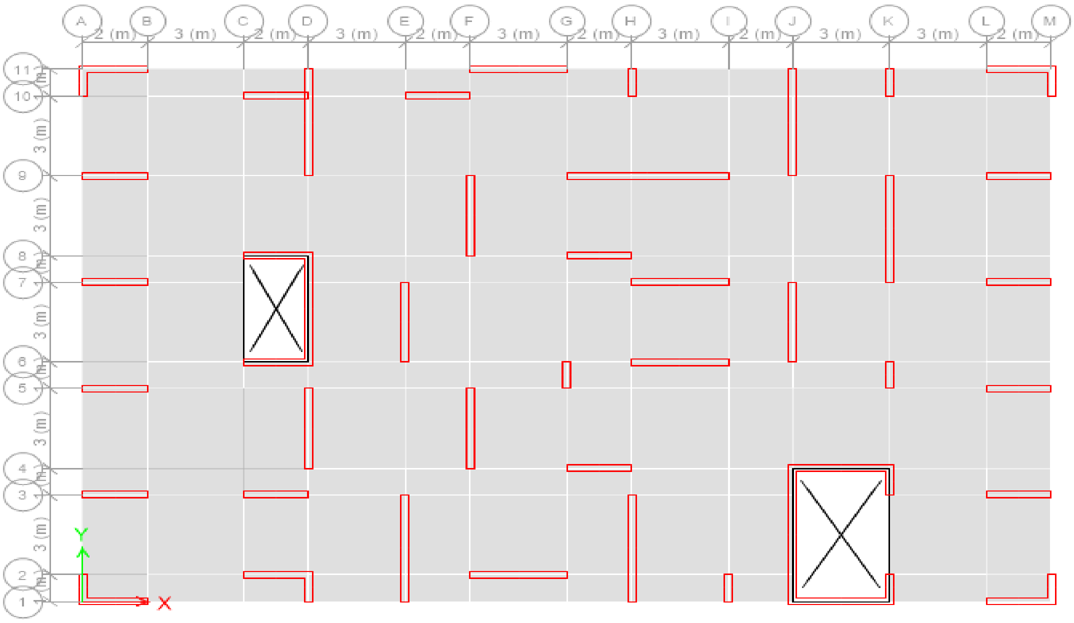


Figure 4.12: (a) plan view of tunnel formwork building

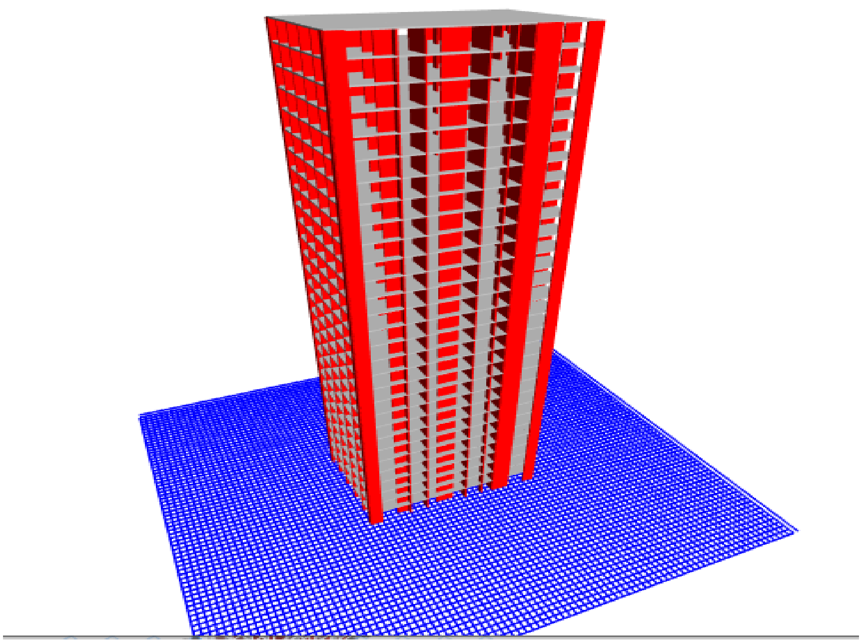


Figure 4.12: (b) Tunnel formwork building

Table 4.21. Tunnel formwork building specification

Building dimension in X-axis (m)	Building dimension in Y-axis (m)	Building dimension in Z-axis (m)	No. Of stories	Each story Height (m)	Type of structure
30	20	99	30	3.3	Multi-story RC tunnel formwork structure

Table 4.22. Tunnel formwork building element's properties

Section Properties	Thickness (m)	Material
Slab	0.30	RCC
Shear Wall	0.35	RCC
Beam	No Beam	
Column	No Column	

Here, the structure assigned a gravity load composed of a dead load of (1.53 KN/m^2 + self-weight) and 1.44 KN/m^2 live load assigned on each floor. Also, the structure has a mass source combination of (1.0 Dead Load + 0.25 Live Load).

Table 4.23. Tunnel formwork building material properties

Material Properties		Slab	Modulus of Elasticity (E)
	Fc`	4000 Psi	24855 MPa

Design properties for Rebars (Fy)	60 Ksi	Minimum yield strength (Fy)	413.69 MPa
		Minimum tensile strength (Fu)	620.53 MPa
		Expected yield strength (Fy)	455.05 MPa
		Expected tensile strength (Fu)	682.58 MPa

Table 4.24. Tunnel formwork building slab and shear wall properties

Define Slab section	Floor Slab thickness	300 mm	As per design

Define Shear Wall	Shear wall thickness	350 mm	Marked as wall

Table 4.25. Loads for tunnel formwork building

Define Mass source	For Dead load	1
	For Live load	0.25

Table 4.26. Code references for tunnel formwork building

References	RCC Frame design	ACI-318-19
	Slab	
	Shear wall	

Model	Supports	Fixed
Type of occupancy	Residential	

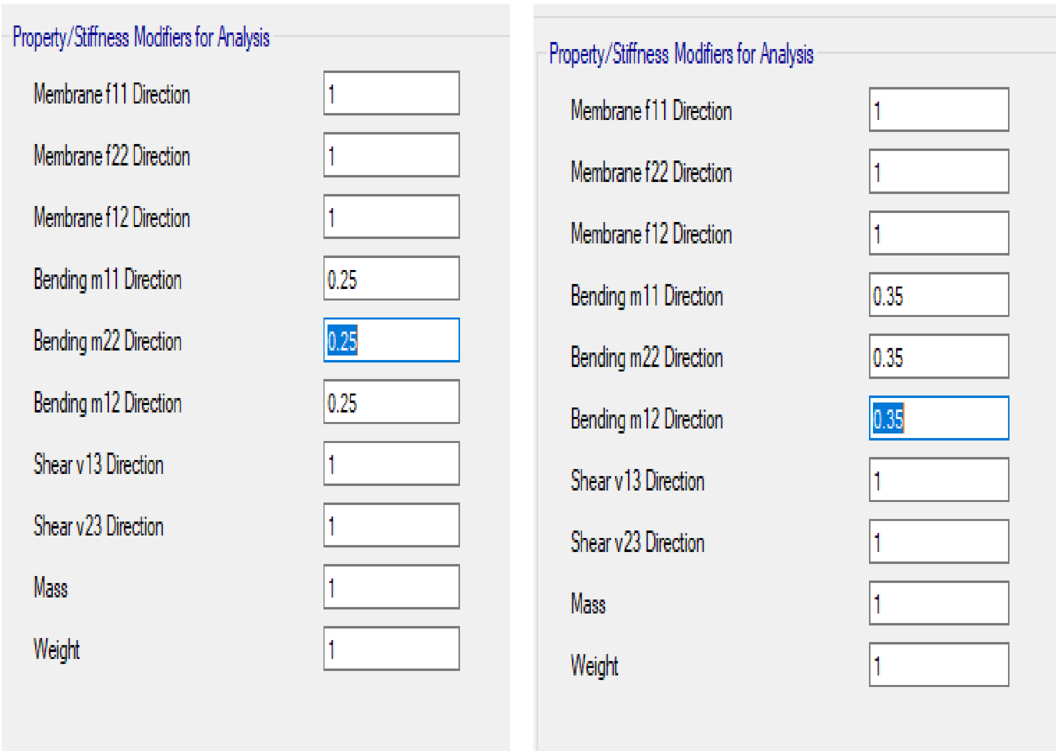


Figure 4.13: Stiffness modifiers for slabs and shear walls

4.2.2.1. Shear wall reinforcement details

Tunnel formwork building which have shear walls only as a lateral and vertical load resisting elements and a flat slab without any beam and column. Which have good stiffness due to having numerous shear walls. The following figure displays all the shear walls existed in tunnel formwork building.

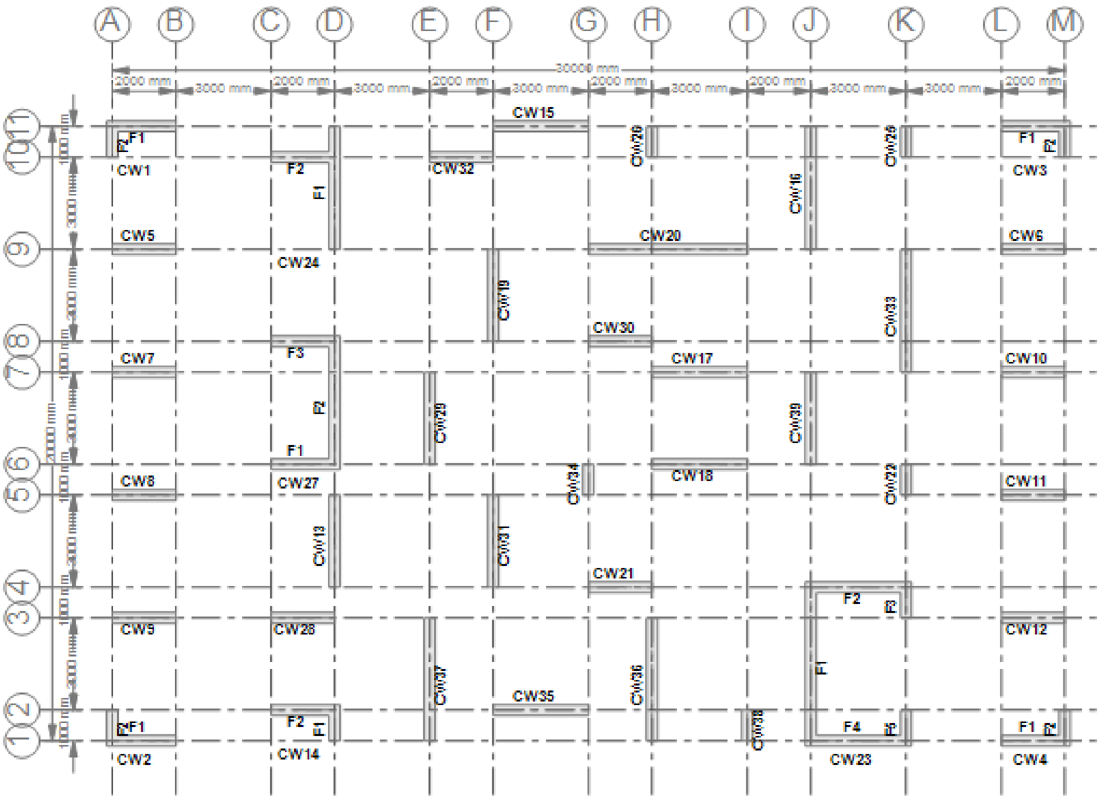


Figure 4.14: Tunnel formwork building shear walls layout

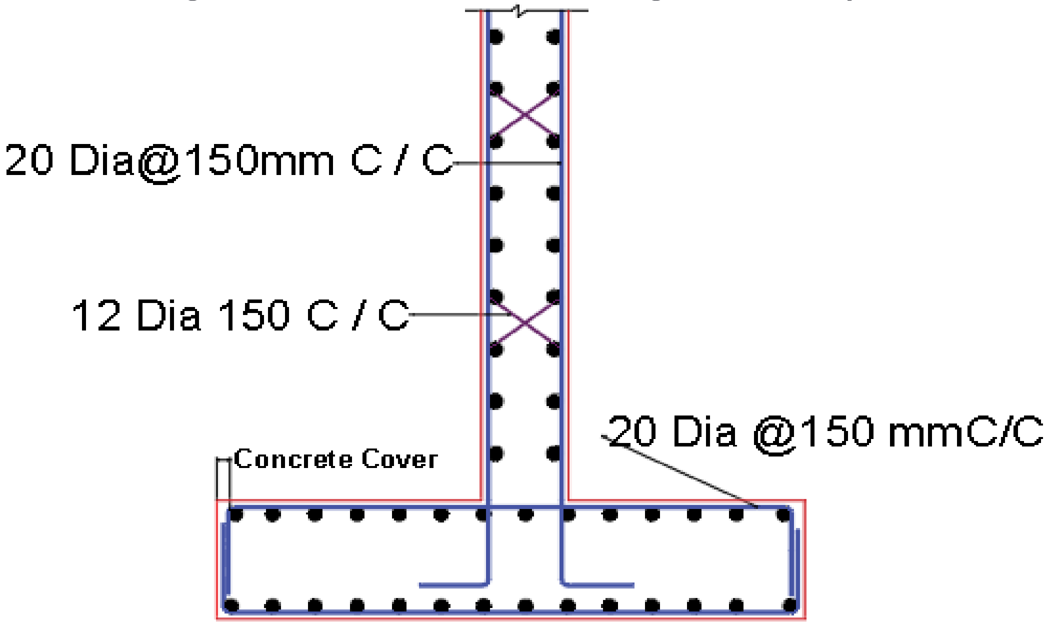


Figure 4.15: Typical Shear wall in tunnel formwork building

The shear walls in tunnel formwork building have been designed according to the minimum requirement of the reinforcement. All the shear walls in tunnel formwork building are having the same thickness only the length is different and reinforcement is variable in upper floors but they are following the same design strategy.

Table 4.27. Axial force and Moment capacity (5m x 0.35 m)

Point	P (KN)	M2 (KN-m)	M3 (KN-m)
1	25795.55	0	0
2	25795.55	8605.256	-8605.26
3	24621.16	13745.61	-13745.6
4	21349.33	17698.46	-17698.5
5	17864	20505.13	-20505.1
6	14008.95	22297.68	-22297.7
7	11699.63	25945.63	-25945.6
8	8460.563	27888.62	-27888.6
9	2984.296	21825.76	-21825.8
10	-2481.18	12723.84	-12723.8
11	-8188.74	0	0

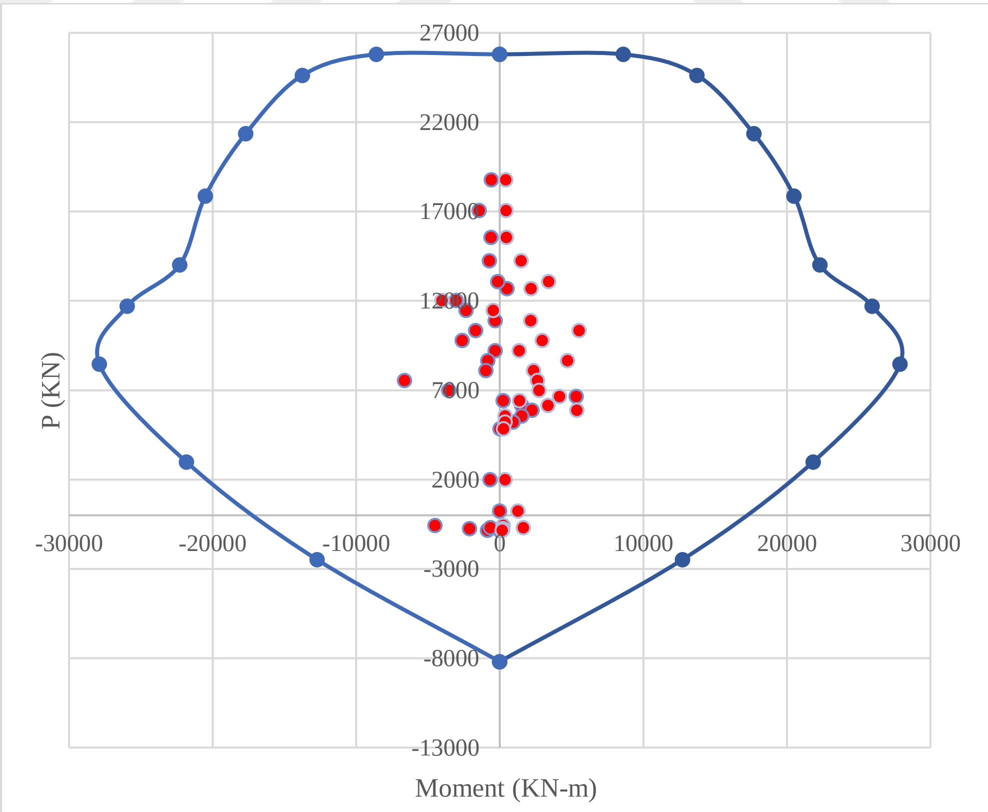


Figure 4.16: Interaction curve of (5 m x 0.35 m) shear wall

Table 4.28. Axial force and Moment capacity (4 m x 0.35 m)

Point	P (KN)	M2 (KN-m)	M3 (KN-m)
1	15545.96	0	0
2	15545.96	3737.585	-3737.59
3	15273.88	6387.715	-6387.72
4	13305.83	8315.387	-8315.39
5	11291.99	9531.718	-9531.72
6	9202.59	10054.71	-10054.7
7	8279.88	11364.08	-11364.1
8	6983.482	11957.55	-11957.6
9	4105.454	9187.067	-9187.07
10	1223.803	5216.445	-5216.44
11	-1684.36	0	0

From the above data of axial force and moment capacity data plot the axial force and moment relation curve.

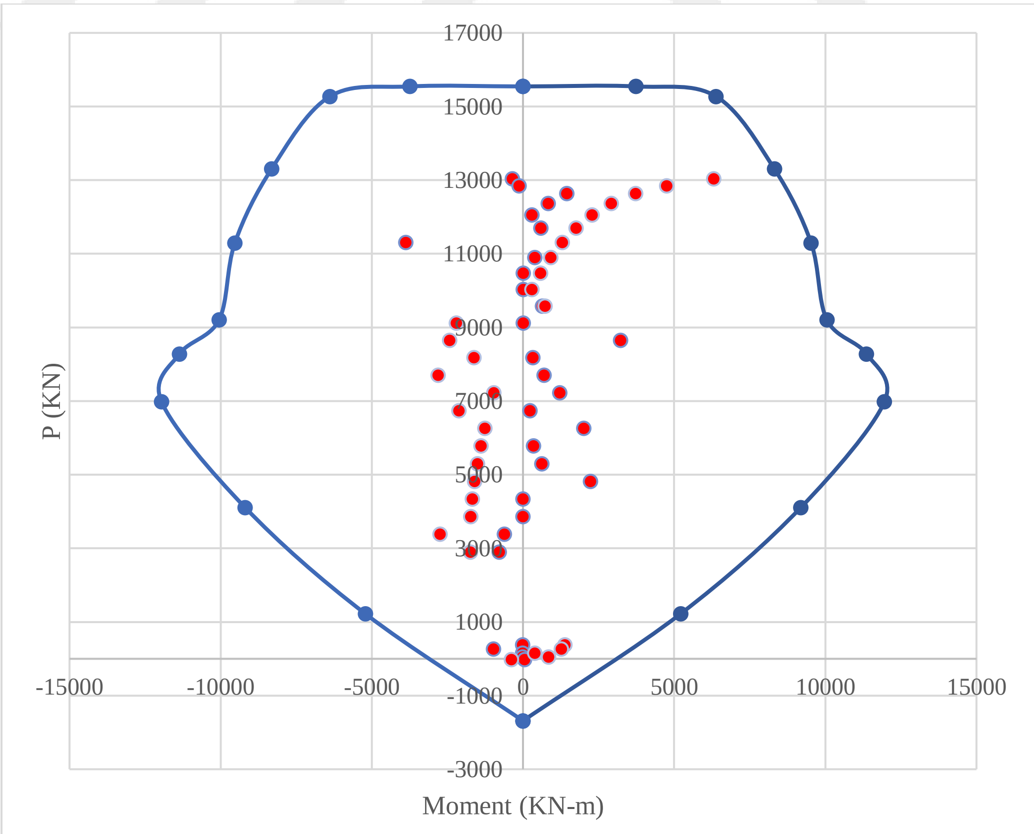


Figure 4.17: Interaction curve of (4 m x 0.35 m) shear wall

Table 4.29. Axial force and Moment capacity (3 m x 0.35 m)

Point	P (KN)	M2 (KN-m)	M3 (KN-m)
1	11934.88	0	0
2	11934.88	2197.681	-2197.68
3	11646.58	3709.196	-3709.2
4	10125.6	4814.822	-4814.82
5	8558.544	5521.892	-5521.89
6	6907.092	5848.075	-5848.08
7	6126.987	6634.525	-6634.53
8	5032.937	6996.493	-6996.49
9	2769.477	5380.774	-5380.77
10	502.3487	3048.512	-3048.51
11	-1768.58	0	0

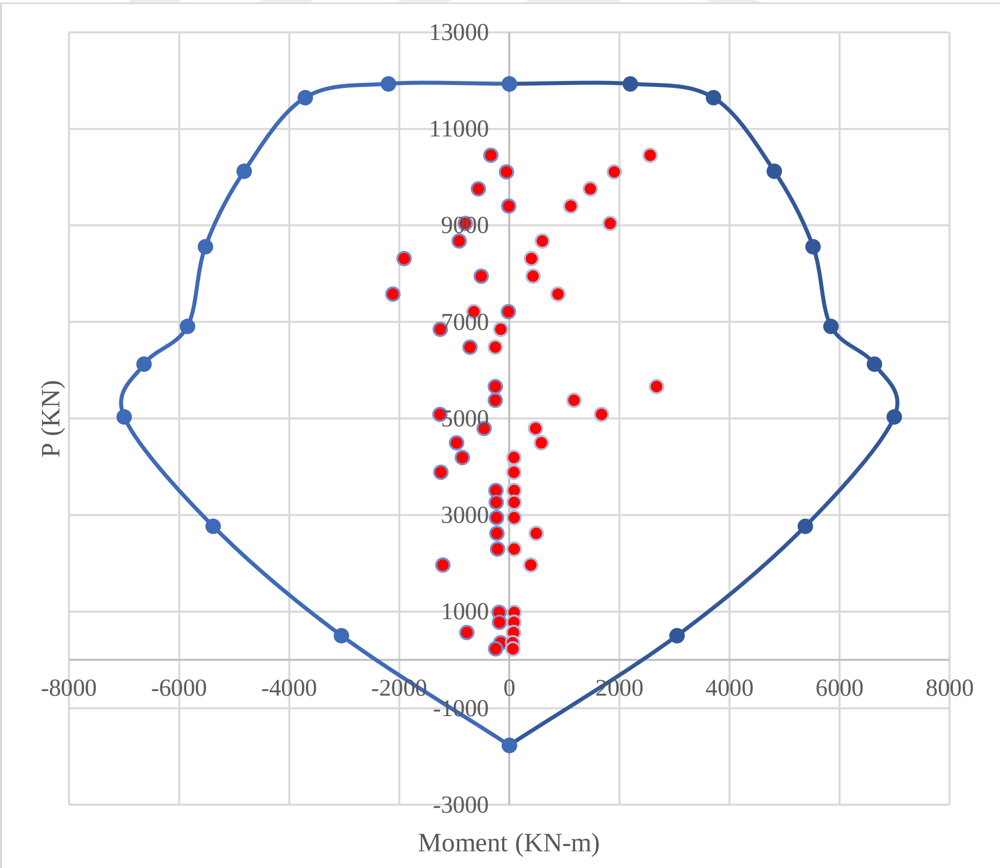


Figure 4.18: Interaction curve of (3 m x 0.35 m) shear wall

Table 4.30. Axial force and Moment capacity (2 m x 0.35 m)

Point	P (KN)	M2 (KN-m)	M3 (KN-m)
1	7864.783	0	0
2	7864.783	967.9136	-967.914
3	7675.031	1639.416	-1639.42
4	6659.554	2127.002	-2127
5	5616.251	2433.083	-2433.08
6	4523.732	2562.491	-2562.49
7	4040.567	2885.211	-2885.21
8	3361.675	3028.559	-3028.56
9	1909.201	2322.221	-2322.22
10	462.9068	1323.932	-1323.93
11	-1010.62	0	0

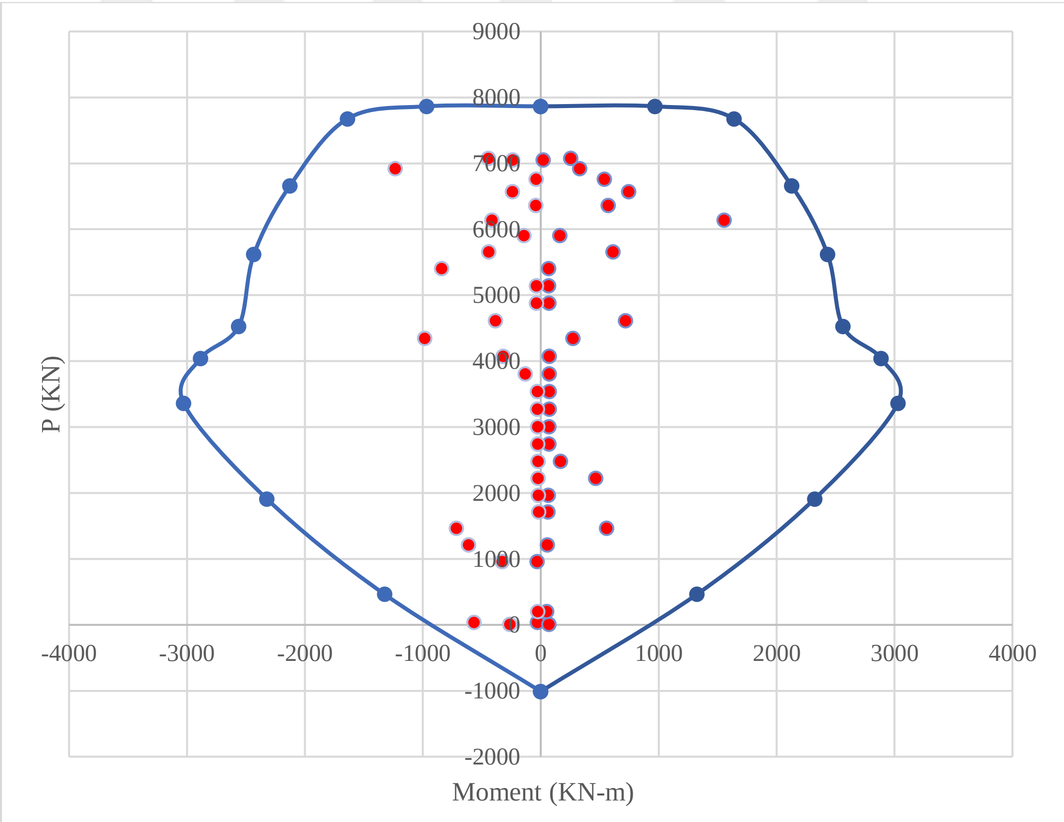


Figure 4.19: Interaction curve of (2 m x 0.35 m) shear wall

Table 4.31. Axial force and Moment capacity (1 m x 0.35 m)

Point	P (KN)	M2 (KN-m)	M3 (KN-m)
1	5414.148	0	0
2	5414.148	389.1362	-389.136
3	5037.32	608.9692	-608.969
4	4303.996	778.0073	-778.007
5	3496.307	902.3138	-902.314
6	2578.374	985.9933	-985.993
7	2081.67	1122.029	-1122.03
8	1373.371	1199.231	-1199.23
9	303.9457	947.8139	-947.814
10	-909.838	529.2729	-529.273
11	-2105.68	0	0

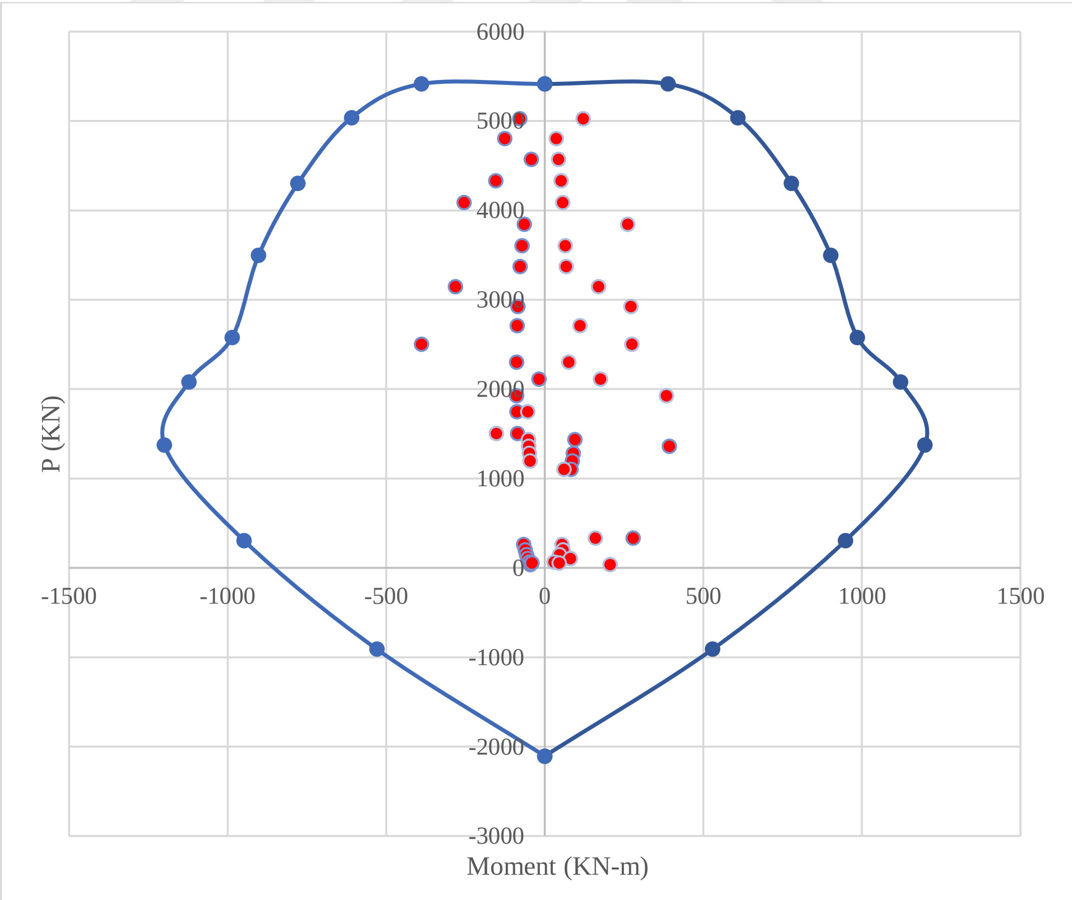


Figure 4.20: Interaction curve of (1 m x 0.35 m) shear wall

Table 4.32. Pier shear forces in tunnel formwork building

	P-5		P-4		P-3		P-2		P-1	
Story	**V2 (KN)**	**V3 (KN)**	**V2 (KN)**	**V3 (KN)**	**V2 (KN)**	**V3 (KN)**	**V2 (KN)**	**V3 (KN)**	**V2 (KN)**	**V3 (KN)**
Story30	112.98	-1.6	-55.09	143.14	34.03	41.92	15.10	-7.22	5.29	3.54
Story29	52.79	0.56	-35.87	107.77	34.57	31.25	13.64	-5.12	1.57	3.19
Story28	14.68	-0.3	-32.81	120.10	36.34	33.57	10.16	-5.15	4.12	3.57
Story27	-29.08	0.18	-30.17	125.81	41.58	34.00	11.33	-4.79	3.43	3.99
Story26	-66.81	0.01	-26.27	133.52	45.75	34.85	10.41	-4.44	4.55	4.48
Story25	-102.92	0.21	-21.92	141.35	50.76	35.73	10.57	-4.02	4.83	5.01
Story24	-130.17	0.40	-17.37	149.27	55.53	36.66	10.30	-3.58	5.60	5.58
Story23	-132.39	0.59	-12.70	157.02	60.53	37.60	11.16	-3.11	6.18	6.17
Story22	-128.46	0.78	-8.10	164.43	65.46	38.55	12.29	-2.63	6.89	6.77
Story21	-123.99	0.97	-3.60	171.36	70.41	39.47	13.49	-2.12	7.54	7.38
Story20	-119.40	1.15	0.70	177.69	75.27	40.35	14.68	-1.61	8.22	7.98
Story19	-114.59	1.34	4.78	183.33	80.06	41.18	15.90	-1.10	8.86	8.56
Story18	-109.53	1.53	8.58	188.20	84.71	41.92	17.15	-0.59	9.48	9.13
Story17	-104.16	1.72	12.07	192.23	89.20	42.57	18.43	-0.09	10.04	9.67
Story16	-98.43	1.90	15.23	195.35	93.49	43.11	19.74	0.40	10.55	10.17
Story15	-92.26	2.09	18.02	197.49	97.54	43.50	21.09	0.87	10.99	10.63
Story14	-85.61	2.28	20.40	198.57	101.32	43.74	22.48	1.31	11.34	11.03
Story13	-78.40	2.47	22.34	198.51	104.76	43.79	23.92	1.71	11.59	11.37
Story12	-70.57	2.65	23.77	197.22	107.83	43.62	25.41	2.08	11.70	11.62
Story11	-62.05	2.84	24.64	194.55	110.45	43.19	26.97	2.39	11.70	11.78
Story10	-52.79	3.03	24.83	190.37	112.60	42.46	28.62	2.63	11.46	11.83
Story9	-42.73	3.22	24.25	184.46	114.13	41.37	30.35	2.81	11.20	11.74
Story8	-31.85	3.41	22.70	176.58	115.13	39.88	32.23	2.88	10.41	11.47
Story7	-20.19	3.60	19.99	166.40	115.18	37.88	34.18	2.85	9.97	11.00
Story6	-7.81	3.80	15.74	153.53	114.96	35.30	36.50	2.69	8.16	10.27
Story5	4.83	4.00	9.64	137.42	113.05	32.01	38.59	2.37	8.12	9.23
Story4	17.51	4.22	0.82	117.44	112.93	27.87	42.25	1.84	3.83	7.79
Story3	27.99	4.42	-10.41	92.68	109.34	22.65	43.73	1.11	6.64	5.87
Story2	37.18	4.51	-21.40	62.02	120.50	16.42	55.98	-0.16	-5.08	3.17
Story1	34.75	3.22	1.53	25.15	175.33	7.11	55.89	0.04	10.65	0.93

Table 4.33. Pier D/C ratio in tunnel formwork building

Story	Station	D/C Ratio	D/C Ratio	D/C Ratio	D/C Ratio	D/C Ratio
	Pier Section	**P-5**	**P-4**	**P-3**	**P-2**	**P-1**
Story30	Top	0.181	0.197	0.954	0.316	0.661
Story30	Bottom	0.286	0.147	0.694	0.199	0.523
Story29	Top	0.271	0.447	0.5	0.171	0.459
Story29	Bottom	0.24	0.434	0.475	0.143	0.437
Story28	Top	0.323	0.454	0.484	0.167	0.497
Story28	Bottom	0.275	0.405	0.422	0.149	0.46
Story27	Top	0.333	0.406	0.42	0.16	0.482
Story27	Bottom	0.271	0.359	0.368	0.164	0.453
Story26	Top	0.305	0.376	0.384	0.183	0.492
Story26	Bottom	0.233	0.335	0.338	0.188	0.458
Story25	Top	0.248	0.362	0.371	0.207	0.481
Story25	Bottom	0.23	0.315	0.351	0.212	0.449
Story24	Top	0.244	0.352	0.384	0.232	0.469
Story24	Bottom	0.253	0.321	0.364	0.237	0.437
Story23	Top	0.263	0.345	0.397	0.261	0.444
Story23	Bottom	0.272	0.336	0.377	0.267	0.415
Story22	Top	0.281	0.362	0.409	0.291	0.442
Story22	Bottom	0.289	0.355	0.396	0.297	0.442
Story21	Top	0.296	0.377	0.287	0.322	0.466
Story21	Bottom	0.305	0.376	0.282	0.327	0.468
Story20	Top	0.239	0.27	0.301	0.261	0.328
Story20	Bottom	0.246	0.273	0.3	0.266	0.331
Story19	Top	0.249	0.288	0.317	0.284	0.344
Story19	Bottom	0.255	0.291	0.316	0.291	0.347
Story18	Top	0.258	0.308	0.333	0.309	0.358
Story18	Bottom	0.265	0.315	0.332	0.316	0.361
Story17	Top	0.274	0.335	0.348	0.335	0.37
Story17	Bottom	0.278	0.342	0.347	0.341	0.374
Story16	Top	0.292	0.362	0.362	0.36	0.388
Story16	Bottom	0.299	0.369	0.362	0.367	0.393
Story15	Top	0.314	0.389	0.376	0.385	0.416
Story15	Bottom	0.321	0.396	0.375	0.392	0.421
Story14	Top	0.336	0.416	0.388	0.411	0.444
Story14	Bottom	0.343	0.423	0.388	0.418	0.449
Story13	Top	0.357	0.444	0.40	0.436	0.473
Story13	Bottom	0.365	0.451	0.402	0.443	0.477
Story12	Top	0.379	0.471	0.415	0.461	0.501
Story12	Bottom	0.39	0.478	0.422	0.468	0.506
Story11	Top	0.401	0.499	0.399	0.486	0.529
Story11	Bottom	0.415	0.505	0.405	0.493	0.534

Story10	Top	0.385	0.479	0.42	0.461	0.506
Story10	Bottom	0.404	0.485	0.426	0.467	0.512
Story9	Top	0.405	0.504	0.441	0.483	0.538
Story9	Bottom	0.431	0.51	0.447	0.489	0.544
Story8	Top	0.425	0.528	0.463	0.504	0.57
Story8	Bottom	0.458	0.534	0.469	0.51	0.576
Story7	Top	0.462	0.552	0.484	0.524	0.603
Story7	Bottom	0.493	0.558	0.49	0.53	0.609
Story6	Top	0.503	0.575	0.505	0.543	0.636
Story6	Bottom	0.531	0.581	0.511	0.549	0.642
Story5	Top	0.55	0.597	0.526	0.561	0.675
Story5	Bottom	0.573	0.605	0.532	0.567	0.68
Story4	Top	0.603	0.619	0.547	0.577	0.715
Story4	Bottom	0.617	0.633	0.554	0.583	0.721
Story3	Top	0.664	0.639	0.568	0.59	0.754
Story3	Bottom	0.67	0.663	0.581	0.596	0.76
Story2	Top	0.734	0.666	0.588	0.601	0.793
Story2	Bottom	0.74	0.7	0.613	0.607	0.799
Story1	Top	0.802	0.705	0.619	0.604	0.829
Story1	Bottom	0.808	0.76	0.662	0.621	0.835

Shear capacities for shear wall in tunnel formwork building has been demonstrated in the following tables.

Table 4.34. Pier-1 shear capacity along the story height (5 m x 0.35 m)

Story Height (m)	Pier -1	Demand from load combination (KN)	Shear capacity-V3 (KN)	Shear capacity-V3 (KN)	
99	P-5	1277.15	587.81	1089.0	
95.7	P-5	853.38	633.96	1135.2	Response modification coefficient (R) = 6.5
92.4	P-5	616.8	633.96	1227.5	
89.1	P-5	388.37	633.96	1181.3	
85.8	P-5	270.73	792.46	1283.3	
82.5	P-5	394.01	726.27	1227.5	
79.2	P-5	530.36	633.96	1135.2	
75.9	P-5	649.94	633.96	1135.2	Overstrength factor (D) = 3
72.6	P-5	753.31	680.12	1181.3	
69.3	P-5	844.07	633.96	1227.5	
66	P-5	924.02	633.96	1135.2	
62.7	P-5	995.59	772.43	1186.0	Vt / R = V_{tn} x D
59.4	P-5	1060.65	633.96	1135.2	

56.1	P-5	1121.16	633.96	1135.2	
52.8	P-5	1178.96	633.96	1135.2	
49.5	P-5	1235.92	633.96	1135.2	
46.2	P-5	1293.94	633.96	1135.2	
42.9	P-5	1355	633.96	1135.2	
39.6	P-5	1421.28	633.96	1135.2	
36.3	P-5	1494.97	633.96	1135.2	
33	P-5	1578.7	633.96	1135.2	
29.7	P-5	1674.58	633.96	1135.2	
26.4	P-5	1785.68	633.96	1135.2	
23.1	P-5	1911.83	633.96	1135.2	
19.8	P-5	2055.16	633.96	1135.2	
16.5	P-5	2201.96	633.96	1135.2	
13.2	P-5	2345.24	633.96	1127.3	
9.9	P-5	2400.43	633.96	1107.9	
6.6	P-5	2304.91	587.81	1103.3	
3.3	P-5	1477.13	577.29	1085.4	

Table 4.35. Pier-2 shear capacity along the story height (4 m x 0.35 m)

Story Height (m)	**Pier -2**	Demand from load combination (KN)	Shear capacity-V3 (KN)	Shear capacity-V3 (KN)	
99	P-4	225.31	496.38	883.94	
95.7	P-4	235.05	507.17	886.52	Response modification coefficient (R) = 6.5
92.4	P-4	260.79	507.17	891.05	
89.1	P-4	295.67	507.17	888.66	
85.8	P-4	323.68	517.32	908.13	
82.5	P-4	350.47	507.17	908.13	
79.2	P-4	373.39	507.17	908.13	
75.9	P-4	394.18	507.17	908.13	Overstrength factor (D) = 3
72.6	P-4	412.32	507.17	908.13	
69.3	P-4	428.41	507.17	997.48	
66	P-4	442.4	507.17	1038.46	
62.7	P-4	454.54	507.17	918.74	V_t / R = V_{tn} x D
59.4	P-4	464.91	507.17	922.90	
56.1	P-4	473.65	567.78	992.31	
52.8	P-4	480.88	599.48	925.20	
49.5	P-4	486.7	507.17	919.67	
46.2	P-4	491.27	507.17	908.13	
42.9	P-4	494.73	507.17	898.90	

39.6	P-4	497.29	507.17	861.97	
36.3	P-4	499.13	545.02	913.66	
33	P-4	500.64	512.71	912.74	
29.7	P-4	502.04	645.63	908.13	
26.4	P-4	504.21	647.48	908.13	
23.1	P-4	507.25	507.17	881.77	
19.8	P-4	513.99	581.02	881.82	
16.5	P-4	523.65	507.17	859.67	
13.2	P-4	547.9	507.17	861.97	
9.9	P-4	583.02	471.14	835.67	
6.6	P-4	689.55	466.09	831.97	
3.3	P-4	706.84	461.86	813.51	

Table 4.36. Pier-3 shear capacity along the story height (3 m x 0.35 m)

Story Height (m)	**Pier -3**	Demand from load combination (KN)	Shear capacity-V3 (KN)	Shear capacity-V3 (KN)	
99	P-3	134.7	378.53	681.10	
95.7	P-3	135.61	383.37	681.10	Response modification coefficient (R) = 6.5
92.4	P-3	147.65	380.38	681.10	
89.1	P-3	168.81	380.38	681.10	
85.8	P-3	185.16	388.12	681.10	
82.5	P-3	202.78	390.07	690.33	
79.2	P-3	218.47	392.17	681.10	
75.9	P-3	223.64	380.38	727.25	Overstrength factor (D) = 3
72.6	P-3	247.45	380.38	681.10	
69.3	P-3	260.2	380.38	729.10	
66	P-3	271.7	380.38	681.10	
62.7	P-3	281.96	406.44	681.10	Vt / R = V_{tn} x D
59.4	P-3	290.9	380.38	685.71	
56.1	P-3	298.53	401.95	681.17	
52.8	P-3	304.92	380.38	681.10	
49.5	P-3	310.01	380.38	681.10	
46.2	P-3	313.71	380.38	726.95	
42.9	P-3	315.99	380.38	681.10	
39.6	P-3	316.85	380.38	681.10	
36.3	P-3	316.2	380.38	681.10	
33	P-3	314.08	380.38	743.87	
29.7	P-3	310.26	396.39	681.10	
26.4	P-3	305	397.18	681.10	
23.1	P-3	297.59	380.38	681.10	

19.8	P-3	289.4	380.38	681.10	
16.5	P-3	277.76	380.38	681.10	
13.2	P-3	269.65	380.38	681.10	
9.9	P-3	255.03	380.38	678.61	
6.6	P-3	270.44	376.24	676.52	
3.3	P-3	376.41	380.38	646.71	

Table 4.37. Pier-4 shear capacity along the story height (2 m x 0.35 m)

Story Height (m)	**Pier -4**	Demand from load combination (KN)	Shear capacity-V2 (KN)	Shear capacity-V3 (KN)	
99	P-2	64.26	219.51	415.44	
95.7	P-2	61.51	221.76	420.23	Response modification coefficient (R) = 6.5
92.4	P-2	74.15	228.23	428.70	
89.1	P-2	87.67	228.23	428.70	
85.8	P-2	100.66	228.23	428.70	
82.5	P-2	112.62	228.23	428.70	
79.2	P-2	123.8	228.23	428.70	
75.9	P-2	134.1	230.22	428.70	Overstrength factor (D) = 3
72.6	P-2	143.58	251.76	428.70	
69.3	P-2	152.25	255.00	433.32	
66	P-2	160.13	228.23	436.58	
62.7	P-2	167.25	259.61	437.87	Vt / R = V_{tn} x D
59.4	P-2	173.62	228.23	428.70	
56.1	P-2	179.27	228.23	441.60	
52.8	P-2	184.21	228.23	446.51	
49.5	P-2	188.49	228.23	428.70	
46.2	P-2	192.1	228.23	437.94	
42.9	P-2	195.09	228.23	439.73	
39.6	P-2	197.48	263.30	441.63	
36.3	P-2	199.29	265.75	442.82	
33	P-2	200.55	258.69	443.31	
29.7	P-2	201.31	228.23	443.83	
26.4	P-2	201.54	228.23	428.70	
23.1	P-2	201.68	226.38	428.70	
19.8	P-2	200.68	224.07	428.70	
16.5	P-2	200.04	222.69	428.70	
13.2	P-2	198.43	221.63	428.70	
9.9	P-2	198.06	218.12	428.70	
6.6	P-2	209.17	217.15	428.70	
3.3	P-2	243.53	214.71	428.70	

Table 4.38. Pier-5 shear capacity along the story height (1 m x 0.35 m)

Story Height (m)	**Pier -5**	Demand from load combination (KN)	Shear capacity-V2 (KN)	Shear capacity-V3 (KN)	
99	P-1	113.15	79.94	184.77	
95.7	P-1	47.24	84.53	184.77	Response modification coefficient (R) = 6.5
92.4	P-1	50.45	87.35	184.77	
89.1	P-1	69.43	84.53	184.77	
85.8	P-1	82.82	87.76	184.77	
82.5	P-1	80.55	91.45	194.12	
79.2	P-1	86.74	84.53	196.31	
75.9	P-1	88.28	84.53	184.77	Overstrength factor (D) = 3
72.6	P-1	92.11	91.94	184.77	
69.3	P-1	94.38	84.53	203.45	
66	P-1	97.09	84.53	194.00	
62.7	P-1	99.08	84.53	184.77	Vt / R = V_{tn} x D
59.4	P-1	100.9	84.53	184.77	
56.1	P-1	102.19	89.31	207.99	
52.8	P-1	103.08	93.76	184.77	
49.5	P-1	103.46	84.53	184.77	
46.2	P-1	103.31	84.53	184.77	
42.9	P-1	102.6	84.53	184.77	
39.6	P-1	101.24	84.53	184.77	
36.3	P-1	99.31	84.53	184.77	
33	P-1	96.49	84.53	184.77	
29.7	P-1	93.24	84.53	184.77	
26.4	P-1	88.49	84.53	184.77	
23.1	P-1	84.06	84.53	184.77	
19.8	P-1	76.25	84.53	184.77	
16.5	P-1	71.93	84.53	182.66	
13.2	P-1	57.6	84.53	180.24	
9.9	P-1	69.04	81.76	179.82	
6.6	P-1	25.98	80.83	177.80	
3.3	P-1	56.24	78.78	176.03	

The concrete compressive and steel rebar tensile strains (mm/mm) of different shear wall pier sections in tunnel formwork building are illustrated in below table.

Table 4.39. Strains for shear wall piers in tunnel formwork building

	Comp strain	Tensile Strain	Comp strain	Tensile Strain	Comp strain	Tensile Strain	Comp strain	Tensile Strain	Comp strain	Tensile Strain
	P-05		P-04		P-03		P-02		P-01	
1	0.00051	0.0029	0.0003	0.0047	0.00066	0.0035	0.0004	0.0049	0.0006	0.0025
2	0.00047	0.0034	0.0002	0.0049	0.00058	0.0037	0.0003	0.0051	0.0005	0.0028
3	0.00042	0.0040	0.0002	0.0048	0.00052	0.0039	0.0003	0.0053	0.0005	0.0032
4	0.00038	0.0045	0.0002	0.0049	0.00046	0.0041	0.0003	0.0054	0.0005	0.0035
5	0.00035	0.0049	0.0002	0.0050	0.00042	0.0044	0.0003	0.0056	0.0004	0.0038
6	0.00031	0.0052	0.0003	0.0051	0.00037	0.0046	0.0003	0.0058	0.0004	0.0041
7	0.00029	0.0057	0.0003	0.0053	0.00034	0.0048	0.0003	0.0060	0.0004	0.0044
8	0.00030	0.0061	0.0002	0.0055	0.00030	0.0050	0.0003	0.0062	0.0003	0.0047
9	0.00028	0.0063	0.0003	0.0057	0.00027	0.0053	0.0003	0.0063	0.0003	0.0049
10	0.00025	0.0065	0.0003	0.0059	0.00024	0.0056	0.0002	0.0065	0.0003	0.0052
11	0.00022	0.0068	0.0003	0.0062	0.00021	0.0060	0.0002	0.0068	0.0003	0.0053
12	0.00020	0.0070	0.0003	0.0064	0.00019	0.0060	0.0002	0.0069	0.0002	0.0057
13	0.00017	0.0072	0.0003	0.0066	0.00018	0.0064	0.0002	0.0071	0.0002	0.0061
14	0.00015	0.0077	0.0003	0.0069	0.00018	0.0068	0.0002	0.0073	0.0002	0.0065
15	0.00013	0.0076	0.0003	0.0071	0.00018	0.0075	0.0002	0.0076	0.0002	0.0068
16	0.00011	0.0077	0.0002	0.0073	0.00018	0.0076	0.0001	0.0078	0.0003	0.0073
17	0.00009	0.0078	0.0002	0.0075	0.00017	0.0077	0.0001	0.0077	0.0003	0.0075
18	0.00013	0.0079	0.0002	0.0076	0.00023	0.0078	0.0001	0.0078	0.0002	0.0076
19	0.00014	0.0079	0.0002	0.0078	0.00022	0.0080	0.0002	0.0079	0.0002	0.0077
20	0.00013	0.0081	0.0002	0.0079	0.00022	0.0081	0.0002	0.0080	0.0002	0.0078
21	0.00013	0.0086	0.0002	0.0084	0.00021	0.0082	0.0001	0.0085	0.0002	0.0082
22	0.00012	0.0088	0.0002	0.0086	0.00020	0.0086	0.0002	0.0087	0.0002	0.0086
23	0.00011	0.0090	0.0002	0.0088	0.00019	0.0087	0.0001	0.0087	0.0002	0.0088
24	0.00010	0.0090	0.0002	0.0090	0.00017	0.0088	0.0001	0.0100	0.0002	0.0090
25	0.00009	0.0090	0.0002	0.0088	0.00009	0.0090	0.0001	0.0100	0.0001	0.0090
26	0.00008	0.0090	0.0001	0.0086	0.00007	0.0086	0.0001	0.0100	0.0001	0.0089
27	0.00006	0.0090	0.0001	0.0079	0.00006	0.0090	0.0001	0.0100	0.0001	0.0091
28	0.00005	0.0090	0.0001	0.0076	0.00005	0.0089	0.0001	0.0100	0.0001	0.0090
29	0.00003	0.0090	0.0002	0.0070	0.00003	0.0089	0.0001	0.0100	0.0001	0.0090
30	0.00001	0.0100	0.0001	0.0066	0.00003	0.0090	0.0001	0.0100	0.0001	0.0090

4.2.2.2. Slab reinforcement details

Tunnel formwork building have a flat slab as a reinforced concrete (RC) slab supported directly on and built monolithically with the shear walls. The flat slab is divided into strips; The size of each strip is well-defined using certain rules. Totally, some examination strategies are utilized to work on the investigation of the analysis of slabs. Initial, a rigid diaphragm model exclusive of slabs from the lateral resisting system is utilized to transfer the lateral forces to the vertical resisting elements in direct extent to the stiffness of vertical components.

Flat slab has numerous advantages such as a reduced and simpler formwork, adaptability, and simpler space partitioning. Flat slab in this kind of building system is a significant structural element which is constructed to make smooth and suitable surfaces such as floors, roofs, and ceilings. Flat slab structures possess major advantages because of the free design space, flexibility in room layout, partition walls and others. This type of construction is very attractive in appearance as compare to beam slab construction also the flat is inexpensive to construct as it requires less stuffs of formwork

The figure underneath shown the upper and lower rebar reinforcement of flat slab in tunnel formwork building with rebars numbers and specification.

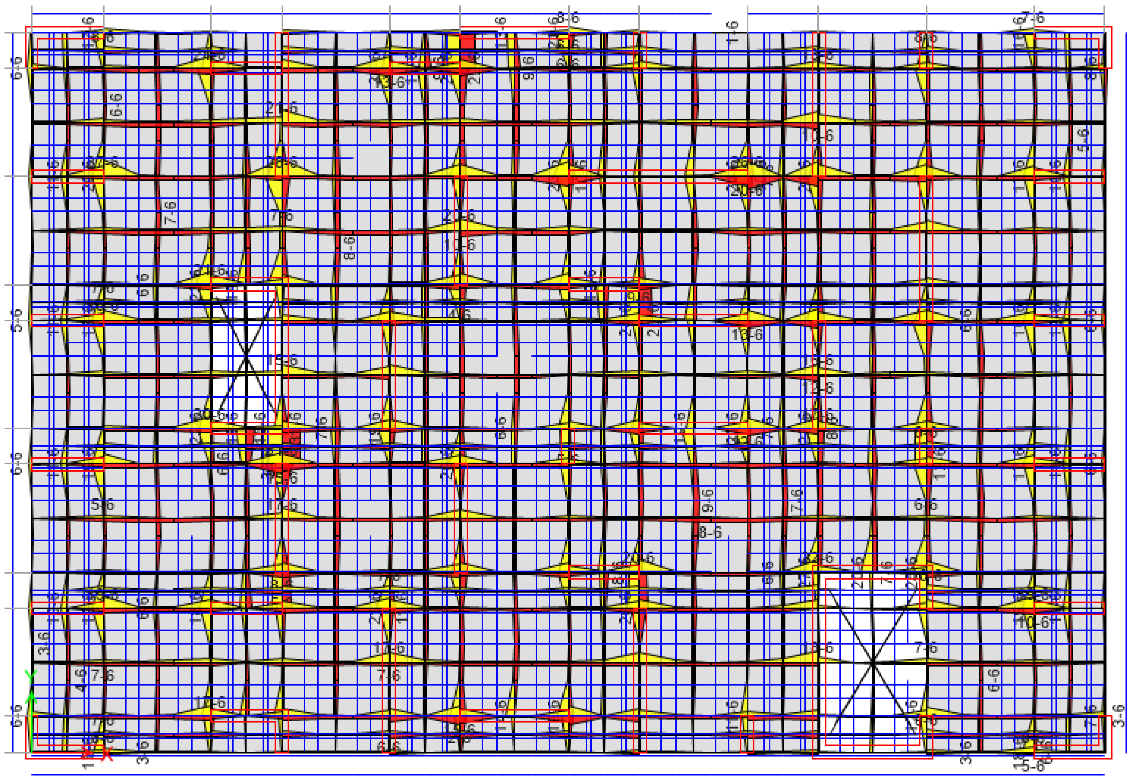


Figure 4.21: Top and bottom layer reinforcement in slab

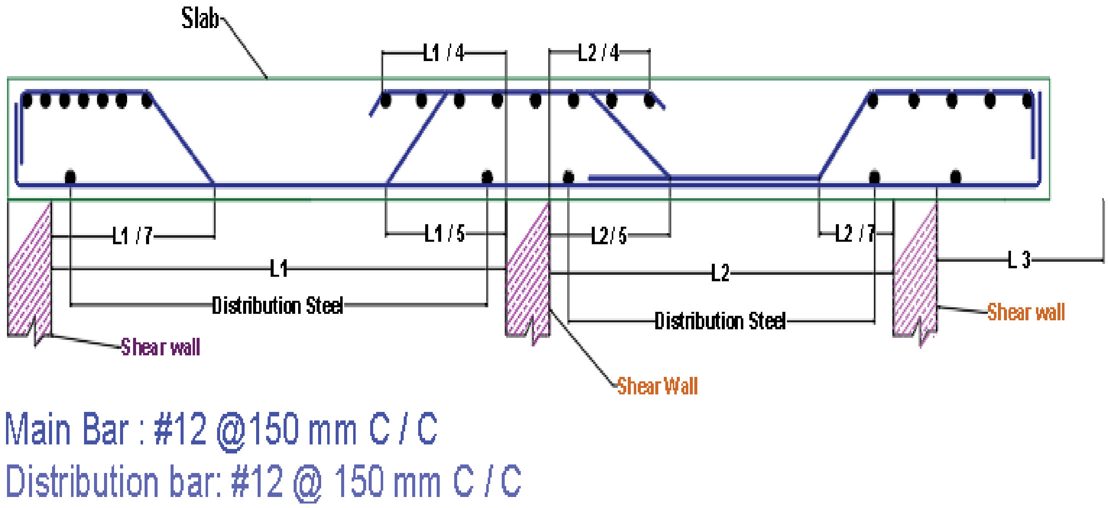


Figure 4.22: Typical Slab of Tunnel formwork building

4.3. Absolute story acceleration

Fundamentally growing construction in the earthquake vulnerable regions needs appropriate handling of seismic prone buildings so that the response of all the structural members of the building should be precisely checked; Absolute story acceleration of core wall and tunnel formwork buildings systems have indicated in the figures underneath.

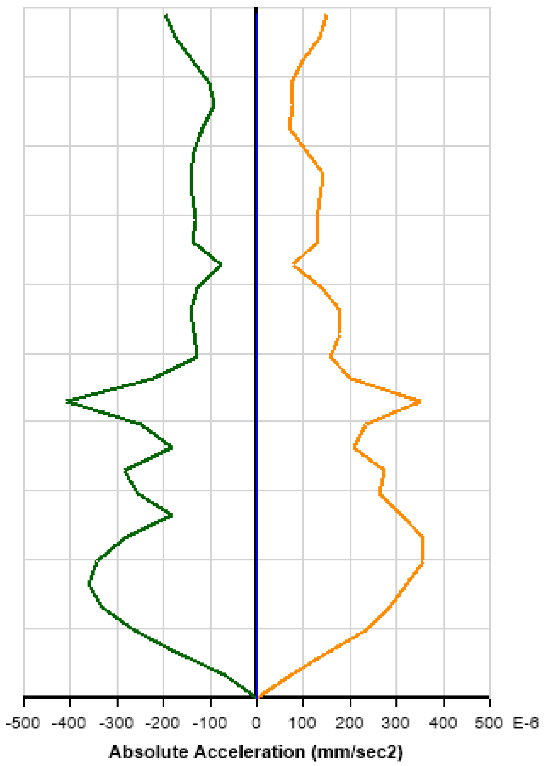


Figure 4.23: Absolute story Acc.
For core wall building

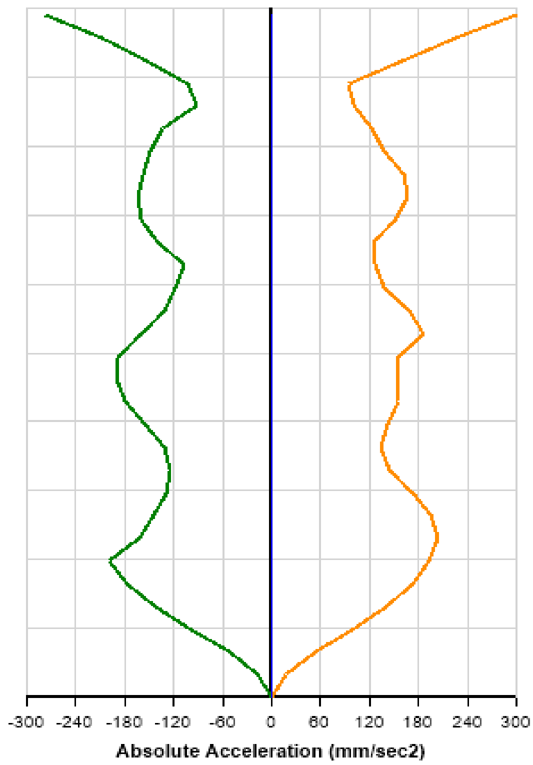


Figure 4.24: Absolute story Acc.
for tunnel formwork building

4.4. Cost of the buildings

Costs for core wall and tunnel formwork buildings are calculated separately and their quantities has been represented here related to their consecutive price; for the building costs extensive enlightened calculation you can check the Appendix-A1 and Appendix-A2 for core wall and tunnel formwork building respectively.

Table 4.40. (A) Costs of the buildings

Quantities for Core Wall & Tunnel Formwork Buildings						
S. No	**Item Description**	**Core Wall Building (m^3)**	**Tunnel Formwork Building (m^3)**	**Rate / m^3 (\$)**	**Price for Core wall building (\$)**	**Price for Tunnel Formwork building (\$)**
3	Total Quantity of concrete in footing	8.63	22.5			
4	Total Quantity of concrete in columns (Plinth Level)	49.01				
5	Total Quantity of concrete in shear wall (PL)	37.26	70.35			
7	Total Quantity of concrete in DPC (PL)	2.5	13.2			
9	Total Quantity of concrete in Columns (SS)	1345.41				
10	Total Quantity of concrete in shear wall (SS)	1229.58	1989.9			
11	Total Quantity concrete in Beams	2614.5				
12	Total Quantity of concrete in Slabs	2754.6	4761.81			
	Total quantity of Concrete	8041.49	6857.76			
	Total Cost			50	402074.5	342888
S. No	**Formwork / Shuttering**	**Core Wall Building (m^2)**	**Tunnel Formwork Building (m^2)**	**Rate / m^2 (\$)**	**Price for Core wall building (S)**	**Price for Tunnel Formwork building (\$)**
1	Footing Shuttering	51.3	90.6			
2	Columns Shuttering (Sub-Structure)	356.4				
3	SW Shuttering (Sub Structure)	259.2	977.2			

6	RCC Beam Shuttering	11928				
7	RCC Columns Shuttering (SS)	11761.2				
8	Shear Walls Shuttering (SS)	8553.6	27640.8			
9	RCC Slab Shuttering	16785	17475			
	Total quantity of shuttering	49694.7	46237.7			
	Total Cost			25	1242367.5	1155942.5
S. No	**Steel Rebar Quantity**	**Core Wall Building (Kg)**	**Tunnel Formwork Building (Kg)**	**Rate / ton ($)**	**Price for core wall building ($)**	**Price for Tunnel formwork building ($)**
1	Steel Qt. in footing	3320.37	6785.47			
2	Steel Qt. In Columns (Sub-str.)	20620.05				
3	Steel Qt. In Columns (SS)	304266.82				
4	Steel Qt. In SW (Sub-Str.)	2027.88	10607.66			
5	Steel Qt. In SW (SS)	52989.01	306717.14			
6	Steel Qt. In Beam	563659.66				
7	Steel Qt. In Slab	39558523.2	44004596			
	Total quantity of steel rebar (Kg)	40496327	44328706			
	Total quantity of steel rebar (Ton)	40496.33	44328.71			
	Total Cost			700	28353784.89	31030094.39

*PL: Plinth Level
**SS: Super Structure

Table 4.40. (B) Total costs of the buildings

S. No	Building Description	Cost of building (USD)	Million (USD)
1	Core wall building	29998226.89	30.00
2	Tunnel Formwork building	32527572.39	32.53

From the above calculations it is clear that the cost for core wall building is around 30.00 million USD or ($ 29998226.89) and cost for the tunnel formwork building is around 32.53 million USD ($ 32527572.39). But both types of building systems have good earthquake performance but we expect very minor damages for tunnel formwork building after earthquake happening.

It is considered that for more details of the buildings costs you can look at Appendix A1 for core wall building and Appendix A2 for tunnel formwork building respectively.

4.5. Closing observations

After experiencing all the related design and modelling consequences of core wall and tunnel formwork buildings systems it has been realized that both structures systems shown the correct results. Also, other than the general procedures and required designing steps the most important specification such as reinforcing details of different structure members, axial forces and moment capacity relationships, interaction curves for column and shear walls, shear forces for columns and shear walls, story response for each building system, D/C ratios, shear design capacities for columns and different shear walls legs and finally, the total quantities and cost estimation with cost comparison for core wall and tunnel formwork building have be illustrated. The construction cost of tunnel formwork building is more than the construction cost of core wall building but it is considered that both types of buildings have good earthquake performance and very suitable to withstand against earthquake loads.

CHAPTER 5

ANALYSES OF THE BUILDINGS

5.1. General

After complete designing of core wall and tunnel formwork buildings in ETABS software, the designs are transferred with their all specifications to PERFORM-3D software and performed the nonlinear dynamic analysis. The behaviour of the elements is taken into account with fibre element sections. The (LATBSDC-2020) documents were utilized as a kind of perspective while determining the expected material strength, characteristic material strengths, and the elasticity mode of high strength concrete.

The three-dimensional (3D) geometric core wall building designs were developed in PERFORM-3D software. The beams and columns were modelled as beam and column elements respectively. The floor slabs were modelled as a plate shell element. Also, the three-dimensional (3D) geometric tunnel formwork building was developed with shear walls and with the floor flat slabs without any beams and columns. For the accurate results of the structural systems, different analyses are made.

5.2. Defining earthquake records

The buildings were analysed within the scope of the study and according to the criteria specified in the American Society of Civil Engineering (ASCE-7-16) and American Concrete Institute (ACI-318) codes as taken into account. In addition, the level of danger determined within the extent of the investigation of the (ASCE-7-16) updates and the earthquake records well-suited with the selected target spectrum. Their corresponding target acceleration spectrums for the field. It is noticeable that the database of Pacific Earthquake Engineering Research Centre (PEER) was used. The characteristics of the 11 earthquakes records and the scaling coefficients for various transformation periods of the DD-1 earthquake records.

Table 5.1. User entries of the location

Address	Skopje city
Earthquake Ground Motion Level	DD-1
Local Ground Class	ZB

Latitude	41.9981° N
Longitude	21.4254° E

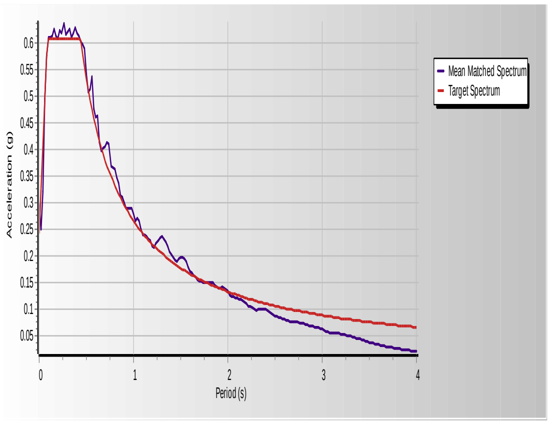
Table 5.2. Earthquake records, properties and scaling coefficients
DD-1 Earthquakes

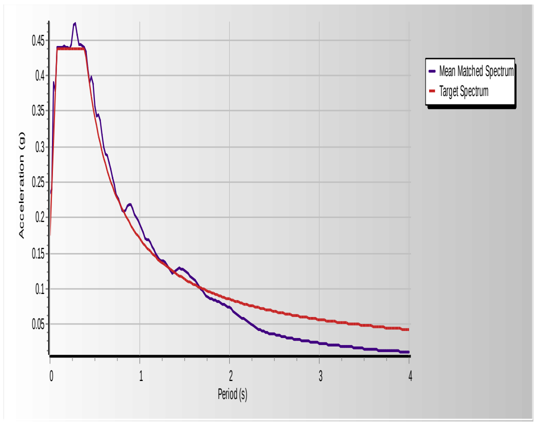
Record No	Earthquake Name	Year	Station Name	Earthquake Magnitude	Scale
EQ1	Kocaeli, Turkey	1999	Ambarli	7.51	4.95
EQ2	Chi-Chi, Taiwan	1999	HWA025	7.62	7.81
EQ3	Landers	1992	Forest Falls Post Office	7.28	4.38
EQ4	Irpinia, Italy-01	1980	Sturno (STN)	6.90	2.20
EQ5	Parkfield-02, CA	2004	Fresno – NSMP USGS Office	6.00	10
EQ6	Kobe, Japan	1995	Amagasaki	6.90	1.81
EQ7	Chi-Chi, Taiwan-04	1999	CHY015	6.20	4.90
EQ8	Imperial Valley-06	1979	Delta	6.53	2.11
EQ9	Kern County	1952	Pasadena – CIT Athenaeum	7.36	10.41
EQ10	San Fernando	1971	Whittier Narrows Dam	6.61	4.90
EQ11	Morgan Hill	1984	UCSC Lick Observatory	6.19	13.51

Scaling of the earthquake ground motion time history at different PGA has been scaled

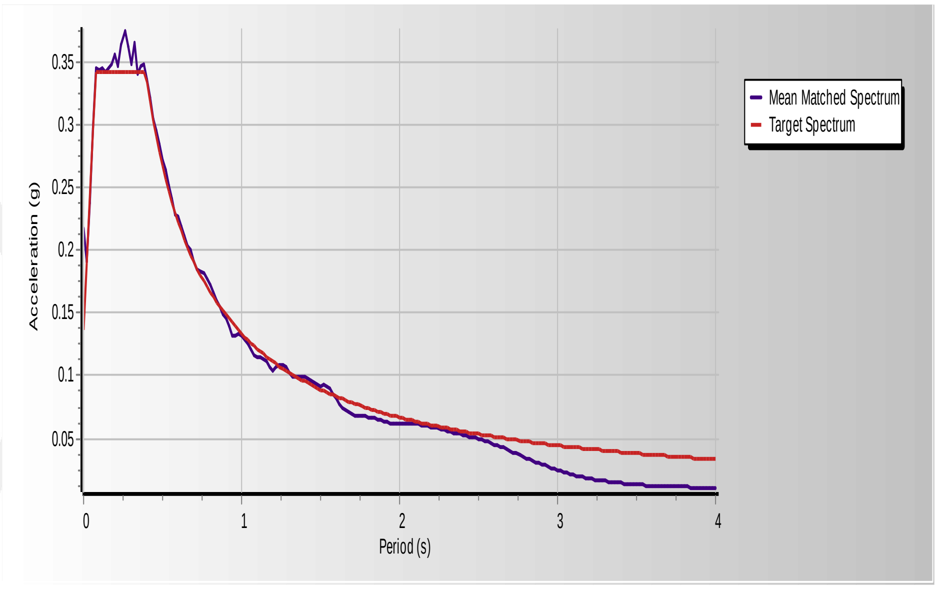
Scale factor = Required PGA / Current PGA

For instance Scale Factor (S) = 0.5 PGA / 0.35 PGA = 1.4



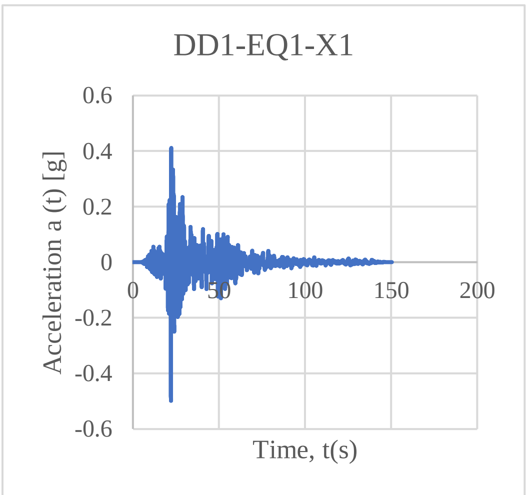


(a): MCE level **(b): DBE level**



(C): SLE level

Figure 5.1: Comparison of the Average spectral acceleration of ground motions to the target spectrum (5% damping rate)



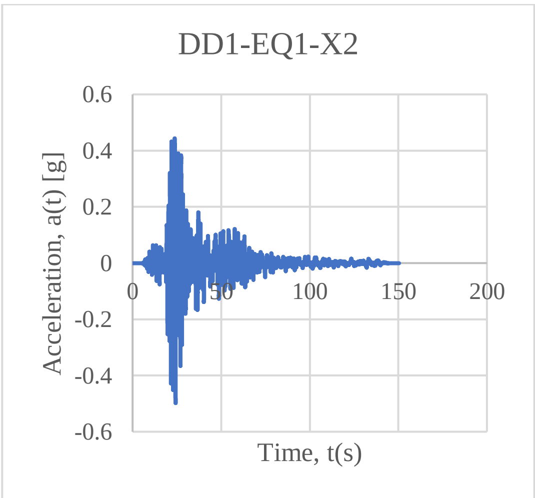
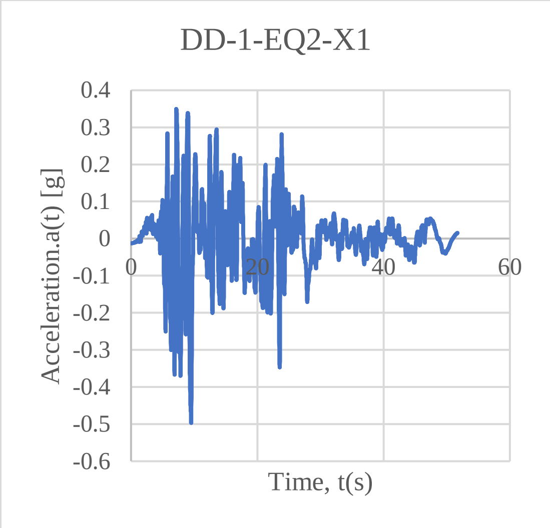
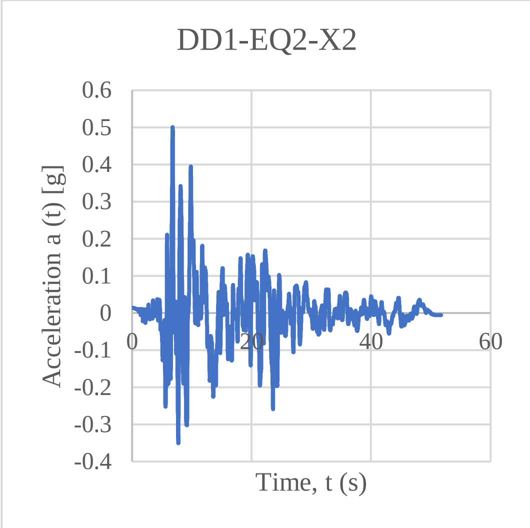
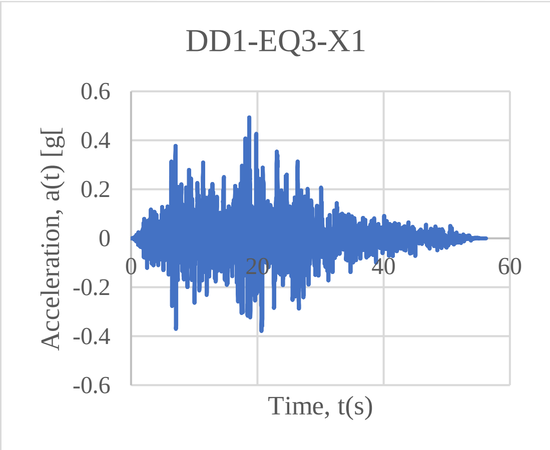
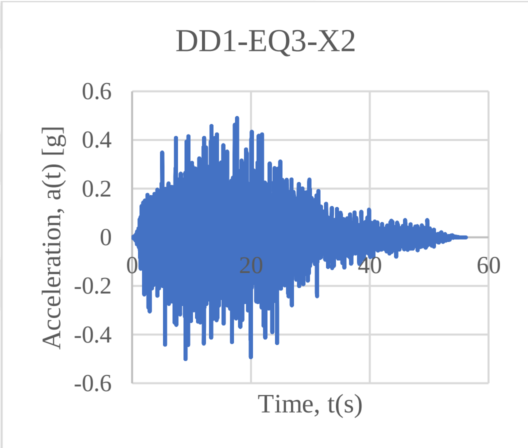


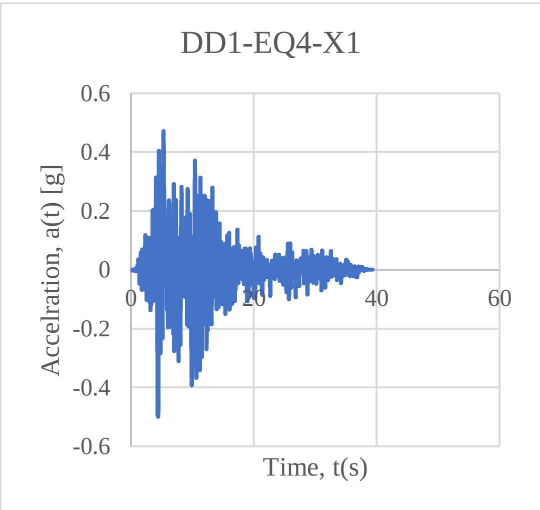
Figure 5.2: Scaled acceleration-time graphs for 2475-year return period

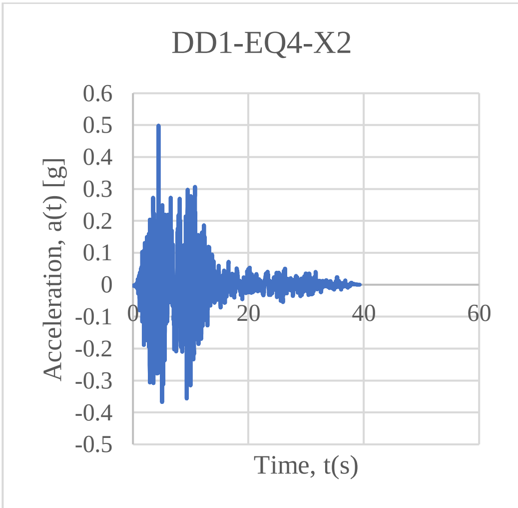

DD-1-EQ2-X1
Acceleration.a(t) [g]
Time, t(s)

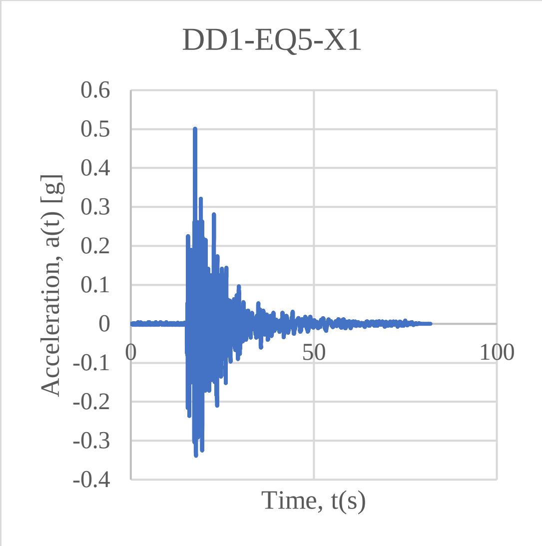

DD1-EQ2-X2
Acceleration a (t) [g]
Time, t (s)

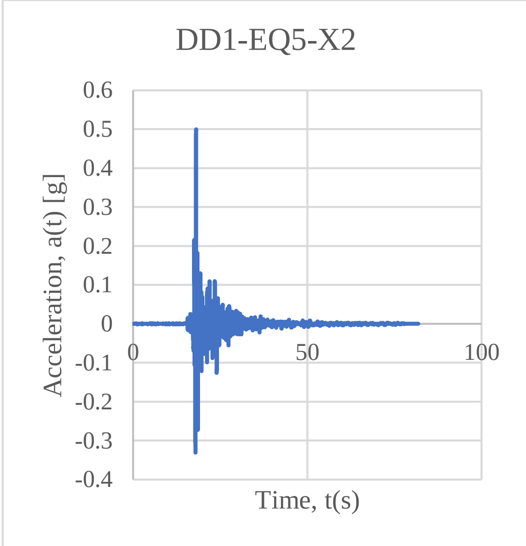

DD1-EQ3-X1
Acceleration, a(t) [g[
Time, t(s)

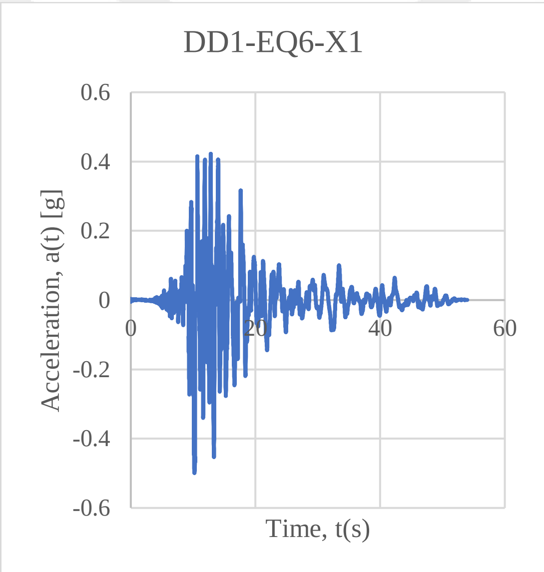

DD1-EQ3-X2
Acceleration, a(t) [g]
Time, t(s)

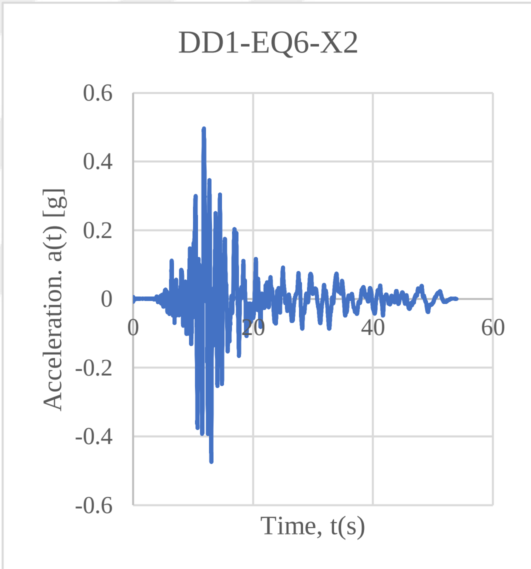

DD1-EQ4-X1
Accelration, a(t) [g]
Time, t(s)

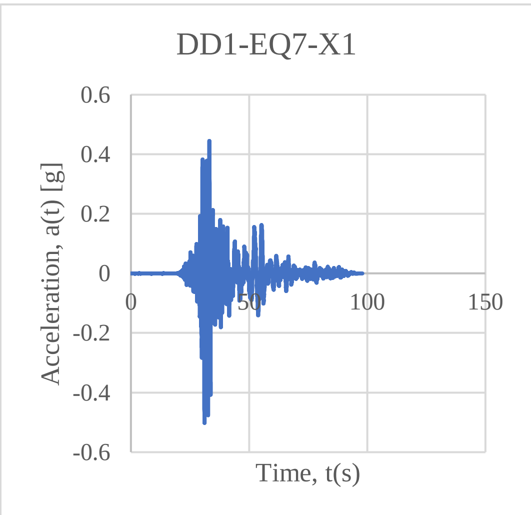

DD1-EQ4-X2
Acceleration, a(t) [g]
Time, t(s)

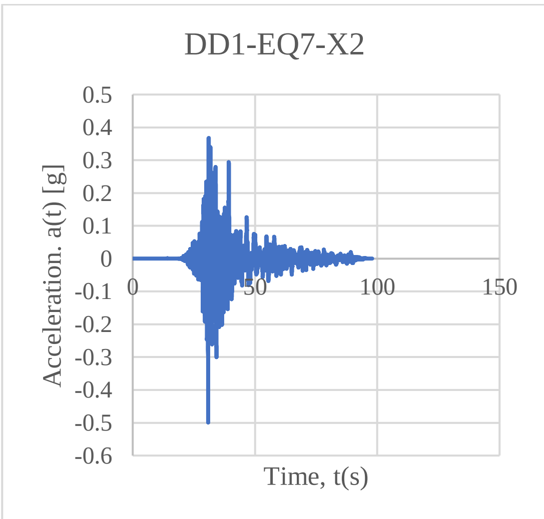

DD1-EQ5-X1
Acceleration, a(t) [g]
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
50
100
Time, t(s)


DD1-EQ5-X2
Acceleration, a(t) [g]
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
50
100
Time, t(s)


DD1-EQ6-X1
Acceleration, a(t) [g]
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
0
20
40
60
Time, t(s)


DD1-EQ6-X2
Acceleration. a(t) [g]
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
0
20
40
60
Time, t(s)


DD1-EQ7-X1
Acceleration, a(t) [g]
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
0
50
100
150
Time, t(s)


DD1-EQ7-X2
Acceleration. a(t) [g]
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
0
50
100
150
Time, t(s)

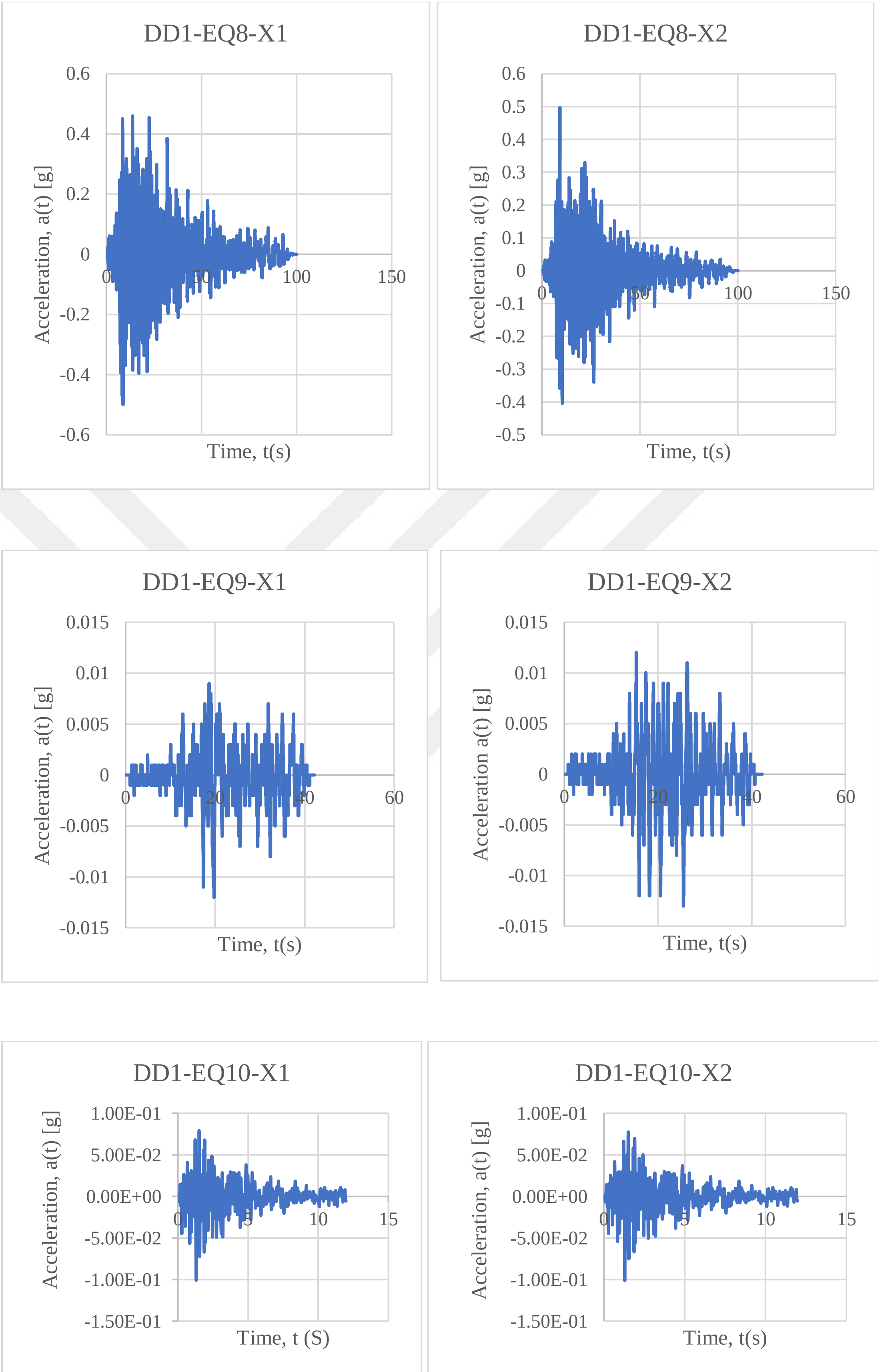
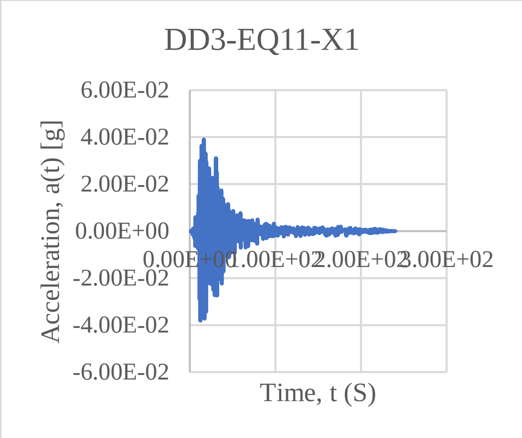


Figure 5.3: Scaled acceleration-time graph for the 475-years return period



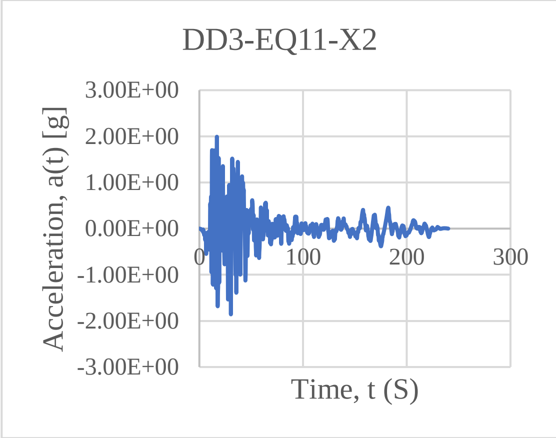


Figure 5.4: Scaled acceleration-time graph for the 72-years return period

5.3. Earthquake hazards of the area

The arbitrary area and location of the structures is in Skopje city of North Macedonia. Core wall and tunnel formwork structures of 30-Story were analysed using the time history analysis in PERFORM-3D. As it is the capital city of North Macedonia which is widely populated with tall structures. It is considered that the Skopje location is considered a high seismic zone due to the seismic history of Macedonia Vader zones where regular earthquakes happen, and Skopje is part of it

The building’s specific location is (41.9981° N, 21.4254° E) at Skopje city Macedonia.

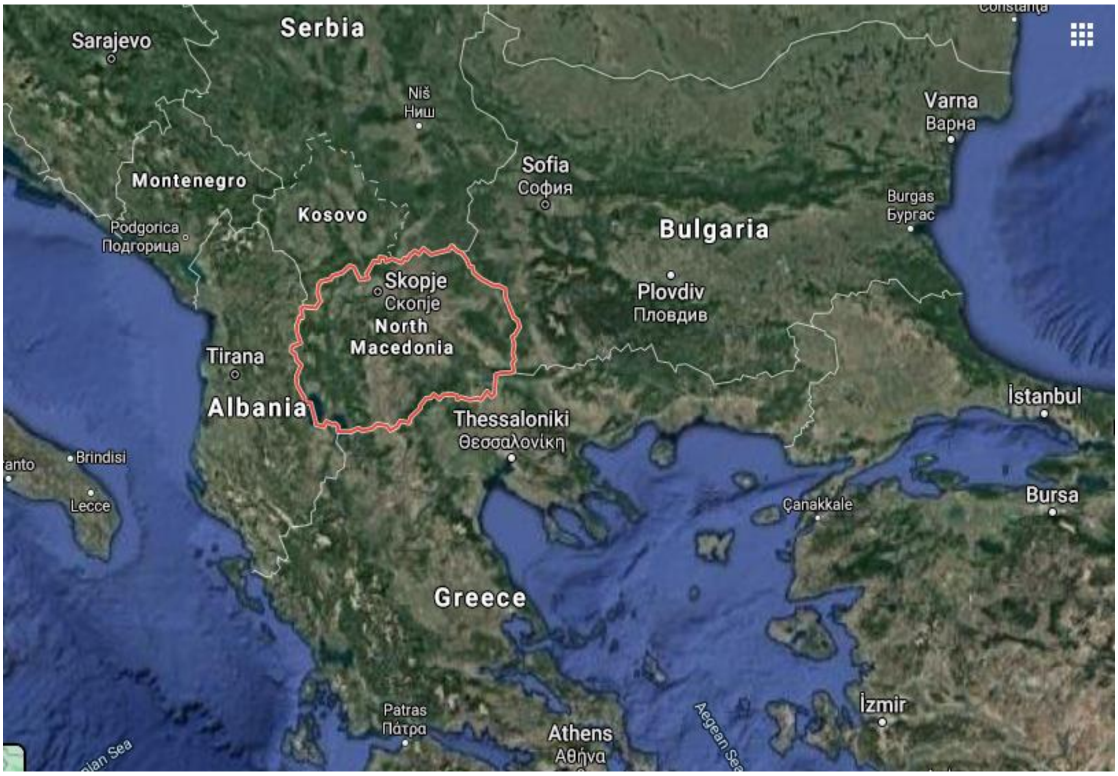


Figure 5.5: Skopje city Macedonia

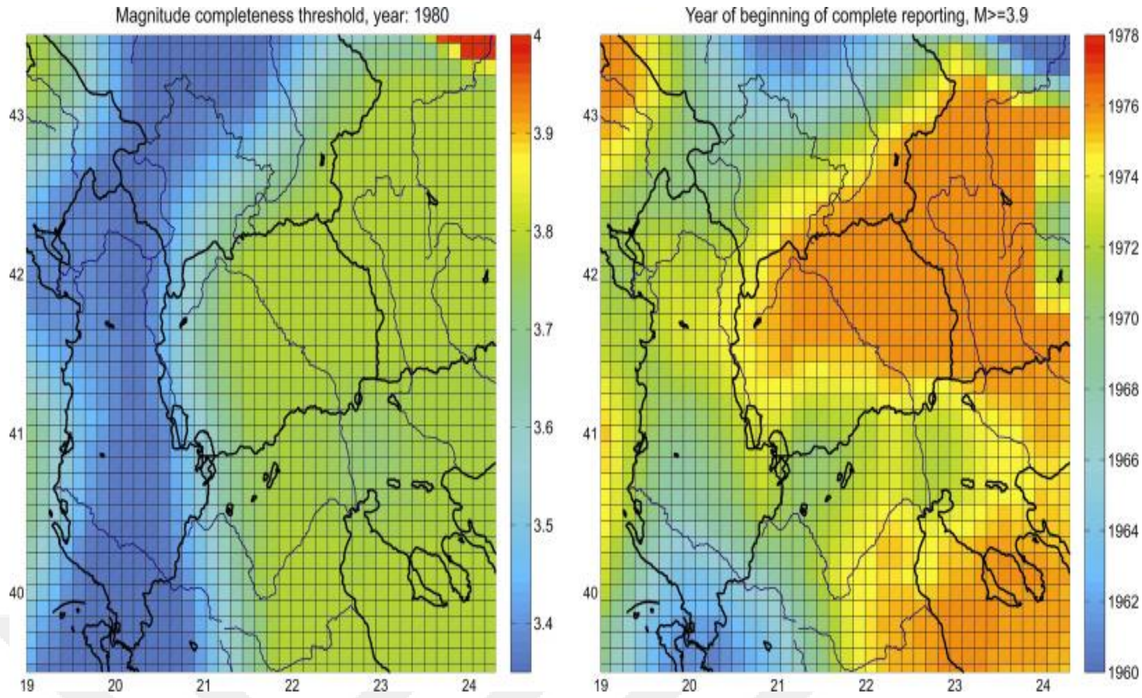


Figure 5.6: Seismic hazard maps of Macedonia

Using the inputs from Table 5.1. into the website for Earthquake Hazard Map of Skopje city Macedonia. The following data has been figured-out

S_S =1.145 S_1= 0.5

F_A = 0.9 F_V = 0.8

$S_{DS} = (2/3) * F_A * S_S \rightarrow S_{DS} = (2/3) * 0.9 * 1.145 \quad S_{DS} = 0.687$

$S_{D1} = (2/3) F_V * S_1 \rightarrow S_{D1} = (2/3) * 0.8 * 0.5 \quad S_{D1}= 0.2667$

Where:

S_S: Short period map spectral acceleration coefficient [dimensionless]

S_1: Map spectral acceleration coefficient [dimensionless] for 1.0 second period

S_{DS}: Spectral design acceleration short period coefficient [dimensionless]

S_{D1}: Spectral acceleration design coefficient [dimensionless] for 1.0 second period.

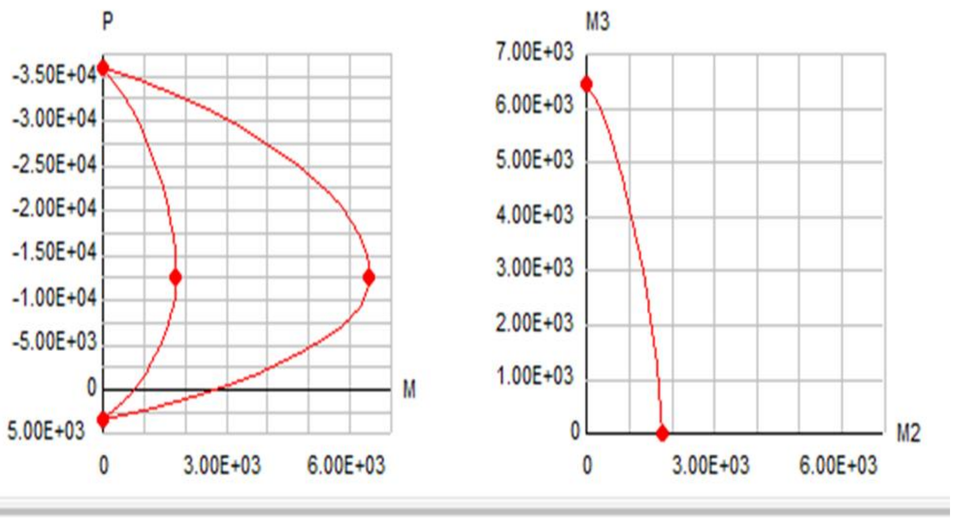
5.4. Core wall building analysis

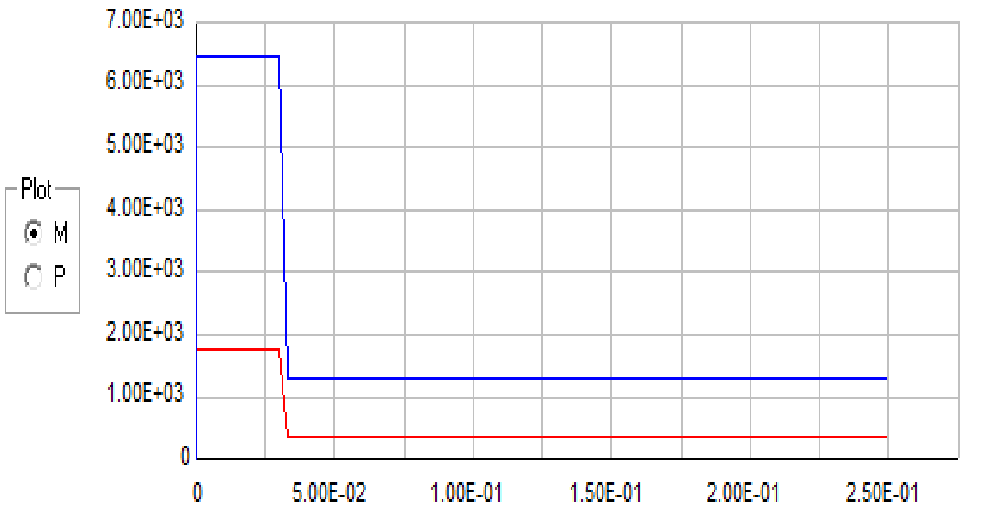
At the point when we get the core wall building design imported from ETABS software to PERFORM-3D, then perform certain procedures to accomplish our focused-on results of the structure. These methods few procedures are represented as underneath In PERFORM-3D software, there are two phases one is a modelling phase which is somewhat already completed in ETABS software instead, and another phase is the analysis phase. Where after all the modelling stages completion the analysis phase will conduct. All supports of the building are fixed and slaves are provided for each story. Slaving shows that if one node in a story moves then all other nodes can move together in that specific story which can help in story displacement reduction. Also, lumped all floors of the building in a single point which indicates that all weight of that specific story consolidated and collected in a single point. Furthermore, elastic, inelastic, and many compound cross-sections are modelled precisely to prolong the analysis.

Table.5.3. Material acceptance criteria

	Characteristic Compressive Strength	Expected Strength	Magnification Rate
Concrete	27.58 MPa	36 MPa	1.30
	Characteristic Yield Strength	Expected Yield Strength	
Reinforcement	413.69 MPa	455.05 MPa	1.09

In the below Figure 5.6. The linear behaviour of columns is considered, presented with normal strength-moment and moment-rotation relationships.





(Moment Vs Plastic rotation)

Figure 5.7: linear behaviour of columns

Moment hinge rotation relationship for a connection is a way to better understand the connection behaviour under loading and bearing capacity for beams.

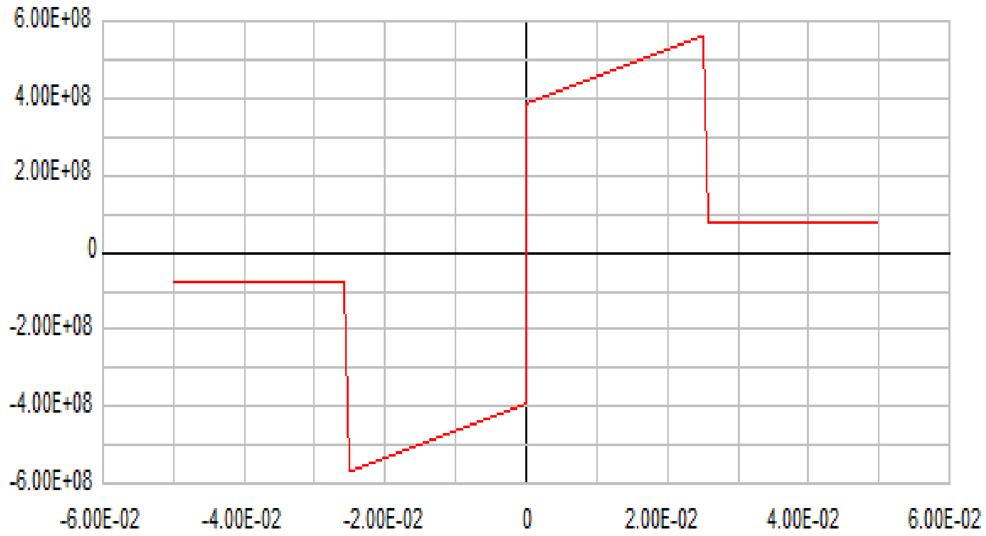


Figure 5.8: Moment rotation curve in beam

The dampening rate under the earthquake (DD-1, DD-2, DD-3 earthquakes), which is the 2475-years, 475-years, and 72-years returning period respectively of the structure was again accepted as a reference to (LATBSDC 2020) and received 2.5% damping, where 2.4% of this rate is defined as fixed dampening and 0.1% of the Rayleigh dampening rate as shown in Figure 5.8 and figure 5.9 for all vertical elements of the structure. Also, the internal forces arising from the second-level effects are taken into account.

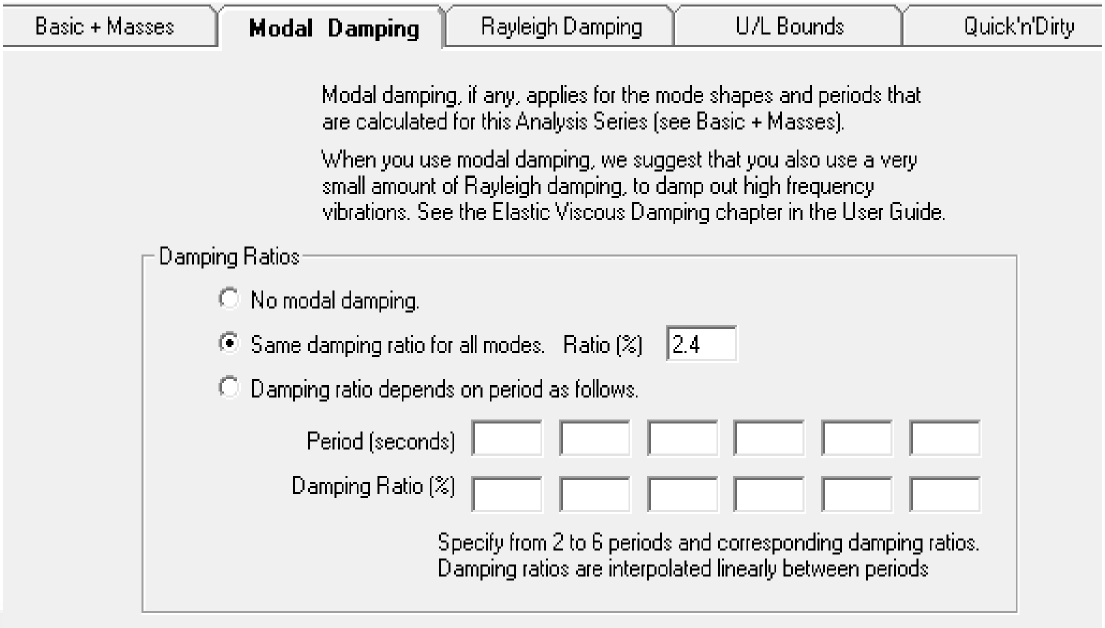


Figure 5.9: Modal damping ratio

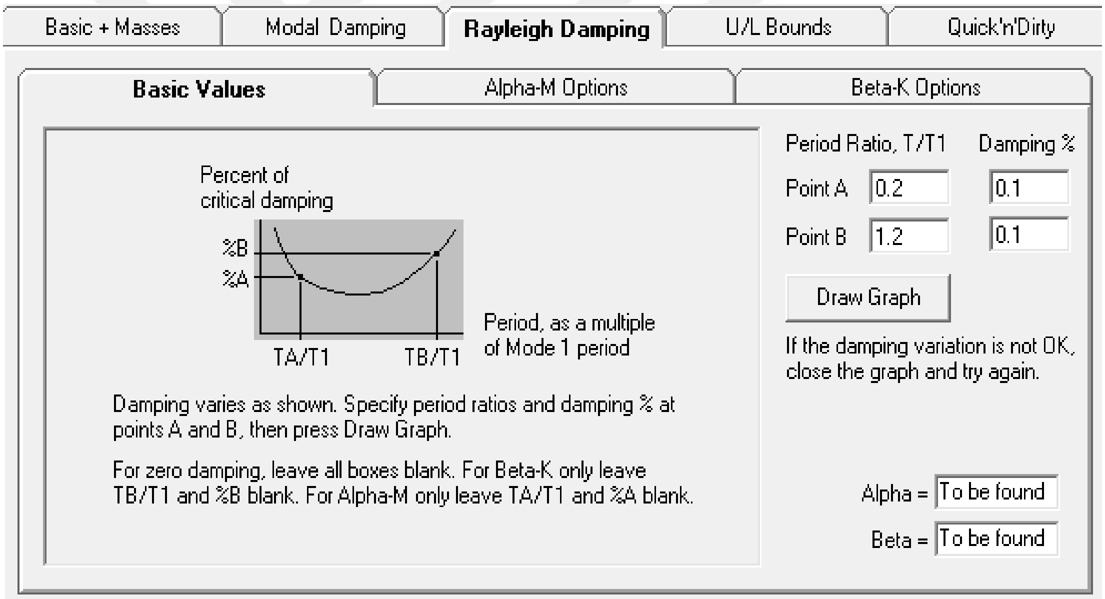


Figure 5.10: Rayleigh damping assigned to the structures

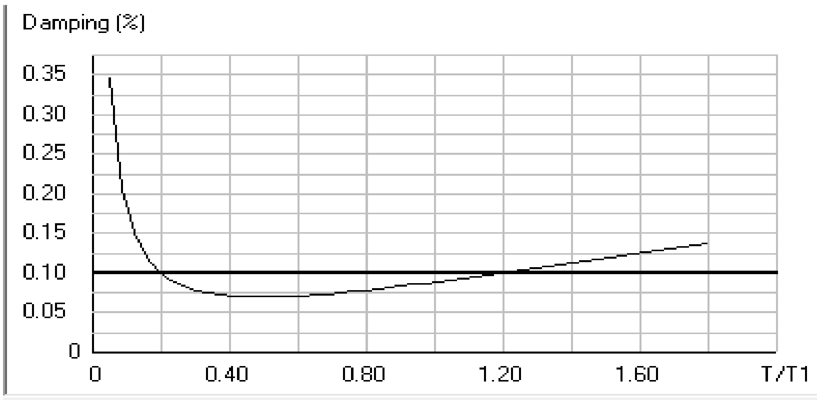


Figure 5.11: Building period ratio and damping relationship

P-Delta effect which generally becomes prevalent as a geometric nonlinearity in multi-story building structures that is experiencing gravity loads and large lateral displacement and involves the equilibrium compatibility relationships of a structural system loaded about its deflected configuration. The influence of P-Delta effects is fundamentally importance for buildings responding in a highly inelastic manner.

In this study the P-delta effects up to 50 mode shapes are intended for analysis.

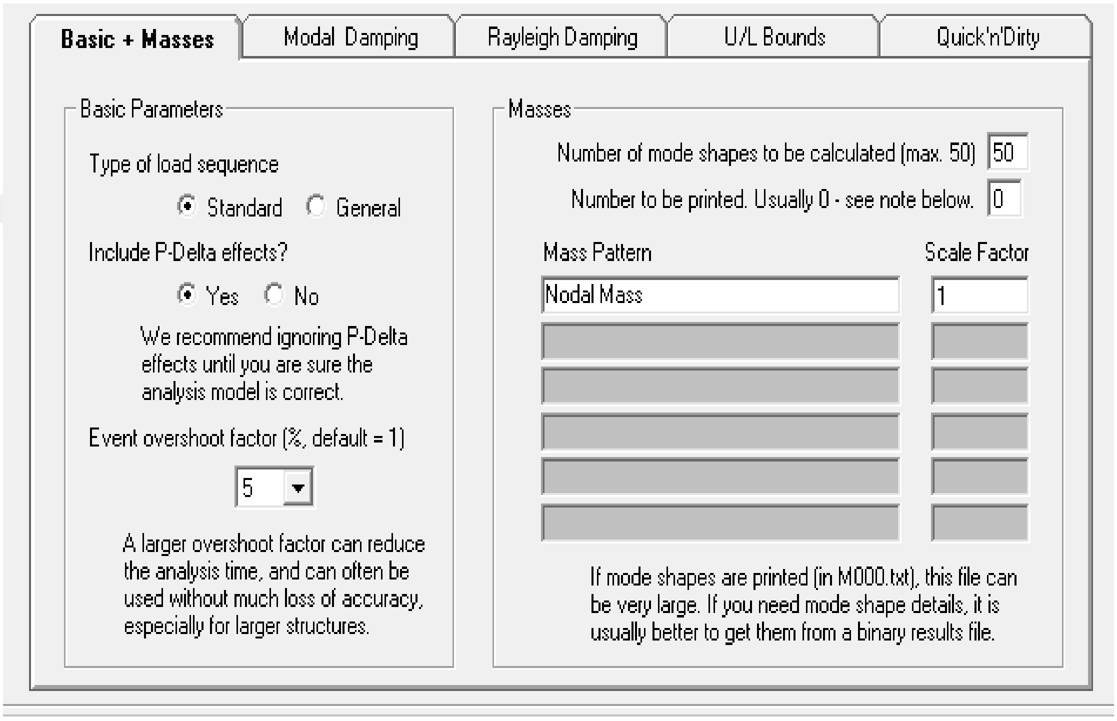


Figure 5.12: Identification of the mode shape & damping rate

Additionally, provide all the necessary steps for instance limit state for deformation of the beam, column, and shear wall in tension and compression case; Also, make the limit state for drifts in both x and y directions respectively. The building is analyzed for different earthquake records, one of their related plots has been dedicated as shown in below figure for instance justification.

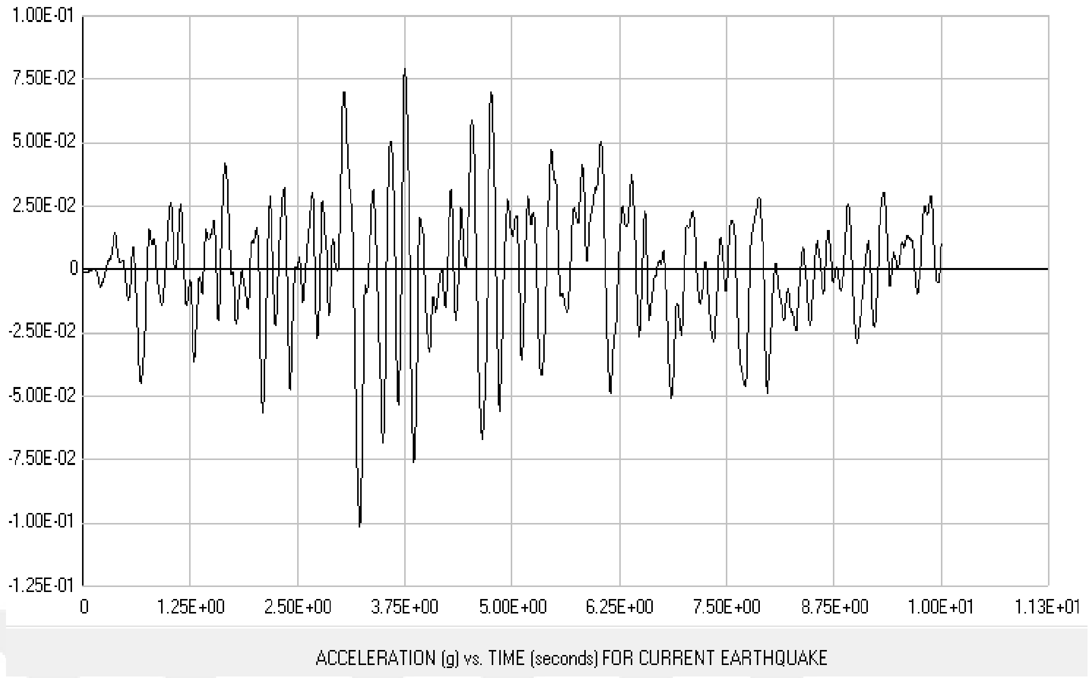


Figure 5.13: Earthquake record for analysis

5.5. Tunnel Formwork building analysis

The tunnel formwork building also been analysed in PERFORM-3D software after designing in ETABS software; The analysis procedures and steps for the tunnel formwork building has followed the same as in core wall building. Here we will not repeat the same analysis procedure for the tunnel formwork building again, because the tunnel formwork building analysis procedures follows the same method as illustrated in the above section 5.4. for core wall building.

From the above analysis of the core wall and tunnel formwork building systems it is clear that both building systems shown good results and the analyzation procedure for core wall and tunnel formwork buildings are correct.

CHAPTER 6

RESULTS AND DISCUSSION

This chapter deals with the results of the study of core wall and tunnel formwork RC high rise buildings such as base shear, buildings period, mode shapes, drifts, drifts ratio, maximum displacement, and finally, the comprehensive buildings performance comparison, performance evaluation, and other specification of the buildings will be discussed. The detailed illustration of the buildings systems has already been described in chapter-3, chapte-4 and chapter-5.

6.1. Base shear

Base shear and story shear play a significant role in lateral stiffness and strength of the structures and weight of steel. A substantial base shear will increase the percentage of steel in columns and shear walls, and it indicates that the building is over stiffened laterally. Therefore, to get a desired base shear optimization in a building is essential. In the current study the buildings have been analysed by the time history analysis method. As per code provision of ASCE, ACI codes and others guidelines.

In this study the core wall building has minimum base shear (6763.2 KN) and the tunnel formwork building has a maximum base shear (9396.6 KN). Based on the results obtained from the study it indicates that a higher base shear indicates a stiffer building and will undergo lesser lateral displacement with less period.
The following table shows the base shears of both types of buildings.

Table.6.1. Base shear

Building Type	Base Shear (KN)
Core Wall Building	6763.2
Tunnel Formwork Building	9396.6

6.2. Building Period

At the point when the ground shakes, the foundation of a structure moves with the ground, and the structure swings forward-backward. If the structure were rigid each point in it would move by a similar sum as the ground; If a structure is flexible then various parts of the structure move forward-backward by various sums. Generally, earthquake shaking of the ground has waves whose periods differ in the range of (0.03-33 sec), period of a structure demonstrates the time that a building completes a cycle when oscillating due to earthquake. Period of the structure indicates its flexibility, more of period means that the building is laterally more flexible and less period of a building indicates that the building is laterally stiff.

Table.6.2. Period of buildings

Mode	Period	Core Wall Building (Sec)	Tunnel Formwork Building (Sec)
Mode.1	Period.1	3.40	3.15
Mode.2	Period.2	3.0	2.87
Mode.3	Period.3	2.82	2.61
Mode.4	Period.4	0.95	0.88
Mode.5	Period.5	0.79	0.79
Mode.6	Period.6	0.77	0.73
Mode.7	Period.7	0.47	0.43
Mode.8	Period.8	0.38	0.37
Mode.9	Period.9	0.36	0.32
Mode.10	Period.10	0.29	0.24
Mode.11	Period.11	0.22	0.21
Mode.12	Period.12	0.22	0.18
Mode.13	Period.13	0.21	0.15
Mode.14	Period.14	0.18	0.14
Mode.15	Period.15	0.15	0.12
Mode.16	Period.16	0.15	0.11
Mode.17	Period.17	0.14	0.10
Mode.18	Period.18	0.13	0.09

Mode.19	Period.19	0.12	0.08
Mode.20	Period.20	0.11	0.07
Mode.21	Period.21	0.11	0.06
Mode.22	Period.22	0.10	0.06
Mode.23	Period.23	0.09	0.06
Mode.24	Period.24	0.09	0.05
Mode.25	Period.25	0.09	0.05
Mode.26	Period.26	0.09	0.04
Mode.27	Period.27	0.08	0.04
Mode.28	Period.28	0.07	0.04
Mode.29	Period.29	0.07	0.04
Mode.30	Period.30	0.07	0.03
Mode.31	Period.31	0.07	0.03
Mode.32	Period.32	0.06	0.03
Mode.33	Period.33	0.06	0.03
Mode.34	Period.34	0.06	0.03
Mode.35	Period.35	0.06	0.03
Mode.36	Period.36	0.06	0.03
Mode.37	Period.37	0.05	0.02
Mode.38	Period.38	0.05	0.02
Mode.39	Period.39	0.05	0.02
Mode.40	Period.40	0.05	0.02

It is shown that the periods for Tunnel formwork building is less than the periods for Core wall building. In total 50 periods and mode shapes of buildings have been obtained in these analyses. It is considered that the tunnel formwork building having an initial period of (3.15 Sec) and the core wall building has the initial period of (3.40 Sec). The structural system which has the maximum period means that it is more flexible than the structure which has a lesser period.

6.2.1. Mode shapes

A mode shape is a deformity that the component would show while vibrating at the natural frequency while vibrating at the regular recurrence. The terms mode shape or normal vibration shape are utilized in structural dynamics. In this manner, the natural frequencies and mode shapes specify how the structure acts under a dynamic load.
The analysesis which are carried out individually for the core wall and tunnel formwork buildings and up to fifty (50) different mode shapes are obtained as expected.

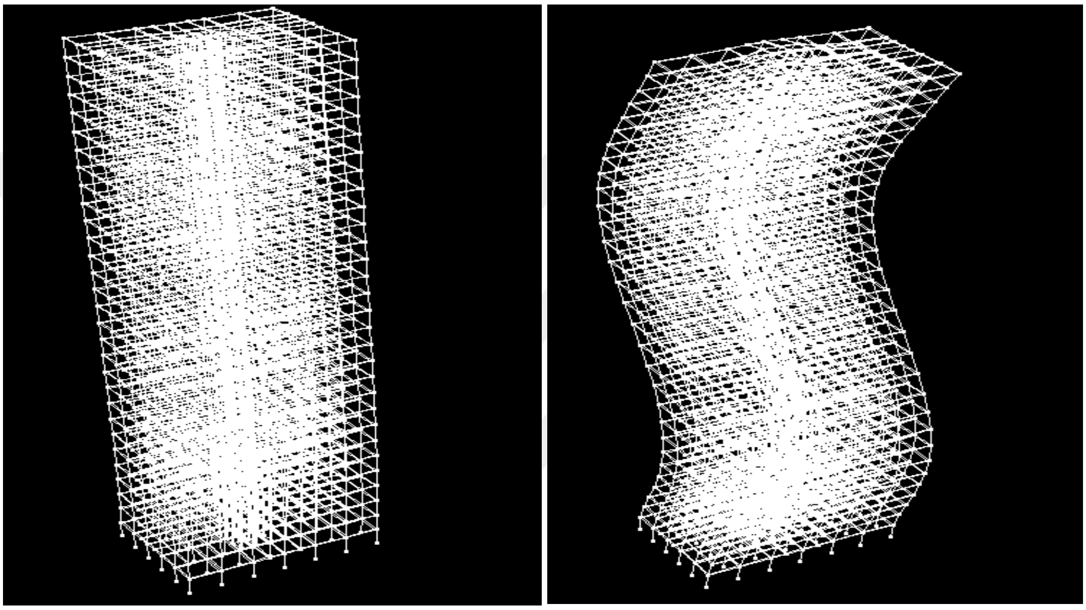


Figure 6.1: 2nd & 5th Mode shapes of core wall building

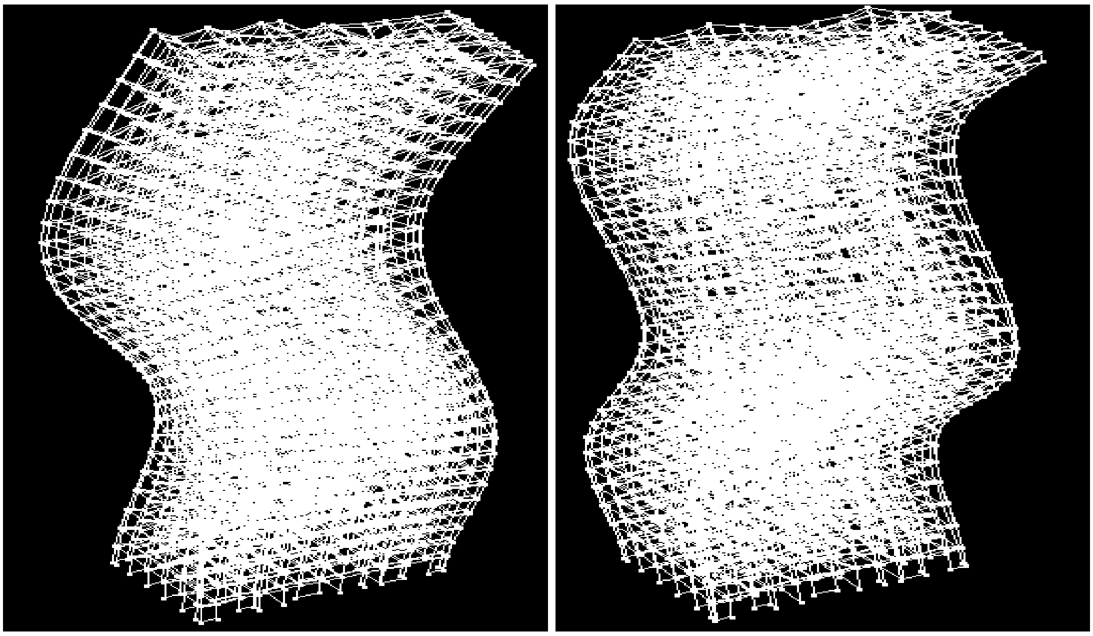


Figure 6.2: 7th & 9th Mode shapes of tunnel formwork building

6.3. Maximum story forces

Maximum internal forces along the story height of core wall and tunnel formwork buildings have been indicated in below table.

Table 6.3. Max. Story forces of the buildings

	Maximum story forces Core wall building		Maximum story forces Tunnel formwork building	
Story	H_1 force (KN)	H_2 force (KN)	H_1 force (KN)	H_2 force (KN)
30	1068.8	34.27	1010.5	151.1
29	2727.4	45.66	1834.2	262.94
28	3996.2	40.53	2555.9	345.73
27	5048.4	43.13	3142.6	396.31
26	5678.8	52.82	3627.2	448.99
25	6107.9	37.60	4137.2	478.71
24	6390.9	49.21	4630.7	487.76
23	6630.2	39.42	5127.2	495.96
22	6560.4	57.17	5568.7	539.1
21	6165.5	39.69	5950.4	579.82
20	5088.1	60.58	6295.6	621.08
19	4264.8	40.46	6567.7	653.88
18	3089.8	33.69	6770	674.3
17	3706.8	29.88	6909.1	690.25
16	4069.9	27.09	6998.9	703.27
15	4271.8	39.41	7027.4	726.51
14	4380.9	40.23	7297.2	775.62
13	4510.3	45.70	7646.5	818.61
12	5202.5	33.92	7917.5	853.25
11	5974.9	43.93	8100.1	883.53
10	6259.5	39.35	8186.9	907.93
9	6575.2	33.65	8181.8	926.3
8	7478.8	33.98	8084.9	938.5
7	7912.4	34.97	8279.1	948.12
6	8548.1	32.14	8565.3	962.24
5	9123.4	33.86	8819.6	996.05
4	9560.2	29.48	9063.1	1034.3
3	10262	36.55	9247.3	1059.9
2	11175	45.97	9360.8	1083.3
1	11454	63.55	9396.6	1094.5

6.4. Maximum displacement

Limiting lateral displacement is a very significant factor in the design and construction of high-rise buildings. Lateral displacement specifies the flexibility of a building; when it is subjected to lateral loads the more lateral displacement shows the more flexible building and the less lateral displacement indicates the stiffer building.

Displacement of core wall and tunnel formwork buildings systems have indicted in table 6.4.

Table 6.4. Max. displacement of the buildings

Story	Core wall building Displacement (mm)	Tunnel formwork building Displacement (mm)
	x-axis	y-axis
Story30	147.54	44.02
Story29	142.69	42.79
Story28	137.69	41.53
Story27	132.60	40.23
Story26	127.37	38.89
Story25	122.01	37.48
Story24	116.52	36.02
Story23	110.90	34.51
Story22	105.17	32.92
Story21	99.33	31.29
Story20	93.39	29.59
Story19	87.43	27.84
Story18	81.40	26.05
Story17	75.35	24.21
Story16	69.29	22.35
Story15	63.25	20.46
Story14	57.26	18.57
Story13	51.34	16.67
Story12	45.54	14.79
Story11	39.88	12.94
Story10	34.39	11.14
Story09	29.17	9.39
Story08	24.19	7.73
Story07	19.51	6.17
Story06	15.19	4.73
Story05	11.26	3.43
Story04	7.798	2.30
Story03	4.859	1.36
Story02	2.518	0.64
Story01	0.847	0.17

6.5. Drift ratio

Drift is the displacement of one floor relative to the floor above or below due to the design lateral forces. The drift is a dominant feature in tall building design that is why the structural characterization and drift minimization of high-rise buildings under lateral load is extremely important.

Story drift ratio is the story drift divide by story height. The analyses offered seeks to establish a direct relationship between the limiting drift ratio, and the corresponding material and structural properties of reinforced concrete elements. The core wall and tunnel formwork buildings drift ratios are presented below.

Table 6.5. Drift ratio of buildings

	Core wall building		Tunnel formwork building	
Story	Drift ratio (H1)	Drift ratio (H2)	Drift ratio (H1)	Drift ratio (H2)
Story30	0.0075	-0.0075	0.0015	-0.0015
Story29	0.0080	-0.0080	0.0037	-0.0037
Story28	0.0084	-0.0084	0.0048	-0.0048
Story27	0.0090	-0.0090	0.0055	-0.0055
Story26	0.0095	-0.0095	0.0060	-0.0060
Story25	0.0100	-0.0100	0.0064	-0.0064
Story24	0.0104	-0.0104	0.0068	-0.0068
Story23	0.0107	-0.0107	0.0070	-0.0070
Story22	0.0109	-0.0109	0.0071	-0.0071
Story21	0.0111	-0.0111	0.0072	-0.0072
Story20	0.0111	-0.0111	0.0072	-0.0072
Story19	0.0112	-0.0112	0.0072	-0.0072
Story18	0.0112	-0.0112	0.0072	-0.0072
Story17	0.0113	-0.0113	0.0071	-0.0071
Story16	0.0113	-0.0113	0.0070	-0.0070
Story15	0.0113	-0.0113	0.0070	-0.0070
Story14	0.0113	-0.0113	0.0070	-0.0070
Story13	0.0113	-0.0113	0.0070	-0.0070
Story12	0.0113	-0.0113	0.0069	-0.0069
Story11	0.0112	-0.0112	0.0069	-0.0069
Story10	0.0110	-0.0110	0.0068	-0.0068
Story9	0.0108	-0.0108	0.0066	-0.0066
Story8	0.0105	-0.0105	0.0064	-0.0064
Story7	0.0101	-0.0101	0.0062	-0.0062
Story6	0.0096	-0.0096	0.0059	-0.0059

Story5	0.0090	-0.0090	0.0055	-0.0055
Story4	0.0083	-0.0083	0.0051	-0.0051
Story3	0.0073	-0.0073	0.0048	-0.0048
Story2	0.0058	-0.0058	0.0043	-0.0043
Story1	0.0032	-0.0032	0.0016	-0.0016

Inter storey drift is a significant constraint of structure behaviour in seismic analysis. Beating impact in building basically means collision between nearby structures because of earthquake load caused by out of phase vibration of adjoining structures.

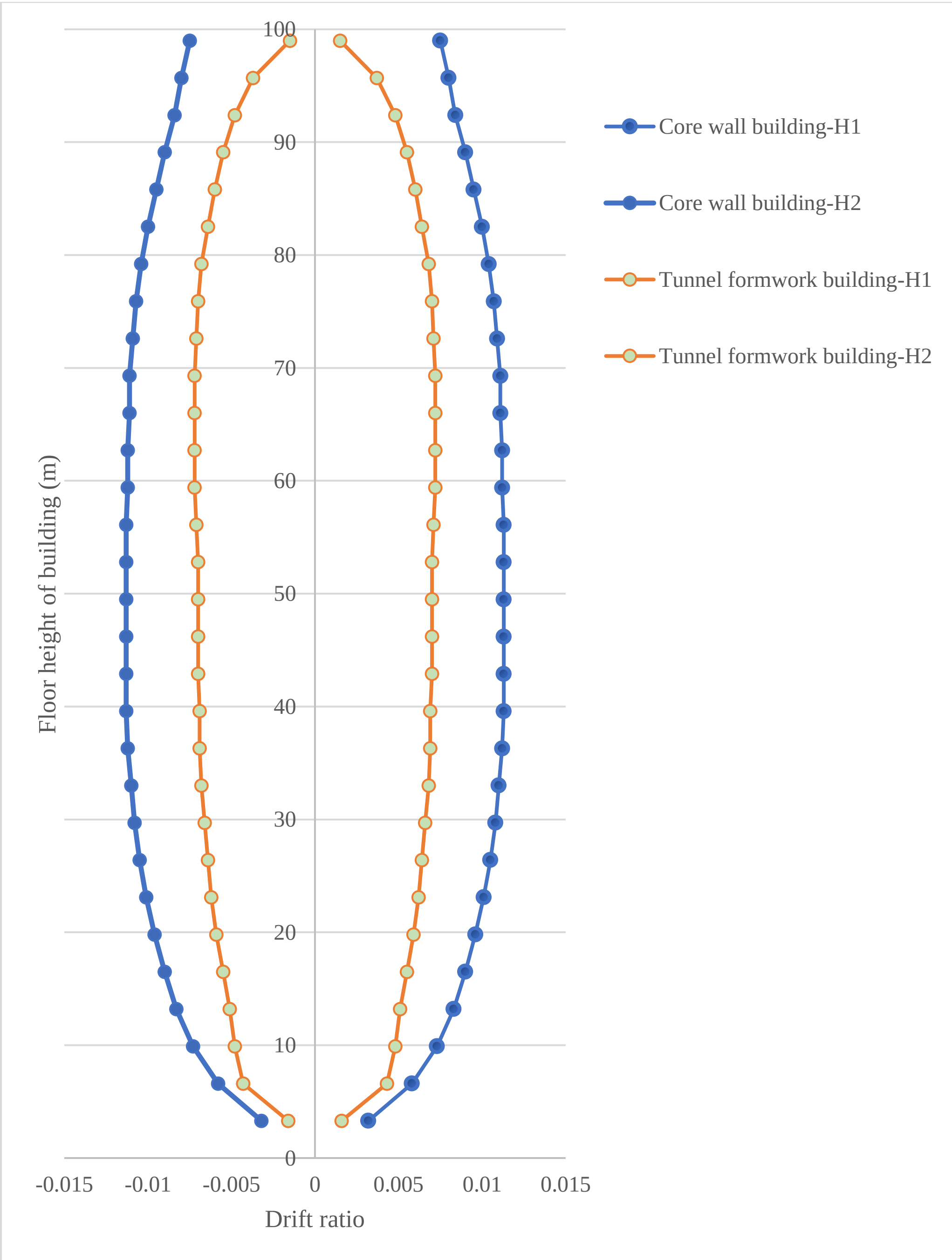


Figure 6.3: Drift ratio plots for core wall & Tunnel formwork building

6.6. Moment curvature relationship

Moment-curvature (MC) relationship characterizes the cross-sectional behaviour and nonlinear properties of the RC members. Following are the MC relationship at mid height of shear wall in core wall and tunnel formwork buildings respectively.

6.6.1. Moment curvature at mid height of buildings

Moment curvature relationship of shear wall in core and tunnel formwork buildings are shown respectively as below for maximum credible earthquake (MCE) level.

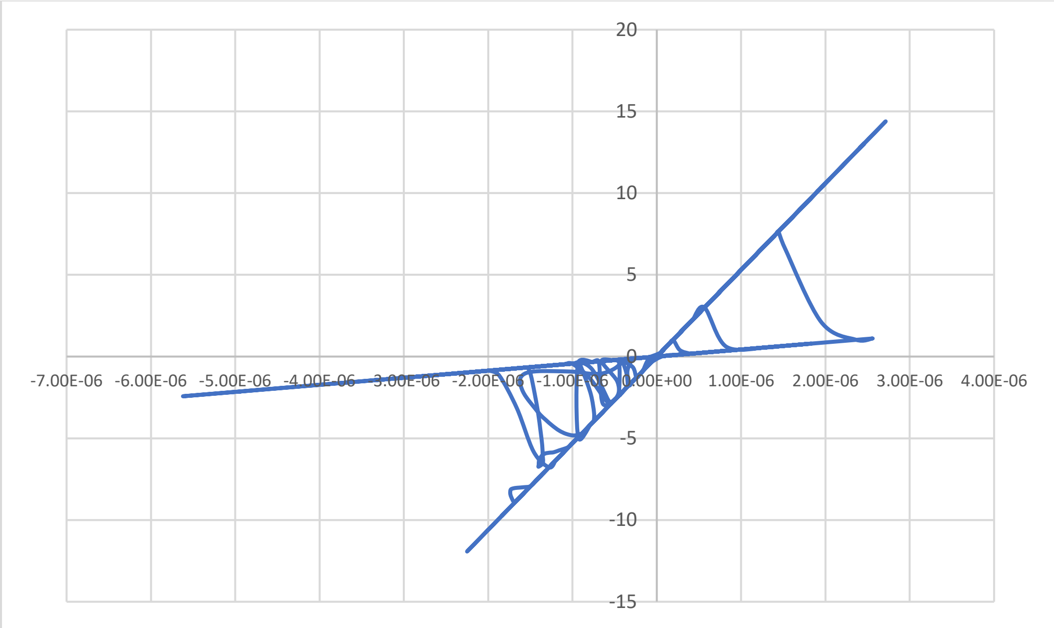


Figure 6.4: Moment -curvature at mid height in core wall building

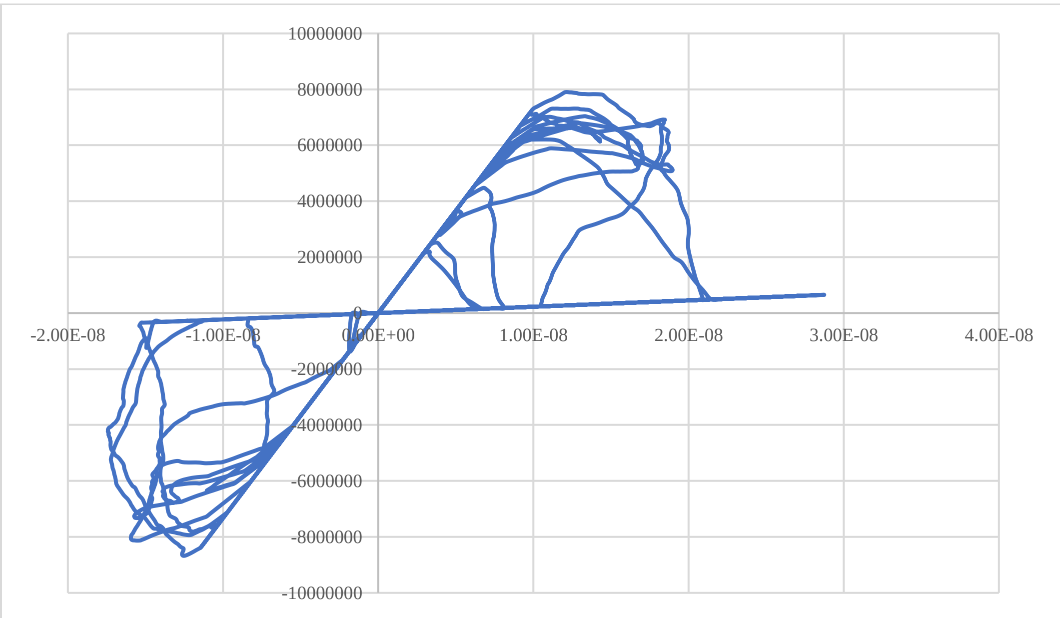


Figure 6.5: Moment - curvature at mid height in tunnel formwork building

Moment curvature investigation is a strategy to precisely govern the load deformation behaviour of a concrete section using nonlinear material stress strain relationships.

The stress-strain relation is a graphical illustration to show the reaction of a material when a load is applied which is used to study the behaviour of materials under different loading conditions. The stress and strain signify the lifetime of a component. It also indicates its endurance limit; the number of stresses rounds it can tolerate. This assistances in keeping a limit on the use of that specific component as protracted use after the detailed limit of time might result in failure. The axial force and strain relation for shear walls is exposed in the following figure.

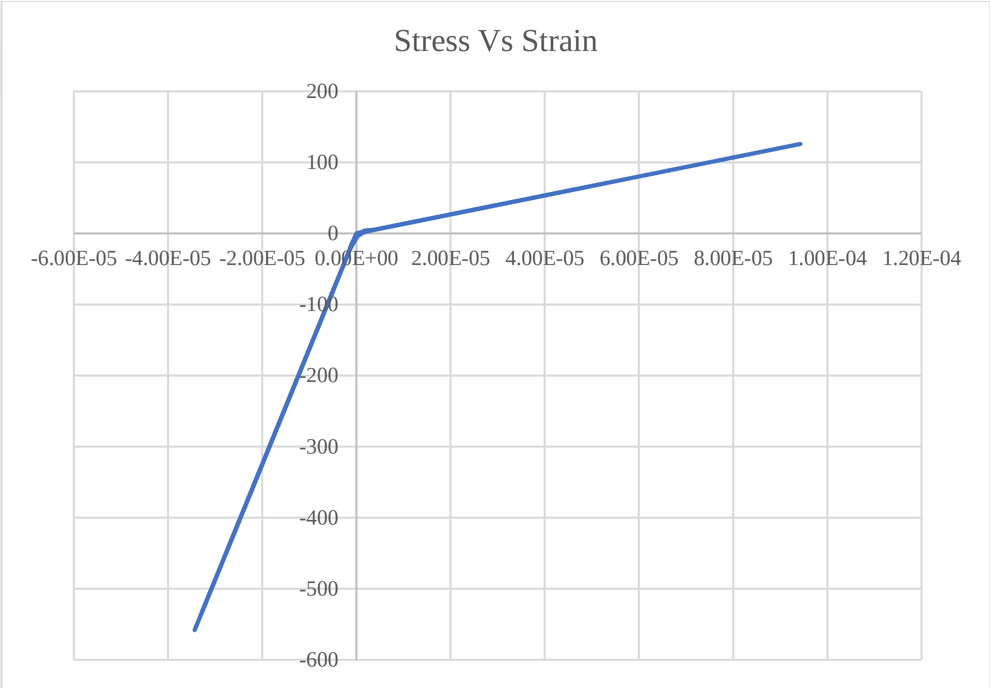


Figure 6.6: Stress strain curve in shear wall

6.7. Column and beam rotation

Column rotation capacity and end moment for core wall building have been displayed in the following figures under MCE level Earthquake.

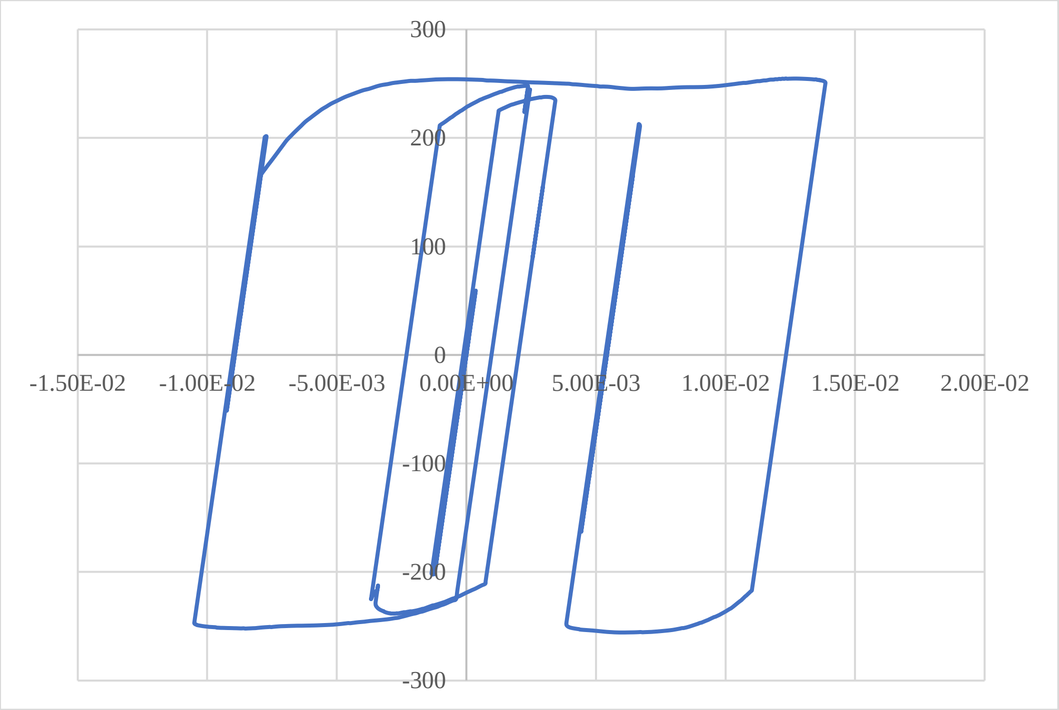


Figure. 6.7: Column rotation capacity in axis-1 of core wall building

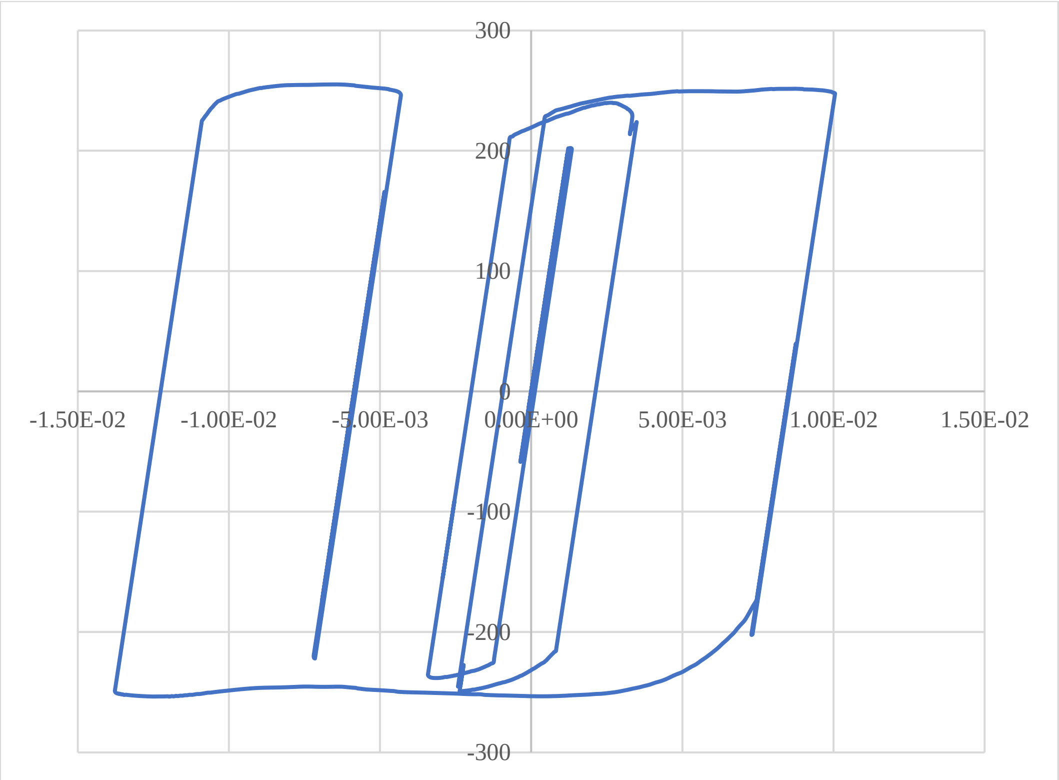


Figure. 6.8: Column rotation capacity in axis-2 of core wall building

Beam rotation capacity in core wall building has been shown in the following figures

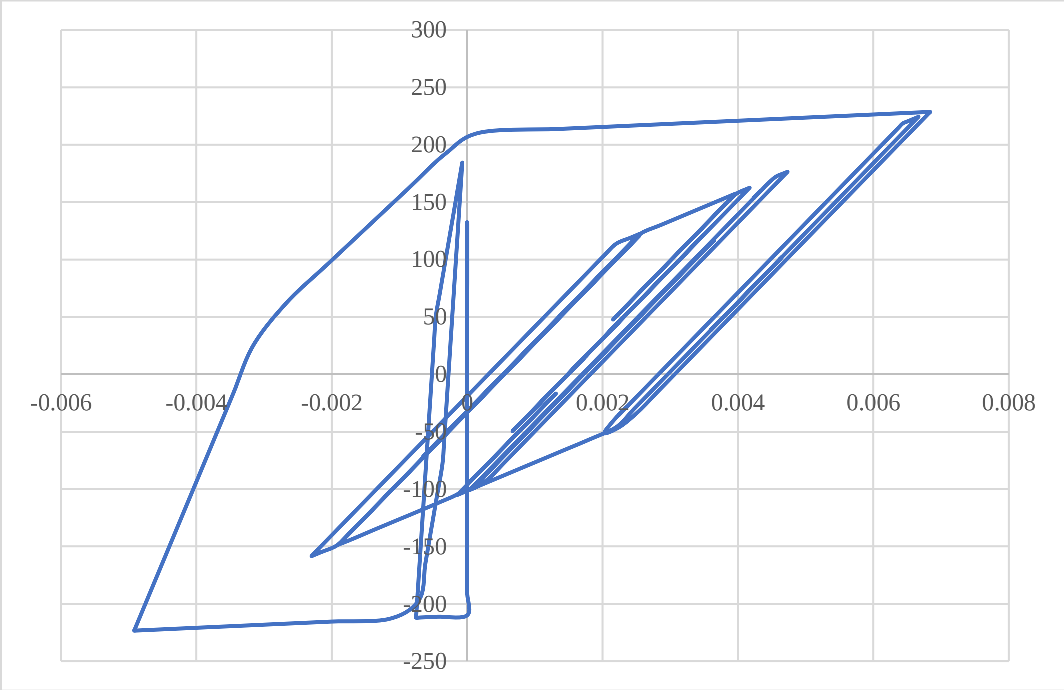


Figure. 6.9: (a) Beam rotation capacity in core wall building

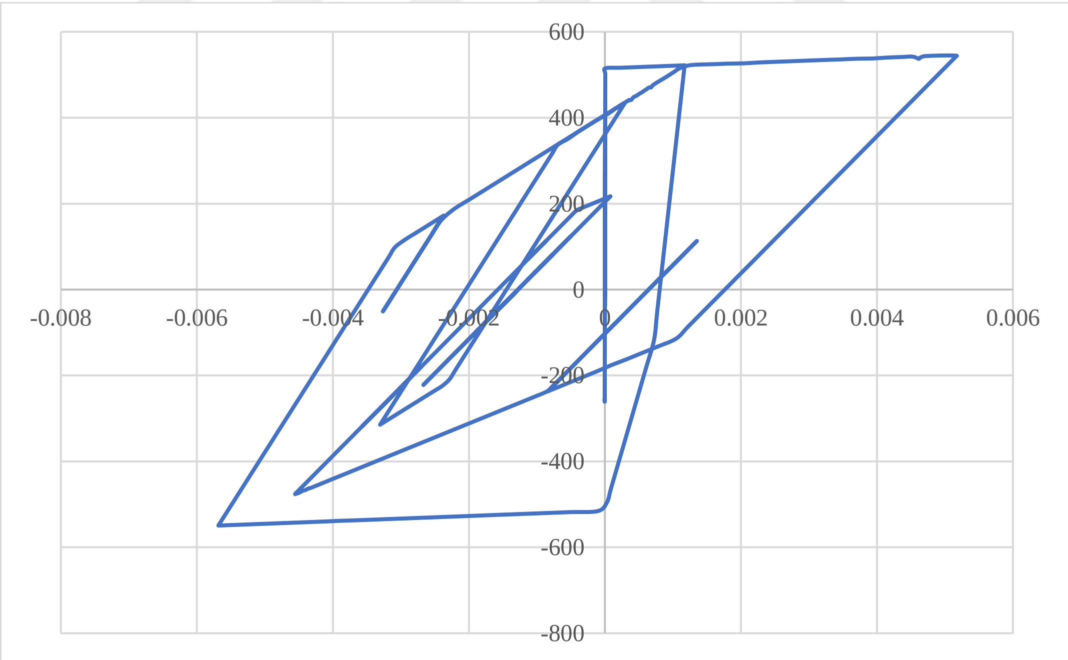


Figure. 6.9: (b) Beam rotation capacity in core wall building

End moment of the column in core wall building system has been shown in the following graph under MCE earthquake level.

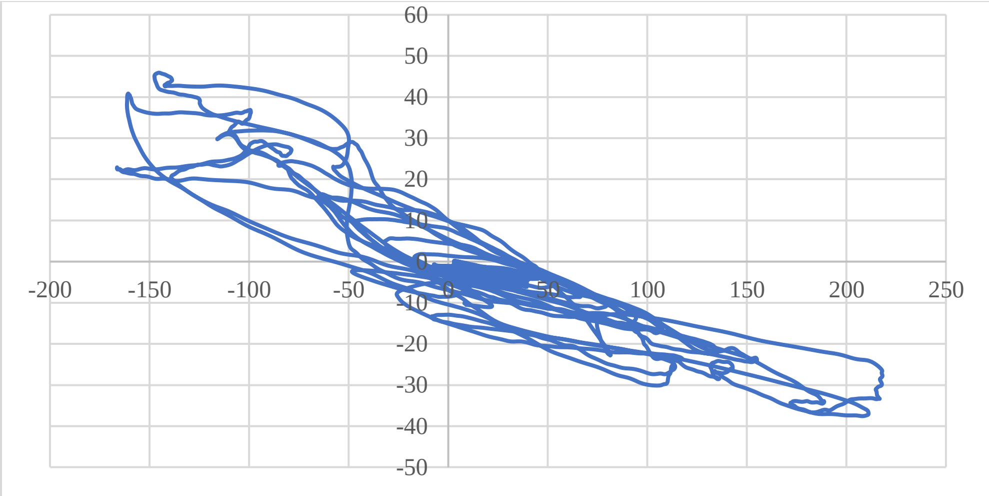


Figure 6.10: End moments of column in core wall building

6.8. Earthquake performance acceptance of 2475 earthquake return period

The buildings performance targets expected under 2475 years earthquake return period for all structural elements. The average of the maximum of the effects obtained from nonlinear time history analyses using (11x2 = 22) different earthquake records in x and y directions respectively. Under this average effect, all the building elements are expected to meet the building performance target. If the building elements meet these earthquake performance targets, it will be assumed that the structure provides the performance of "Collapse Prevention" under the earthquake, which is a return period of 2475 years.

Table 6.6. Earthquake performance targets expected from buildings elements

Elements	**Earthquake Performance Target**	
	Impact type	**Limits**
Shear wall -Concrete	Unit deformation	0.018
Columns	Plastic rotation	0.04-0.06*
Beams	Plastic rotation	0.05
Tie-Beams	Plastic rotation	0.05
All Floors	Avg. Drift ratio	3.5%**
Columns, Beams, Shear walls	Shear Force	Shear force capacities defined by ASCE-7-16***

* Taken from ASCE 41-17 documents depending on the column axial force.
** The average drift ratio rate on the floor.
*** ASCE-7-16 regulations of American Society of Civil Engineering Code

CHAPTER 7

CONCLUSIONS

7.1. About present study

In current research study, the designing, analysis and comparison of seismic performance of core wall and tunnel formwork buildings systems of (G+29) story RC high rise buildings have been performed. The buildings designed with the help of ETABS software and the buildings analyses have been completed in PERFORM-3D analysis software. Also, the buildings costs comparison has been deliberated. The results described in chapter 5 and chapter 6 shown that the specific objectives of the study have been achieved. So, this chapter presents the conclusion derived from the study.

The following conclusions have been made

- First the buildings are designed in ETABS software.
- Second the buildings analysed in PERFORM-3D analysis software.
- Nonlinear dynamic or time history analysis which is the most suitable type of analysis for high rise buildings is used for the analysis of core wall and tunnel formwork buildings.
- Interaction curves of different elements of core wall and tunnel formwork building have been attained.
- Internal forces of columns in core wall building and internal forces for shear wall in both types of buildings obtained.
- Both types of buildings systems either core wall building and tunnel formwork building have shown decent earthquake performance which can frequently be used in the construction industry for the resisting of lateral loads.
- The buildings designs were obtained by an iterative process until the acceptable designs terminated; Which fulfilled all the codes requirements Hence, the buildings are ok.
- The core wall building construction process taking more time and needs many labours but the tunnel formwork building construction process does not need that much time but needs many machineries.
- Both types of building are cost effective.

- The buildings have shown good performance results against lateral earthquake forces.
- Tunnel formwork building has a maximum base shear (9396.6 KN) and the core wall building system has the minimum base shear (6763.2 KN)
- Tunnel formwork building has lower period which is (3.15 Sec), but the core wall building has the higher period of (3.40 Sec).
- 50 mode shapes of each structure have been achieved
- Maximum internal force for both type of buildings system is within a limit
- Maximum lateral displacement or deflection values for each story are obtained for both type of buildings systems.
- Drifts and drift ratios for the core wall and for the tunnel formwork buildings systems are in the acceptable range and revealed the correct results in both x-direction and y-direction respectively.
- Drift in core wall building is more than drift in tunnel formwork building both H1 and H2 directions
- Drifts ratio graphs along the height of the building has been plotted for both types of buildings.
- Drift ratio along the story height is more in core wall building as compare to tunnel formwork building
- Maximum displacement is more in core wall building as compare to tunnel formwork building
- Shear along the height for core wall and tunnel formwork buildings have been accomplished, which shown the acceptable results.
- Moment curvature relationship of columns and shear walls for core wall and tunnel formwork building have been checked individually.
- Based upon the interaction curves of different structural members in core wall and tunnel formwork buildings and based on the capacities of different elements of the buildings it is clear that both type of buildings is under designed. And balanced reinforcements have been used.
- Axial force and strain relationship in shear wall section have been attained

- Shear forces and bending moments envelopes in H1 and H2 directions for core wall and tunnel formwork building have been extracted.
- Stress strain relationship have been obtained
- The total quantities of concrete, formwork or shuttering, and steel rebars quantities have been calculated for core wall and tunnel formwork building respectively.
- The total construction cost for the core wall building is around 30.0 million USD or ($ 29998226.89) and the total construction cost for the tunnel formwork building is around 32.53 million USD or ($ 32527572.39), which is more than the construction cost of core wall building system.
- Both building systems have good earthquake performance
- As the construction cost of tunnel formwork building is more than the construction cost of core wall building that is why we expect the minimum damages after earthquake occurring
- Tunnel formwork building stiffness is more as compare to core wall building
- Both buildings systems responses against lateral loads have been checked which exposed better outcomes
- Finally, both buildings systems are cost-effective, efficient, reliable, durable and shown good earthquake performance.

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APENDICES

Appendix A1: Complete cost estimation for Core wall building

Quantities for Core Wall Building							
Sr. No	Item Description	No	Length (m)	Width/Breadth (m)	Hight/Depth (m)	Quantity	
	Concrete Quantity						
1	**Footing**						
	Footing-1	54	0.55	0.55	0.3	**4.90**	**m^3**
	L/W = 0.55 m, H = 250 mm (0.25 m)						
	Footing-2	3	7.2	0.3	0.3	**1.94**	**m^3**
	L = 7.2 m, W = 0.3 m, H = 0.25 m						
	Footing-3	3	6.6	0.3	0.3	**1.78**	**m^3**
	L = 6.6 m, W = 0.3 m, H = 0.25 m						
				Total Footing Quantity		**8.63**	**m^3**
2	**RCC Columns and Shear Wall at Plinth Level**						
	Pedestal Column-1	54	0.55	0.55	3	**49.01**	**m^3**
	Size = (550 mm x 550 mm)						
	H = 3.3 m (Up to Plinth Level)						
			Total Quantity of Columns (PL)			**49.01**	**m^3**
	Pedestal Shear wall- 1	3	7.2	0.3	3	**19.44**	**m^3**
	Size = (7200 mm x 300 mm)						
	H = 3.3 m (up to Plinth Level)						
	Shear Wall-2	3	6.6	0.3	3	**17.82**	**m^3**
	Size = (6600 mm x 300 mm)						
	H = 3.3 m (up to Plinth Level)						
			Total Quantity of Shear Walls (PL)			**37.26**	**m^3**

3	**DPC at Plinth Level**						
	Beam-1 W = 0.5 m, L = 20 m	1	20	0.5	0.1	**1**	**m^3**
	Beam -2 W = 0.5 m, L = 30 m	1	30	0.5	0.1	**1.5**	**m^3**

				Total Quantity of DPC		**2.5**	**m^3**
4	**RCC Columns in Superstructure**						
	Column -1	54	0.55	0.55	3.3	53.91	**m^3**
	Size = (550 mm x 550 mm)					**539.055**	**m^3**
	H = 3.3 m						
	Column-2	54	0.5	0.5	3.3	44.55	**m^3**
	Size = (500 mm x 500 mm)					**445.5**	**m^3**
	H = 3.3 m						
	Column-3	54	0.45	0.45	3.3	36.0855	**m^3**
	Size = (450 mm x 450 mm)					**360.855**	**m^3**
	H = 3.3 m						
		Total Quantity of Columns (Super structure)				**1345.41**	**m^3**
5	**Shear Walls in Super Structure**						
	Shear Wall-1	3	7.2	0.3	3.3	**21.384**	**m^3**
	Size = (7200 mm x 300 mm)						
	H = 3.3 m						
	Shear Wall-2	3	6.6	0.3	3.3	**19.60**	**m^3**
	Size = 6600 mm x 300 mm						
	H = 3.3 m						
		Total Quantity of Shear Walls (Single Floors)				40.99	**m^3**
		Total Quantity of Shear Walls (All Floors)				**1229.58**	**m^3**
6	**RCC Beam at Slab level**						
	Beam-1 Size = (500 mm x 500 mm)	14	4.5	0.5	0.5	**15.75**	**m^3**
	L = 4.5 m						
	Beam-2 Size = (500 mm x 500 mm)	16	4	0.5	0.5	**16**	**m^3**
	L = 4 m						
	Beam-3 Size = (500 mm x 500 mm)	7	2.9	0.5	0.5	**5.075**	**m^3**
	L = 2.9 m						
	Beam-4 Size = (500 mm x 500 mm)	8	3.6	0.5	0.5	**7.2**	**m^3**
	L = 3.6 m						
	Beam-5 Size = (500 mm x 500 mm)	16	3.4	0.5	0.5	**13.6**	**m^3**
	L = 3.4 m						
	Beam-6 Size = (500 mm x 500 mm)	16	3.5	0.5	0.5	**14**	**m^3**
	L = 3.5 m						
	Beam-7 Size = (500 mm x 500 mm)	12	3.3	0.5	0.5	**9.9**	**m^3**
	L = 3.3 m						

	Beam-8 Size = (500 mm x 500 mm)	9	2.5	0.5	0.5	**5.625**	**m^3**
	L = 2.5 m						
		Total Quantity of Single Floor (Beams)				**87.15**	**m^3**
		Total Quantity of All Floors (Beams)				**2614.5**	**m^3**

7	**RCC Slab**						
	L = 29.30 m, W = 19.30 m	1	29.3	19.3	0.17	**96.13**	**m^3**
	Thickness = 170 mm (0.17 m)						
		Total Quantity of Slab without Deduction				**96.13**	**m^3**
X1	**Deductions**						
	Column						
	Size = (550 mm x 550 mm)	54	0.55	0.55	0.15	**2.45**	**m^3**
	H = 150 mm (0.15 m)						
	Shear Wall						
	Size = (7200 mm x 300 mm)	3	7.2	0.3	0.15	**0.97**	**m^3**
	Size = 6600 mm x 300 mm	3	6.6	0.3	0.15	**0.89**	**m^3**
			Total Deduction			**4.31**	**m^3**
		Actual Total Quantity of Slab (Single Floor) = 96.13 - 3.5				**91.82**	**m^3**
		Actual Total Quantity of Slabs (All Floors)				**2754.60**	**m^3**
Formwork / Shuttering							
01	**Footing Shuttering**						
	Footing-1	54	2.2		0.25	**29.7**	**m^2**
	L = 2 x (0.55+ 0.55) = 2.2 m						
	Footing-2	3	15		0.25	**11.25**	**m^2**
	L = 2 x (7.20 + 0.3) = 15 m						
	Footing-3	3	13.8		0.25	**10.35**	**m^2**
	L = 2 x (6.6 + 0.3) = 13.8 m						
				Total Footing Quantity		**51.30**	**m^2**
02	**Columns Shuttering (Sub-Str.)**						
	Pedestal Column-1	54	2.2		3	**356.4**	**m^2**
	Size = (550 mm x 550 mm)						
	L = 2 x (0.55 + 0.55) = 2.2 m						
			Total Area of Columns (PL)			**356.4**	**m^2**

03	**SW Shuttering (Sub Structure)**						
	Shear Wall-1	3	15		3	**135**	**m²**
	Size = (7200 mm x 300 mm)						
	L = 2 x (7.2 + 0.3) = 15 m						
	Shear Wall-2	3	13.8		3	**124.2**	**m²**
	Size = 6600 mm x 300 mm						
	L = 2 x (6.6 + 0.3) = 13.8 m						
			Total Area of Shear Walls (Plinth Level)			**259.2**	**m²**
04	**RCC Beam Shuttering**						
	Beam-1 Size = (500 mm x 500) mm	14	10		0.5	**70**	**m²**
	L = 2 x (4.5 + 0.5) = 10 m						
	Beam-2 Size = (500 mm x 500) mm	16	9		0.5	**72**	**m²**
	L = 2 x (4 + 0.5) = 9 m						
	Beam-3 Size = (500 mm x 500) mm	7	6.8		0.5	**23.8**	**m²**
	L = 2 x (2.9 + 0.5) = 6.8 m						
	Beam-4 Size = (500 mm x 500) mm	8	8.2		0.5	**32.8**	**m²**
	L = 2 x (3.6 + 0.5) = 8.2 m						
	Beam-5 Size = (500 mm x 500) mm	16	7.8		0.5	**62.4**	**m²**
	L = 2 x (3.4 + 0.5) = 7.8 m						
	Beam-6 Size = (500 mm x 500) mm	16	8		0.5	**64**	**m²**
	L = 2 x (3.5 + 0.5) = 8 m						
	Beam-7 Size = (500 mm x 500) mm	12	7.6		0.5	**45.6**	**m²**
	L = 2 x (3.3 + 0.5) = 7.6 m						
	Beam-8 Size = (500 mm x 500) mm	9	6		0.5	**27**	**m²**
	L = 2 x (2.5 + 0.5) = 6 m						
	L = 2 x (5 + 0.5) = 11 m						
				Total Area of Beams		**397.6**	**m²**
		Total Area of Beams (All Floor)				**11928**	**m²**

05	**RCC Columns Shuttering (SS)**						
	Column-1	54	2.2		3.3	**392.04**	**m²**
	Size = (550 mm x 550 mm)						
	L = 2 x (0.55 + 0.55) = 2.2 m						
		Total Area of Columns (Single Floor)				**392.04**	**m²**
		Total Area of Columns (All Floors)				**11761.2**	**m²**
	Shear Walls Shuttering (SS)						

	Shear Wall-1	3	15		3.3	**148.5**	**m^2**
	Size = (7200 mm x 300 mm)						
	L = 2 x (7.2 + 0.3) = 15 m						
	Shear Wall-2	3	13.8		3.3	**136.62**	**m^2**
	Size = 6600 mm x 300 mm						
	L = 2 x (6.6 + 0.3) = 13.8 m						
		Total Area of Shear Walls (Single Floor)				**285.12**	**m^2**
		Total Area of Shear Walls (All Floors)				**8553.6**	**m^2**
06	**RCC Slab Shuttering**						
	L = 29.30 m, W = 19.30 m	Area = Area - Deduction = 565.5 - 23 = 542.5 Shuttering Area = 542.5 + 17 = 559.5 Sq. m					
	Slab Shuttering = (Area of Slab + Sides x Height)						
	Area = 29.30 x 19.30 = 565.5 Sq. m						
	L = 2 x (30 + 20) = 100 m						
	H = 0.17 m						
	Sides x Height =100 m x 0.17 =17 m2						
		Total Shuttering Slab (Single Floor)				**559.5**	**m^2**
		Total Slab Shuttering (All Floors)				**16785**	**m^2**
Steel Quantity							
	Steel Quantity in Footing						
01	**Footing-1**	**nos. Footing**	**L (m)**	**No of bars**	**W /m**	**Quantity**	
	Main Bars	54	0.85	6	1.58	**435.13**	**Kg**
	L = 0.55 m, H = 250 mm (0.25 m)						
	Concrete Cover = 50 mm						
	C/C distance = 150 mm						
	L = 0.55 - (2 x Side Cover) + 2 x 0.25 - (2 x top and bottom cover) L = 0.55 - (2x0.05) +0.5-(2x0.05) =0.85 m						
	nos of bars = (Total length - 2 x Conc Cover/ CC) + 1 = (0.85 - 2x0.05) / 0.15 =6						
	Unite weight = D^2 / 162 = 1.58						
	Distribution Bars	54	0.85	6	1.58	**435.13**	**Kg**
	W = 0.85 m, H = 250 mm (0.25 m)						

	Footing-2						
	Main Bars	3	7.5	37	1.58	**1315.35**	**Kg**
	L = 7.5 m, W = 0.3 m, H = 0.25 m						
	Distribution Bars	3	0.6	4	1.58	**11.376**	**Kg**
	W = 0.6 m, H = 0.25 m						
	Footing-3						
	Main Bars	3	6.9	34	1.58	**1112.0**	**Kg**
	L = 6.9 m, W = 1.2 m, H = 0.25 m						
	Distribution Bars	3	0.6	4	1.58	**11.376**	**Kg**
	W = 0.6 m, H = 0.25 m						
			Total Steel Quantity in footing			**3320.37**	**Kg**

02	**Steel Quantity in Columns (Plinth Level)**						
		nos. Col	**L (m)**	**No of bars**	**W/m**	**Quantity**	
	Column-1	54	4.5	22	3.85	**20582.10**	**Kg**
	Size = (550 mm x 550 mm)						
	L = 3.0 + 0.25 - bottom Cover + bent up bars + Closed bar on top - 2 x Side cover = 3.3 + 0.25 - 0.05 + 0.70 + 0.70 - 2 x 0.05 L = 4.5 m						
	H = 3.0 m						
	Stirrups/ Lateral ties						
		nos. Stirrups	**L (m)**		**W/m**	**Quantity**	
		22	1.96		0.88	**37.95**	**Kg**
	Dia 150 mm C/c						
	L = 550 - 2 x Side cover - 2 x 1/2 dia of Stirrup L= 550-2 x50 - 2 x 1/2 x 12= 438 mm now 4 x 438 + 2x hooks 2560 + 18 * 12 = 1968 = 1.96 m						
	nos of Stirrups = (3000+250-50-50/150) +1 = 22						
		Total Quantity of Steel in Pedestal Columns				**20620.05**	**Kg**
03	**Steel Quantity in Columns (SS)**						

		nos. Col	L (m)	No of bars	W/m	Quantity	
	Column-1	54	4.3	16	3.85	**14303.52**	**Kg**
	Size = (550 mm x 550 mm)						
	L = 3.3 + bent up bars + Closed bar on top - 2 x Side cover = 3.3 + 0.55 + 0.55 - 2 x 0.05 L = 4.3 m						
	Column-2	54	4.2	16	2.46	**8926.848**	**Kg**
	Size = (500 mm x 500 mm)						
	L = 3.3 + bent up bars + Closed bar on top - 2 x Side cover = 3.3 + 0.75 + 0.75 - 2 x 0.05 L = 4.2 m						
	Column-3	54	4.1	16	2	**7084.8**	**Kg**
	Size = (450 mm x 450 mm)						
	L = 3.3 + bent up bars + Closed bar on top - 2 x Side cover = 3.3 + 0.45 + 0.45 - 2 x 0.05 L = 4.1 m						
	Stirrups/ Rings						
		nos. Stirrups	**L (m)**		**W/m**	**Quantity**	
	Stirrups-1	24	1.96		0.88	**41.40**	**Kg**
	Dia 150 mm C/c						
	L = 550 - 2 x Side cover - 2 x 1/2 dia of Stirrup L= 550-2 x50 - 2 x 1/2 x 12= 438 mm now 4 x 438 + 2x hooks 1752 + 18 * 12 = 1968 = 1.96 m						
	nos of Stirrups = 3300+250-50-50)/150 +1 = 24						
	Stirrups-2	24	1.76		0.88	**37.17**	**Kg**
	Dia 150 mm C/c						
	L = 500 - 2 x Side cover - 2 x 1/2 dia of Stirrup L= 550-2 x50 - 2 x 1/2 x 12= 388 mm now 4 x 388 + 2x hooks 1752 + 18 * 12 = 1768 = 1.76 m						

	nos of Stirrups = 3300+250-50-50)/150 +1 = 24						
	Stirrups-3	24	1.56		0.88	**32.95**	**Kg**
	Dia 150 mm C/c						
	L = 450 - 2 x Side cover - 2 x1/2 dia of Stirrup L= 550-2 x50 - 2 x 1/2 x 12= 338 mm now 4 x 338 + 2x hooks 1352 + 18 * 12 = 1568 = 1.56 m						
	nos of Stirrups = 3300+250-50-50)/150 +1 = 24						
		Total Quantity of Steel in Cols. (Single Floor)				**30426.68**	**Kg**
		Total Quantity of Steel in Cols. (All Floor)				**304266.82**	**Kg**

04	**Steel Quantity in SW (Plinth Level)**						
		nos. Sw	**L (m)**	**nos. Bars**	**W/m**	**Quantity**	
	Shear Wall-1	3	4.8	24	1.57	**542.59**	**Kg**
	Size = (7200 mm x 300 mm)						
	L = 3.0 + 0.25 - bottom Cover + bent up bars + Closed bar on top - 2 x Side cover = 3.0 + 0.25 - 0.05 + 0.3 + 1.25+0.15 - 2 x 0.05 L = 4.8 m						
	H = 3.0 m						
	Shear Wall-2	3	4.8	24	1.57	**542.592**	**Kg**
	Size = 6600 mm x 300 mm						
	L = 3.0 + 0.25 - 0.05 + 0.3 + 1.25 + 0.15 - 2 x 0.05 = 4.8 m						
	H = 3.0 m						
	Stirrups/ Rings						
		nos. Stirrups	**L (m)**		**W/m**	**Quantity**	
	Stirrups-1	25	29.5		0.88	**649**	**Kg**
	12 Dia 150 mm C/c						

	L = 7200 - 2 x Side cover - 2 x 1/2 dia of Stirrup L1 = 2500-2 x50 - 2 x 1/2 x 12 L1 = 7088 mm = 7.08 m now 2 x 7088 + 2x hooks 14176 + 18 * 12 = 14392 = 14.39 m L2 = 300 - 2 x 50 - 2 x 1/2 x 10 =190 now 4 x 190 +180 = 940 L2 = 0.94 m L = L1 + L2 = 14.39 + 0.94 = 15.33 m						
	nos of Stirrups = 3500+125-50-50)/150 +1 = 25						
	Stirrups-2	25	13.35		0.88	**293.70**	**Kg**
	L1 = 6600 - 2 x 50 - 2 x 1/2 x 12 = 6488						
	L2 = 300 - 2 x 50 - 2 x 1/2 x 10 = 190						
	Total L = 2 x (6488+190) = 13356 = 13.35						
		Total Quantity of Steel in SW (PL)				**2027.88**	**Kg**
05	**Steel Quantity in Shear Walls (Super Structure)**						
		nos. SW	**L (m)**	**nos. Bars**	**W/m**	**Quantity**	
	Shear Wall-1	3	5.3	25	1.57	**624.08**	**Kg**
	Size = (7200 mm x 300 mm)						
	L = 3.5 + 0.25 - bottom Cover + bent up bars + Closed bar on top - 2 x Side cover = 3.3 + 0.25 - 0.05 + 0.3 + 1.25+0.15 - 2 x 0.05 L = 5.3 m						
	H = 3.3 m						
	Shear Wall-2	3	5.3	25	1.57	**624.075**	**Kg**
	Size = 6600 mm x 300 mm						
	L = 5.3						
	Stirrups/ Rings						
		nos. Stirrups	**L (m)**		**W/m**	**Quantity**	
	Stirrups-1	24	14.98		0.88	**316.38**	**Kg**
	Dia 150 mm C/c						

	L = 7200 - 2 x Side cover - 2 x1/2 dia of Stirrup L1 = 2500-2 x50 - 2 x 1/2 x 12 L1 = 7088 mm now 2 x 7088 + 2x hooks 9560 + 18 * 10 = 14392 = 14.39 m L2 = 300 - 2 x 50 - 2 x 1/2 x 10 =190 now 2 x 190 + 18 x 12 = 596 L2 = 0.59 m L = L1 + L2 = 14.39+0.59 = 14.98 m						
	nos of Stirrups = 3300+125-50-50)/150 +1 = 24						
	Stirrups-2	24	13.56		0.62	**201.77**	**Kg**
	L1 = 6600 - 2 x 50 - 2 x 1/2 x 12 = 6488 mm						
	L2 = 300 - 2 x 50 - 2 x 1/2 x 12 = 188						
	Total L = 2 x (6488+188) + 18 x 12 = 13568 = 13.56 m						
		Total Quantity of Steel in SW. (Single Floor)				**1766.30**	**Kg**
		Total Quantity of Steel in SW. (All Floor)				**52989.01**	**Kg**
06	**Steel Quantity of Beam**						
	Beam-1 **= (500 mm x 500 mm)**			**No of bars**	**Unit Weight (Kg/m)**		
	Bottom Bar	14	5.76	7	2.46	**1388.62**	**Kg**
	L= 4.5 + 0.7 + 0.7 - 2 x Side Cover + 2 hooks L = 4.5 + 0.7 + 0.7 - 2 x 0.25+ 18 x 0.02 = 5.76 m						
	Top Bar	14	5.76	7	2.46	**1388.62**	**Kg**
	Stirrups (12 mm Dia 200 mm C/C	14	1.75	28	0.62	**425.32**	**Kg**
	L = Beam Dimension - 2 x covers - 2 x (1/2 x Dia of Stirrup = 500- 2 x 25 - 2 x (1/2 x 12) =438 mm Total length = 4 x 438 = 1752 mm						
	nos. of Stirrups = (4.5 + 0.7 + 0.7 - 2 x 0.25 / Spacing) + 1						

	= 4500 + 1400 - 50)/190 = 27 + 1 = 28						
	Beam-2 Size = (500 mm x 500 mm)						
	Bottom Bar	16	5.26	7	2.46	**1449.24**	**Kg**
	L = 4						
	L= 4+ 0.7 + 0.7 - 2 x Side Cover + 2 hooks L = 5 + 0.7 + 0.7 - 2 x 0.25+ 18 x 0.02 = 5.26 m						
	Top Bar	16	5.26	7	2.46	**1449.24**	**Kg**
	Stirrups (12 mm Dia 200 mm C/C	16	1.75	26	0.62	**451.36**	**Kg**

	L = Beam Dimension - 2 x covers - 2 x (1/2 x Dia of Stirrup = 500- 2 x 25 - 2 x (1/2 x 12) =438 mm Total length = 4 x 438 = 1752 mm						
	nos. of Stirrups = (4 + 0.7 + 0.7 - 2 x 0.25 / Spacing) + 1 = 4500 + 1400 - 50)/200 = 25 + 1 = 26						
	Beam-3 Size = (500 mm x 500 mm)						
	L = 2.9 m						
	Bottom Bar	7	4.16	7	2.46	**501.45**	**Kg**
	L= 2.9 + 0.7 + 0.7 - 2 x Side Cover + 2 hooks L = 2.9 + 0.7 + 0.7 - 2 x 0.25+ 18 x 0.02 = 4.16 m						
	Top Bar	7	4.16	7	2.46	**501.45**	**Kg**
	Stirrups (12 mm Dia 200 mm C/C	7	1.75	20	0.62	**151.90**	**Kg**
	L = Beam Dimension - 2 x covers - 2 x (1/2 x Dia of Stirrup = 500- 2 x 25 - 2 x (1/2 x 12) =438 mm Total length = 4 x 438 = 1752 mm						
	nos. of Stirrups = (2.9 + 0.7 + 0.7 - 2 x 0.25 / Spacing) + 1 = 4500 + 1400 - 50)/190 = 19 + 1 = 20						
	Beam-4 Size =						

	(500 mm x 500 mm)						
	L = 3.6 m						
	Bottom Bar	8	4.86	7	2.46	**669.51**	**Kg**
	L= 3.6 + 0.7 + 0.7 - 2 x Side Cover + 2 hooks L = 4.5 + 0.7 + 0.7 - 2 x 0.25+ 18 x 0.02 = 4.86 m						
	Top Bar	8	4.86	7	2.46	**669.51**	**Kg**
	Stirrups (12 mm Dia 200 mm C/C	8	1.75	23	0.62	**199.64**	**Kg**
	L = Beam Dimension - 2 x covers - 2 x (1/2 x Dia of Stirrup = 500- 2 x 25 - 2 x (1/2 x 12) =438 mm Total length = 4 x 438 = 1752 mm						
	nos. of Stirrups = (3.6 + 0.7 + 0.7 - 2 x 0.25 / Spacing) + 1 = 4500 + 1400 - 50)/200 = 22 + 1 = 23						
	Beam-5 Size = (500 mm x 500) mm						
	L = 3.4 m						
	Bottom Bar	16	4.66	7	2.46	**1283.92**	**Kg**
	L= 3.4 + 0.7 + 0.7 - 2 x Side Cover + 2 hooks L = 3.4 + 0.7 + 0.7 - 2 x 0.25+ 18 x 0.02 = 4.66 m						
	Top Bar	16	4.66	7	2.46	**1283.92**	**Kg**
	Stirrups (12 mm Dia 200 mm C/C	16	1.75	22	0.62	**381.92**	**Kg**
	L = Beam Dimension - 2 x covers - 2 x (1/2 x Dia of Stirrup = 500- 2 x 25 - 2 x (1/2 x 12) =438 mm Total length = 4 x 438 = 1752 mm						
	nos. of Stirrups = (3.4 + 0.7 + 0.7 - 2 x 0.25 / Spacing) + 1 = 4500 + 1400 - 50)/200 = 21 + 1 = 22						
	Beam-6 Size = (500 mm x 500) mm						
	L = 3.5						
	Bottom Bar	16	4.76	7	2.46	**1311.48**	**Kg**

	L= 3.5 + 0.7 + 0.7 - 2 x Side Cover + 2 hooks L = 3.5 + 0.7 + 0.7 - 2 x 0.25+ 18 x 0.02 = 4.76 m						
	Top Bar	16	4.76	7	2.46	**1311.48**	**Kg**
	Stirrups (12 mm Dia 200 mm C/C	16	1.75	23	0.62	**399.28**	**Kg**
	L = Beam Dimension - 2 x covers - 2 x (1/2 x Dia of Stirrup = 500- 2 x 25 - 2 x (1/2 x 12) =438 mm Total length = 4 x 438 = 1752 mm						
	nos. of Stirrups = (3.5 + 0.7 + 0.7 - 2 x 0.25 / Spacing) + 1 = 4500 + 1400 - 50)/200 = 22 + 1 = 23						
	Beam-7 Size = (500 mm x 500) mm						
	L = 3.3						
	Bottom Bar	12	4.56	7	2.46	**942.28**	**Kg**
	L= 3.3 + 0.7 + 0.7 - 2 x Side Cover + 2 hooks L = 3.3 + 0.7 + 0.7 - 2 x 0.25+ 18 x 0.02 = 4.56 m						
	Top Bar	12	4.56	7	2.46	**942.28**	**Kg**
	Stirrups (12 mm Dia 200 mm C/C	12	1.75	22	0.62	**286.44**	**Kg**
	L = Beam Dimension - 2 x covers -2 x (1/2 x Dia of Stirrup = 500- 2 x 25 - 2 x (1/2 x 12) =438 mm Total length = 4 x 438 = 1752 mm						
	nos. of Stirrups = (3.3 + 0.7 + 0.7 - 2 x 0.25 / Spacing) + 1 = 21 + 1 = 22						
	Beam-8 Size = (500 mm x 500 mm)						
	L = 2.5 m						
	Bottom Bar	9	3.76	7	2.46	**582.72**	**Kg**
	L= 2.5 + 0.7 + 0.7 - 2 x Side Cover + 2 hooks L = 2.5 + 0.7 + 0.7 - 2 x 0.25+ 18 x 0.02 = 3.76 m						
	Top Bar	9	3.76	7	2.46	**582.72**	**Kg**

	Stirrups (12 mm Dia 200 mm C/C	12	1.75	18	0.62	**234.36**	**Kg**
	L = Beam Dimension - 2 x covers - 2 x (1/2 x Dia of Stirrup = 500- 2 x 25 - 2 x (1/2 x 12)=438 mm Total length = 4 x 438 = 1752 mm						
	nos. of Stirrups = (2.5 + 0.7 + 0.7 - 2 x 0.25 / Spacing) + 1 = 17 + 1 = 18						
		Total Steel Quantity of Beam (Single Floor)				**18788.66**	**Kg**
		Total Steel Quantity of Beam (Whole Building)				**563659.66**	**Kg**

07	**Steel Quantity in Slab**						
	Main Bars	No Slab	L	No of bars	Weight (Kg/m)		
	L = 29.30 m, W = 19.30 m	1	3802	147	1.58	883052.52	Kg
	C/c = 200 mm						
	No of M.B = (Length / C/c) +1 = (29.3 / 0.2) + 1 = 147						
	Total length of Main bar = 147 x 19.3 = 3802						
	Distribution bar	1	2842	97	1.58	435564.92	Kg
	nos of bars= (19.30 / 0.2) +1 = 130	Total Quantity of Steel in Slab				**1318617.44**	**Kg**
	L = 97 x 29.3 = 2871	Total Quantity of Steel in Slab (whole Building)				**39558523.2**	**Kg**

Appendix A2: Complete cost estimation of tunnel formwork building

Quantities for Tunnel Formwork Building							
Sr. No	Item Description	No	Length (m)	Width/Breadth (m)	Hight/Depth (m)	Quantity	
1	**Footing**						
	Footing-1	12	1.5	0.6	0.25	**2.7**	**m^3**
	L = 1.5 m, W = 0.6 H = 200 mm						
	Footing-2	20	2.5	0.6	0.25	**7.5**	**m^3**
	L = 2.5 m, W = 0.6 m, H = 0.20 m						
	Footing-3	11	3.5	0.6	0.25	**5.78**	**m^3**
	L = 3.5 m, W = 0.6 m, H = 0.20 m						
	Footing-4	6	4.5	0.6	0.25	**4.05**	**m^3**
	L = 4.5 m, W = 0.6 m, H = 0.20 m						
	Footing-5	3	5.5	0.6	0.25	**2.48**	**m^3**
	L = 5.5 m, W = 0.6 m, H = 0.20 m						
				Total Footing Quantity		**22.50**	**m^3**
2	**RCC Shear Wall at Plinth Level**						
	Shear Wall-1	12	1	0.3	3.5	**12.6**	**m^3**
	L = 1.0 m, W = 0.3 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-2	20	2	0.3	3.5	**42**	**m^3**
	L = 2.0 m, W = 0.3 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-3	11	3	0.3	3.5	**34.65**	**m^3**
	L = 3 m, W = 0.3 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-4	6	4	0.3	3.5	**25.2**	**m^3**
	L = 4 m, W = 0.3 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-5	3	5	0.3	3.5	**15.75**	**m^3**
	L = 5 m, W = 0.3 m						
	H = 3.50 m (up to Plinth Level)						
			Total Quantity of SW up to PL			**70.35**	**m^3**

3	**DPC at Plinth Level**						
	Wall-1	6	23	0.5	0.1	**6.9**	**m^3**
	L = 23 m, W = 0.5 m, H = 0.7 m						
	Wall-2						
	L = 18m, W = 0.5 m, H = 0.7 m	7	18	0.5	0.1	**6.3**	**m^3**
			Total Quantity of PCC (PL)			**13.2**	**m^3**
4	**PCC in floor**						
	L = 29.30 m, W = 19.30 m	1	29.3	19.3	0.1	**56.55**	**m^3**
	Thickness = 300 mm (0.30 m)						
			Total Quantity of PCC in Floor			**56.55**	**m^3**
5	**Shear Walls in Super Structure**						
	Shear Wall-1	12	1	0.3	3.3	**11.88**	**m^3**
	L = 1.0 m, W = 0.3 m, H = 3.3 m						
	Shear Wall-2	20	2	0.3	3.3	**39.6**	**m^3**
	L = 2.0 m, W = 0.3 m, H = 3.3 m						
	Shear Wall-3	11	3	0.3	3.3	**32.67**	**m^3**
	L = 3 m, W = 0.3 m, H = 3.3 m						
	Shear Wall-4	6	4	0.3	3.3	**23.76**	**m^3**
	L = 4 m, W = 0.3 m, H = 3.3 m						
	Shear Wall-5	3	5	0.3	3.3	**14.85**	**m^3**
	L = 5 m, W = 0.3 m, H = 3.3 m						
		Total Quantity of Shear Wall (Single Floor)				**66.33**	**m^3**
		Total Quantity of Shear Wall (All Floors)				**1989.9**	**m^3**
6	**RCC Slab**						
	L = 29.30 m, W = 19.30 m	1	29.3	19.3	0.3	**169.65**	**m^3**
	Thickness = 300 mm (0.30 m)						
		Total Quantity of Slab without Deduction				**169.65**	**m^3**
X1	**Deductions**						
	Shear Wall-1	12	1	0.3	0.2	**0.72**	**m^3**
	L = 1.0 m, W = 0.3 m						
	H = 150 mm (0.15 m)						
	Shear Wall-2	20	2	0.3	0.2	**2.4**	**m^3**

	L = 2.0 m, W = 0.3 m						
	H = 150 mm (0.15 m)						
	Shear Wall-3	11	3	0.3	0.2	**1.98**	**m^3**
	L = 3 m, W = 0.3 m						
	Shear Wall-4	6	4	0.3	0.2	**1.44**	**m^3**
	H = 150 mm (0.15 m)						
	L = 4 m, W = 0.3 m						
	Shear Wall-5	3	5	0.3	0.2	**0.9**	**m^3**
	L = 5 m, W = 0.3 m						
	H = 150 mm (0.15 m)						
	Opening-1	1	5	3	0.3	**4.5**	**m^3**
	Opening-2	1	4	2	0.3	**2.4**	**m^3**
		Total Deduction				**10.92**	**m^3**
		Actual Total Quantity of Slab (Single Floor) = 169.647 - 9.465				**158.73**	**m^3**
		Actual Total Quantity of Slab (All Floors)				**4761.81**	**m^3**

7	**Formwork / Shuttering**						
	Footing Shuttering						
	Footing-1	12	4.2		0.25	**12.6**	**m^2**
	L = 2 x (1.5+ 0.6) = 4.2 m						
	Footing-2	20	6.2		0.25	**31**	**m^2**
	L = 2 x (2.5 + 0.6) = 6.2 m						
	Footing-3	11	8.2		0.25	**22.55**	**m^2**
	L = 2 x (3.5 + 0.6) = 8.2 m						
	Footing-4	6	10.2		0.25	**15.3**	**m^2**
	L = 2 x (4.5 + 0.6) = 10.2 m						
	Footing-5	3	12.2		0.25	**9.15**	**m^2**
	L = 2 x (5.5 + 0.6) = 12.2 m						
			Total quantity of Footing Shuttering			**90.6**	**m^2**
	Shear Wall Shuttering (Sub Structure)						
	Shear Wall-1	12	2.6		3.5	**109.2**	**m^2**
	L = 2 x (1.0 + 0.3) = 2.6 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-2	20	4.6		3.5	**322**	**m^2**
	L = 2 x (2.0 + 0.3) = 4.6 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-3	11	6.6		3.5	**254.1**	**m^2**
	L = 2 x (3 + 0.3) = 6.6 m						

	H = 3.50 m (up to Plinth Level)						
	Shear Wall-4	6	8.6		3.5	**180.6**	**m^2**
	L = 2 x (4 + 0.3) = 8.6 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-5	3	10.6		3.5	**111.3**	**m^2**
	L = 2 x (5 + 0.3) = 10.6 m						
	H = 3.50 m (up to Plinth Level)						
		Total Area of SW Shuttering up to PL				**977.2**	**m^2**
	DPC Shuttering						
	Wall-1	6	47		0.1	**28.2**	**m^2**
	L = 2 x (23 + 0.5) = 47 m						
	Wall-2						
	L = 2 x (18 + 0.5) = 37 m	7	37		0.1	**25.9**	**m^2**
			Total Area of DPC Shuttering			**54.1**	**m^2**
	Shear Wall Shuttering (SS)						
	Shear Wall-1	12	2.6		3.3	**102.96**	**m^2**
	L = 2 x (1.0 + 0.3) = 2.6 m						
	H = 3.3 m						
	Shear Wall-2	20	4.6		3.3	**303.6**	**m^2**
	L = 2 x (2.0 + 0.3) = 4.6 m						
	H = 3.3 m						
	Shear Wall-3	11	6.6		3.3	**239.58**	**m^2**
	L = 2 x (3 + 0.3) = 6.6 m						
	H = 3.3 m						
	Shear Wall-4	6	8.6		3.3	**170.28**	**m^2**
	L = 2 x (4 + 0.3) = 8.6 m						
	H = 3.3 m						
	Shear Wall-5	3	10.6		3.3	**104.94**	**m^2**
	L = 2 x (5 + 0.3) = 10.6 m						
	H = 3.3 m						
		Total Area of SW Shuttering (Single Floor)				**921.36**	**m^2**
		Total Area of SW Shuttering (All Floors)				**27640.8**	**m^2**

	RCC Slab Shuttering						
	L = 29.30 m, W = 19.30 m	Area = Area - Deduction = 565.5 - 30 = 535.5 Shuttering Area = 535.5 + 17 = 552.5 Sq. m					
	Slab Shuttering = (Area of Slab + Sides x Height)						

	Area = 29.30 x 19.30 = 565.5 Sq. m						
	L = 2 x (30 + 20) = 100 m						
	H = 0.17 m						
	Sides x Height =100 m x 0.17 =17 m2						
		Total Shuttering Slab (Single Floor)				**582.5**	**m^2**
		Total Slab Shuttering (All Floors)				17475	**m^2**
8	**Steel Quantity**						
	Steel Quantity in Footing						
	Footing-1	**nos. Footing**	**L (m)**	**nos. Bars**	**W (Kg/m)**	**Quantity**	
	Main Bars	12	1.8	11	1.58	**375.41**	**Kg**
	L = 1.5 m, W = 0.6 m, H = 200 mm						
	L = 1.5 - (2 x Side Cover) + 2 x 0.25 - (2 x top and bottom cover) L = 1.5 - (2x0.05) +0.5-(2x0.05) =1.8						
	nos of bars = (Total length - 2 x Conc Cover/ CC) + 1 = (1.5 - 2x0.05) / 0.15 =11						
	Unite weight = 16^2 / 162 = 1.58						
	Distribution Bars	12	0.9	7	1.58	**119.45**	**Kg**
	Footing-2						
	Main Bars	20	2.8	19	1.58	**1681.12**	**Kg**
	L = 2.5 m, W = 0.6 m, H = 0.20 m						
	Distribution Bars	12	0.9	7	1.58	**119.45**	**Kg**
	Footing-3						
	Main Bars	11	3.8	26	1.58	**1717.14**	**Kg**
	L = 3.5 m, W = 0.6 m, H = 0.20 m						
	Distribution Bars	11	0.9	7	1.58	**109.49**	**Kg**
	Footing-4						
	Main Bars	6	4.8	33	1.58	**1501.63**	**Kg**
	L = 4.5 m, W = 0.6 m, H = 0.20 m						
	Distribution Bars	6	0.9	7	1.58	**59.72**	**Kg**
	Footing-5						
	Main Bars	3	5.8	39	1.58	**1072.19**	**Kg**
	L = 5.5 m, W = 0.6 m, H = 0.20 m						
	Distribution Bars	3	0.9	7	1.58	**29.86**	**Kg**
			Total Steel Quantity in Footing			**6785.47**	**Kg**

	Steel Quantity in Shear Wall at Plinth Level						
	Shear Wall-1	12	4.55	15	2.46	**2014.74**	**Kg**
	L = 1.0 m, W = 0.3 m						
	L = 3.5 + 0.25 - bottom Cover + bent up bars + Closed bar on top - 2 x Side cover = 3.5 + 0.25 - 0.05 + 0.3 + 0.5+0.15 - 2 x 0.05 L = 4.55 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-2	20	5.05	15	2.46	**3726.9**	**Kg**
	L = 2.0 m, W = 0.3 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-3	11	5.55	15	2.46	**2252.75**	**Kg**
	L = 3 m, W = 0.3 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-4	6	6.05	15	2.46	**1339.47**	**Kg**
	L = 4 m, W = 0.3 m						
	H = 3.50 m (up to Plinth Level)						
	Shear Wall-5	3	6.55	15	2.46	**725.085**	**Kg**
	L = 5 m, W = 0.3 m						
	H = 3.50 m (up to Plinth Level)						
		SW Steel Quantity				**10058.94**	**Kg**

	Stirrups/ Rings						
	Stirrups-1	**nos. Stirrups**	**L (m)**		**W/m**	**Quantity**	
	Size = (1000 mm x 300 mm)	26	4.68		0.62	**75.44**	**Kg**
	L = 1000 - 2 x Side cover - 2 x 1/2 dia of Stirrup L1 = 1000-2 x50 - 2 x 1/2 x 10 L1 = 890 mm now 4 x 890 + 2x hooks 3560 + 18 * 10 = 3740 = 3.74 m L2 = 300 - 2 x 50 - 2 x 1/2 x 10 =190 now 4 x 190 + 180 = 940 L2 = 0.94 m L = L1 + L2 = 3.74+0.94 = 4.68 m						

	nos of Stirrups = 3500+250-50-50)/150 +1 = 26						
	Stirrups-2	26	4.34		0.62	**69.96**	**Kg**
	L1 = 2000 - 2 x 50 - 2 x 1/2 x 10 = 1890 L2 = 300 - 2 x50 - 2 x 1/2 x 10 =190 Total L = 2 x (1890 + 190) + 2 hooks Total L = 4160+180 = 4340 = 4.34						
	Stirrups-3	26	6.34		0.62	**102.20**	**Kg**
	L1 = 3000 - 2 x 50 - 2 x 1/2 x 10 = 2890 mm L2 = 300 - 2 x50 - 2 x 1/2 x 10 =190 Total L = 2 x (2890 + 190) + 2 hooks Total L = 6160+180 = 6340 = 6.34						
	Stirrups-4	26	8.34		0.62	**134.44**	**Kg**
	L1 = 4000 - 2 x 50 - 2 x 1/2 x 10 = 3890 mm L2 = 300 - 2 x50 - 2 x 1/2 x 10 =190 Total L = 2 x (3890 + 190) + 2 hooks Total L = 8160+180 = 8340 = 8.34						
	Stirrups-5	26	10.34		0.62	**166.68**	**Kg**
	L1 = 5000 - 2 x 50 - 2 x 1/2 x 10 = 4890 mm L2 = 300 - 2 x50 - 2 x 1/2 x 10 =190 Total L = 2 x (4890 + 190) + 2 hooks Total L = 10160+180 = 10340 = 10.34						
		Stirrups Steel Quantity				**548.72**	**Kg**
		Total Quantity of Steel in SW(PL)				**10607.66**	**Kg**
	Steel Quantity in Shear Walls (Super Structure)						
	Shear Wall-1	12	4.35	15	2.46	**1926.18**	**Kg**
	L = 1.0 m, W = 0.3 m,						
	L = 3.3 + 0.25 - bottom Cover + bent up bars + Closed bar on top - 2 x Side cover = 3.5 + 0.25 - 0.05 + 0.3 + 0.5+0.15						

	- 2 x 0.05 L = 4.35 m						
	H = 3.30 m						
	Shear Wall-2	20	4.85	15	2.46	**3579.3**	**Kg**
	L = 2.0 m, W = 0.3 m						
	H = 3.30 m						
	Shear Wall-3	11	5.35	15	2.46	**2171.565**	**Kg**
	L = 3 m, W = 0.3 m						
	H = 3.30 m						
	Shear Wall-4	6	5.85	15	2.46	**1295.19**	**Kg**
	L = 4 m, W = 0.3 m						
	H = 3.30 m						
	Shear Wall-5	3	6.35	15	2.46	**702.945**	**Kg**
	L = 5 m, W = 0.3 m						
	H = 3.30 m						
		SW Steel Quantity				**9675.18**	**Kg**
	Stirrups/ Rings						
	Stirrups-1	**nos. Stirrups**	**L (m)**		**W/m**	**Quantity**	
	Size = (1000 mm x 300 mm)	26	4.68		0.62	**75.44**	**Kg**
	L = 1000 - 2 x Side cover - 2 x 1/2 dia of Stirrup L1 = 1000-2 x50 - 2 x 1/2 x 10 L1 = 890 mm now 4 x 890 + 2x hooks 3560 + 18 * 10 = 3740 = 3.74 m L2 = 300 - 2 x 50 - 2 x 1/2 x 10 =190 now 4 x 190 + 180 = 940 L2 = 0.94 m L = L1 + L2 = 3.74+0.94 = 4.68 m						
	nos of Stirrups = 3500+250-50-50)/150 +1 = 26						
	Stirrups-2	26	4.34		0.62	**69.96**	**Kg**
	L1 = 2000 - 2 x 50 - 2 x 1/2 x 10 = 1890 L2 = 300 - 2 x50 - 2 x 1/2 x 10 =190 Total L = 2 x (1890 + 190) + 2 hooks Total L = 4160+180 = 4340 = 4.34						
	Stirrups-3	26	6.34		0.62	**102.20**	**Kg**

	L1 = 3000 - 2 x 50 - 2 x 1/2 x 10 = 2890 mm L2 = 300 - 2 x50 - 2 x 1/2 x 10 =190 Total L = 2 x (2890 + 190) + 2 hooks Total L = 6160+180 = 6340 = 6.34						
	Stirrups-4	26	8.34		0.62	**134.44**	**Kg**
	L1 = 4000 - 2 x 50 - 2 x 1/2 x 10 = 3890 mm L2 = 300 - 2 x50 - 2 x 1/2 x 10 =190 Total L = 2 x (3890 + 190) + 2 hooks Total L = 8160+180 = 8340 = 8.34						
	Stirrups-5	26	10.34		0.62	**166.68**	**Kg**
	L1 = 5000 - 2 x 50 - 2 x 1/2 x10 = 4890 mm L2 = 300 - 2 x50 - 2 x 1/2 x 10 =190 Total L = 2 x (4890 + 190) + 2 hooks Total L = 10160+180 = 10340 = 10.34						
		Stirrups Steel Quantity				**548.72**	**Kg**
		Total Quantity of Steel in SW (Single Floor)				**10223.90**	**Kg**
		Total Quantity of Steel in SW (All Floor)				**306717.14**	**Kg**

	Steel Quantity in Slab						
	Main Bars	No Slab	L	No of bars	Wt (Kg/m)		
	L = 29.30 m, W = 19.30 m	1	3802	147	1.58	883052.52	Kg
	C/c = 200 mm						
	No of M.B = (Length / C/c) +1 = (29.3 / 0.15) + 1 = 197						
	Total length of Main bar = 197 x 19.3 = 3802						
	Distribution bar	1	3809	97	1.58	583767.34	Kg
	nos of bars= (19.30 / 0.15) +1 = 97	Total Quantity of Steel in Slab				**1466819.9**	**Kg**
	L = 130 x 29.3 = 3809	Total Quantity of Steel in Slab (whole Building)				**44004596**	**Kg**